**OREGON DEPARTMENT OF TRANSPORTATION**
**HIGHWAY DIVISION**

# HYDRAULICS DESIGN MANUAL

**PREPARED BY**
**ENGINEERING AND ASSET MANAGEMENT UNIT**
**GEO-ENVIRONMENTAL SECTION**
**April 2014**

**OREGON DEPARTMENT OF TRANSPORTATION**
**PREFACE**

This Hydraulics Manual provides practices and procedures used by the Oregon Department of Transportation for the hydraulic design of highway facilities. It is not intended to be a textbook of hydraulic engineering.

The Hydraulics Manual presents valuable engineering information based on established Oregon Department of Transportation practices in providing satisfactory drainage facilities for a highway project. Sound engineering judgment remains a vital part in the process of applying the design criteria in this manual to individual projects. The Hydraulics Manual is not a legal document.

Susan Haupt,
Geo-Environmental Section Manager

# CHAPTER 1

## INTRODUCTION

## 1.1 General

Various types of drainage facilities are required to convey both subsurface and surface water under, along, or away from the highway. These facilities must be economical and efficient, and they must convey the discharge without damaging the highway or endangering the public.

This manual provides hydrologic and hydraulic analysis procedures for most situations encountered in highway design. The project hydraulics engineer should be contacted for assistance when a user encounters a situation that is not described in this manual. Users are encouraged to request assistance as soon as questions or problems arise. This will reduce the amount of redesign and it provides the time needed to develop a satisfactory solution.

Users should always keep in mind the legal and ethical obligations of the facility owner concerning hydraulic issues. The final project design should be carefully examined to determine if the project causes any significant changes to stormwater quantity and quality, or changes to runoff patterns both upstream and downstream of the project. Care must be taken to ensure that the construction does not interfere with or damage any adjacent or nearby drainage facilities and conforms to applicable environmental regulations.

Updating this manual is a continuing process and revisions are issued as required to enhance content clarity and reflect changes in the regulatory landscape. Technical bulletins may be issued between official chapter updates that address content clarity or errors, and changes in regulations. Future manual updates would supersede outstanding technical bulletins. Users should continually consult Geo-Environmental's website to ensure the most current guidance is being used to design ODOT drainage facilities.

Questions or suggestions for revisions should be addressed to the Hydraulics Engineer in the Engineering and Asset Management Unit of the Geo-Environmental Section, (503) 986-3365.

# CHAPTER 2

# LEGAL ASPECTS

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## 2.1 Introduction

Oregon drainage law and regulations that apply to highway drainage facilities are discussed in this chapter. The intention is to only provide information and guidance on the engineer's role in the legal aspects of highway drainage. This chapter should not be treated as a manual upon which to base legal advice or make legal decisions. It is also not a summary of all existing drainage laws, and most emphatically, this chapter is not intended as a substitute for legal counsel.

The following generalizations can be made in reaching the proper conclusions regarding liability:

- a goal in highway drainage should be to perpetuate the natural drainage, insofar as practical, and
- the courts look with disfavor upon infliction of injury or damage that could be avoided by a prudent designer, even where some alteration of flow is legally permissible.

Additional information about highways and drainage law is in American Association of State Highway and Transportation Officials (AASHTO) Highway Drainage Guidelines. This reference gives advice on expert testimony and the role of the hydraulics engineer during litigation. This manual can be ordered from the AASTHO website.

## 2.2 Oregon Drainage Law

Oregon drainage law, which originates from common law or case law, has developed without legislative action, and it is embodied in the decisions of the courts. Therefore, there are no Oregon Revised Statues to cite pertaining to Oregon drainage law.

Oregon has adopted the civil law doctrine of drainage. Under this doctrine, adjoining landowners are entitled to have the normal course of natural drainage maintained. The lower owner must accept water that naturally comes to his land from above, but he is entitled to not have the normal drainage changed or substantially increased. The lower landowner may not obstruct the runoff from the upper land if the upper landowner is properly discharging the water.

For a landowner to drain water onto lands of another in the State of Oregon, one of two conditions must be satisfied initially: (1) the lands must contain a natural drainage course; or, (2) the landowner must have acquired the right of drainage supported by valuable consideration (i.e. a purchased drainage easement). In addition, because Oregon has adopted the civil law doctrine of drainage, the following three basic elements must be followed.

1. A landowner may not divert water onto adjoining land that would not otherwise have flowed there. "Divert water" includes but is not necessarily limited to:

    a. water diverted from one drainage area to another, and,

    b. water collected and discharged which normally would infiltrate into the ground, pond, and/or evaporate.

2. The upper landowner may not change the place where the water flows onto the lower owner's land. (Most of the diversions not in compliance with this element result from grading and paving work and/or improvements to water collection systems.)

3. The upper landowner may not accumulate a large quantity of water, then release it, greatly accelerating the flow onto the lower owner's land. This does not mean that the upper landowner cannot accelerate the water at all; experience has found the drainage to be improper only when the acceleration and concentration were substantially increased.

Subsurface waters which percolate to the surface can be intercepted and diverted for the protection of the highway without regard for the loss of these waters to the adjacent landowners. In those cases where wells and springs are involved, the right-of-way agent should contact the affected owner(s) to prevent any misunderstanding over damage that could be claimed.

Drainage designs should satisfy Oregon drainage law to avoid claims or litigation resulting from improper drainage design. When it is apparent that the drainage design will not satisfy the law, then drainage easements should be obtained from the affected property owners. The legal staff should be consulted in those situations that appear to be unique and could result in litigation.

Where certain drainage patterns have been established over long periods of time (i.e. in excess of at least 10 years), that are not the original natural drainage, there may be legal rights acquired which allow the continuance of the altered drainage pattern. Again, legal staff should be consulted in such situations.

## 2.3 Common Drainage Complaints

Most complaints about highway-related drainage result from alleged diversion, collection and concentration, augmentation, obstruction, erosion and sedimentation, and groundwater interference. The hydraulics engineer should have a thorough understanding of both the basis of individual complaints when investigating damage claims, and the causes of complaints when evaluating drainage alternatives during highway design. Again, the proper utilization of legal counsel cannot be overstressed. It should be noted that ODOT, by being an owner of property, is in neither a worse or better position than any other property owner in the State of Oregon.

Therefore, the complaints discussed below are also complaints that ODOT can bring against upper and lower property owners.

Complaints regarding drainage conditions should be investigated as soon as possible. If the investigation concludes that the complaint is warranted, then it is advisable to take corrective action.

### 2.3.1 Diversion

Diversion means unauthorized detention or changing the course of a stream or watercourse from the natural or existing condition. ODOT can be held liable for diverting water from a natural waterway; however, courts generally do not disapprove of a change in drainage in which waters are taken out of their natural course and are later returned to such course without material injury to abutting property owners. Where a diversion is necessary, purchase of drainage easements over the lands adversely affected by the diversion will substantially reduce the risk of complaints.

Highway designers often choose to discharge surface waters into the most convenient watercourse. This practice is generally unquestioned if those waters were naturally tributary to the watercourse. However, if all or part of the surface water has been diverted from one watershed to another, any lower landowner may complain and recover for any damage directly attributable to the diversion.

Consider the situation where the upper landowner diverts the stream or conducts other activities that places water on the highway right-of-way from a different place or position than it came originally. The landowner will be responsible to go back and:

- adjust the drainage pattern either to its original position,
- or so that the water does not reach the highway at a point where it would damage the highway or be costly to ODOT to construct new culverts or ditches to drain the water off of the highway right-of-way.

### 2.3.2 Collection and Concentration

A common complaint made regarding new or expanded highways is that surface waters are collected more efficiently, and, therefore, the peak flows at the points of discharge from the highway are greater. The cuts and fills required for highways and the associated ditches and culverts that comprise the highway drainage system collect and concentrate the surface water. The courts have recognized the collection of surface water as an economic necessity to a highway facility. The courts have also imposed limitations on such collection based upon both its reasonableness and upon considerations of significant damage to the landowner.

The highway designer should always analyze points of collection and discharge to see if any unreasonable condition is being imposed upon adjacent property and provide such corrective measures as may be necessary. An example of providing a design measure to preclude damage to downstream property would be to secure a drainage easement or additional right-of-way on the upstream side of the highway for the construction of a detention pond. The detention pond would be designed to limit the outflow to the pre-construction discharge. Obtaining drainage easements on the downstream side of the highway can also preclude liability for property damage and would eliminate the need for the upstream detention pond.

### 2.3.3 Augmentation

An increase in peak flow or volume caused by development is often referred to as augmented flow and also as accelerated flow. As with diversion and collection, a certain amount of augmentation can occur from a highway. The increase in peak flow caused by the highway can be quantified by calculating the peak flow from the watershed with and without the highway. In order for the highway to cause a significant increase in the peak flow, the highway must comprise a large portion of the drainage area in the watershed.

Problems or complaints sometimes occur when the upper landowner discharges more water to the highway right-of-way and ODOT simply passes the water downstream. The lower landowner complains because they are getting more water than they got before and they try to blame ODOT for the increased flow. ODOT should inform the lower landowner that the problem is caused by the upper landowner and it is not ODOT's responsibility to take legal action against the upper landowner. The exception would be if the increased flow were causing damage to the highway, in which case ODOT could take legal action against the upper landowner for the damage to the highway.

Another problem that occurs from land development is that ODOT drainage structures that were once adequate are now inadequate due to increased flows from the development. If the upper landowner increases the flow such that the highway drainage facilities are no longer adequate, it is the upper landowner's responsibility for paying for enlarging the highway drainage facilities to adequately pass the increased flow. They are also responsible for damages that the lower landowners may claim were caused by the increased flow.

### 2.3.4 Obstruction

Backwater from a bridge or culvert is a common basis of complaint involving obstruction. A basis for complaint may exist when damage to private property results from undersized drainage structures in highway embankments. In the case of a highway paralleling a stream, a roadway embankment that encroaches on the stream's floodplain may obstruct the stream flow in the floodplain. Lack of maintenance of an otherwise adequate opening can be considered obstruction.

Two important legal aspects the hydraulics engineer should consider when evaluating the effects of obstruction are:

- courts have held that the extent of liability due to obstructing flow is limited to the amount of the increase in damages attributable to the obstruction, and
- regardless of the design frequency and discharge used, ODOT can be held liable for the backwater damage. From a legal standpoint, the determination of liability is based on whether the waterway opening was adequate for a flow that reasonably could have been anticipated at the time the opening was constructed (this will include considering the likelihood of possible flood conditions). Since the determination of liability is made "after-the-fact" in a court of law, it is important when selecting the size of an opening to evaluate the backwater damage potential for discharges that exceed the design discharge.

### 2.3.5 Erosion and Sedimentation

Erosion of private property that can be attributed to the highway can be a basis for a claim. Sediment originating from a highway and deposited off the highway right-of-way can also generate complaints. Therefore, it is important to establish the natural erosion and sedimentation conditions in order to assess the possibility that the observed erosion and sedimentation would have occurred under natural conditions. Erosion and sedimentation problems from highway construction and operation can be minimized through proper design and construction of temporary and permanent erosion and sediment control features. These features are discussed in ODOT's Erosion and Sediment Control Manual.

### 2.3.6 Groundwater Interference

Groundwater is often encountered during highway construction. Similar to the laws governing surface waters, any temporary or permanent interference with the flow, quality or level of groundwater can be a basis for a complaint.

Excavations and de-watering operations may deplete groundwater previously available for irrigation and domestic supply. Embankments may compress underlying water bearing soils and restrict the circulation of groundwater, thereby depriving users of normal flow. Another complaint is an alleged decrease in the quality of groundwater as a result of contaminants from highway runoff.

Where groundwater interference is a potential basis for complaint, the groundwater level and quality should be investigated and documented prior to the beginning of construction.

## 2.4 National Flood Insurance Program

In 1968 the U.S. Congress established the National Flood Insurance Program (NFIP) with passage of the National Flood Insurance Act. The act authorized the Federal Emergency Management Agency (FEMA) to conduct flood studies. The purpose of the studies is to support state and local efforts to:

- regulate the development of land which is exposed to flood damage, where appropriate,
- guide proposed construction away from locations which are threatened by flood hazards,
- assist in reducing damages caused by floods, and
- otherwise improve the long-range land management and use of floodprone areas.

The NFIP is designed to benefit individuals and communities. It enables property owners to purchase flood insurance at reasonable rates, and it requires participating cities and counties to adopt and administer legally enforceable local floodplain management measures aimed at protecting lives and new construction from flooding.

FEMA publishes the flood insurance studies and administrates the NFIP. Once a Flood Insurance Study is published any revisions to the study must be approved by FEMA.

NFIP regulations that are administered by FEMA are contained in Title 44, Chapter 1 of the Code of Federal Regulations (CFR), Parts 59 to 75. The regulations along with a wealth of other information about the NFIP can be found at FEMA's website.

### 2.4.1 Flood Insurance Studies and Floodways

The published flood insurance studies contain water surface profiles and floodplain maps of flood hazard areas. The water surface profiles should include the backwater effects of existing highway facilities that are located in the study area. For waterways studied in detail, water surface profiles are published for the 10-year, 50-year, 100-year and 500-year floods. For the 100-year flood, a floodway is usually established and the water surface profile associated with the floodway is published in the flood study. The floodway is the channel of the stream, plus any adjacent flood plain areas that must be kept free of encroachment so that the 100-year flood can be conveyed without increasing the water surface elevation more than 1.0 foot. Figure 2.1 shows the floodway schematic. In some instances, local agency officials have adopted a floodway that allows less than a 1.0 foot rise. Therefore, the applicable city or county floodplain ordinance should be consulted for the standards that are in effect.

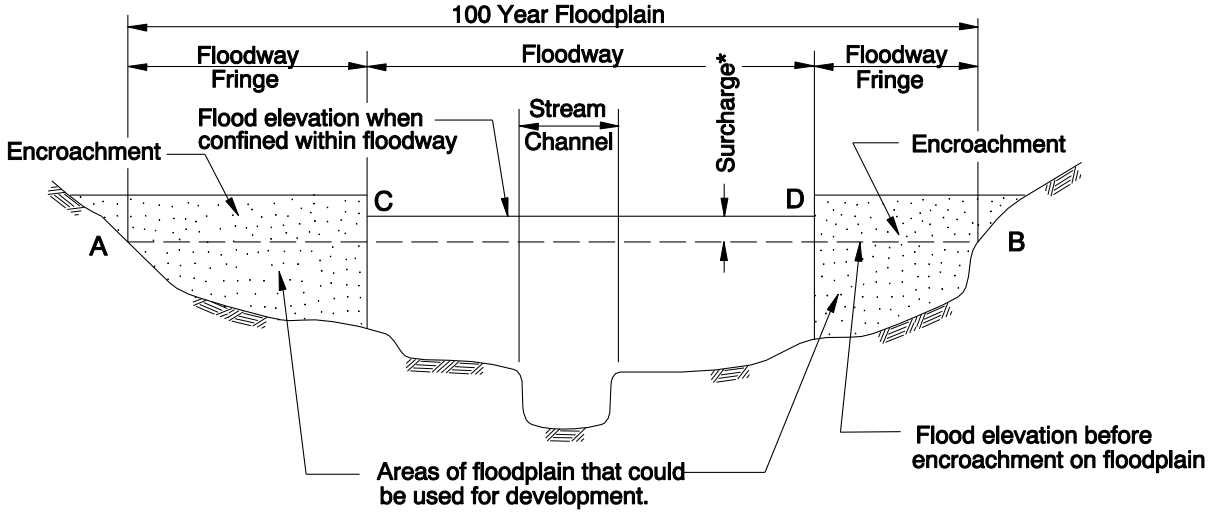
### 2.4.2 NFIP Maps

Three types of NFIP maps are published:

- Flood Hazard Boundary Maps (FHBM)
- Flood Boundary and Floodway Maps (FB&FM) and
- Flood Insurance Rate Maps (FIRM)

A FHBM is generally not based on a detailed hydraulic study and, therefore, the floodplain boundaries shown are approximate. A FB&FM is derived from a detailed hydraulic study and it should provide reasonably accurate information. The FB&FM will show the boundaries of the floodway, 100-year floodplain, and 500-year floodplain. The FIRM map will usually show the boundaries of the floodway, 100-year floodplain, and 500-year floodplain as well as the flood insurance rate zones and the 100-year flood elevations. The FIRM map is the basis for establishing flood insurance rates. Recent Flood Insurance Studies do not include the FB&FM maps. The floodway elevation and boundary information for studies since 1990 are included on the FIRM maps.

Cities and counties may or may not have published one or more of the above maps depending on their level of participation in the NFIP. The Geo-Environmental Section's Engineering and Asset Management Unit has one copy of each available map. Maps are also available on FEMA's website.




Line A - B is the flood elevation before encroachment
Line C - D is the flood elevation after encroachment

* MAXIMUM SURCHARGE 1.0 FOOT. A LESSER MAXIMUM MAY BE SPECIFIED BY FEMA OR THE LOCAL JURISDICTION. CONSULT FLOOD INSURANCE STUDY. IN MANY LOCATIONS A PORTION OF THE SURCHARGE HAS BEEN USED AS A RESULT OF EXISTING ENCROACHMENTS.

**Figure 2-1 Floodway Schematic**

### 2.4.3 Coordination with Local Jurisdictions and FEMA

There should be ODOT coordination with local jurisdictions and FEMA in situations where administrative determinations are needed involving a regulatory floodway or where flood risks in NFIP communities are significantly impacted. The circumstances that would ordinarily require coordination with local jurisdictions and FEMA include the following:

- when a proposed crossing encroaches on a regulatory floodway and it would require an amendment to the floodway map,
- when a proposed crossing encroaches on a floodplain where a detailed study has been performed but no floodway designated and the maximum 1.0 foot increase in the 100-year flood elevation would be exceeded in the vicinity of insurable buildings,

- when a local agency is expected to enter into the regular program within a reasonable time period and detailed floodplain studies are underway, and
- when a local agency is participating in the emergency program and the 100-year flood elevation in the vicinity of insurable buildings is increased by more than 1.0 foot. Where insurable buildings are not affected, it is sufficient to notify FEMA of changes to the 100-year flood elevation as a result of highway construction.

The Draft Environmental Impact Statement or Environmental Assessment (EIS/EA) should indicate the NFIP status of affected communities, the encroachments anticipated, and the need for a floodway revision. Coordination means furnishing to FEMA the draft EIS/EA and, upon selection of an alternative, furnishing to FEMA, through the local agency, a preliminary site plan and water surface elevation information and technical data in support of a floodway revision request as required. If a determination by FEMA would influence the selection of an alternative, a commitment from FEMA should be obtained prior to the Final Environmental Impact Statement (FEIS) or a finding of no significant impact (FONSI). Otherwise, this later coordination may be postponed until the design phase.

### 2.4.4 Floodway Revisions

In most situations, it is possible to design and construct highways in a cost-effective manner such that their components are excluded from the floodway. This is the simplest and preferred way to be consistent with the standards and should be the initial alternative evaluated. If a project element encroaches on the floodway but has a very minor effect on the floodway water surface elevation (such as piers in the floodway), the project may normally be considered as being consistent with the standards. Hydraulic conditions, however, must be improved so that no water surface elevation increase is reflected in the computer output for the new conditions. One way to compensate for the effect of piers in the floodway is to lengthen the bridge to offset the backwater created by the piers. Other less environmentally friendly solutions include channel excavation or a reduction of Mannings “n” value by removing the vegetation on the channel banks and overbank areas.

A second alternative would be a modification of the floodway itself, where it is not cost-effective to design a highway crossing to avoid encroachment on an established floodway. Since the local agency selects and adopts the floodway, the local agency must agree to modify the floodway, and the request to FEMA to revise the floodway must come from the local agency. Often, the local agency will be willing to accept an alternative floodway configuration to accommodate a proposed crossing provided NFIP limitations on increases in the 100-year flood elevation are not exceeded. This approach is useful where the highway crossing does not cause more than a 1.0 foot rise in the 100-year flood elevation. In some cases, it may be possible to enlarge the floodway or otherwise increase conveyance in the floodway above and below the crossing in order to allow greater encroachment. Such planning is best accomplished when the floodway is

first established. However, where the local agency is willing to amend an established floodway to support this option, the floodway may be revised. Floodway revisions are a lengthy process.

The responsibility for demonstrating that an alternative floodway configuration meets NFIP requirements rests with the local agency. This responsibility will be borne by ODOT for proposed ODOT highway construction projects that impact floodways. ODOT will supply the local agency with revised flood profiles, floodway and floodplain mapping, and background technical data required by FEMA to revise the floodway. After the local agency receives the data necessary to revise the floodway, it is the local agency's responsibility to forward the data to FEMA with a request to revise the floodway.

Floodway modifications resulting in a base flood increase greater than 1.0 feet are eligible for Federal Highway Administration (FHWA) funding in some instances. FHWA floodplain development requirements are in federal regulation 23 CFR 650 Subpart A. This regulation along with explanatory supplements can be viewed on the FHWA website http://www.fhwa.dot.gov.

### 2.4.5 Geo-Environmental Section's Engineering and Asset Management Unit and Region Responsibilities

Relative to the NFIP, the ODOT Geo-Environmental Section's Engineering and Asset Management Unit has the responsibility to advise and assist ODOT personnel with NFIP and FEMA related issues.

The ODOT Regions have the following responsibilities:

- coordinate with FEMA in situations where administrative determinations are needed involving a regulatory floodway or where flood risks in NFIP communities are significantly impacted,
- coordinate with FEMA and the local agency and prepare the data necessary for the local agency to submit to FEMA for floodway revisions,
- provide the engineering analysis necessary for projects to conform to the local floodplain regulations, and
- provide engineering certification required to obtain the floodplain permit from the local agency. The required engineering certification will be one or more of the following:

    - a no-rise certificate or statement of no rise where increased flood elevations are not allowed or desired,
    - a statement that proposed development is limited to the floodway fringe or that the proposed development does not increase surcharge above the published values,
    - a statement that proposed changes within a floodway do not reduce floodway conveyance, or

- a floodway boundary revision request if the proposed changes raise floodway water surface elevations above the published values.

Calculations and other documentation may be required or desired to support the preceding statements.

*Note: The seal of a professional engineer is required on many certification documents, as discussed in* ***Chapter 3****.*

# CHAPTER 3

# POLICY

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## 3.1 Introduction

The purpose of this chapter is to:

- define the hydraulic responsibilities of the ODOT Geo-Environmental Section's Engineering and Assets Management Unit, the Bridge Preservation Unit, and Region Technical Centers,
- define responsibilities of the hydraulics engineer, hydraulics designer, and designer; and
- present general ODOT policies and practice for the hydraulic design of highways,
- present an overview of ODOT fish passage policy.

Policy is defined as a definite course of action that guides and determines present and future decisions. Design practice and criteria are developed to implement the specific policy. Policy, practice, and criteria that are specific to a particular topic are presented in the individual Chapters.

## 3.2 Core Hydraulic Responsibilities, ODOT Geo-Environmental Section's Engineering and Asset Management Unit

The Geo-Environmental Section's Engineering and Asset Management Unit, is responsible for developing and maintaining the ODOT Hydraulics Manual, standards of practices, standard drawings, and specifications in the Hydraulics discipline. Core functions of this unit include:

- develop and maintain ODOT hydraulic engineering standards, practice, and standard drawings,
- review and approve design software,
- provide leadership on technical committees,
- promote transfer of technology,
- provide training,
- assistance and advise department personnel and consultants involved in critical drainage related matters,
- perform quality assurance review of agency personnel and consultant design products,
- review hydraulic design service Request-for-Proposals (RFP's), Agreements-to-Agree (ATA's), and Work Order Contracts (WOC's),
- advise and assist department personnel and consultants with matters pertaining to local jurisdiction regulations,
- advise and assist the Department of Justice on legal matters pertaining to ODOT,
- maintain the hydraulics library and files, and
- provide technical advice on roadway drainage, storm drains, culverts, water quality treatment, retention, detention, control systems (gates, valves, and weirs), pump stations,

energy dissipators, fish passage, trenchless technology, and temporary water management.

## 3.3 Core Hydraulic Responsibilities, ODOT Bridge Standards & Practices Unit

The ODOT Bridge Section Standards & Practices Unit manages the bridge scour program and non-National Bridge Inventory large culvert program. The Unit has core functions for hydraulics and structural design, as follows:

- support ODOT hydraulic engineering standards, practice, and standard drawings,
- review and approve design software,
- provide leadership on technical committees,
- promote transfer of technology,
- provide training,
- assist and advise department personnel and consultants involved in critical drainage related matters,
- perform quality assurance review of agency personnel and consultant design products,
- review hydraulic design service RFP's, AGA's, and WOC's,
- advise and assist department personnel and consultants with matters pertaining to local jurisdiction regulations,
- advise and assist the Department of Justice on legal matters pertaining to ODOT,
- forward Oregon Division of State Lands Fill and Removal permit applications to Region staff for their review and comment,
- maintain the scour program files and a library of structural design references for hydraulic facilities, and
- provide technical advice on bridges, scour, revetment, large culverts, open-channel hydraulics, stream bank stabilization, and coastal erosion.

## 3.4 Core Hydraulic Responsibilities, ODOT Region Technical Centers

ODOT divides its highway operation into five geographical regions. Each region is responsible for developing, designing and managing the construction of highway projects. Each region's technical center's unit with a hydraulics engineer on staff is responsible for the design activities considered to be hydraulic engineering such as:

- design of all hydraulic features to be included on project plans, and
- design of all hydraulic features to be included on maintenance repair or other plans,
- certification to verify hydraulic structures replaced in kind meet current hydraulic design standards and are appropriate for the application,

- preparation of preliminary reports and recommendations (see Chapter 4),
- preparation of engineering material supporting permit applications (see Chapter 4), and
- services as an expert witness representing ODOT during litigation,
- project scoping and planning where detailed written recommendations are required for engineered hydraulic structures, and
- technical review of Environmental Impact Statements, Biological Assessments, and other environmental documents addressing engineered hydraulic structures.

## 3.5 Professional Engineer

Many aspects of ODOT hydraulics are the responsibility of a professional engineer (P.E.) registered in Oregon. These engineers are employed by ODOT, consultants and their employees working under contract to ODOT, or others. Professional engineering involvement is required by both agency policy and practice.

### 3.5.1 Policy

Final engineering documents such as plans, specifications, calculations, and reports, either within the agency or externally from the agency, shall be stamped by the registrant who prepared the design, or in the case of a non-registered person, by the registrant in technical supervision and control of the design. For more detailed information use the following web page link to access the ODOT Professional of Record Guidance: Technical Guidance Web Page

### 3.5.2 Practice

A licensed professional with sufficient expertise in hydraulic engineering should be in responsible charge of the following ODOT project design activities:

- hydraulics reports,
- stormwater reports,
- operation and maintenance manuals,
- culverts, medium and large (span, diameter, or equivalent diameter greater than 4 feet),
- structural design of culverts, pipe, and arches including headwalls and wingwalls,
- fish passage (culverts, waterway enhancement),
- bridge hydraulics including waterway openings, channel widths, scour depths, backwater and floodwater elevations,
- bridge deck drainage,
- floodplain hydraulics and documentation such as “no-rise” certifications and floodway revisions,
- large open channel designs (Q is greater than 50 cubic feet per second or grades steeper than 10 percent),
- in-stream channel modifications or restoration,

- outlet or grade control structures such as tide gates, weirs, etc.,
- stormwater control facility designs (excluding design of low impact development approaches),
- underground injection control facilities (drywells),
- storm drain system designs (diameter greater than 24 inches),
- structural scour and erosion protection designs,
- streambank stabilization (including bio-stabilization approaches),
- complex energy dissipators,
- coastal engineering,
- water budget analysis,
- pump stations,
- temporary water management designs, and
- trenchless technology designs

These project design activities are considered to be hydraulic engineering based on ODOT practice and they can be done under the seal of an engineer of another specialty. The engineer, however, must have sufficient expertise to either perform or oversee the work.

- Design of culverts 4 feet or less span, diameter, or equivalent diameter, with no fish passage issues.
- Design of small channels with design discharges equal to or less than 50 cubic feet per second on grades of 10 percent or less, with no fish passage issues.
- Stormwater treatment (Best Management Practices, only).
- Simple energy dissipators.

These activities do not require the seal of an engineer. The person doing the activity, however, must have sufficient expertise to do the work, or they must do it under the immediate supervision of someone who does.

- Support to ODOT maintenance for drainage problems and complaints where expert witness services or designs are not needed.
- Review of drainage designs submitted to ODOT Maintenance by private interests and others requesting to discharge runoff to ODOT drainage facilities.
- Project hydraulic scoping and planning where general comments are needed.
- Review and comment on Oregon Department of State Lands Fill and Removal permit applications.

## 3.6 General ODOT Policies

General ODOT policies that pertain to highway drainage are as follows,

1. Drainage facilities shall be designed to convey surface water through, along, or away from the highway by the most direct means consistent with economy, ease of maintenance, and the least disturbance of natural conditions without damaging the highway or adjacent property.

2. Safety of the general public is an important consideration in the hydraulic design of highways.

3. Hydraulic design shall be coordinated with other Federal, State, or local agencies concerned with water resources planning.

4. All drainage designs shall satisfy the precedent established by Oregon drainage law.

5. During the design phase the appropriate maintenance personnel shall be contacted to identify past maintenance problems and other maintenance concerns. The drainage facility shall be designed to accommodate the unique maintenance requirements of the site. The design of the drainage facility shall consider the frequency and type of maintenance expected, and to make allowance for access of maintenance equipment and personnel.

6. Any ODOT maintenance activities that may alter a stream or river's hydraulics shall be designed or reviewed by a professional engineer with expertise in hydraulics. Activities include but are not limited to bank stabilization projects; design of stream barbs or other flow deflectors; in-stream boulder and root wad placement; and retrofitting culverts for fish passage. A professional engineer, upon review of the in-stream work, may authorize maintenance or other ODOT personnel to complete the work without a sealed design.

7. Bank stabilization methods that provide an environmental benefit shall be considered and used when feasible to protect highway embankments and bridge abutments/approach fills. Biostabilization is not to be used as the sole method of protection in areas where loss of the unprotected highway is likely.

8. All hydraulic designs will strive to meet the letter, spirit, and intent of environmental laws and regulations. During the design process, coordination should occur between the designer and the appropriate State or Federal agencies to ensure that the proposed design satisfies the applicable environmental laws and regulations. If possible, the design should avoid the resource. If the resource cannot be avoided, the hydraulic design should

minimize the impact to the resource. Where the resource cannot be avoided, and where minimization harms the resource, then the design must incorporate mitigation measures to compensate for the harmful impacts.

9. Hydraulic Reports or memoranda should be prepared for all culverts larger than 4 feet in diameter, all bridges, channel changes on large streams, and other significant hydraulic structures such as energy dissipators and riverine or tidal control structures. Stormwater Reports should be prepared for all stormwater conveyance facilities larger than 24 inches in diameter. Stormwater Reports and Operation and Maintenance Manuals should be prepared for all stormwater treatment and/or storage facilities, pump facilities, and control structures. The detail of the studies, reports, and memoranda should be commensurate with the risk associated with the drainage facility and with other economic, engineering, social, or environmental concerns. A registered professional engineer shall stamp all hydraulic or stormwater reports or memoranda.

## 3.7 Design Recurrence Interval

Recurrence intervals are a means of placing a statistical value on rainfall events. A recurrence interval (RI) of 50 years means that an event of that magnitude would, over a long period of record, occur on an average of once in 50 years. It is not correct to say that a 50-year event will occur at 50-year intervals since such an event could occur twice in one year although the statistical chance of this happening is low. Rainfall events can also be expressed in terms of the exceedance probability that the event will be equaled or exceeded during any year. The exceedance probability, expressed in percent, equals 100 / RI. For example, there is a 2 percent chance that a 50-year event will be equaled or exceeded in any year.

Design Recurrence Intervals are listed in Table 3-1. The design event should not inundate the travel lane. There may be situations where the standard design event is not appropriate for the site.

**Table 3-1 Design Recurrence Interval (Years)**

| Drainage Facility | Freeways | Highways Other Than Freeways | |
|---|---|---|---|
| | | ADT[1] less than 750 | ADT[1] greater than or equal to 750 |
| Bridge Openings[2,4] | 50 | 25 | 50 |
| Bridge Scour | See Chapter 10 | See Chapter 10 | See Chapter 10 |
| Bank Protection | 50 | 25 | 50 |
| Culverts[2,4] | 50 | 25 | 50 |
| Ditches, Inlets and Gutters | 10 | 10 | 10 |
| Depressed Roadways | 50 | 25 | 50 |
| Energy Dissipators[3] | 50 | 25 | 50 |
| Storage Facilities | See Chapter 12 | See Chapter 12 | See Chapter 12 |
| Water Quality Facilities | See Chapter 14 | See Chapter 14 | See Chapter 14 |
| Storm Drains | 10 | 10 | 10 |
| Storm Drain Outfalls from Sags | 50 | 25 | 50 |
| Temporary Drainage Facilities[5] | See Section 3.10 | See Section 3.10 | See Section 3.10 |
| Channel Changes[2,4] | 50 | 25 | 50 |

**Table 3-1, Cont. Design Recurrence Interval (Years)**

**Notes**

[1]Average daily traffic (ADT). Part 1 of the project prospectus will give the existing and proposed ADT. The proposed ADT should be used to determine the design event.

[2]Designs should be analyzed for the base flood. This is done to assess the potential effects of inundation from floods that exceed the design event. Designs should be checked for either the 500-year flood if more frequent than the incipient roadway overtopping event, or the incipient roadway overtopping event if more frequent than the 500-year flood. The check is done to assess the potential scour and other damage to the installation from floods that exceed the design event.

[3]The design should be checked for a range of discharges from the 2-year through the 500-year event to ensure that the dissipator will perform as intended.

[4]A 100-year design flood recurrence interval is used for facilities in floodplains subject to federal National Flood Insurance Program regulations and facilities on U.S. Department of Agriculture Forest Service land. The 100-year flood is also known as the base flood.

[5]Temporary drainage facilities used for erosion control use design and check discharges as per ODOT erosion control policies, guidelines, and criteria.

The following risk factors should be considered when justifying a non-standard design event.

1. The impacts associated with interrupting traffic caused by flooding of the highway. The situation may be evaluated by considering the average daily traffic volume (ADT); the traffic delay incurred; the overall importance of the route, including the provision of emergency supply and rescue; and the availability of alternate routes. The importance of the route to national defense and to the economic well being of a community should play a large role in evaluating the risk of traffic interruptions.

2. The potential for the loss of life. Factors to be considered in potential loss of life situations should include the duration, depth, and velocity of hazardous floodwaters, the dependability of adequate warning systems, sight distances, potential for loss of structure or road washout, and the availability of detours.

3. The potential for property damage whether it is public or private. Flood damage includes eroded highway embankments, loss of highway structures, and damage to adjacent property.

## 3.8 Fish Passage

State agencies including ODOT have been directed by the Governor to operate within the provisions of the Oregon Plan for Salmon and Watersheds. The Oregon Plan represents an unprecedented undertaking on the part of the State of Oregon to restore salmon and trout resources. ODOT's goal is to help restore populations and fisheries to productive and sustainable levels that will provide substantial environmental, cultural, and economic benefits by removing all fish passage barriers caused by ODOT structures.

The Oregon Transportation Commission authorized the ODOT Fish Passage program in 1997. Barriers to fish passage for salmonids and other species of interest will be removed from the state highway system as funding permits. The current funding level is $3,000,000 per year to replace structures on ODOT right of way that are a barrier to fish passage.

The main regulatory agencies for fish passage are the Oregon Division of State lands (DSL) and the US Army Corps of Engineers (USACE). The principal resource agencies that provide biological opinions and other environmental data to the regulatory agencies are the Oregon Department of Fish and Wildlife (ODFW) and the National Oceanic Atmospheric Administration (NOAA). ODOT has been working with these regulators as well as US Fish and Wildlife who regulates other non migratory species.

An impasse site is a location on a stream where fish cannot pass due to a full or partial blockage

from an ODOT structure. Each site is classified as high, medium or low priority for replacement/improvement. ODOT has committed to repair all of the high priority sites first and then work through the medium and low priorities.

There are basically two options for addressing impasse sites. One is to replace the structure, and the other is to retrofit the existing structure. Replacement involves removing the impasse and replacing it with a structure such as a bridge or very large culverts that fully spans the stream channel and is not a barrier to fish passage. Replacement is done on many sites.

The cost of replacing all impasse culverts with bridges is in excess of the available and projected funding levels. An alternative to replacement is retrofitting. Retrofitting is temporarily providing passage at a site by enhancing or modifying the existing structure. The advantage of retrofits is the lower cost, the ability to give improved passage capability to more sites and thus, expand the number of stream miles that the fish can access. The disadvantages of retrofits are that the passage improvements are typically not as good as full replacements. The ODFW considers retrofits to be a temporary solution, and the retrofits should be identified, monitored, and scheduled for future replacement.

## 3.9 Temporary Construction Within a FEMA Floodway

Roadway detour structures are often used in highway construction when bridges or culverts are repaired or replaced. Temporary structures such as falsework, cofferdams, and workbridges are used when bridges are repaired or replaced. Often these temporary installations are in a regulatory floodway defined in the National Flood Insurance Program (NFIP) as administered by the Federal Emergency Management Agency (FEMA).

ODOT general policy is to satisfy regulations such as those of the NFIP, and FEMA has a policy interpretation about temporary construction within an NFIP regulatory floodways. These requirements have been provided by the FEMA regional office to ODOT in written correspondence about a past project. Excerpts from this correspondence follow.

Temporary construction practices are defined as "… anything necessary during the construction phase to build the final structure and will definitely be removed; and, the only reasonable course of action on the part of the local government would be to permit only the final structure design regardless of the shape or timing of the temporary construction practice. It is assumed that the constructing agency would make preliminary calculations on the effects of the practice to assure no significant water level increases. It is also assumed that the constructing agency would assume liability for any flooding its temporary practices would cause. As a final policy note, all practicable efforts must be spent to design the construction phase so that the temporary practices and temporary crossings are not in place during the flood season (November 1 – May 31)."

Temporary crossings, as pertaining to FEMA permit policy, are defined as follows. "Temporary crossings can be defined as temporary construction practices so the above interpretation could also apply. The real difference here, however, is that, based on economics, and the political atmosphere that can be created based on the benefits of having two crossings where there was previously one crossing, the temporary crossing can likely become permanent.

Therefore, the course of action of permitting only the final structure also applies for temporary crossings with the following caveat: twelve months after the initial placement of the temporary crossing, the constructing agency must reevaluate the status of the crossing and if the temporary crossing will remain in place for an additional 12 months or more, the constructing agency will apply to FEMA for a Conditional Letter of Map Revision according to 60.3(d)4 and 65.12 of the National Flood Insurance regulations (44 CFR) within 30 days."

## 3.10 Design and Check Recurrence Intervals and Exceedance Discharges for Temporary Installations

Temporary structures such as roadway detour bridges, falsework, cofferdams, and workbridges are used when bridges or culverts are repaired or replaced. Temporary water management also uses structures and operating features such as dams, pumps, pipes, ditches, etc. Discharge estimates are needed to design these installations and to assure they resist flood damage. ODOT does not have a policy listing design and check discharge recurrence intervals for these temporary installations. The following guidance is provided.

Considerable judgment is used when selecting design and check discharges for temporary facilities. A significant consideration is the damage that would occur if the facility was lost or damaged. An additional consideration is the accuracy of the check and design discharge predictions. These discharge estimates are often based on flows from nearby gaged watersheds during average conditions. Discharges may be significantly higher or lower in the subject watershed due to local watershed characteristics and variations in year-to-year climatic conditions.

**Temporary Installations In Place During the Flood Season –** The 5-year event is the typical design flow for installations in place through the flood season. Ten or 25-year discharges are often used as check flows. A larger design or check event may be warranted for an installation where there will be significant damage or adverse environmental impacts if a larger flood occurs while it is in place. The 5, 10, and 25-year flows can be determined by the hydrologic methods in Chapter 7.

The 100-year base flood discharge is often used as a design or check discharge for temporary structures within regulatory floodways during the flood season, such as detour structures and falsework providing critical support to structures.

**Temporary Installations In Place Outside of the Flood Season -** The design and check flows for these installations are based on either maximum predicted discharges based on station records and envelope curves, or mean daily exceedance discharges, for the months they will be in place. Usually these facilities are designed to be in place during the Oregon Department of Fish and Wildlife in-water work period.

Maximum predicted discharge estimates based on station records and envelope curves require hydraulic data that is not widely available. These estimates are done by hydraulics designers or others having access to this data. The procedures for calculating these discharges, as well as the exceedance discharges, are described in Chapter 7.

The design and check discharges for temporary installations are based on how critical it is that the facility convey the discharges or resist hydraulic damage, as follows.

- **Highly Critical Temporary Installations –** Check flow is the maximum predicted discharge based on station records and an envelope curve. Design discharge is the 5 percent mean daily exceedance discharge.

- **Moderately Critical Temporary Installations** – Check flood discharge is the 5 percent mean daily exceedance discharge, and the design discharge is the 25 percent mean daily exceedance discharge.

- **Less Critical Temporary Installations** – Check flood discharge is the 25 percent mean daily exceedance discharge, and the design discharge is the 50 percent mean daily exceedance discharge.

## 3.11 Cooperative Agreements

Cooperative hydraulic projects between ODOT and local agencies may include, but are not limited to:

- storm drains,
- flood control and/or flood protection, and
- water quality treatment.

These cooperative projects may be justified where both a mutual economic benefit and a demonstrated need exist. Early coordination with the government entity involved is necessary to determine the project scope. Each cooperative project may be initiated by a resolution adopted by the local agency governing body either (1) requesting the improvements and/or indicating its willingness to share the cost of a state project, or, (2) indicating the local agency's intent to make certain improvements and requesting state cost participation in the local agency project.

Participation in cooperative projects must be approved by the Chief Engineer with the extent of ODOT participation being restricted to the amount of benefit accruing to ODOT. No commitments should be made prior to approval by the Chief Engineer and the amount of participation shall be documented by a formal agreement. Actual work performed by ODOT under such agreements shall be limited to ODOT right-of-way or easements unless otherwise approved by the Chief Engineer.

## 3.12 Hydraulics Design Deviations

A hydraulic design deviation is a request to obtain a deviation from a design requirement(s) stipulated in the current version of the ODOT Hydraulics Manual. It also allows for introducing innovative designs or methods that may differ substantially from those discussed in the ODOT Hydraulics Manual.
A few possible design requirements that would require deviation approval include but are not limited to:

- Reducing documentation requirements
- Replacement “in-kind” (replace without calculations)
- Reducing design storm
- Reducing barrel material design life
- Exceed culvert allowable headwater
- Use or removal of a tidegate
- Reduce bridge opening clearance
- Eliminate or reduce scour protection, or
- Exceed water spread on pavement

### 3.12.1 Hydraulics Design Deviation Process

The submittal and concurrence process for hydraulic design deviations is presented below:

**Step 1 -** Project teams determine justification for design deviations(s) at scoping, prospectus, design phases, or planning process

**Step 2 -** Project Team Leader or Consultant Project Manager coordinates with Project Hydraulic or Roadway Designer to prepare Hydraulics Design Deviation Request form with supporting justification such as:

- an explanation of the issues,

- a brief description of the project,
- detailed location (bridge number, highway number, and milepost),
- a justification for the deviation, and
- supporting documentation.

Hydraulics design deviation approvals must be obtained as early as possible and prior to the design acceptance package (DAP).

Use the following link to access the Hydraulics Design Deviation Request form. The form is an Adobe Acrobat fillable form. Open the form and Save a copy. Fill in all the fields and then print:

Hydraulics Design Deviation Request Form

**Step 3 -** Submit completed Hydraulics Design Deviation Request forms prepared by individuals outside of the agency to the Region Hydraulics Engineer for review and concurrence

**Step 4 -** Region Hydraulics Engineer submits recommendations to Technical Services Geo-Environmental and/or Bridge Section Senior Hydraulics Engineer(s) [as appropriate] for final review. Hydraulics engineering staff in Technical Services will conduct the review and provide concurrence, suggested revisions, or deny the design deviation request. *NOTE: Design deviations formally obtained in writing during the Planning, Environmental or Survey phase need not be requested again.*

**Step 5 -** State Bridge Engineer or Geo-Environmental Section Manager provides approval of deviations receiving Region and Technical Services concurrence.

# OREGON DEPARTMENT OF TRANSPORTATION
## HYDRAULIC DESIGN DEVIATION REQUEST

| | | | |
|---|---|---|---|
| **Section:** | | **Route #:** | |
| **Highway Name:** | | **Highway #:** | |
| **County:** | | **Milepost:** | |
| **Region:** | | **Key #:** | |
| **Bridge #:** | | **EA #:** | |

## PROJECT DATA

| | | | | | |
|---|---|---|---|---|---|
| **Design Frequency** | | **Design Precipitation** | | **Design Flow** | |
| **Design Life** | | **Current ADT** | | **Design ADT** | |
| **DAP Date** | | **Bid Date** | | | |

## HYDRAULIC DESIGN DEVIATIONS

| **Documentation** (Reference Ch. 4) | **Culverts** (Reference Ch. 9) | **Bridges** (Reference Ch. 10) | **Energy Dissipation** (Reference Ch. 11) |
|---|---|---|---|
| ☐ Reduce documentation requirements | ☐ Replacement "in kind" (replace w/ calculations)<br>☐ Reduce design storm<br>☐ Reduce barrel material design life (ref. Ch. 15)<br>☐ Exceed allowable headwater<br>☐ Use or remove tidegate | ☐ Reduce opening clearance<br>☐ Reduce design storm | ☐ Reduce design storm<br>☐ Eliminate protection<br>☐ Reduce protection |

| **Storage Facilities** (Reference Ch. 12) | **Storm Drainage** (Reference Ch. 13) | **Water Quality** (Reference Ch. 14) | **Bank Protection** (Reference Ch. 15) |
|---|---|---|---|
| ☐ Reduce freeboard<br>☐ Eliminate soil amendment<br>☐ Reduce design storm | ☐ Exceed water spread on pavement<br>☐ Allow pressure flow-storm drains<br>☐ Reduce design storm | ☐ Reduce freeboard<br>☐ Eliminate soil amendment<br>☐ Reduce design storm | ☐ Reduce design storm<br>☐ Eliminate protection<br>☐ Reduce protection |

| **Other** | |
|---|---|
| ☐ | Use an experimental or new technology or material |
| ☐ | |

# OREGON DEPARTMENT OF TRANSPORTATION
## HYDRAULIC DESIGN DEVIATION REQUEST

| | |
|---|---|
| **Description of Hydraulic Design Exception:** | |
| **Description of Project:** | |
| **Location of Feature:** | |
| **Hydraulic/Hydrologic History:** | |
| **Reasons for Not Attaining Standard:** | |
| **Effect on Other Standards:** | |
| **Compatibility with Adjacent Sections:** | |
| **Probable Time before Reconstruction:** | |
| **Mitigation for Design Deviation Included in the Design:** | |
| **Supporting Documentation:** | |

**ATTACHMENTS:**

- ☐ Vicinity map
- ☐ Plan sheets
- ☐ Photos
- ☐ Charts - use to compare options, design elements, costs, etc.
- ☐ Design Manual Figure or Section that shows Design Standard

# OREGON DEPARTMENT OF TRANSPORTATION
## HYDRAULIC DESIGN DEVIATION REQUEST

**SIGNATURES:**

**Prepared by:** _____ **Date:** _____

(Engineer of Record)

**Print Name:** **Phone:**

**Company Name:**

**Company Address:**

**City:** **State:**

**Concurred by:** _____ **Date:** _____

(Region Hydraulic Engineer)

(Print Name)

**Concurred by:** _____ **Date:** _____

(Geo-Enviro. Senior Hydraulics Eng. or Senior Bridge Hydraulics Eng.)

(Print Name)

**Approved by:** _____ **Date:** _____

(State Bridge Engineer or Geo-Environmental Section Manager)

(Print Name)

**PREPARED BY:**

| ENGINEER OF RECORD PROFESSIONAL ENGINEERS STAMP |
| --- |
| |

# CHAPTER 4
# DOCUMENTATION

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## 4.1 Introduction

This chapter provides hydraulic study documentation guidelines for bridges, culverts, stormwater drainage systems, stormwater water quality treatment facilities, and stormwater storage facilities. These guidelines are for designers within the agency and consultants designing either ODOT projects or local jurisdiction projects administered by ODOT. Additional documentation guidelines for other topics, such as channel and storage facilities, are in their respective chapters.

Proper documentation provides:

- essential information for planning and environmental studies, regulatory and permit documents, and design,
- data for operation and maintenance,
- justification for the expenditure of public funds and the selection of structure type,
- future reference for engineers and others if an improvement, change, or rehabilitation is needed,
- information for the development of defense in matters of litigation, and
- public information.

The documentation level is often proportional to the study importance and complexity. Documentation almost always consists of:

- the Preliminary Hydraulic Recommendations (hydraulic and/or stormwater),
- the Report(s) (hydraulics and/or stormwater),
- the supporting data, and
- the documentation storage file.

Hydraulic and stormwater studies and documentation are almost always done by a professional engineer registered in Oregon or people working under the direct supervision of the engineer. These requirements are discussed in Chapter 3. Consultants preparing recommendations and reports for ODOT should submit copies of the listed documentation to ODOT for review. The reviewer may request additional documentation in some instances. The review comments are to be considered and addressed before the report is finalized and distributed.

Many hydraulic and stormwater studies for federally funded non-ODOT projects are prepared by consultants and reviewed by ODOT. Typically an engineer outside of the agency will use the same procedures as an ODOT engineer to compile and calculate the data. There are no requirements that the engineer must use ODOT methods. The consulting engineer should be aware that the ODOT reviewer may check the values using ODOT design procedures, and if the values in the report are significantly different than values determined by ODOT methods, the reviewer may express concern. Following the guidelines in this chapter will simplify the review process.

## 4.2 Bridge and Large Culvert Study Documentation

Many aspects of a bridge or culvert design are based on the Preliminary Hydraulic Recommendations and the Hydraulics Report. As a result, if the structure is to perform as intended, this information must be complete and accurate. Following these guidelines will help to assure the documentation is complete. These guidelines apply to hydraulic recommendations and reports for:

- bridges,
- circular culverts with diameters more than or equal to 6 feet,
- pipe-arches with equivalent diameters more than or equal to 6 feet,
- box culverts with spans more than or equal to 6 feet,
- multiple barrel culverts with cumulative span widths more than or equal to 6 feet,
- open-bottom culverts of all sizes,
- culverts providing fish passage, and
- complex hydraulic structures such as cast-in-place energy dissipators, flumes, etc.

These guidelines go step-by-step through typical ODOT hydraulic study documentation with checklists of common items. These guidelines are for typical bridges or culverts on streams or rivers and they may not be applicable for structures at other locations. Some studies may address more or fewer items than those listed, as discussed in the specific chapters in this manual. Engineering judgment should be used when preparing documentation, and any variances from these guidelines should be discussed with the Geo-Environmental Engineering and Asset Management Unit prior to submitting materials for review.

### 4.2.1 Recommendations and Report Distribution

Hydraulic information is used by numerous members of the project team. The Preliminary Recommendations, the Hydraulic Report, and revisions and amendments to these documents, are distributed as follows.

- Ten copies to the project team leader for distribution among the project team.
- One copy to the Federal Highway Administration Division Bridge Engineer in Salem for projects on the National Highway System involving new construction and reconstruction of facilities when the total project cost exceeds one million dollars, or rehabilitation (3R) projects when total project costs exceed five million dollars. The Hydraulics Report and its revisions and amendments are sent to the FHWA. The Preliminary Hydraulic Recommendations are not submitted.
- One copy to the Geo-Environmental Section's Engineering and Asset Management Unit. Submit in Adobe Acrobat portable document format (pdf) by completing the project

report submittal form.

Project Report - Submittal Form 734-5056[A1]

- The original copy to the project files.

**Items to Check**

_____Does the distribution list include ten copies for the Project Team Leader?
_____Does the distribution list include the FHWA if the project is subject to federal review?
_____Is a copy sent to the Geo-Environmental Section and the Standards and Practices Unit and the original kept in the project file?

### 4.2.2 Preliminary Hydraulic Recommendations

The Preliminary Hydraulic Recommendations are a one or two page memorandum accompanied by one or more drawings. The recommendations are issued after the initial site visit and before the hydraulic survey request. This information gives the project team a preliminary idea of the hydraulic structure type, size, location, and special features or concerns. An example is in Appendix A.

### 4.2.3 Hydraulics Report

The Hydraulics Report provides detailed information for many tasks, such as structure design, roadway design, environmental documents, and permit applications. It is also of interest to maintenance personnel, and in the case of larger projects with federal funding, the FHWA. Excerpts of several Hydraulic Reports are included in the chapter text to provide guidance to the reader on specific situations. A complete report is in Appendix B.

#### 4.2.3.1 Introduction

A typical introduction includes a brief description of the project scope stating the reason the structure is being built or replaced. ODOT bridge numbers are included where applicable. New bridge numbers are required for replacement bridges and larger culverts. Also included is a description of the reference datum for the elevations in the report, i.e. 1929 NGVD with the 1947 adjustment, assumed datum, 1988 NAVD, etc. Examples follow.

The existing bridge (Bridge # 12586A) is being replaced by a new widened structure on a different alignment (Bridge # 18463) during the reconstruction of the highway. The existing bridge is too narrow and in the wrong location to accommodate the widened roadway. All elevations in this report are based on the project datum, the 1929 National Geodetic Vertical Datum with the 1947 adjustment.

or

A culvert with two 48-inch span by 24-inch rise reinforced concrete box barrels was recommended in the original 22 December 1995 Hydraulics Report. These culvert barrels have projecting ends and open channels at the inlet and outlet. Recently, it was decided that a safety grate is needed at the inlet and the culvert will discharge into an enclosed storm drain system manhole. In addition, a wetland will be constructed immediately upstream from the culvert, and a special drop style inlet will be needed to keep water pooled in the wetland. This amended report provides the hydraulic information for this revised culvert.

**Items to Check**

_____Is the report scope defined?
_____Are the bridge numbers included?
_____Is the elevation datum described?

### 4.2.3.2 Regulatory Requirements and Permit Data

There are numerous regulations and requirements for hydraulic structures in addition to ODOT design standards. These topics are presented in the Regulatory Requirements and Permit Data section. The most common regulations and requirements are described below.

**Floodplain Development Regulations -** Cities and counties often require an engineer to certify the project meets their floodplain development regulations, as discussed in Chapters 2 and 3. The Hydraulic Data sheets or a suitable reference should list the water surface elevations and other data that show the proposed structure is in compliance. This data should be supported by narrative comments. Two examples follow.

Bear Creek has a floodway designated in the National Flood Insurance Program, and the bridge widening and roadway work on this project are designed to meet applicable floodway standards and regulations. Changes to the Bear Creek floodway caused by the proposed project are addressed in the Request for Conditional Letter of Map Revision submitted by ODOT to the City of Medford.

or

City of Beaverton regulatory standards require the extended culvert to have headwater elevations no higher than the existing culvert. As per agreement with the city, discharges listed in the hydraulic study for the nearby TRI-MET train bridges were used in this study. As shown in the Hydraulic Data tables, the extended culvert produces lower headwater elevations than the existing culvert. The increased hydraulic performance of the extended culvert is due to the beveled top edge on the upstream extension. The square top edge of the existing culvert is not as efficient as the beveled edge.

**Fish Passage -** Fish passage is a concern at all crossings of fish-bearing streams. Passage requirements are based on NMFS, Endangered Species Act and ODFW requirements, OAR Div. 412. Appropriate environmental permitting and authorizations should be listed as a reference, if needed. Specific design data and a description of fish passage features are required. An example is shown on the following page.

**Riparian Habitat –** Projects in riparian areas are subject to regulations pertaining to both construction procedures and the finished product. Several regulations are listed in the Data Collection chapter. Hydraulically related construction methods and design features required by these regulations should be mentioned in the Hydraulics Report. As an example, a culvert was designed to meet the FHWA "Federal Aid to Highways Programmatic" (FAHP). FAHP requires large woody debris to be included in the project design. The Hydraulics Report says wood will be included as described in the narrative at the bottom of the following page.

**Other Environmental Regulations and Requirements -** Other environmental concerns, regulations, or requirements can affect structure type, size, or location. Region permit or environmental specialists are good sources of information about these issues. The Biological Assessment, Environmental Impact Statement, or other documents should be listed as references, as needed. These requirements should be mentioned if they affect the waterway opening characteristics. Examples are wild and scenic river requirements, the need to provide animal passage along streambanks, etc.

**Fish Passage Data Table Example**

The extended culvert is designed to pass fish, as per 'Oregon Department of Fish and Wildlife Standards and Criteria for Stream-Road Crossings.' An extended culvert at this site needs fish weirs in order to pass fish. There would be insufficient flow depth in the barrel during the low design flow if weirs are not used. The Oregon Department of Fish and Wildlife requires data on fish passage and a description of fish passage features when they review the project plans. The fish passage information below applies to the culvert described in this Hydraulics Report.

Fish Passage Data

| | |
|---|---|
| Design Species: | Cutthroat trout less than 20 inches long |
| Low Design Flow: | Less than 1 cubic foot per second |
| Minimum Depth at Low Design Flow: | 9 inches |
| High Design Flow: | 57 cubic feet per second |
| Average Velocity at High Design Flow: | 3 feet per second |
| Maximum Jump Height: | 6 inches |
| Jump Pool Length: | 28 inches |

Design features include.

1. The upstream edge of the upstream apron is below the elevation of the stream bottom and the water surface to maintain sufficient water depth at low flows. This requires a negative slope on the upstream apron, as shown in Figure 1.
2. Fish weirs are incorporated into the downstream apron to maintain sufficient water depth during low flows.
3. Two extra fish weirs are included to maintain fish passage if there is degradation of the channel bottom during the design life of the culvert. These weirs are too low to provide pools when the streambed is at its current elevation. These weirs will provide jump pools if the streambed degrades.
4. The bottom of the basin is 1.5 feet lower than the water surface elevation during the low design flow to provide fish passage through the stilling basin at the outlet.

The proposed culvert design will also incorporate large woody debris in the pond area created by beaver dams upstream from the culvert location. The quantity and exact location of the woody debris will be determined at a later date.

**In-Water Work Period -** Information on the in-water work period is useful for people designing and administering the project. It can be included in the report as follows.

The in-water work period for Hall Creek is July 1 through September 30 unless specified otherwise by ODFW.

**Ordinary High Water -** Water surface elevations during the ordinary high water event are often required for permit applications. Methods to determine this elevation are located in Chapter 6. This information is included in the Hydraulic Data sheet and mentioned in the report as follows.

To provide data for the Division of State Lands permit, the ordinary high water elevations based on a field survey are listed in Tables 2 and 3.

**Navigational Clearance -** Vertical and horizontal clearance can be an issue on navigable waterways, as discussed in Chapter 10. These requirements should be mentioned in the narrative if they affect the type, size, or location of the replacement structure. An example follows.

Boats travel through this crossing. As a result, the Oregon State Marine Board has requested that passage clearance under the proposed crossing be maintained at the present level.

**Other Regulations and Requirements -** In some cases there are other regulations or requirements that affect the type, size, or location of a bridge or culvert. As an example, the irrigation districts or U. S. Bureau of Reclamation have hydraulic requirements or regulations about structures on their waterways. Region permit specialists are good sources of information about these issues. These requirements should be mentioned if they affect the waterway opening characteristics.

### Items to Check

_____Is it mentioned if the structure is in a regulatory floodway or floodplain?
_____Is it mentioned if the design must satisfy applicable flood plain development ordinances?
_____Is it noted if FIS data was used to design the structure?
_____Are fish passage concerns mentioned if they affect the hydraulic design?
_____Are the needed fish passage data listed?
_____Are riparian habitat or other environmental regulations mentioned if they affect the hydraulic design?
_____Are design features mentioned if they are included to meet riparian area use regulations?

_____Is the in-water work period listed?
_____Are the ordinary high water elevations listed?
_____Is navigational clearance mentioned if it is required or if it affects the design?
_____Are other regulations governing the type, size, and location of the structure mentioned if they affect the hydraulic design?

### 4.2.3.3 Hydrology Information

The hydrology section lists the flood flows expected for various recurrence intervals. Hydrology methods are discussed in Chapter 7. The hydrology section should include, but is not limited to:

- the flooding source (i.e. runoff, high tides, etc.), the contributing drainage area at the site, and the time of year when floods usually occur,
- the method used to determine the hydrology, and
- the flood recurrence interval versus peak discharge relationship at the site. This data is usually listed in a table, and it includes the 2, 5, 10, 25, 50, 100, and 500-year flows,
- historical flood data, if available, and
- drainage maps with drainage boundaries and flow directions noted. This is needed for complex drainages where the boundaries are not obvious, such as urban areas.
- The fish passage high and low flow design discharges with accompanying narrative. Presentation of this data is discussed in Chapter 7. These discharges are also listed on the fish passage design data table presented in Subsection 4.2.3.2.

**Items to Check**

_____Is the drainage area listed?
_____Is the method of determining the hydrology noted?
_____Are 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year discharges listed?
_____If flows from an FIS report or gaging station are used and adjusted to the drainage area at the bridge site, is this mentioned along with a description of the adjustment method?
_____Are significant historic floods or flood damage mentioned?
_____Are maps included for complex drainages?
_____Are fish passage design discharges presented?

### 4.2.3.4 Ice and Debris

This section mentions whether or not ice or debris passage are a problem or concern with a structure. Some types of bridges that are adequate for conveying water may be poor choices for

conveying water containing debris or ice. In addition, a proposal to replace a bridge with a culvert on a stream carrying debris or ice deserves careful review. Debris and/or ice passage should be mentioned if it affects the type, size, or location of the proposed structure. It should also be noted if ice or debris passage has not been a problem.

**Items to Check**

_____Is debris passage adequately described if it is a problem?
_____Is ice passage described if it is a problem?
_____Is it noted that ice or debris passage is not a concern if this is the case?

### 4.2.3.5 Hydraulic Design

The report includes a section on hydraulic design. This section presents most of the hydraulic information used to design the structure, including a design narrative, hydraulic data sheets, and drawings.

### 4.2.3.6 Design Narrative

The design narrative describes the type, size, and location of the existing and proposed structures. Designers will use this information to verify the structure described in the report matches the vertical and horizontal alignment of the proposed roadway. This is very important because the proposed highway alignment may be changed several times during the design process. An example follows.

> The existing structure is a 63-foot long 3-span reinforced concrete deck with timber superstructure bridge supported by pile trestles. The hydraulic performance of the existing bridge is summarized in Table 1.
>
> The proposed structure is a 69-foot long single-span combination spillthrough/vertical abutment bridge. This report is based on an alignment with the centerline of the new road approximately 0.33 feet higher and 28.83 feet north of the centerline of the existing road. The hydraulic performance of the proposed bridge is summarized in Table 2 and the waterway opening is shown in Figure 1. The inside face of the west abutment is at Station 937+67.06 and the face of the east abutment is at Station 938+35.96.

The design narrative describes flood characteristics if this information is needed to clarify the information listed on the Hydraulic Data sheet. An example follows.

The existing bridge does not convey the entire flow of Oak Creek during large floods. Water overtops the creek banks upstream from the highway and bypasses the bridge during floods of a 17-year or greater recurrence interval. Most of this bypass flow overtops a low section of the roadway 850 feet west of the bridge. The remainder of the bypass flow crosses under the highway through two 30-inch diameter concrete pipes 1,050 feet west of the bridge. The bypass flow rejoins Oak Creek downstream from the bridge crossing.

The design narrative discusses special features and miscellaneous topics which affect the design. Drawings should be provided as needed. This information will be used to incorporate the features into the design. Special features and miscellaneous topics include, but are not limited to the following.

- Channel changes.
- Debris control structures.
- Streambank or embankment protection.
- Upstream settling basins.
- Recommended end treatments for culverts. Dimensioned drawings should be included for inlets not shown on standard plans, such as drop inlets, slope-tapered inlets, etc.
- Recommended barrel materials for culverts, in conformance to ODOT alternate materials policy, as discussed in Chapter 5. Occasionally, alternate materials cannot be specified because only certain types will work. If so, the narrative should mention why the alternate material cannot be used.
- Invert protection for culverts.

A narrative description of a special inlet follows.

The recommended style of safety grate is shown in Figure 2a. The longitudinal bars should be on the top face and the transverse bars should be on the bottom face, as shown in Figure 2b.

The design narrative provides construction and maintenance information to the designers, project management, and maintenance personnel. A typical or frequent maintenance should be described in detail. Examples follow.

Silt and gravel have accumulated in the streambed upstream from the existing culvert. The stream channel should be regraded to the profile shown in Figure 1. Upstream from the culvert the streambed should be relocated to match the inlet to the arch. Within the culvert there should be at least 8 feet of clearance between the channel bottom and the top of the culvert. The channel should be restored to resemble the natural channel.

or

During floods the proposed culvert will convey more water than the existing culverts and flow depths and velocities will increase at the outlet of the culvert and in the downstream canal. These greater flows may accelerate the erosion of the mudstone ledge and the embankment slope. As a result, the embankment slope near the outlet should be inspected periodically and revetment installed as needed.

The narrative should also mention if the existing structure blocks fish passage and the reasons. This information will aid in the preparation of environmental documents. As an example:

The existing single, smooth barrel culvert is hydraulically inadequate for upstream fish passage. Due to the small diameter of the existing culvert, the flow velocities in the culvert can be too high for fish passage during almost all high flow events. Additionally, during low flow events there is a small jump between the culvert outlet and the scour pool at the outlet.

Any changes to the hydraulic characteristics of nearby structures caused by the proposed bridge or culvert should be mentioned in the narrative. An example follows.

The proposed fish passage culvert is much larger than the existing culvert. It eliminates the need to replace the overflow culvert at milepoint (M.P.) 40.18. This culvert can be removed or left in place and filled with grout.

**Items to Check**

_____Are the types, sizes, and locations provided for existing and proposed structures?
_____Are flood characteristics addressed if this information helps support the Hydraulic Data sheets?
_____Are channel changes, debris control structures, streambank, or embankment protection

described?
_____Are other special design features described?
_____Is frequent or atypical maintenance addressed?
_____Are culvert features described, such as end treatments, barrel materials, and invert protection?
_____Is fish passage through the existing culvert described?
_____Are any changes to hydraulic conditions at nearby structures described?

### 4.2.3.7 Hydraulic Data Sheets

All hydraulic reports should include Hydraulic Data sheets for the existing and proposed structures. Blank and completed sheets for a bridge and a culvert are shown in Figures 4-1 through 4-4. Figure rounding guidelines are listed in Subsection 4.2.3.12.

Many times the water surface elevation in an adjacent water body can affect the hydraulic characteristics of the river or stream that flows through the structure, as discussed in Chapter 10. Two data sheets are needed for the existing structure and each alternative replacement structure if the water surface elevations at the crossing are affected by water surface elevations in a downstream water body. Typically, one data sheet summarizes the hydraulic performance during the discharge-tailwater conditions that result in the greatest velocities through the structure, and the other sheet describes the discharge-tailwater combinations that cause the highest water surface elevations. A single data sheet can be used if the elevations during both conditions are clearly listed. This is done in the report included in the Appendix. A summary of the methods for calculating hydraulic data follows.

**Discharges and Recurrence Intervals -** The first and second rows are for discharges and recurrence intervals, respectively. Selection of these values is discussed in Chapter 9 for culverts and Chapter 10 for bridges.

The Design event discharge is listed in the first column. As discussed in Chapter 3, 25 and 50-year flows are the most common design discharges. In most cases, the design flows are used to determine the minimum recommended waterway areas and bottom of beam elevations for bridges, and the recommended barrel sizes for culverts.

The Base Flood discharge is listed in the second column **and it is always the 100-year flow**. In regulatory floodplains and floodways it can control the minimum recommended waterway areas and bottom of beam elevations of bridges and the minimum recommended barrel sizes for culverts. It is also used in scour analyses.

## BRIDGE

**PROJECT:** **COMPILED BY:**

**DATE:**

| TABLE _: | HYDRAULIC DATA | | | |
|---|---|---|---|---|
| | DESIGN EVENT | BASE FLOOD | ROADWAY OVERTOPPING FLOOD | ORDINARY HIGH WATER |
| Discharge (cubic feet per second) | | | | |
| Recurrence Interval (years) | | 100 | | - |
| Approach Section H.W. Elevation w/Natural Channel[1] (feet) | | | | - |
| Approach Section H.W. Elevation w/ Bridge[1] (feet) | | | | |
| Backwater (feet) | | | | - |
| H.W. Elev. at Upstream Face of Bridge[2] (feet) | | | | |
| H.W. Elev. At Downstream Face of Bridge[3] (feet) | | | | |
| Waterway Area at Downstream Face of Bridge[4] (square feet) | | | | - |
| Average Velocity at Downstream Face of Bridge (feet per second) | | | | - |

[1] Approach section is one waterway opening width upstream from upstream face of bridge.

[2] Located at upstream face of bridge along the embankment.

[3] Located at downstream face of bridge opening.

[4] Area normal to channel centerline. Projected area of piers has been subtracted.

**REMARKS:**

Manning's "n" bridge opening
Manning's "n" main channel
Manning's "n" overbanks

**Figure 4-1 Blank Bridge Data Sheet**

## BRIDGE

**PROJECT:** Oak Creek Bridge # 18463
Pacific Hwy - Lebanon (RR Tracks) Section
Corvallis - Lebanon Highway, M.P. 12.48
Linn County Key # 06420

**COMPILED BY:** _____

**DATE:** 6 October 1997

| TABLE 2: Proposed Bridge | HYDRAULIC DATA | | | |
|---|---|---|---|---|
| | DESIGN EVENT | BASE FLOOD | ROADWAY OVERTOPPING FLOOD | ORDINARY HIGH WATER |
| Discharge (cfs) | 1,850 | 2,390[6] | 2,060[5] | - |
| Recurrence Interval (yrs) | 50 | 100 | 80 | |
| Approach Section H.W. Elevation w/Natural Channel[1] (ft) | 277.0 | 277.5 | 277.2 | - |
| Approach Section H.W. Elevation w/ Bridge[1] (ft) | 277.8 | 278.4 | 278.0 | 274.7 |
| Backwater (ft) | 0.6 | 0.9 | 0.8 | - |
| H.W. Elev. at Upstream Face of Bridge[2] (ft) | 277.7 | 278.3 | 277.9 | 274.6 |
| H.W. Elev. At Downstream Face of Bridge[3] (ft) | 276.5 | 276.9 | 276.7 | 274.4 |
| Waterway Area at Downstream Face of Bridge[4] ($ft^2$) | 290 (331) | 290 (331) | 290 (331) | -- |
| Average Velocity at Downstream Face of Bridge (ft/s) | 6.4 | 7.0 | 6.4 | - |

[1] Approach section is one waterway opening width upstream from upstream face of bridge.
[2] Located at upstream face of bridge along the embankment.
[3] Located at downstream face of bridge opening.
[4] Area normal to channel centerline. Area in parentheses is parallel to roadway centerline.
[5] 1,870 cfs flows under the bridge and 190 cfs flows through culverts west of the bridge.
[6] 2,030 cfs flows under bridge, 170 cfs flows over road to west of bridge, and 190 cfs flows through culverts to the west of the bridge.

**REMARKS:**

The spillthrough structure is a 70-foot long single-span bridge with combination spillthrough/vertical abutments. The structure is skewed 29 degrees. Three 36-inch diameter culverts are located under the roadway to the west of the bridge.

Manning's "n" bridge opening = 0.04
Manning's "n" main channel = 0.04
Manning's "n" overbanks = 0.04

**Figure 4-2 Example Bridge Data Sheet**

## CULVERT

**PROJECT:** **COMPILED BY:**

**DATE:**

| TABLE _: | HYDRAULIC DATA | | | |
|---|---|---|---|---|
| | DESIGN EVENT | BASE FLOOD | 500-YEAR FLOOD | ORDINARY HIGH WATER |
| Discharge (cubic feet per second) | | | | - |
| Recurrence Interval (years) | | 100 | 500 | |
| Highwater Elevation of Natural Channel, at Culvert Inlet (feet) | | | | - |
| Headwater Elevation at Culvert Inlet (feet) | | | | |
| Backwater Depth at Culvert Inlet (feet) | | | | - |
| Tailwater Elevation at Culvert Outlet (feet) | | | | |
| Average Velocity at Culvert Outlet (feet per second) | | | | - |

**REMARKS:**

**$K_e$ =**

**Upstream invert elevation =**
**Downstream invert elevation =**

**Figure 4-3 Blank Culvert Data Sheet**

## CULVERT

**PROJECT: Little Oak Creek Culvert Bridge 18496**
**Pacific Highway - Lebanon (RR Tracks) Section**
**Corvallis - Lebanon Highway, M.P. 15.40**
**Linn County**

**COMPILED BY: _____**

**DATE: 13 November 1997**

| TABLE 2 Proposed Culvert | HYDRAULIC DATA | | | |
|---|---|---|---|---|
| | DESIGN EVENT | BASE FLOOD | 500-YEAR FLOOD | ORDINARY HIGH WATER |
| Discharge (cfs) | 220 | 260 | 350 | - |
| Recurrence Interval (yrs) | 50 | 100 | 500 | - |
| Highwater Elevation of Natural Channel, at Culvert Inlet (ft) | 313.2 | 313.3 | 313.5 | - |
| Headwater Elevation at Culvert Inlet (ft) | 314.4 | 314.8 | 316.1 | 312.4 |
| Backwater Depth at Culvert Inlet (ft) | 1.2 | 1.5 | 2.6 | - |
| Tailwater Elevation at Culvert Outlet (ft) | 312.5 | 312.6 | 312.8 | 311.7 |
| Average Velocity at Culvert Outlet (ft/s) | 8.9 | 9.4 | 10.4 | - |

**REMARKS:**

**The proposed culvert is a 10-foot span by 6-foot rise by 84-foot long reinforced concrete box culvert with a beveled inlet, wingwalls, aprons, and cutoff walls. The culvert invert is buried 2 feet below the channel bottom. The culvert centerline crosses road centerline at Station 812+08.0 with a 16 degree right hand forward skew.**

**$K_e$ = 0.2**

**Upstream and downstream invert elevations = 308.4 ft**

**Figure 4-4 Example Culvert Data Sheet**

The 500-year or Roadway Overtopping Flood discharges are listed in the third column. An exception is when the roadway overtopping flood is the design event, in which case the third column is left blank. This large flow is often called the “check discharge” as described in Chapter 10. This discharge is used for the revetment design and the scour depth calculations. Roadway overtopping flood data, if listed, should be valid for the specific structure summarized on the data sheet. It is quite common for existing and replacement structures to have two different roadway overtopping discharges. If flow occurs over road or through an overflow structure, these discharges and the discharge through the bridge opening are listed in the "Remarks" on the data sheet, as shown in Figure 4-2.

The Ordinary High Water elevations at the upstream and downstream faces of the proposed structure are listed in the last column. These elevations are determined by hydraulic analyses for sites where hydraulic modeling has been performed. The elevations are determined by field observations and surveys in the absence of hydraulic modeling. These elevations are used in environmental documents and permits.

**Hydraulic Data Sheets for Bridges -** The third through fifth rows summarize backwater data. These depths and elevations are determined by methods in Chapter 10. The backwater depths listed in the fifth row equal the elevations in the fourth row minus the elevations in the third row.

The sixth and seventh rows list water surface elevations at the upstream and downstream faces of the bridge. The elevation at the upstream face is the water surface along the embankment. It is the surface elevation at the approach section (with bridge in-place) minus the friction loss between the approach section and the bridge. The elevation at the downstream face is the water surface as it passes under the downstream edge of the bridge deck.

The eighth row lists the flow areas at the downstream face of the bridge opening. The areas are normal to the channel centerline regardless of the angle of intersection between the stream and road centerlines. The flow area excludes the projected areas of piers, if piers are present.

The ninth row lists the average flow velocities at the downstream face of the bridge opening. The average velocities are the discharges in the first row divided by the areas in the eighth row unless there is roadway overtopping or flow through an overflow structure. The velocities are based on discharges through the bridge opening, only, if overtopping or overflow occurs. Make sure that Q = V/A. Note any overtopping or overflow amounts under comments.

The bridge length, type, number of spans, and skew angle are listed in the "Remarks" section of the data sheet.

**Hydraulic Data Sheets for Culverts -** The first and second rows summarize the hydrology. The third through fifth rows list water surface elevation data determined by methods in Chapter 9. The surface elevations in the natural channel without the road or culvert in-place are listed in the third row. These elevations are in the natural channel at the location of the proposed culvert inlet. They are "highwater" elevations. The surface elevations at the inlet with the culvert in-place are listed in the fourth row. They are "headwater" elevations. The backwater depths in the fifth row are equal to the headwater elevations in the fourth row minus the highwater elevations in the third row.

The sixth row lists the tailwater elevations at the culvert outlet.

The seventh row lists the average outlet velocities. Flows through the culvert, only, are used in the calculations. Unless there is roadway overtopping flow, the average velocities are the discharges in the first row divided by the flow areas at the outlet. The actual flow area at the outlet should be used in the calculations if the outlet is partially full.

The inlet and outlet invert elevations, the culvert length and skew angle, the inlet loss coefficient, $K_e$, and the inlet configuration (projecting, mitered, beveled, slope tapered, or side tapered) are listed in the "Remarks" section of the data sheet.

ODOT policy is to recommend alternate culvert barrel materials if they can be used, as discussed in Chapter 5. Often the alternatives have different hydraulic characteristics. The hydraulic performance of each alternative is listed on a separate data sheets if this is the case.

**Items to Check**

(Data sheets for bridges and culverts.)

_____Are Hydraulic Data sheets included?

_____Can fluctuating water surface elevations in a downstream water body affect hydraulic characteristics at the structure? If so, are Hydraulic Data sheets included which reflect the range of tailwater elevations at the structure?

_____Are appropriate discharges and recurrence intervals listed on the data sheets?

_____Does the roadway overtopping flow and recurrence interval apply to the structure described on each data sheet?

_____Are ordinary high water elevations listed for all proposed structures?

_____Are the roadway overtopping flows listed if overtopping occurs more frequently than every 500 years?

_____Are flows over the road excluded from the flows used to calculate the average velocities?

_____Is the information on the data sheet consistent with the data shown on the drawings and the narrative discussion?

**Items to Check**
(Data sheets for bridges.)

_____Is it noted if the information is for the existing or proposed structure?
_____Are the backwater depths calculated at the right locations?
_____Are water surface elevations at the bridge opening calculated at the correct locations?
_____Are the waterway areas normal to the channel centerline?
_____Are the structure type, length, skew angle, etc noted in the "Remarks"?

**Items to Check**
(Data sheets for culverts.)

_____Are the outlet velocities based on actual flow areas?
_____Are invert elevations, culvert length, skew angle, inlet loss coefficient, and inlet configuration listed?
_____Are data sheets included for alternate materials if the hydraulic characteristics of the alternatives differ?

### 4.2.3.8 Drawings

A hydraulic report includes drawings of the proposed bridges and/or culverts. At a minimum, these drawings show the bridge waterway opening cross-section or the elevation view of the culvert. Headwalls, wingwalls, retaining walls, and the revetment protecting these structures are also shown, along with any special features. Often additional drawings are needed. This could include a drawing of a change in channel alignment for a bridge. For a culvert, this may be a drawing of a special inlet or special details required for fish passage.

**Bridge Waterway Opening Drawings –** A typical waterway opening drawing is shown in Plate 4-1. The drawing should show the following.

- The recommended waterway opening and the existing groundline at the opening, including stationing and elevations.
- The recurrence interval and elevation of the design flood.
- The water surface elevation at the downstream face of the bridge during design flood.
- The width of the waterway at the downstream face of the bridge. The width is the distance parallel to the roadway centerline from edge of water to edge of water during the design event.
- The area below the design event water surface elevation normal to the channel centerline. This area is the same as the design event waterway area in the Hydraulic Data table.
- The area below the design event water surface elevation parallel to the roadway centerline if the crossing is skewed (a crossing where the roadway and waterway centerlines are perpendicular has a skew angle of 0 degrees).

- The skew angle in degrees.
- A note verifying the projected pier area has been subtracted from the waterway opening area shown, if the bridge has piers.
- The width and elevation of the channel bottom if a trapezoidal waterway opening is recommended.
- The minimum recommended bottom of beam elevation. The bottom of beam elevations at both abutments should be noted if the bridge is on a grade.
- The abutment type, including the end slopes if spillthrough abutments are recommended.
- The revetment protection, including details.
- The potential scour elevations (or evidence solid rock is present, i.e. Geotechnical Report).
- The datum for the elevations.

**Culvert Elevation View Drawings -** A typical culvert elevation view drawing is shown in Plate 4-2. The following should be shown.

- The longitudinal section of the culvert along the barrel centerline including the existing and proposed groundlines and channel bottoms. Elevations and distances right and left of the roadway centerline should be shown on the drawing axes.
- The profile of the channel bottom near the culvert.
- The recurrence interval, headwater elevation, and tailwater elevation of the design flood.
- The barrel length, invert elevations, and slope.
- The degree of skew of the crossing. (A crossing where the roadway and culvert centerlines are perpendicular has a skew angle of 0 degrees.)
- The barrel size, number of barrels, and type.
- The inlet configuration (mitered, projecting, beveled, etc.),
- The minimum elevations of revetment on the embankments around the culvert ends.
- Revetment on or in the channel bottom upstream or downstream from the proposed culvert.
- The burial depth if the culvert invert is buried below the channel bottom.
- The burial depth if revetment is buried below the channel bottom.
- The datum for the elevations.

**Items to Check**
(Bridges and Culverts)

_____Are drawings included for all proposed alternative structures with differing dimensions or hydraulic characteristics?
_____Is all applicable information included on the drawing(s)?
_____Is the data on the drawing consistent with the data on the Hydraulic Data Sheet and in the narrative discussion?
_____Are additional drawings included that show special features?

### 4.2.3.9 Scour

This section describes possible long term changes in channel bottom elevation due to either aggradation or degradation, possible shifts in channel alignment due to lateral instability, clear-water or live-bed contraction scour, and pier scour, as discussed in Chapter 10. In addition, any past problems with aggradation, degradation, lateral stability, or scour are discussed. Also, this section presents the methods and assumptions used to determine the potential scour elevations shown on the waterway drawings.

ODOT bridges are rated for susceptibility to scour using "National Bridge Inventory System Item 113 – Scour Critical Bridges," as discussed in Chapter 10. If the modified existing structure or proposed structure will have a scour rating of 6 or lower, this should be mentioned in the report with reasons for the low rating.

**Items to Check**

_____Are past problems with scour described?
_____Is channel aggradation/degradation addressed?
_____Is lateral stability of the channel addressed?
_____Are the types of scour that compose the total scour mentioned?
_____Based on available information, will the proposed modified or new structure have an adequate rating for susceptibility to scour?

### 4.2.3.10 Revetment Design

This section recommends revetment protection in the vicinity of the structure, as discussed in Chapter 10 for bridges, Chapters 9 and 11 for culverts and storm drain outfalls.

**Revetment Around Bridges -** The riprap protection for a bridge is discussed in the narrative and it is shown on the waterway opening drawings and additional drawings if needed. A typical recommendation for riprap protection around a spillthrough abutment follows. Design discharges for end fills (embankments) are the 50-year event. Larger flows are used as check discharges, as discussed in Chapter 3. Riprap recommendations for vertical abutments, piers, and other applications use modified versions of the following language.

The abutment and embankment protection for the proposed bridge is designed to withstand the 50-year* event. Recommended protection of the 1V : 2H abutment end slopes are a 3.0 foot thick blanket of Class 700 loose riprap with riprap backing, as shown in Figure 1. The riprap should conform to Section 00390.00 of the ODOT 2008 Oregon Standard Specifications for Construction. The riprap backing should be Type 2 riprap geotextile conforming to Section 00390.10 of the Standard Specifications. A granular filter blanket is not recommended.

* by letter from Deputy Director

The riprap should wrap around the bridge abutments and protect the fresh embankment slopes below elevation 1,362.9 feet. 4.0 foot deep toe trenches should be provided. The toe trench is not needed for slopes away from the stream channel. The revetment should extend longitudinally along each embankment face a distance sufficient to protect the end panels from undermining.

Riprap or other protection that is considered to be temporary should be addressed in the report. The expected protection life and the consequences of its failure should be mentioned. Any required periodic maintenance should be described.

**Revetment Design for Culverts -** Recommended revetment around culvert inlets and outlets is discussed in the narrative and shown on the culvert drawing. A typical ODOT recommendation for revetment around a culvert follows. Revetment recommendations for other types of culverts use modified versions of the following language.

Erosion protection consists of a 1.5 foot thick blanket of Class 100 loose riprap around the inlet and outlet up to elevation 76.4 feet. The riprap should conform to Section 00390.00 of the ODOT 2002 Oregon Standard Specifications for Construction. A filter blanket is not needed. The riprap should extend along the embankment at least 10 feet from the outside edge of the wingwalls. The toe trench around the outlet cutoff wall should be backfilled with Class 50 riprap after the wall is constructed, as shown in Figure 1.

**Items to Check**
(Bridges and Culverts)

_____If needed, are riprap backing specifications mentioned?
_____Does the revetment design narrative or drawing note the type or class of revetment, blanket thickness, toe trench dimensions, and minimum elevation of revetment protection?
_____Is wave action a concern and is it addressed?

_____Are narrative and drawings included to describe special or unique revetment design features?

**Items to Check**
(Bridges)

_____Does the narrative or drawing indicate the extent of revetment protection needed on the abutment end slopes, embankment side slopes, and piers?
_____If revetment or other protection is temporary, is expected life, failure consequences, and maintenance discussed?

**Items to Check**
(Culverts)

_____ Is the revetment protection shown or described for the culvert ends, such as revetment around cutoff walls, collars, wingwalls, etc?
_____ Are the vertical and horizontal limits of the revetment on the embankments shown on the drawings or mentioned in the narrative?

### 4.2.3.11 Temporary Water Management

The report should have recommendations for temporary water management (TWM). This data includes, but is not limited to:

- a brief description of the TWM facilities,
- a brief discussion about fish passage through the TWM facilities, noting any limitations such as passage blockage,
- the in-water work period,
- recommended period of the year for the TWM facilities, and
- a TWM discharge table, as shown in the table at the end of this subsection. The methods to calculate these discharges are described in Chapter 7.

**Items to Check**

Additional TWM data is needed if a detour roadway is used, such as the flow area of the detour structure and minimum elevation of the detour roadway. The report should also contain a brief statement about the detour location. Other information about the detour may include a discussion of maintenance needs such as monitoring for debris or scour. Detour structures in floodways are also subject to Federal Emergency Management Agency standards and these hydraulic requirements should be mentioned. Detours are sometimes subject to fish passage and navigational clearance requirements just like permanent structures, and these requirements should also be discussed.

**DRY CREEK AT HWY 99W**

ESTIMATED DISCHARGES FOR

TEMPORARY WATER MANAGEMENT

| | AVERAGE DAILY DISCHARGE IN CUBIC FEET PER SECOND (GALLONS PER MINUTE) | | |
|---|---|---|---|
| **NOTE** | **1** | **2** | **3** |
| JULY | 3.1 (1,400) | 2.0 (900) | 1.4 (630) |
| AUGUST | 2.0 (900) | 1.1 (490) | 0.77 (350) |
| SEPTEMBER | 2.5 (1,100) | 1.3 (580) | 0.68 (310) |
| OCTOBER | 11.0 (5,000) | 3.3 (1,500) | 1.4 (630) |

1) 5 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 2 days each month.)
2) 25 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 8 days each month.)
3) 50 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 16 days each month.)

- In-Water work period extends from 1 July through 15 October.
- Temporary water management shown on plans recommended for 1 July through 15 October.
- Listed discharges are surface water from the upstream watershed.
- The estimated discharges are based on nearby gaged basins.
- Discharges in the subject watershed may differ.

## Items to Check

_____ Are the TWM facilities briefly described?
_____ Is fish passage through the facilities described and any limitations noted?
_____Are TWM discharges listed with seasonal limitations?
_____Are minimum waterway opening areas and minimum elevations listed for detour roadway bridges or culverts?
_____If applicable, are other detour structure issues addressed such as maintenance needs, fish passage, and navigational clearance?

#### 4.2.3.12 Rounding-off Guidelines

Guidelines for rounding-off hydraulic data are listed in this subsection.

- Elevations of structural members such as bridge decks or footings: round to the nearest hundredth of a foot.
- Channel and bridge opening widths: round to the nearest foot for structures over 100 feet wide. Round to nearest tenth of a foot for smaller openings.
- Cross-section areas: round to nearest square foot.
- Drainage areas: round to nearest three significant figures.
- Velocities: round to nearest tenth of a foot per second.
- Discharges:
  - round to two significant figures for discharges less than 1000 cubic feet per second,
  - round to three significant figures when discharges are greater than 1000 cubic feet per second, and
  - round to four significant figures when discharges are greater than 10,000 cubic feet per second.
- Pipe and pipe-arch sizes: use sizes listed in Oregon Standard Drawings RD380, RD382, RD384, and RD386.
- Box culvert sizes: use sizes listed in ODOT Standard Drawings BR820, BR825, BR830, BR835, BR840, or BR841.

## 4.3 Bridge and Large Culvert Study Supporting Data

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These guidelines apply to supporting data for hydraulic studies of the facilities listed in Section 4.2. The supporting data in many aspects is as important as the project report. It provides:

- information needed for review prior to approving the report for distribution,
- documentation to justify and support the report recommendations, and
- useful information for maintenance of the proposed facility, the designs of modifications to the facility, and the hydraulic designs of nearby or subsequent facilities, and
- supporting data for resolution of drainage complaints and lawsuits.

These guidelines are for a typical bridge or culvert and they are not applicable for all situations. Engineering judgment should be used when deciding whether or not to retain data. Factors to consider are the availability of the data from other sources, the ease of recreating the data if it is needed, the importance of the data to the study, and the utility of the information.

In general, all data and calculations should be retained that support the data in the Hydraulic

Data Sheets and the details shown on the drawings. Data is retained for the existing structure and the design alternative that is built. Notes and calculations are discarded for alternative structures that are not built. Recommendations for specific topics follow.

**Regulatory Requirements and Permits Supporting Data -** The following should be retained in the project file:

- copies of correspondence, certification letters, applications, and other materials related to the compliance with floodplain regulations, and
- copies of FEMA floodway boundary revision requests.

The critical correspondence and forms are retained. Generic information is not kept, such as the instructions on how to fill out the forms, copies of regulations, etc.

The following material is referenced in the main project file. It is stored separately in the Geo-Environmental Section's Engineering and Asset Management Unit with similar material from other locations and projects:

- the original HEC-2 or HEC-RAS river modeling input data supplied by FEMA, and
- FEMA maps and flood studies.

**Hydrology Supporting Data -** The following should be retained in the project file:

- copies of flood parol evidence such as a witness statement and supporting material, and
- hydrologic calculations (including input and output printouts if computer solutions are used).

The following material is referenced in the main project file. It is stored separately in the Geo-Environmental Section's Engineering and Asset Management Unit with similar material from other locations and projects:

- flood photographs and highwater mark survey information, and
- USGS quadrangle maps showing the drainage area.

**Hydraulic Design, Scour Calculations, and Revetment Design Supporting Data -** The following should be retained in the project file:

- notes and calculations detailing the assumptions used to determine the downstream water surface elevations if variable elevations can occur (see Chapter 10),
- notes and calculations detailing the overtopping flow assumptions used in the analysis of the existing and proposed structures,
- notes and calculations for the hydraulic design, scour calculations, and revetment design

including printouts of the input and output files if a computer solution is used, and
- documentation of past scour problems if the material is not stored in another file, such as the bridge maintenance file.

**Detour Design Supporting Data -** The detour structure design data is retained until the detour is removed and any disputes related to the detour are settled.

**Other Supporting Data -** The following should be retained in the project file:

- the original signed and stamped copy of the Hydraulics Report,
- the bridge vicinity map (typically this map shows the locations of the existing and proposed structures, channel profile location, and cross-section locations),
- notes from the hydraulic survey, such as stream cross-sections, profiles, and other hydraulic topographic data,
- photographs,
- as-constructed plans that show the waterway opening and pile tip or footing elevations, and
- critical correspondence and other information that should be filed and retained.

Paper copies of the previously listed location information can be quite bulky and difficult to file. It is recommended that digital terrain models, other electronic survey data, and hydraulic computer models be copied onto a CD and stored in the project file instead of paper copies.

## 4.4 Small Culvert Design Documentation and Supporting Data

Documentation and supporting data retention guidelines for small culvert hydraulic studies are in this section. These culverts are usually designed by the roadway designer. Small culverts are:

- circular culverts with diameters less than or equal to 4 feet,
- pipe-arches with equivalent diameters less than or equal to 4 feet, and
- box culverts with spans less than or equal to 4 feet.

Report documentation for culverts with diameters or equivalent diameters of 6 feet or larger, culverts designed to pass fish, or open-bottom culverts are outlined in Section 4.2. Report documentation guidelines for culverts with diameters larger than 4 feet, but less than 6 feet, are presented in Section 4.5.

Documentation is described for a typical small culvert. In some cases additional documentation may be warranted. An example would be a small culvert located in a floodway. In this instance, documentation would be needed to demonstrate the replacement culvert would comply with

applicable flood management ordinances. As a general rule, small culvert documentation should include:

- verification that design objectives and standards have been met,
- data needed to incorporate the culverts into the roadway design,
- data about the capacity of the culverts if discharge changes are anticipated during the project design life (such as runoff from anticipated development),
- information on special maintenance needs and anticipated modifications such as extensions, and
- any changes to hydraulic conditions at nearby, upstream, or downstream drainage facilities due to the culvert replacement.

Documentation for small culverts may include many items, such as design sheets, maps, calculations, narrative, tables, and drawings. The following is needed for a typical design.

- Design sheets described in Chapter 9 for the following discharges described in Chapter 3:
  - design events,
  - check floods (cross-culverts, only), and
  - base floods (culverts subject to floodplain management ordinances).
  - Drainage maps showing runoff patterns and drainage area boundaries.
  - Calculations and assumptions for headwater and tailwater elevations, outlet scour protection, fill heights, barrel material life, and invert protection design.
  - Recommended barrel materials in conformance with ODOT alternate materials policy, as discussed in Chapter 5. Occasionally, alternate materials cannot be specified because only one type of barrel material is suitable. If so, it should be documented in the narrative why alternate materials cannot be used.
  - Recommended end treatments as discussed in Chapter 9.
  - Descriptions of special design considerations such as upstream settling basins, etc.
  - Descriptions of maintenance needs if they will be atypical or unusually frequent.
  - If needed, recommended invert protection as discussed in Chapter 5.
  - If needed, details of channel changes, ditches, etc.
  - If needed, outlet erosion protection as discussed in Chapter 11.
  - If needed, details of debris control structures as discussed in Chapter 9.
  - If needed, details of streambank protection as discussed in Chapter 15.
  - Important correspondence.

**Items to Check**

_____Are culvert design sheets included?
_____Are drainage maps included?
_____Are design assumptions and calculations included?
_____Are recommended barrel materials and end treatments listed?

_____Are special design considerations mentioned and described, such as settling basins, atypical or frequent maintenance, invert protection, channel changes, outlet erosion protection, debris control structures, etc?
_____Is important correspondence included?
_____Are changes to hydraulic characteristics at nearby structures noted?

## 4.5 Medium Culvert Design Documentation and Supporting Data

Medium culverts have diameters or equivalent diameters greater than 4 feet, but less than 6 feet. These culverts are designed by a professional engineer registered in Oregon or people under the direct supervision of the engineer, as discussed in Chapter 3.

Medium culverts where fish passage is not required, and with no special design features, can use the level of documentation appropriate for small culverts. Medium culverts requiring fish passage or having special design features need the level of documentation required for large culverts. Examples of special features are:

- roughening rings inside the culvert barrel,
- cast-in-place reinforced concrete drop, side tapered, or slope-tapered inlets, or
- cast-in-place reinforced concrete energy dissipators.

Medium culverts in critical locations may also justify a "large culvert" design study. An example is a culvert to be jacked or rammed through a deep fill under a wide highway. This culvert would be a very expensive installation, and a more comprehensive "large culvert" study is justified.

## 4.6 Stormwater Design Documentation

These documentation guidelines are for all projects that include stormwater designs. They are primarily intended for facilities conveying, treating or storing runoff from ODOT property.

Projects that include ODOT right-of-way improvements along with other improvements (private development, local agency, or other) will need to comply with ODOT and/or the local or governmental agency with jurisdiction for stormwater treatment. These projects will need to also comply with all requirements for detention of runoff from sources draining onto ODOT property or into ODOT drainage systems. The report writing guidelines for these systems are included in Appendix C.

### 4.6.1 Water Resources Impact Assessment

The Water Resources Impact Assessment is a document stating whether or not water quality treatment is needed. If the determination is that treatment is required, then the document states what level (engineered facility or BMP only) of treatment is needed. The water resources impact assessment is a short report developed early in the design process by environmental personnel.

The need for peak flow control is also determined early in the design process. It is sometimes stated in the water resources assessment when it is required by environmental regulations. Peak flow control requirements are often determined by the hydraulics engineer when local regulations or liability considerations are identified.

The documentation for water quality treatment and peak flow control requirements are retained as a reference for subsequent recommendations and reports.

### 4.6.2 Preliminary Stormwater Recommendations

This is a formal document that outlines the scope of the project, the basic storm water conveyance system layout, pipe outfall locations, treatment and/or storage requirements, and the recommended treatment and/or storage concepts. These recommendations do not contain facility designs. It is a tool to assist in the selection of the types and locations of the facilities to be designed and is an important tool to assist other personnel (environmental, right-of-way, hazardous materials experts, etc). These recommendations are prepared after the receipt of the Water Resources Impact Assessment and a determination of the need for storage, and before the preliminary engineering and facility designs. Preliminary facility design commences after the team approves the design concepts in the recommendations.

The preliminary stormwater recommendations should be prepared by, or under the direct supervision of a registered professional engineer and the report should be sealed by the engineer, as discussed in Chapter 3. Copies of the recommendations should be provided to the project development team for review and comment. Typically, this includes the project team leader, the environmental representative, the roadway and/or bridge designers, right-of-way agent, hazardous materials specialist, and if involved, the local agency representative. The review comments are to be considered and addressed, and the recommendations revised as needed. The revised recommendations should be approved by all team members before the start of facility design.

The contents of typical preliminary stormwater recommendations are presented in the remainder of this subsection. Specific guidance for storage and water quality designs is included in Chapters 12 and 14, respectively.

**Introduction and Title -** The preliminary recommendations starts with a title that lists the project name, highway name and number, beginning and ending milepoints, county, and date of the report. The introduction mentions the people who prepared the recommendations, the purpose of the report, a brief description of the project, and a summary of treatment/storage concepts and recommendations on their use. An example follows.

PRELIMINARY STORMWATER RECOMMENDATIONS
Oregon Coast Hwy. @ Cannon Beach N. Entrance Section
Oregon Coast Highway (OR # 6), M.P. 27.96 to 28.68
Clatsop County
3 June 2001

1.0 INTRODUCTION

This recommendations were prepared by _____P.E. with special thanks to _____ for preparing a significant amount of the technical information.

1.1 PURPOSE

The purpose of these recommendations are to outline potential solutions for improving the quality of storm water runoff from the facilities located within the Cannon Beach North Entrance project. It also outlines possible solutions for detaining the additional runoff from the added impervious surface.

1.2 JOB DESCRIPTION

The primary purpose of the project is to reconstruct the intersection at the north entrance to Cannon Beach from the Oregon Coast Highway. A southbound on and off-ramps will be constructed as well as a northbound on-ramp. The northbound on-ramp will pass underneath a new structure to be built on the realigned Oregon Coast Highway.

1.3 SUMMARY

The following information has been summarized for quick reference. Assumptions and design criteria used to develop these recommendations are described in more detail in subsequent sections.

## WATER QUALITY MITIGATION LOCATIONS

Total = 2

| Location # | Station | Description |
|---|---|---|
| 1 | 1496+06 to 1499+34 | NB On-ramp O'xing |
| 2 | 1509+19 to 1515+75 | Ecola Creek |

| | | Location Number 1 | | 2 | |
|---|---|---|---|---|---|
| Mitigation Alternatives | | Recommended | Acceptable | Recommended | Acceptable |
| A | Extended Detention Dry Pond[1] | Yes | Yes | No | Yes |
| B | Bioretention Pond | No | No | No | No |
| C | Constructed Wetland | No | No | No | No |
| D | Infiltration Trench / Dry Well[2] | No | No | No | No |
| E | Infiltration / Retention Basin[2] | No | No | No | No |
| F | Bioslope | No | No | No | No |
| G | Biofiltration Swale | No | Yes | Yes | Yes |
| H | Filter Strip | No | No | No | Yes |
| I | Porous Pavement | No | No | No | No |
| J | Proprietary Structure | No | Yes | No | Yes |

## STORAGE MITIGATION LOCATIONS

Total = 1

| Location# | Station | Description |
|---|---|---|
| 1 | 1496+06 to 1499+34 | NB On-ramp O'xing |

| | | Location Number 1 | | 2 | |
|---|---|---|---|---|---|
| Mitigation Alternatives | | Recommended | Acceptable | Recommended | Acceptable |
| I | Dry Pond[1] | Yes | Yes | N.A. | N.A. |
| II | Vault[3] | No | Yes | N.A. | N.A. |
| III | Tank[3] | No | Yes | N.A. | N.A. |

[1]A single dry pond could satisfy both treatment and storage requirements at Location 1.
[2]Mitigation Alternatives D and E could eliminate the need for storage at Location 1.
[3]A vault or a tank would be built at Location 1 in addition to the selected treatment facility.

**Existing and Proposed Conditions Narrative –** The introduction is followed by a narrative that describes the proposed changes to the existing conditions. The pollutant removal and storage targets should also be mentioned. An example follows.

2.0 EXISTING AND PROPOSED CONDITIONS

2.1 GEOGRAPHY

The project runs at a steady 6 percent slope from north to south down to Ecola Creek. It appears that the entire project is within the same drainage basin. Approximately 4.3 acres of new impervious surface will be constructed on this project.

- Location 1 is the northbound on-ramp overcrossing which is the approximate middle of the project. Approximately 2.5 acres of new impervious area will be constructed from the beginning of the project to here. Storage in the form of detention will be required at this location.
- Location 2 is Ecola Creek which is at the end of the project. Approximately 1.85 acres of new impervious area will be constructed from Location 1 to here. Detention is not required at this location.

2.2 TARGET POLLUTANT REMOVAL

There are no specific constituents identified by regulations for storm water on this project. Total suspended solids (TSS) will be targeted because it is the most common and measurable constituent. A XX-percent TSS removal rate from the new impervious surface area will be the goal.

2.3 TARGET DETENTION RELEASE RATE

The runoff at Location 1 enters an ODOT cross-culvert that has limited capacity for increased peak discharge. This culvert is not within the limits of this project. As a result, at Location 1 for the 50-year runoff event, the post-construction release rate from detention should not exceed the pre-construction runoff rate of 2.45 cubic feet per second.

**Proposed Mitigation Alternatives -** The third section in the Preliminary Stormwater Recommendations is a brief generic discussion of all proposed mitigation alternatives. Topics addressed are location, removal efficiency, storage capacity, constructibility, maintenance, and cost. BMP and storage alternatives from Chapters 14 and 12, respectively, are evaluated and listed.

Example recommendations follow. Three different mitigation alternatives were discussed in the example; extended detention dry ponds, biofiltration swales, filter strips, proprietary treatment structures, vaults, and tanks. These alternatives were listed as either recommended or acceptable in the preceding summary table. The discussion of one alternative, the extended detention dry pond, is included in the following text.

3.0 PROPOSED MITIGATION ALTERNATIVES

A total of four different water quality BMP's and three storage facilities are described in the following sections. Six BMP's are not included due to their high costs or other factors making them less desirable. Additional information is available on these BMP's if requested.

**MITIGATION ALTERNATIVES**

| | Mitigation Alternatives | Included | Excluded |
|---|---|---|---|
| 3.1 | Extended Detention Dry Pond (for treatment and storage) | * | |
| | Bioretention Pond (treatment) | | * |
| | Constructed Wetland (treatment) | | * |
| | Infiltration Trench / Dry Well (treatment and storage) | | * |
| | Infiltration / Retention Basin (treatment and storage) | | * |
| | Bioslope (treatment) | | * |
| 3.2 | Biofiltration Swale (treatment) | * | |
| 3.3 | Filter Strip (treatment) | * | |
| | Porous Pavement (treatment) | | * |
| 3.4 | Proprietary Structure (treatment) | * | |
| 3.5 | Vaults or Tanks (storage) | * | |

3.1 EXTENDED DETENTION DRY POND

Extended dry detention pond basins are depressions that treat and provide temporary storage (also called detention) of a portion of the stormwater runoff following a storm event. The extended detention time of the stormwater in the basin provides an opportunity for pollutants carried by the runoff to settle out. A 24- to 48-hour retention period is needed after each rainfall event. Water quality design volume is in addition to detention storage volume.

3.1.1 LOCATION

A dry pond could be placed at the outfall of the storm drain system at each location. The pond at Location 1 would be designed to provide both treatment and detention. The pond at Location 2 would be designed to provide treatment, only. The ponds can be placed "in-line" or "off-line."

3.1.2 CONSTITUENT REMOVAL EFFICIENCY

80% to 90% TSS Removal

3.1.3 DETENTION RELEASE RATE

Less than or equal to the pre-construction runoff rate of 2.45 cubic feet per second during the 50-year runoff event.

3.1.4 CONSTRUCTABILITY

A pond would need to be excavated large enough to hold the water quality storm (1/3 of the 2-yr/24-hour storm) and to detain the excess runoff from the 50-year storm. An access road would have to be provided as well as access to the bottom of the pond. The size of the pond is not affected by whether it is placed "in-line" or "off-line."

3.1.5 MAINTENANCE

Typical maintenance is expected to be:

- mowing to keep weeds under control,
- debris removal from control device, and
- sediment removal (every 1 or 2 years).

3.1.6 COST

Low to medium cost. Work included in construction will include, but not be limited to:

- excavation,
- seeding,
- outlet control structures (an orifice-standpipe-inlet at Location 1 and two Type "D" inlets at Location 2),
- scour protection near outlet ends of pipes discharging into ponds,
- scour protection near outlet ends of pipes discharging from ponds,
- scour protection (riprap) on auxiliary outlet spillways, and
- landscaping and planting trees around pond perimeters for shading.

**Other Issues –** The fourth section discusses mitigation issues that are not addressed in the previous section on alternatives. The following discussion is from the example recommendations.

4.0 OTHER ISSUES

Curbs vs Ditches – In the current design, the entire project area drains into roadside ditches and cross-culverts. All of this runoff would enter the treatment facility, and the water contains both runoff from pervious surfaces off of the roadway as well as impervious roadway surfaces. This is substantially more water than the mitigation alternative is required to treat. Treatment is needed for runoff from the new impervious surface area, only.

*Note: The impervious surface area runoff can be collected from both existing and new impervious surfaces if the total impervious surface area to be treated is equivalent to the calculated new impervious surface area.*

The proposed typical sections show no curbs. Curbed sections with inlets can be used to convey roadway runoff that requires treatment to the water quality facilities, and ditches can convey runoff from other areas that do not require treatment to the outfall. As a result, the use of curbs and inlets may substantially reduce the inflow of water into the treatment facilities, and smaller and less costly facilities may be adequate.

**Recommendations –** This section has preliminary recommendations about the proposed alternatives. Many aspects are addressed, such as dependability, ease of construction, ease of maintenance, cost, and appearance. An example follows.

5.0 RECOMMENDATIONS

The following table in addition to technical references were used to develop the recommended solutions for water quality mitigation and storage on this project:

| | Mitigation Alternative | Dependability | Construction | Maintenance | Cost | Appearance |
|---|---|---|---|---|---|---|
| A&I | Extended Dry Detention Pond | High | Med | Med | Low-Med | Fair |
| G | Biofiltration Swale | High | Med | Low-Med | Low | Good |
| H | Filter Strip | Med | Easy | Low-Med | Low | Good |
| J | Proprietary Structure | High | Easy | Med | High | None |

An Extended Dry Detention Pond as described in Subsection 3.1 is the recommended alternative at Location 1, and a Biofiltration Swale (Subsection 3.2) is the recommended alternative at Location 2.

**Items to Check**

_____Is the project and its purpose adequately described in the Introduction, Purpose, and Job Description sections?
_____Is a schematic concept design plan that clearly shows locations of each facility included?"
_____Are pipe outfall locations identified?
_____Are all of the mitigation alternatives that were considered summarized and categorized as "Recommended", "Not Recommended", "Acceptable" or "Not Acceptable?"
_____Are the existing and proposed conditions described, including target pollutant removal rates and storage system requirements?
_____Are the viable alternatives described with their proposed locations and estimates of their pollutant removal efficiency or storage adequacy?
_____Are constructability, maintenance, cost, appearance and other issues addressed?

_____Are recommendations included that address dependability, construction, maintenance, cost, and appearance?

### 4.6.3 Standard Stormwater Design Documentation

These documentation guidelines are for standard stormwater designs. Standard stormwater designs include roadway inlets, small storm drains and small channels or ditches. This information is part of the drainage design provided to the roadway designer who incorporates the drainage features into the roadway design. It also may be part of the work done by the roadway designer if the drainage and roadway designs are done concurrently.

Documentation for the typical standard stormwater design is included in this section. In some cases additional documentation should be submitted. An example would be documentation for a city in a cost-sharing agreement with ODOT, when it must be shown that the drainage system performs adequately during the "full buildout" discharge. As a general rule, the documentation for a stormwater design should provide all information needed to:

- review the drainage design and verify that design objectives and standards have been met,
- incorporate the drainage design into the roadway design,
- provide data about the capacity of the drainage system (this is used to evaluate future requests to discharge additional flow into the system), and
- aid in the maintenance of the facility, the design of modifications to the system, and the hydraulic design of nearby or subsequent facilities.

The standard stormwater design documentation should be prepared by, or under the direct supervision of, a registered professional engineer as discussed in Chapter 3. The package of materials should be bound together with a cover letter sealed by the engineer.

Documentation for stormwater systems includes many items, such as design sheets, maps, calculations, narrative, tables, and drawings. The following documentation is needed for a typical design.

- ODOT "Storm Sewer Design sheets (reference Chapter 13) for all storm drains shown on the map(s). The top of manhole and downstream invert elevations are listed on the design sheets. The elevations of other critical components should also be provided, such as gutters, ditch bottoms, upstream pipe inverts, manhole bottoms, etc. This data can be provided by profiles, tables, sketches, etc. Computer output can be provided in lieu of design sheets.
- A drainage system map(s). A small system can often be shown by a single map. A large system may require several maps. These maps show the location and type of the following features.

  - The existing utilities and structures which could be in conflict with the proposed storm drain.
  - The direction of overland flow into the drainage system,
  - The boundaries of drainage areas which contribute runoff to the system, including drainage areas for tributary drain systems. The divisions between different types of land use should be shown on the map (pavement, cultivated fields, etc.) along with the assigned runoff coefficients.
  - The surface contours at least within the drainage limits.
  - Streams, lakes, and other watercourses, past highwater and existing water surface elevations at the outfalls of the storm drains.
  - The estimated limits of flooding that would occur
  - The elevations of drainage divides, sags, and low points.
  - Drainage features such as:
    - gutters and ditches,
    - inlets,
    - pipes, including sizes (use standard sizes) and discharges,
    - manholes,
    - culverts such as driveway, road approach, and cross,
    - drain outfalls to existing streams, lakes, or other watercourses, and
    - additional features such as water quality facilities, detention facilities, energy dissipators, etc.
- Calculations and assumptions for hydrology including time-of-concentration, discharge elevations and velocities, tailwater elevations at outfall, barrel material life, and fill heights.
- Location of significant utilities which may require relocation or may dictate the final design configuration.
- Sketches showing junction structures (manholes, inlet boxes, etc) are of sufficient size to accommodate planned pipe connections.
- Tables, sketches, or narrative as needed to describe special features, allowable or recommended alternate materials, and invert protection.
- Important correspondence.

**Items to Check**

_____Are storm drain design sheets included?
_____Is additional elevation data provided?
_____Are drainage maps included which show all features?
_____Are design calculations and assumptions included?
_____Are recommended barrel materials and end treatments listed?
_____Are detention facilities required and/or included?
_____Are stormwater treatment facilities required and/or included?
_____Are junction structures sized appropriately?

_____Are special design considerations mentioned and described, such as settling basins, atypical or frequent maintenance, invert protection, channel changes, outlet erosion protection, debris control structures, energy dissipators, etc?
_____ Is important correspondence included?

### 4.6.4 Stormwater Design Report

These documentation guidelines are intended for stormwater designs that include significant facilities and large or critical projects. The documentation for these projects is greater than the standard stormwater design documentation. Significant facilities include, but are not limited to, any of the following:

- storm drain systems with pipes larger than 24 inches in diameter,
- stormwater quality facilities, and
- stormwater control facilities (detention, retention, split-flow structures, etc.)

This is the final report that describes in detail the facilities approved by the project development team after their review of the Preliminary Stormwater Recommendations. This report provides facility design information such as the type, size, location, critical dimensions, and features. It is usually completed after the advance plans and concurrently with the preparation of the project final plans.

The stormwater report should be prepared by, or under the direct supervision of, a registered professional engineer, and it should be sealed by the engineer, as discussed in Chapter 3. Copies of the stormwater report should be provided to the project development team. Usually this includes the project team leader, the environmental representative, the roadway and/or bridge designers, and if involved, the local agency representative. Also submit a copy in Adobe Acrobat portable document format (pdf) to Geo-Environmental's Senior Hydraulics Engineer by completing the project report submittal form located at the following website:

http://www.oregon.gov/ODOT/HWY/GEOENVIRONMENTAL/hyd_data_resources.shtml

The facility design(s) incorporated in the final plans should comply with the information in the stormwater report unless approval for any change has been obtained from the engineer who sealed the report. The contents of a typical stormwater design report are presented in the remainder of this subsection.

STORMWATER DESIGN REPORT
for
XXXXXXXXXXXXXXXXXX
BRIDGE (PORTLAND) SECTION

MT. HOOD HIGHWAY (OR #26), M.P. 0.31 to 1.02
CITY OF PORTLAND, MULTNOMAH COUNTY, OREGON

Key # 00000

Contents

- ☒ Inlets
- ☒ Storm Drains
- ☒ Detention
- ☐ Small Culverts
- ☒ Water Quality
- ☐ Small Channels
- ☐ Energy Dissipaters
- ☐ Pipe Rehabilitation

OCTOBER 1999

DFI No. D00001
DFI No. D00002
DFI No. D00003

Prepared by _____

(Seal affixed here)

**Cover Sheet and index-** The report cover sheet includes the title, the project name, the highway name and number, the beginning and ending milepoints, the county, and the unique drainage facility identification number (DFI) for every facility proposed on a project. This information should match with the data provided on the title sheet for the plans. The cover sheet also carries the seal of the engineer of record. An example follows.

*Note: See Chapter 17 for details on obtaining and assigning a DFI.*

1. **Overview**, including the following.

    a. **Project description**, including the overall project scope, including the need for the project.

    b. **Purpose of the study**, including a brief description of the facility design objectives, including the source of the objectives (i.e. environmental regulations, local drainage requirements such as drainage master plans, liability concerns, etc). This topic will be discussed in more detail in the body of the report. This discussion also includes the following.

        i. **Statement** that the design objectives have been met.

        ii. **Explanation** about why any design objectives have not been met, if this is the case.

    c. **Key issues** affecting project scope, need, or design.

    d. **Summary of the results**, as would be desired by a casual reader of the report, including abbreviated tables of pipe sizes and other facilities for quick reference.

2. **Background**, including information about the existing conditions and factors influencing the design. It includes the following.

    a. **Watershed characteristics**, both pre-construction, post-construction, and at the level of buildout expected at the end of the facility design life. Topics to be discussed are drainage area sizes, land uses, and other characteristics affecting drainage.

    b. **Project area** characteristics, with emphasis on the drainage systems.

        i. **Pre-construction** conditions, including the following.

            - **Description** of the existing drainage facilities.

- **Description** of existing drainage problems. This is especially important if a purpose of the project is to rectify an existing drainage deficiency.

- **Condition** of the existing system. This is especially important if a purpose of the project is to replace or upgrade a deteriorated drainage facility.

**ii. Post-construction** conditions, including a description of the proposed facilities.

**c. The outfall**, including the following.

**i. Description** of the outfall, including condition.

**ii. Discharges** expected at the outfall in the pre-construction, post-construction, and buildout at end of design life land use conditions.

**iii. Discussion** of the ability of the outfall to satisfactorily convey the three previously listed discharges.

**d. Utilities**, including the following.

**i. Summary** of the utility location information available and used in the design. Mention the limitations of the utility location data. Mention if utilities are present, there may be conflicts, and the utility locations are not known.

**ii. Description** of any utilities that affected the design, their effects, and how the effects were addressed in the design.

**e. Investigations**, including the following.

**i. Research/previous studies** used in the design should also be referenced. Examples are the hydraulic study for a previous design or a local Drainage Master Plan.

**ii. Site reconnaissance** used to collect design data should be mentioned. An example is the location survey or a video inspection of the existing facilities.

**3. Design,** including detailed design information. The reader is also referenced to the supporting design information listed in Subsection 4.6.3 of this chapter. This information must be included in the Design section of the report.

**a. Design criteria**, mentioning all criteria used in the design. References are made to published material available externally, and also to correspondence and other material retained in the supporting data file.

**b. Analysis methods** used in the design, including the following.

**i. Hydrology**, including method used, with assumptions.

**ii. Hydraulics**, including method used, with assumptions.

**c. Narrative and calculations** used in the design. The descriptive narrative and calculations for simple projects can be included in this report. Otherwise, a summary of the design calculations is appropriate for this section, and references are made to detailed information in the supporting data. Include a separate subsection for each facility or system.

**4. Maintenance** is addressed in this section, if any special activities will be required, or unusually frequent maintenance is expected. An example is a special manhole containing a proprietary filtering system. This system, along with its maintenance requirements, would be mentioned in the report. Detailed information will be included in an "Operation & Maintenance Manual" (see Subsection 4.6.6).

**Items to Check**

_____Is a cover sheet included with all appropriate information and the engineer's seal?
_____Is an overview included?
_____Is a background included?
_____Is design information such as a narrative and calculations included or referenced for each facility or system?
_____Is extraordinary maintenance described, including special activities or the need for unusually frequent service.

### 4.6.5 Stormwater Design Supporting Data

Typical stormwater design data is listed in Subsection 4.6.3 of this chapter. Specific projects may require more or less than the listed data. In general, data is retained that:

- shows the performance of the existing system, it is analyzed,
- shows the performance of the proposed system shown on the project plans,
- provides information for future maintenance,
- provides information of value if there is future litigation, or
- provides information useful for future designs and drainage planning.

Essential data includes, but is not limited to, drainage area maps and summaries of the hydrology and design data.

Data is not retained in the files that is:

- readily available in published references, such as textbooks,
- data pertaining to alternatives that are not built, and
- information used in project development that does not directly support the finished product, such as preliminary recommendations.

### 4.6.6 Stormwater Operation & Maintenance Manual

The Stormwater Operation & Maintenance Manual provides information about facility maintenance and operation. A unique manual must be prepared for every ODOT stormwater treatment and storage facility (low impact development approaches, swales, filter strips, bioretention basins, bioslopes, extended detention dry ponds, proprietary structures, storage dry ponds, tanks, and vaults) and copies of this manual are to be distributed to the personnel who maintain the facility, and the original document is retained in the project files. Also submit a copy of the manual in Adobe Acrobat portable document format (pdf) and in Microsoft Word format, and the operational plan Microstation file to Geo-Environmental's Senior Hydraulics Engineer by completing the project report submittal form located at the following website:

Note: A unique Operation and Maintenance Manual is not required for a pollution control manhole. Always note the location of these manholes on every treatment or storage facility operational plan.

http://www.oregon.gov/ODOT/HWY/GEOENVIRONMENTAL/hyd_data_resources.shtml

An inventory of prepared manuals can be viewed at the following website:

Operation & Maintenance Manuals Website

Two Microsoft Word template documents have been created for detention and water quality facilities. Use these templates to prepare a final Operation and Maintenance Manual. These templates are located in Appendix D. These Microsoft Word documents are "filename.dot" files and have to be saved as a "filename.doc" file. The following steps outline how to create a ".doc" file:

Step 1: Open the appropriate template located in Appendix D. The template for detention facilities is named "OM_Template_DetentionFacility". The template for water quality facilities is named "OM_Template_WaterQualityFacility"

Step 2: Select the "Save" command. Change the file type to "word document (*.doc)" in the field box titled "Save as type".

Step 3: Modify the file name using the drainage facility identification number. For example, the file name for facility #345 would be "OM_DFI_D00345".

Step 4: Select the "Save" button. You now can use the saved document to prepare the final Operation and Maintenance Manual.

The remaining section describes the contents of a typical manual. It proceeds through the manual from beginning to end.

**Cover Sheet -** The report cover sheet includes the title, drainage facility identification number, and facility type. The cover sheet also provides a spot for ODOT Maintenance Districts to add a photo of the constructed facility when applicable. An example follows.

1. **Identification.** This is the first section of the manual. It includes the following.

   a. **Drainage Facility Identification Number (DFI)**, listing the unique DFI number of the specific facility. For example, "D00040". See Chapter 17 for details on obtaining and assigning a DFI.

   b. **Facility Type.** The type of facility. The following is a list of facility types that have been approved for use within ODOT right-of-way, and utilized by ODOT Maintenance. Proprietary facilities or facilities that have been internally designed must be named as either "Proprietary" or "Special" facilities. (i.e. Proprietary Water Quality Catch Basin)

      - Detention Pond
      - Detention Pond/Water Quality Biofiltration Swale Combo
      - Detention Tank/Pipe
      - Detention Vault
      - Dispersion Trench
      - Water Quality Biofiltration Swale
      - Water Quality Bioretention Pond
      - Water Quality Bioslope/Media Filter Strip
      - Water Quality Catch Basin
      - Water Quality Extended Detention Dry Pond
      - Water Quality Filter Strip
      - Water Quality Infiltration Pond
      - Water Quality Manhole
      - Water Quality Porous Pavement
      - Water Quality Sediment Basin
      - Water Quality Treatment/HazMat Containment Facility
      - Water Quality Vault

   c. **Construction Drawings.** The V-file number for the project is listed. As an example: "35V-103."

**d. Location**, listing the facility location by district, highway number, milepoint, and other information needed to locate facility. For example: "District 2A, Hwy 1, MP 289.83 to MP 289.50 – on west side of highway just north of I-5 at Nyberg Road off-ramp. Access via I-5."

**2. Facility Contact Information.** This section provides a general summary of contacts for operational clarification, maintenance clarification, and repair or restoration assistance. This section requires no or little modification.

# OPERATION & MAINTENANCE MANUAL

**DFI No. D00001**

**Facility Type: Detention Pond**

**FACILITY PHOTO**

**(AS-CONSTRUCTED)**

**Prepared: (month, year)**

## INDEX

**1. Identification ..... 1**
**2. Facility Contact Information ..... 1**
**3. Construction ..... 1**
**4. Storm Drain System and Facility Overview ..... 2**
**5. Facility Haz Mat Spill Feature(s) ..... 5**
**6. Auxiliary Outlet (High Flow Bypass) ..... 5**
**7. Maintenance Requirements ..... 6**
**8. Waste Material Handling ..... 6**

**APPENDIX A: Operation Plan and Profile Drawing(s)**
**APPENDIX B: ODOT Project Plan Sheets**
**APPENDIX C: Proprietary Structure Maintenance Requirements**

3. **Construction.** This section lists the engineer of record, the year the project was finished and the name of the contractor. For example: "Engineer of Record: ODOT designer – Region 2 Hydraulics, Alvin Shoblom, (503) 986-3365, Construction was completed in the year 2000. The contractor was Moby Construction Company."

4. **Storm Drain System and Facility Overview.** This section tells how the system works, and it makes reference to detailed material in the remainder of the report and the appendices. Technical discussions and terminology are avoided. This section is written specifically for the personnel responsible for maintaining the system. Include:

   - Brief summary of location of facility
   - Type of facility and treatment function (such as detention pond or water quality biofiltration swale)
   - Overview of conveyance system such as location and features (catch basins, pretreatment structures, and piping)
   - Description of facility inlet and outlet
   - Description of facility access for equipment and maintenance
   - Note if heavy equipment is allowed with no limitation, allowed with limitation, or not allowed into above ground facilities

- Note any special facility features such as amended soils, porous pavers, liners, or underdrains

An example description and formatting preference is provided in Chapter 4, Appendix D.

5. **Facility Haz Mat Spill Feature**. This section tells what facility features are provided or could be modified to assist with capturing or containing hazardous materials. For example, and pond or swale can be used to store a volume of liquid by blocking the outlet pipe located at the outlet of the pond or swale. An example is provided in Chapter 4, Appendix D.

6. **Auxiliary Outlet (High Flow Bypass).** This section describes the overflow system and it tells maintenance personnel about items of concern. Note if the auxiliary outlet feature is designed into the facility or explain the location, type, and function of the auxiliary outlet when not designed into the facility. An example description and formatting preference is provided in Chapter 4, Appendix D.

7. **Maintenance Requirements**. This section addresses the required maintenance activities and when they should be done. References are made to other documents or sections of the manual as needed.

   Include the following summary in every manual. Mark Table 1 in every manual. Then mark the appropriate maintenance table according to facility type. Review the marked maintenance table provided in ODOT's Maintenance Guide. Select the special maintenance box when necessary and detail any additional maintenance requirements.

   Or

   Mark the box next to "Appendix C" when the facility is a proprietary structure. Obtain a copy of the Manufacturer's operation and maintenance manual and include in Appendix C. The Manufacturer's manual must include detailed maintenance instructions and drawings.

| 7. Maintenance Requirements |
|---|
| Routine maintenance table for non-proprietary stormwater treatment and storage/detention facilities have been incorporated into ODOT's Maintenance Guide. These tables summarize the maintenance requirements for ponds, swales, filter strips, bioslopes, and detention tanks and vaults. Special maintenance requirements in addition to the routine requirements are noted below when applicable.<br><br>The ODOT Maintenance Guide can be viewed at the following website:<br><br>http://www.oregon.gov/ODOT/HWY/OOM/MGuide.shtml |

Maintenance requirements for proprietary structures, such as underground water quality manholes and/or vaults with filter media are noted in Appendix C when applicable.

The following stormwater facility maintenance table (See ODOT Maintenance Guide) should be used to maintain the facility outlined in this Operation and Maintenance Manual or follow the Maintenance requirements outlined in Appendix C when proprietary structure is selected below:

☒ Table 1 (general maintenance)
☐ Table 2 (stormwater ponds)
☐ Table 3 (water quality biofiltration swales)
☐ Table 4 (water quality filter strips)
☐ Table 5 (water quality bioslopes)
☐ Table 6 (detention vaults)
☐ Table 7 (detention tanks/pipes)
☐ Appendix C (proprietary structure or special designs)

☐ Special Maintenance requirements:

An example is provided in Chapter 4, Appendix D**.**

**8. Waste Material Handling.** This section discusses handling and disposal of waste material. Contacts are listed along with phone numbers. Include the following summary with every manual. Obtain and adjust the contact number for the Region Hazmat Coordinator according to the appropriate region.

8. Waste Material Handling

Material removed from the facility is defined as waste by DEQ. Refer to the roadwaste section of the ODOT Maintenance Yard Environmental Management System (EMS) Policy and Procedures Manual for disposal options: http://egov.oregon.gov/ODOT/HWY/OOM/EMS.shtml

Contact any of the following for more detailed information about management of waste materials found on site:

| | |
|---|---|
| ODOT Clean Water Unit | (503) 986-3008 |
| ODOT Statewide Hazmat Coordinator | (503) 229-5129 |
| ODOT Region Hazmat Coordinator | (xxx) xxx-xxxx |
| ODEQ Northwest Region Office | (503) 229-5263 |

An example is provided in Chapter 4, Appendix D.

**Operation & Maintenance Manual Appendices:**

Appendices are included to organize supporting information as outlined below.

A cover sheet is included at the front of the appendices. The cover sheet lists the material that is included in each Appendix. An example of appendix cover sheets is provided in Chapter 4, Appendix D.

**Appendix A:**

**An operation plan, profile and details are required in all O&M Manuals**. An Example of an operation plan is provided in Figure 4-5 and several Operation Drawings are shown in Chapter 4, Appendix D. This plan summarizes the facility operation features and details needed to assist personnel who maintain the facility. **Copies of construction plans will not be accepted as a facility's operational plan to be used in O&M Manuals**.

Include the operational plan and profile in Appendix A of the Operation Manual. The operational plan includes:

- Legend
- Title block with the following details:
  - Drainage facility identification number
  - District number
  - Highway number
  - Facility type
  - Highway name
  - Beginning and ending milepoints
  - county

Include a plan view with the following information:

- North Arrow
- Edge of pavement lines
- Adjacent sidewalk lines
- Nearby cross streets, ramps, etc.
- Maintenenance access
- Adjacent traffic features such as barriers, sign, etc.
- Upstream and downstream drainage system (piping, ditches, etc.)
- Upstream and downstream drainage structures (manholes, catch basins, etc.)
- Pond or swale footprint lines:
  - Top of facility line
  - Bottom of facility line

- Pond or swale features:
  - Inlet pipe(s)
  - Inlet structure (flow splitter)
  - Inlet riprap
  - Inlet flow spreader
  - Inlet ditch/channel
  - Forebay
  - Outlet structure(s) (manhole, catch basins)
  - Outlet pipe(s)
  - Outlet riprap
  - Outfall (ditch/channel,pipe, streambank)
- Underground vault, tank or manhole footprint lines:
  - Alignment and footprint of facility
- Underground vault, tank, or manhole features:
  - Inlet pipe(s)
  - Inlet structure (flow splitter)
  - Outlet pipes
  - Outlet riprap
  - Outfall (ditch/channel, pipe, streambank)
- Label the following plan view features:
  - Facility – within box label include:
    - Type of facility
    - Drainage facility identification number
    - Beginning and ending milepoint
  - Bubble label:
    - Facility inlet
    - Facility outlet
    - Inlet/outlet riprap
    - Flow spreaders
    - Flow splitter manholes
    - Flow control manholes
    - Outfall
  - Length of facility (in feet)
  - Travel lanes (e.g. northbound lanes, southbound lanes)
  - Highway number and name
  - Cross streets
  - Tributary creek, ditches, wetland areas
  - Upstream and downstream pipe sizes
  - Off ramps and on ramps
  - Nearby treatment or storage facilities:
    - Type of facility
    - Drainage facility identification number

- Cross-section locations
- Pavement drainage patterns
- Drainage pipe flow direction arrows
- Facility flow direction arrows

Include pond and swale profiles with the following information:

- Existing ground lines (top and bottom of facility)
- Existing ground hatch
- Inlet piping
- Inlet structure
- Inlet riprap
- Flow spreaders
- Outlet structure
- Outlet riprap
- Outfall (ditch/channel, pipe, streambank)
- Label the profile with the following information:
  - Facility – within box label include:
    - Type of facility
    - Drainage facility identification number
  - Pipe sizes
  - Length of facility (in feet)
  - Flow direction
  - Bubble label
    - Facility inlet
    - Facility outlet
    - Inlet/outlet riprap
    - Flow spreaders
    - Flow splitter manhole
    - Flow control manhole
    - Outfall

Include pond and swale cross-section with the following information:

- Existing ground lines (top and bottom of facility)
- Show location and orientation of porous pavers and underdrain when applicable
- Label the cross-section with the following information:
  - Top of facility
  - Bottom of facility
  - Width of facility
  - Porous pavers
  - Underdrain piping

Include detention pipe or vault or manhole structure schematic/profile with the following information:

- Facility footprint lines (length and height)
- Inlet piping
- Inlet structures (manholes, catch basins)
- Outlet piping
- Outlet structures (manholes, catch basins)
- Outlet riprap
- Outfall (ditch/channel, piping, streambank)
- Label:
  - Within the box label include:
    - Type of facility
    - Drainage facility identification number
  - Pipe sizes
  - Length of facility ( in feet)
  - Flow direction
  - Bubble label
    - Facility inlet
    - Facility outlet
    - Flow splitter manhole
    - Flow control manhole
    - Outfall
    - Outlet riprap

Include **plan view and cross-section details** of the facility's flow splitter structure and/or flow control structure with the following information:

- Inlet piping
- Outlet piping
- Diversion features (weirs)
- Flow control features (orifices)
- Orifice screening
- Label structure components:
  - Flow control weir
  - Flow control orifice(s)
  - Flow control orifice(s) screening
  - Riser pipe
  - Riser pipe overflow outlet
  - Pipe sizes

- Flow direction through structure (water quality flow direction, high flow/bypass flow direction)

**Appendix B:**

Include plans and details in Appendix B. Copies of the pertinent project construction plans from the ODOT V-Files are included for reference. The as-constructed versions of the plans are preferred if available. At least include:

- The cover/title sheet
- Water quality/detention plan sheets and/or sometimes designs on included on the roadway plans
- Other details

An example of applicable plan sheets is shown in Chapter 4, Appendix D.

**Appendix C:**

Include Proprietary or Special design structure maintenance instructions, drawings, and plans in Appendix C.

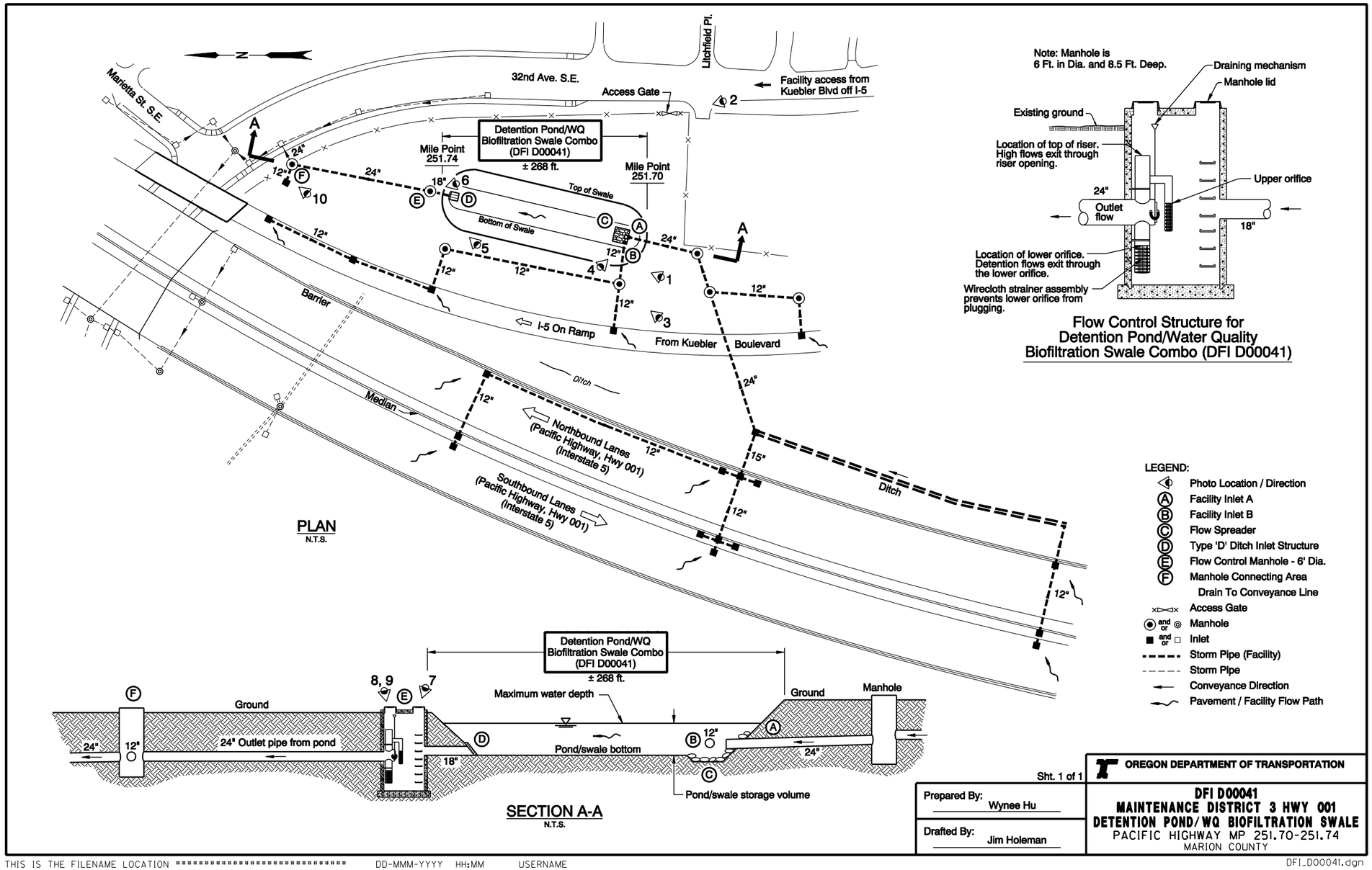



**Figure 4-5 Example O&M Operational Plan**

# APPENDIX A – PRELIMINARY HYDRAULIC RECOMMENDATIONS EXAMPLE

Preliminary Hydraulic Recommendations for Fish Passage Culverts
Peterson Tunnel – Turner Creek Section
Florence – Eugene Highway (OR #62)
Lane County
By _____(name)_____
27 November 2002

Preliminary hydraulic recommendations for the fish passage culverts and embankment protection are presented in this report. The project team, concerned regulators, and other interested parties should review these recommendations. These recommendations will be revised, as needed, based on the review comments. Revised recommendations will be issued if the changes are major. The detailed replacement structure designs will proceed after the recommendations have been approved by the project team. If you have comments or questions, please contact me at (503) 986-____.

All dimensions and quantities in these preliminary recommendations are based on limited survey data. They are rough estimates at best, and they may vary significantly from the final design.

**Fish Passage Culverts -** All fish passage culverts will be designed to have a simulated stream bottom and a service life equal to or in excess of 75 years. Box culverts are recommended for the two larger streams. We are certain boxes will work at these locations. Corrugated metal pipes may also be recommended if the detailed designs show they are feasible.

**Culvert Replacement at M.P. 20.55 -** This culvert is a few hundred feet east of the west end of the project. Currently it is a single 24-inch diameter concrete pipe and it blocks fish passage. During a recent site visit ODFW requested that we restore passage. This culvert replacement will not create fish passage into the entire upstream watershed. A cross-culvert under a private logging road several hundred feet upstream from the highway culvert prevents further fish passage. A replacement fish passage culvert recommendation is shown in Drawing 1. Estimated fish passage characteristics are as follows:

- upstream watershed area is 0.055 square miles,
- an 8-foot diameter 100-foot long aluminum or aluminized steel corrugated metal pipe with mitered ends having reinforced concrete collars,
- the pipe invert is set at a flatter grade than the upstream channel,
- the pipe spans the active flow channel - the active flow channel width varies from 6 to 7 feet and the pipe span is about 7.5 feet at the streambed, and
- the invert would be buried 4 feet into the stream bottom - this allows for at least two feet of cover after an anticipated two feet degradation of the channel bottom.

The pipe would have a simulated natural stream bottom as follows:

- ODOT Class 700 loose riprap placed in a layer over the invert,
- ODOT Class 50 loose riprap placed in a layer over the coarse rock, and
- a layer of natural stream bed material placed over the Class 50 riprap. The natural bed material is expected to be continually washed out of the culvert and be replaced by natural bed material from upstream.

The channel bottom would need to be lowered about 4 feet at the inlet. A roughened channel would be needed upstream from the new culvert to prevent headcuts from undermining the culvert under the private road. Rocks and logs would be used for the roughened channel. It is estimated 50 to 100 feet of channel would need roughening.

We recommend this culvert be replaced with relatively small pipe sized to pass streamflow but not designed to pass fish. We recommend mitigation for this by installing a fish passage culvert just outside the project limits at M.P. 21.3.

**Culvert Replacement at M.P. 20.85 -** This culvert is in the middle of the project. Currently it is a pair of 24-inch diameter concrete pipes. High velocity discharge from these pipes has scoured soils and rock from around the outlets, as shown in Figure 1. This scour and the high velocities in the pipe block fish passage. The recommended replacement culvert is shown in Drawing 2. Estimated fish passage characteristics are as follows:

- 
- upstream watershed area is 0.157 square miles,
- an 8-foot span by 6-foot rise by 96-foot long box culvert with standard ODOT headwalls, wingwalls, and aprons,
- the box invert is set on a flatter slope than the natural stream channel,
- the box spans the active flow channel - the active flow channel width varies from 6 to 8 feet and the box span is 8 feet, and
- the invert would be buried 2 feet into the stream bottom - this provides 2 feet of cover over the invert. Little or no future channel degradation is predicted.

The box would have a simulated natural stream bottom as follows:

- ODOT Class 700 loose riprap placed in a layer over the invert, and
- a layer of natural stream bed material over the Class 700 riprap. The natural bed material will be continually washed out of the culvert and be replaced during floods.

The channel bottom would need to be lowered about 3 feet at the inlet. An estimated 50-foot long transition section will be needed between the culvert inlet and the natural bottom of the existing channel. This transition section can be excavated in the natural bed materials. It is not necessary to make a roughened channel section using rocks and logs.

**Culvert Replacement at M.P. 21.3 -** This culvert is located outside of the project area to the east. Currently it is a pair of 24-inch diameter concrete pipes. These pipes are undersized. ODOT maintenance has reported flooding at this location caused by excessive headwater upstream from the culverts. In addition, the high velocity discharge from these pipes has scoured soils and rock from around the outlets. Figure 2 shows the outlets and Figure 3 shows the side of the scour hole. These pipes block fish passage and they are listed as "High Priority" on the ODFW culvert replacement program. ODOT maintenance personnel have seen several different species of trout and salmon at the outlet of these pipes during floods. The recommended replacement culvert is shown in Drawing 3. Estimated fish passage characteristics are as follows:

- upstream watershed area is 0.240 square miles,
- a 10-foot span by 10-foot rise by 93-foot long box culvert with standard ODOT headwalls, wingwalls, and aprons,
- the box invert and stream grade would be the same,
- the box spans the active flow channel - the active flow channel width varies from 7 to 9 feet and the box span is 10 feet, and
- the invert would be buried 4 feet into the stream bottom - this allows for at least two feet of cover over the invert after up to 2 feet of predicted future channel degradation.

The box would have a simulated natural stream bottom as follows:

- sediment retention plates attached to the culvert bottom,
- ODOT Class 700 loose riprap placed in a layer over the invert and around the plates,
- ODOT Class 50 loose riprap placed in a layer over the coarse rock, and
- a layer of natural stream bed material over the Class 50 riprap. The natural bed material is expected to continually wash out of the culvert and be replaced by natural bed from upstream.

We recommend these pipes be replaced with a fish passage culvert as mitigation. The mitigation would be for replacing the culvert at M.P. 20.55 with a smaller hydraulically adequate culvert rather than a fish passage culvert.

**Bank Protection -** Roadway embankment protection will be needed where Turner Creek flows alongside the roadway. Currently the creek is undercutting the embankment at these places and it will eventually damage the highway. Figure 4 shows the undercutting as seen from the road shoulder. The exact location of this embankment protection will be determined by the geotechnical or foundation engineer. The bank protection recommendations are shown in Drawing 4.

The proposed embankment protection is made from boulders rather than riprap. The relatively large interstitial spaces between the boulders will be filled with topsoil and planted. Plants similar to those growing in the area will be used. Willows will not be used. They do not grow in the shady conditions at this site.

**Temporary Water Management -** It is anticipated the fish passage culverts will be installed when the streams are dry. This is estimated to be from July 15 through October 30. No workplace isolation or stream diversion measures are anticipated.

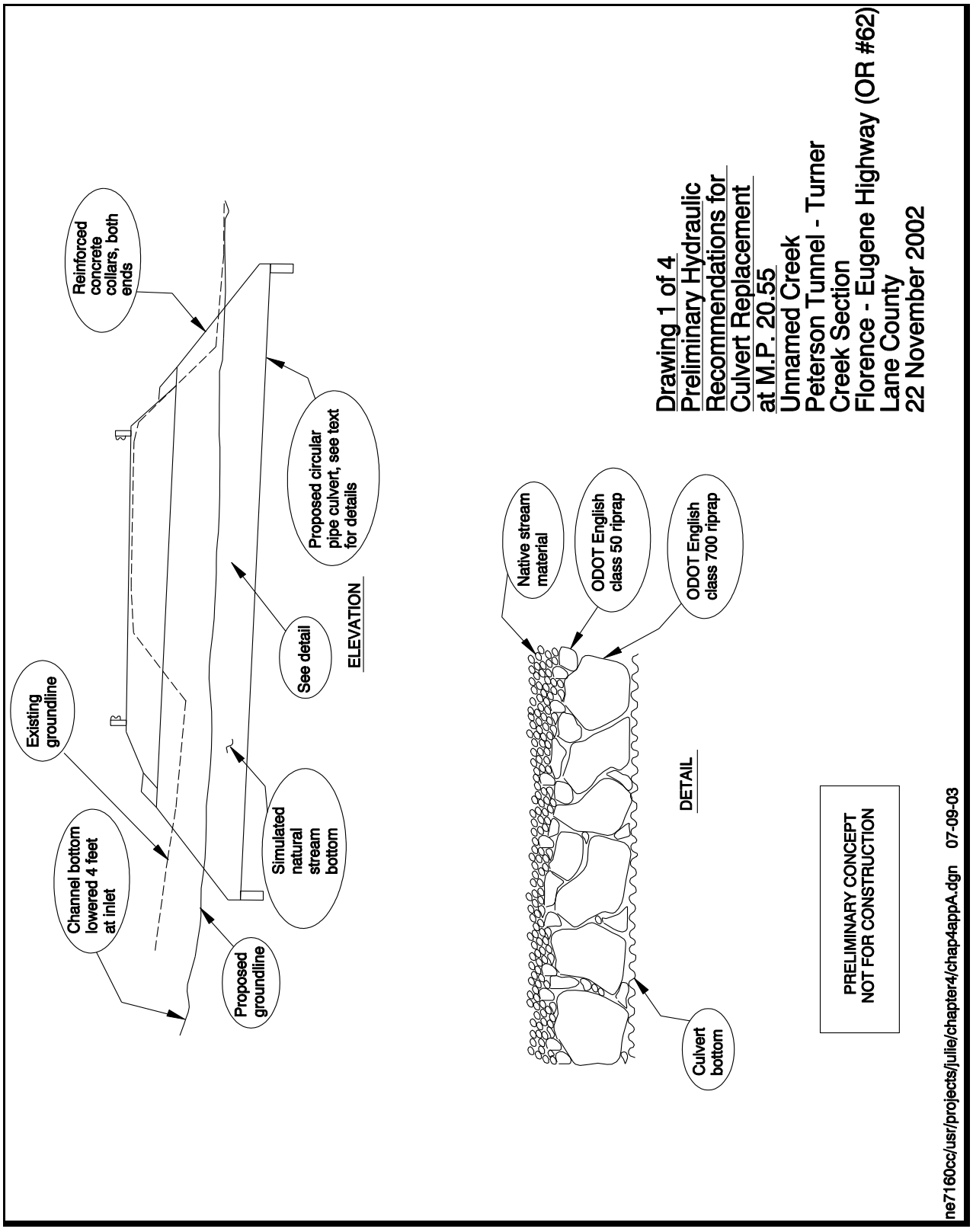
It is also expected there will be little or no flow in Turner Creek when the embankment protection is constructed. It is anticipated the creek will be dammed upstream from the construction area and either pumped past the construction site or allowed to flow through the site in a small culvert. Fish passage would be blocked in both directions if a pump is used. Upstream fish passage would be blocked if a bypass pipe is installed.

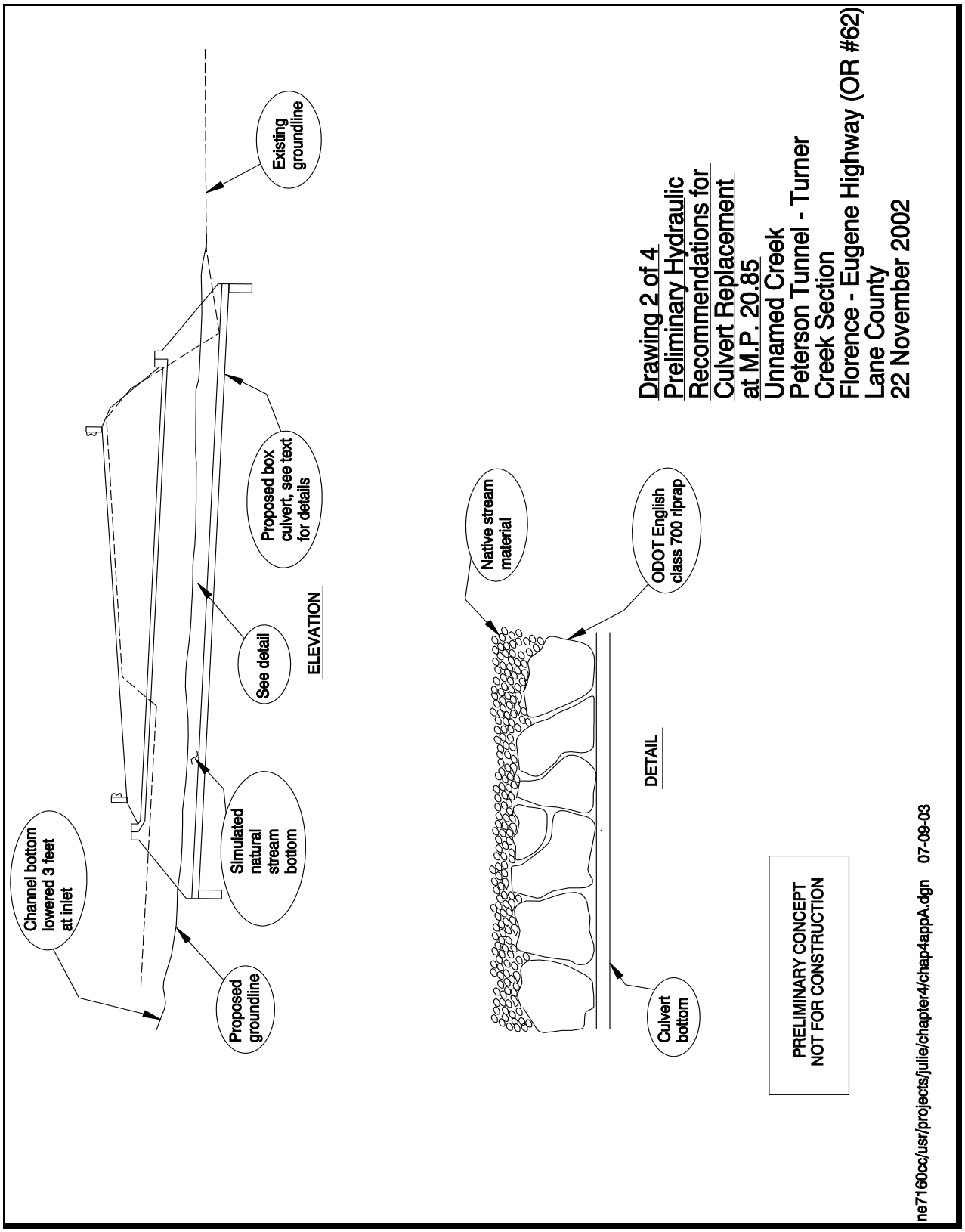
**FIGURE 1**

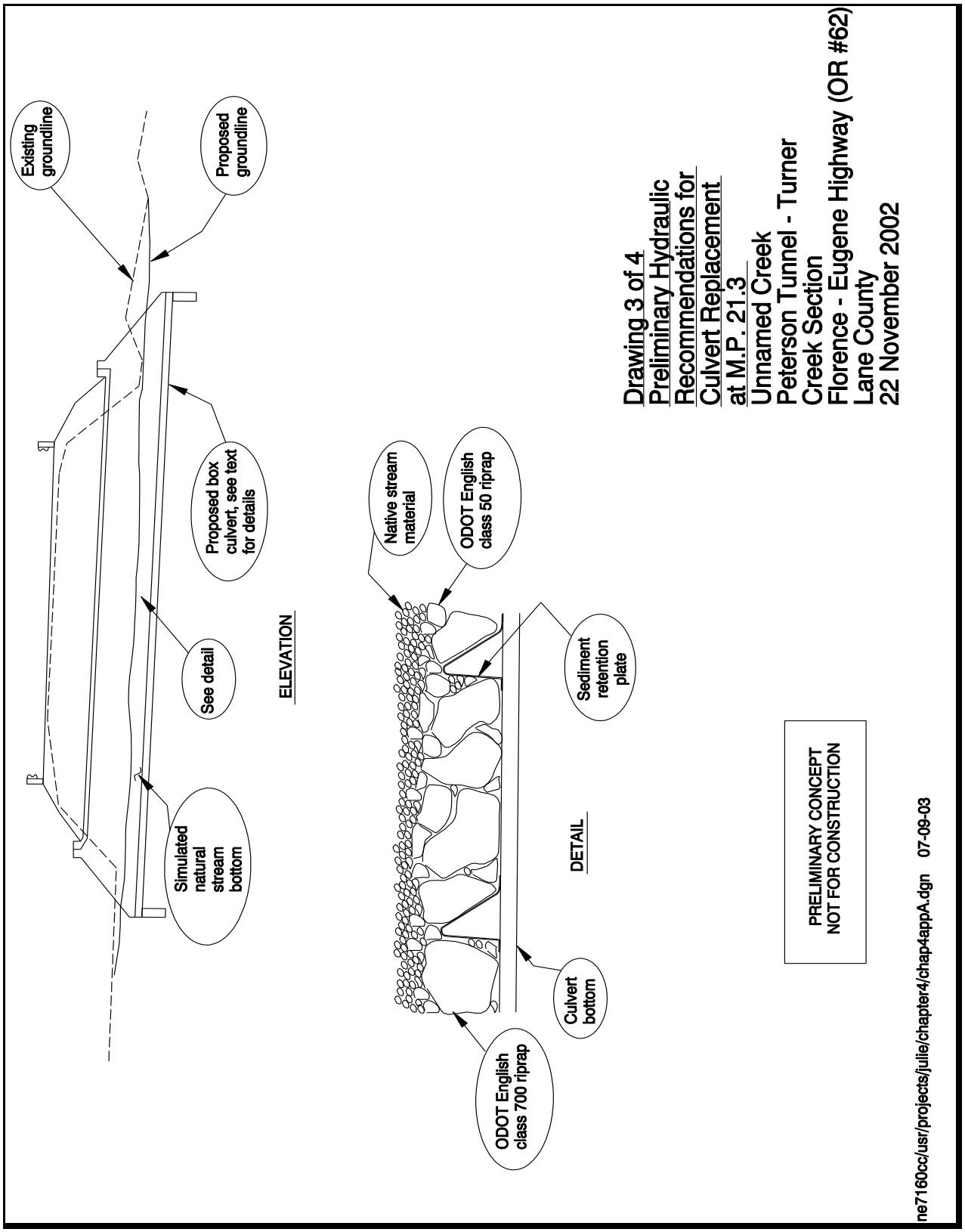
**FIGURE 2**

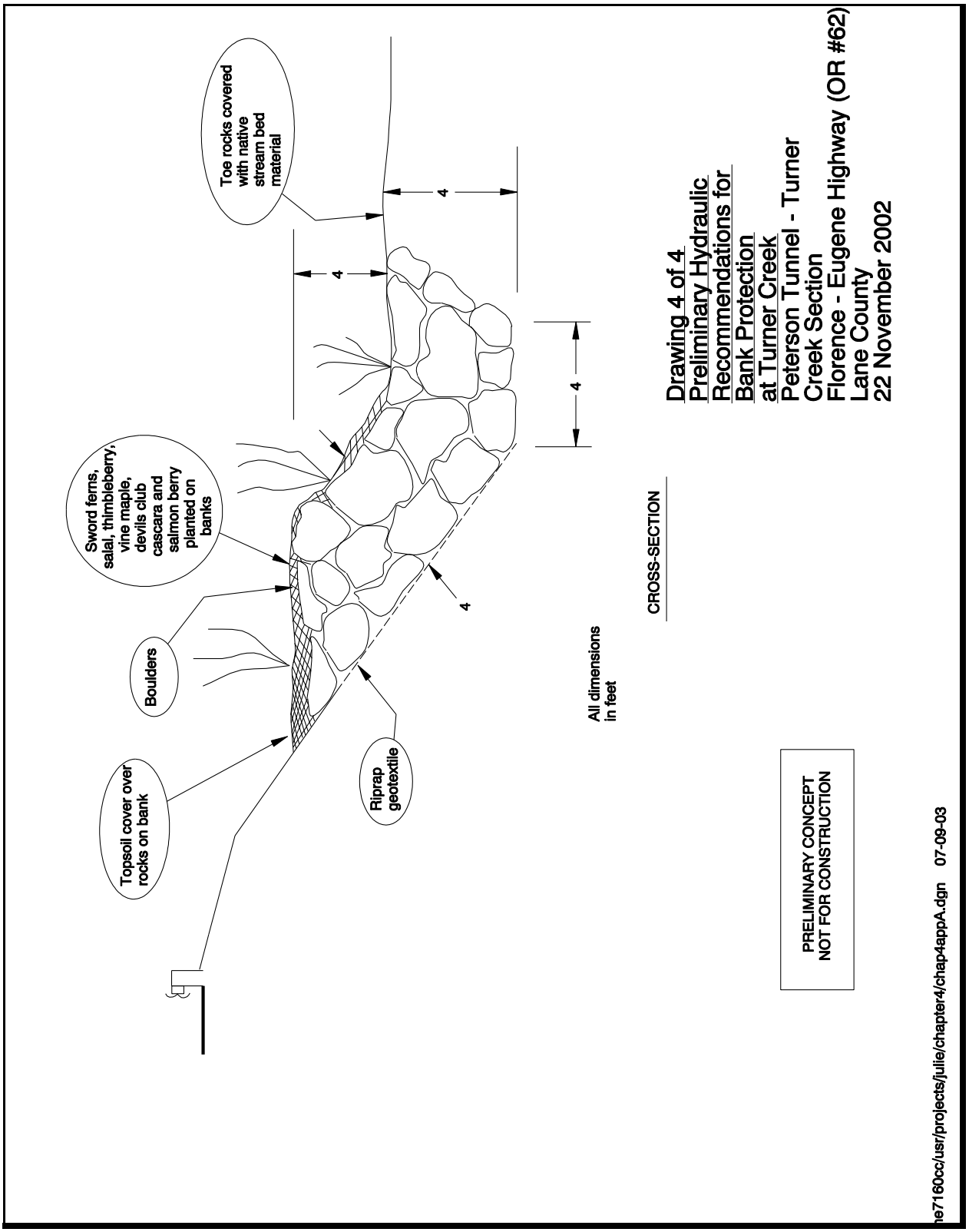
**FIGURE 3**

**FIGURE 4**


Reinforced concrete collars, both ends
Existing groundline
Channel bottom lowered 4 feet at inlet
Simulated natural stream bottom
Proposed groundline
See detail
Proposed circular pipe culvert, see text for details
ELEVATION
Native stream material
ODOT English class 50 riprap
ODOT English class 700 riprap
Culvert bottom
DETAIL
Drawing 1 of 4
Preliminary Hydraulic Recommendations for Culvert Replacement at M.P. 20.55
Unnamed Creek
Peterson Tunnel - Turner Creek Section
Florence - Eugene Highway (OR #62)
Lane County
22 November 2002
PRELIMINARY CONCEPT
NOT FOR CONSTRUCTION
ne7160cc/usr/projects/julie/chapter4/chap4appA.dgn 07-09-03


Drawing 2 of 4
Preliminary Hydraulic
Recommendations for
Culvert Replacement
at M.P. 20.85
Unnamed Creek
Peterson Tunnel - Turner
Creek Section
Florence - Eugene Highway (OR #62)
Lane County
22 November 2002
Existing groundline
Proposed box culvert, see text for details
See detail
ELEVATION
Channel bottom lowered 3 feet at inlet
Simulated natural stream bottom
Proposed groundline
Native stream material
ODOT English class 700 riprap
DETAIL
Culvert bottom
PRELIMINARY CONCEPT
NOT FOR CONSTRUCTION
ne7160cc/usr/projects/julie/chapter4/chap4appA.dgn 07-09-03


Existing groundline
Proposed groundline
Proposed box culvert, see text for details
See detail
Simulated natural stream bottom
ELEVATION
Native stream material
ODOT English class 50 riprap
Sediment retention plate
DETAIL
Culvert bottom
ODOT English class 700 riprap
Drawing 3 of 4
Preliminary Hydraulic Recommendations for Culvert Replacement at M.P. 21.3
Unnamed Creek
Peterson Tunnel - Turner Creek Section
Florence - Eugene Highway (OR #62)
Lane County
22 November 2002
PRELIMINARY CONCEPT
NOT FOR CONSTRUCTION
ne7160cc/usr/projects/julie/chapter4/chap4appA.dgn 07-09-03


Toe rocks covered with native stream bed material
Sword ferns, salal, thimbleberry, vine maple, devils club cascara and salmon berry planted on banks
Boulders
Topsoil cover over rocks on bank
Riprap geotextile
4
4
4
4
All dimensions in feet
CROSS-SECTION
Drawing 4 of 4
Preliminary Hydraulic Recommendations for Bank Protection at Turner Creek
Peterson Tunnel - Turner Creek Section
Florence - Eugene Highway (OR #62)
Lane County
22 November 2002
PRELIMINARY CONCEPT
NOT FOR CONSTRUCTION
ne7160cc/usr/projects/julie/chapter4/chap4appA.dgn 07-09-03

# APPENDIX B
# HYDRAULICS REPORT EXAMPLE

# Hydraulics Report

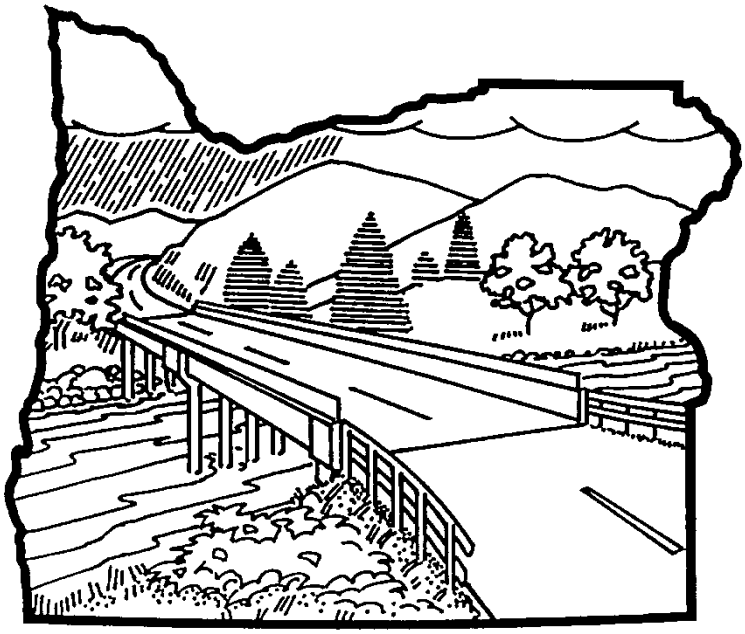
***For District 4 Drainage & Operations Repairs In Support Of:***

## Dry Creek Culvert Replacement

**Pacific Highway West 1W (OR091), MP 88.20**

**BENTON COUNTY**

**Published xxxxx xx, 20XX**
**Bid xxxx, 20XX**
**EA PE001234-567**
**Key # 12345**



**Prepared By:** XXXX
Hydraulic Designer
**Reviewed By:** XXXX
Hydraulic Engineer

(Seal Affixed Here)

**Oregon Department of Transportation**

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# HYDRAULICS REPORT SYNOPSIS

**DRY CREEK CULVERT REPLACEMENT, BRIDGE #19590**

**Benton County**

## PROJECT TYPE

| | | | |
|---|---|---|---|
| ☒ | Bridge | ☒ | Culvert |
| ☐ | Replacement | ☒ | Fish Passage |
| ☐ | Modernization | ☐ | Widening |
| ☐ | Water Quality | ☐ | Water Quantity |

## HYDROLOGY

| | | | |
|---|---|---|---|
| ☐ | Stream Gage Data | ☒ | U.S.G.S. Regression Equa. |
| ☐ | FEMA - FIS | ☐ | Historical Floods |
| ☐ | Parol Evidence | ☐ | Rational Formula |
| ☐ | SCS TR-20 | ☐ | King County Method |

## HYDRAULIC MODEL

| | | | |
|---|---|---|---|
| ☒ | HEC-RAS 4.0 | ☐ | Culvert Master |
| ☐ | HydroCAD | ☐ | StormCAD |

## CONCERNS

| Yes | No | Concern |
|---|---|---|
| ☐ | ☒ | Ice has been a problem. |
| ☐ | ☒ | Debris has been a problem |
| ☐ | ☒ | Erosion has been a problem |
| ☐ | ☒ | Scour has been a problem. |
| ☐ | ☒ | Channel aggradation/degradation has been a problem. |
| ☐ | ☒ | Roadway overtopping has occurred. |
| ☒ | ☐ | Fish passage required. |
| ☐ | ☒ | Threatened and Endangered species on this project. |
| ☒ | ☐ | Flood plain development regulations. |
| ☐ | ☒ | “No rise” in 100 year floodplain regulation. |
| ☐ | ☒ | Navigational clearance is required |

## Introduction

Currently, there are two 4 ft. diameter concrete culverts approximately 98 ft. long with headwalls on both the upstream and downstream ends. Several years ago, part of the east (downstream) slope of the highway failed and blocked off the outlet of the south culvert. Maintenance crews constructed a repair by attaching a CMP extension to the north culvert, constructing a stone embankment and using salvaged concrete bridge rail as a retaining wall. The repair constructed by the maintenance crew is only a temporary solution and the slope on the east side of the highway is unstable and inadequately supports the guardrail and the roadway.

The existing single, smooth barrel culvert is hydraulically inadequate and it blocks upstream fish passage. Due to the small diameter of the existing culvert, the velocity in the culvert can be too high for fish passage during almost all high flow events. Additionally, during lower flow events, there is a small jump between the culvert outlet and the scour pool at the outlet.

Since the existing structure is a pair of culverts each less than 6 feet in diameter, there is no existing bridge number. The ODOT bridge number for the new structure is 19590.

All elevations in this report are based on the project datum, the NGVD 1929.

On Hwy 99W at Airport Road, the 2000 ADT is 6300. The DHV is 0.15 times the ADT; therefore the DHV is 945. Since the DHV is greater than 100, therefore the storm design recurrence interval is 50 years (ODOT Hydraulics Manual).

## Regulatory Requirements and Data for Permits

**Floodplain Development Regulations**

The culvert is in Zone A, "Area of Special Flood Hazard" as designated by the Flood Insurance Rate Maps produced by the Federal Emergency Management Agency (FEMA). The culvert is subject to Benton County floodplain restrictions.

The county floodplain development regulations require that adequate protection (either rock armoring or vegetative cover) be provided to fill slopes exposed to the 100-year flood depending on the velocity of the flood waters. The proposed design will meet these requirements.

Benton Country development codes require that the structure in the floodplain shall not increase the water surface elevation of the base flood more than 1 foot at any point. The proposed culvert design meets these criteria, as is evident upon examination of the base flood headwater elevations calculated for the existing and proposed culverts (see Table 1 and Table 2).

**Aquatic Species Passage and Riparian Habitat**

The culvert is designed to pass fish as per a request from the Oregon Department of Fish and Wildlife (ODFW). It meets the standards in the ODFW "Standards and Criteria for Stream Road Crossings" dated September 16, 1996. The ODFW requires data on fish passage and a description of fish passage design features when they review the project plans. The fish passage information below applies to the culvert described in this Hydraulic Report.

**Fish Passage Data**

Width of Active Channel: 12 ft.
Width of Culvert: 12ft. diameter circular steel pipe

Design Features Include:

1. The proposed culvert will span the active stream channel and incorporate streambed simulation.
2. The steel pipe will be placed on a 1.34% slope.
3. The steel pipe will be filled 4 ft deep with bed material. This will allow a natural channel bottom to form in the culvert barrel.
4. There will be a series of sediment retention baffles in the pipe. See Figure 2.

See Figures 1, 2, and 3 for details of the proposed culvert.

The instream work period for Dry Creek is July 1 to October 15 unless otherwise specified by the ODFW.

**Ordinary High Water**

The ordinary high water elevation is listed in the tables, and is based on a visual field inspection.

**Other Regulations and Requirements**

The culvert is an outfall for the airport/industrial area drainage basin per the South Corvallis Drainage Master Plan (KCM, 1998). The culvert has been checked to verify it will perform adequately during the published design discharge for the airport area basin. This is shown in Table 2.

Dry Creek is not recognized as a navigable waterway by the Division of State Lands pursuant to ORS 274, therefore, navigational clearance is not required.

## Hydrology Information

The drainage basin of Dry Creek is a relatively small area (4.15 sq. mi.) that includes the Corvallis Airport and the farmland surrounding the airport. The basin has flat topography and is located entirely on the floor of the Willamette Valley. Dry Creek empties into the Booneville Slough, which once was a main channel of the Willamette River, but is now a backwater area for the Willamette River.

The Dry Creek drainage basin lies partially inside the Corvallis Urban Growth Boundary. For calculation purposes, build-out conditions were assumed inside the UGB, which is zoned light industrial, intensive industrial, and Public-Institutional (airport). While approximately 57% of the basin is farmland, and will likely continue to remain so since it lies outside of the UGB, the major source of flooding will be runoff from impervious surfaces located in the industrial zones and the airport. To determine the flows during certain storm events, the NRCS TR-55 method was used. The following table summarizes the flows.

| Method | 2-year event (cfs) | 5-year event (cfs) | 10-year event (cfs) | 25-year event (cfs) | 50-year event (cfs) | 100-year event (cfs) | 500-year event (cfs) |
|---|---|---|---|---|---|---|---|
| NRCS TR-55 | 500 | 616 | 664 | 885 | 976 | 1025 | 1250* |
| South Corvallis Drainage Master Plan prepared by KCM, Inc. | N/A | N/A | N/A | N/A | 865 | 901 | |

**Table 1, Peak Flow Data**

*By extrapolation from data.

The South Corvallis Drainage Master Plan drew the Dry Creek watershed boundaries at the Urban Growth Boundary (UGB), even though areas outside of the UGB drain into the watershed. Consequently, the watershed area in the South Corvallis Drainage Master Plan is approximately 1000 acres smaller than the watershed determined by consulting a USGS quad map. Because of this discrepancy, the flow amounts for Dry Creek from the South Corvallis Drainage Master Plan were not considered when determining the flow of the 50-year event. The South Corvallis Drainage Master Plan was consulted for general information about land use in the Dry Creek watershed.

## Ice and Debris

The drainage basin for Dry Creek is located entirely on the Willamette Valley floor, therefore, ice and snow debris will be an unlikely occurrence. The current primary land use in the drainage basin is agriculture and public-institutional (airport). The future land use in the basin will include agriculture, public-institutional, light industrial and intensive industrial. Large debris, such as trees, is not anticipated given the current and future land use in the basin.

## Hydraulic Design

### Existing Culvert

The existing culverts are 4 ft. diameter, approximately 98 ft. long. The south culvert is blocked at the outlet. The culverts are being replaced because they are hydraulically inadequate and block upstream fish passage. The hydraulic performance of the existing culverts is summarized in Table 1.

### Proposed Culvert

The proposed culvert is a 12 ft. diameter steel pipe, approximately 117 ft. long, installed by using trenchless installation techniques. The hydraulic performance of the proposed culvert is summarized in Table 2, and the culvert is shown in Figure 1. The pipe will be installed at a 1.34% slope, which is the overall slope of the stream channel.

The proposed culvert shall have a 30 degree right hand forward skew to the centerline of the highway. Placing the proposed culvert at the skew angle will eliminate the current scour

problem in the stream banks that occurs at the outlet of the existing culverts. See the "Scour" section of this report.

The proposed culvert will also have headwalls and retaining walls for protection at scour critical areas. For approximately 2 pipe diameters (24 feet) upstream and downstream of the culvert openings, the retaining walls shall be constructed of a structurally sound material, not necessarily concrete. For example, Hilfiker welded wire walls are an acceptable alternative to concrete walls at this site. Beyond the two pipe diameter distance, the retaining walls can be more biotechnical in nature, for example, Geocell walls with plantings. See Figure 1.

## Revetment Design

At the inlet and outlet of the proposed culvert, there will be a headwall and retaining walls to restrain adjacent side slopes as previously discussed. See Figure 3 for details on the wall protection.

## Temporary Water Management

During the in-water construction period of July 1 to October 15, flows in Dry Creek will be bypassed around the work area. The conceptual plan for temporary water management (TWM) is to construct one or more temporary dams across the creek upstream from the work area. One or more pumps with fish screens will pump creek flows into a bypass pipe or hose that is run

through the existing culvert to an area in the existing channel downstream from the construction area. There, the water will be released back to Dry Creek. A temporary dam will be constructed if needed, on the downstream side of the construction area. The dam will prevent fish from entering the construction site. As a contingency plan for temporary water management, gravity flow via the existing culvert will remain an option during construction. Fish passage will be blocked throughout the construction period.

**DRY CREEK AT HWY 99W**

ESTIMATED DISCHARGES FOR
TEMPORARY WATER MANAGEMENT

| | AVERAGE DAILY DISCHARGE IN CUBIC FEET PER SECOND (GALLONS PER MINUTE) | | |
|---|---|---|---|
| **NOTE** | **1** | **2** | **3** |
| JULY | 3.1 (1,400) | 2.0 (900) | 1.4 (630) |
| AUGUST | 2.0 (900) | 1.1 (490) | 0.77 (350) |
| SEPTEMBER | 2.5 (1,100) | 1.3 (580) | 0.68 (310) |
| OCTOBER | 11.0 (5,000) | 3.3 (1,500) | 1.4 (630) |

1) Average Exceedance Discharge (Average daily discharge expected to be exceeded 2 days each month.)
2) Average Exceedance Discharge (Average daily discharge expected to be exceeded 8 days each month.)
3) Average Exceedance Discharge (Average daily discharge expected to be exceeded 16 days each month )

**In-Water work period extends from 1 July through 15 October**

Listed discharges are surface water from the upstream watershed. The estimated discharges are based on nearby gaged basins. Discharges in the subject watershed may differ.

**Table 2, Estimated Discharges for Temporary Water Management**

## Scour

There is an existing scour problem at the stream bank downstream of the existing culverts. Due to the alignment of the existing culverts, during high flow events the water exits the existing open culvert and hits the bank nearly perpendicularly. This has caused a large scour hole. Placing the proposed culvert at a 30 degree skew angle will eliminate further scour at the scour hole. In addition, downstream scour from the proposed culvert will be further reduced downstream because it does not constrict the flow like the existing culvert does.

The proposed culvert will accommodate long-term profile changes, such as the future degradation of the channel bed.

## Detour Structure

Because the culvert is being replaced using trenchless construction methods, the highway is anticipated to remain open to traffic during construction and will eliminate the need for a detour structure. Hwy 99W is also the designated detour route for heavy haul trucks until three bridges on I-5 near Eugene are replaced.

Project: District 4 Drainage & OPS Repairs
Dry Creek Culvert
Pacific Highway West 1W (OR 091) at M.P. 88.20
Benton County
Key No. 12366

Compiled By: (name)
Date: 03/04/03

| Existing Culverts | HYDRAULIC DATA | | | |
|---|---|---|---|---|
| | DESIGN EVENT | BASE FLOOD | MAXIMUM PROBABLE FLOOD | ORDINARY HIGH WATER |
| Discharge (cfs) | 976[1] | 1025[2] | 1250[3] | |
| Recurrence (yrs) Interval | 50 | 100 | 500 | Annual |
| Highwater Elevation (feet) Of Natural Channel At Culvert Invert | 217.9 (228.1) | 218.2 (229.4) | 218.8 (232.6) | |
| Headwater Elevation (feet) At Culvert Inlet | 237.7 (237.9) | 237.7 (238.0) | 238.0 (238.3) | 215.5 |
| Backwater Depth (feet) At Culvert Inlet | 19.8 (9.8) | 19.5 (8.6) | 19.2 (5.7) | |
| Tailwater Elevation (feet) At Culvert Outlet | 217.4 (228.1) | 217.6 (229.3) | 218.3 (232.6) | 213.8 |
| Average Velocity (ft/s) At Culvert Outlet | 24.3 (17.3) | 24.9 (16.3) | 24.3 (13.1) | |

**Table 3, Hydraulic Data, Existing Culverts**

The existing culverts are 98 feet long by 4 foot diameter concrete pipe with headwalls on both the upstream and downstream sides. Culvert data assumes that both barrels are operating as designed. Currently, one barrel (the south barrel) is plugged.

Elevations and flow velocities not in parenthesis represent peak flows in Dry Creek with low water elevations in the Booneville Channel (a backwater of the Willamette River). Data in parenthesis represent peak flows in both Dry Creek and coincident highwater elevations in the Booneville Channel. For example, a 100-year discharge occurs in Dry Creek at the same time a 100-year flood elevation occurs in the Booneville Channel.

[1] 607 cfs flows through culvert and 369 cfs flows over the highway.
(434 cfs flows through the culvert and 542 cfs flows over the highway.)
[2] 625 cfs flows through the culvert and 400 cfs flows over the highway.
(409 cfs flows through the culvert and 616 cfs flows over the highway.)
[3] 611 cfs flows through the culvert and 638 cfs flows over the highway.
(328 cfs flows through the culvert and 922 cfs flows over the highway.)

Project: District 4 Drainage & OPS Repairs
Dry Creek Culvert # 19590
Pacific Highway West 1W (OR 091) at M.P. 88.20
Benton County

Compiled By: (name)
Date: 03/31/03

| Proposed Culvert | DESIGN EVENT | BASE FLOOD | MAXIMUM PROBABLE FLOOD | SOUTH CORVALLIS DRAINAGE MASTER PLAN |
|---|---|---|---|---|
| Discharge (cfs) | 976 | 1025 | 1250[1] | 865 |
| Recurrence (yrs) Interval | 50 | 100 | 500 | 50 |
| Highwater Elevation (feet) Of Natural Channel At Culvert Invert | 217.9 (228.1) | 218.2 (229.4) | 218.8 (232.6) | 217.5 |
| Headwater Elevation (feet) At Culvert Inlet | 218.8 (233.2) | 219.4 (234.6) | 222.5 (236.9) | 217.4 |
| Backwater Depth (feet) At Culvert Inlet | 0.9 (5.1) | 1.2 (5.2) | 3.7 (4.3) | 0.1 |
| Tailwater Elevation (feet) At Culvert Outlet | 213.9 (228.1) | 214.1 (229.3) | 214.6 (232.6) | 213.6 |
| Average Velocity (ft/s) At Culvert Outlet | 12.2 (12.2) | 12.8 (12.8) | 15.6 (11.8) | 10.8 |

**Table 4, Hydraulic Data, Proposed Culverts**

Notes:

Elevations and flow velocities not in parenthesis represent peak flows in Dry Creek with low water elevations in the Booneville Channel (a backwater of the Willamette River). Data in parenthesis represent peak flows in both Dry Creek and coincident highwater elevations in the Booneville Channel. For example, a 100-year discharge occurs in Dry Creek at the same time a 100-year flood elevation occurs in the Booneville Channel.

The proposed culvert is a 12 ft. diameter steel pipe, approximately 117 ft. long. The stream bed in the culvert shall be approximately 4 feet above the invert of the pipe, resulting in a cross sectional flow area of 80 $ft^2$. The proposed culvert will have a headwall and retaining wall on both the inlet and outlet sides.

Inlet
Pipe elevation – 204.41 ft.
Channel elevation – 208.4 ft.
Inlet coefficient – $K_e = 0.50$

Outlet
Pipe elevation – 203.09 ft.
Channel elevation – 207.1 ft.

The proposed culvert has a 30-degree right hand forward skew and will cross the centerline at station B 21+414.

[1] 1250 cfs flows through the culvert and zero cfs flows over the highway.
(947 cfs flows through the culvert and 303 cfs flows over the highway.)

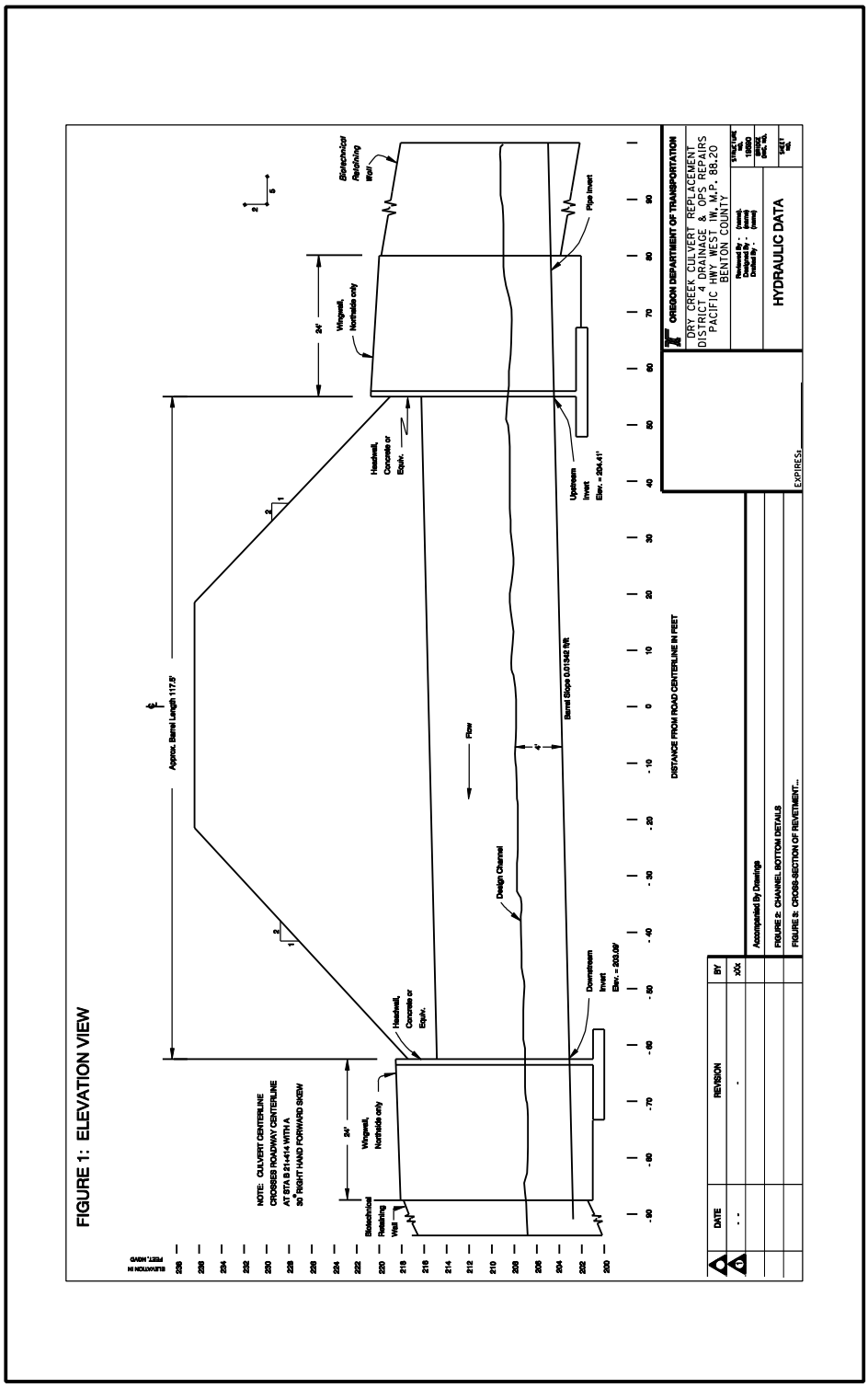
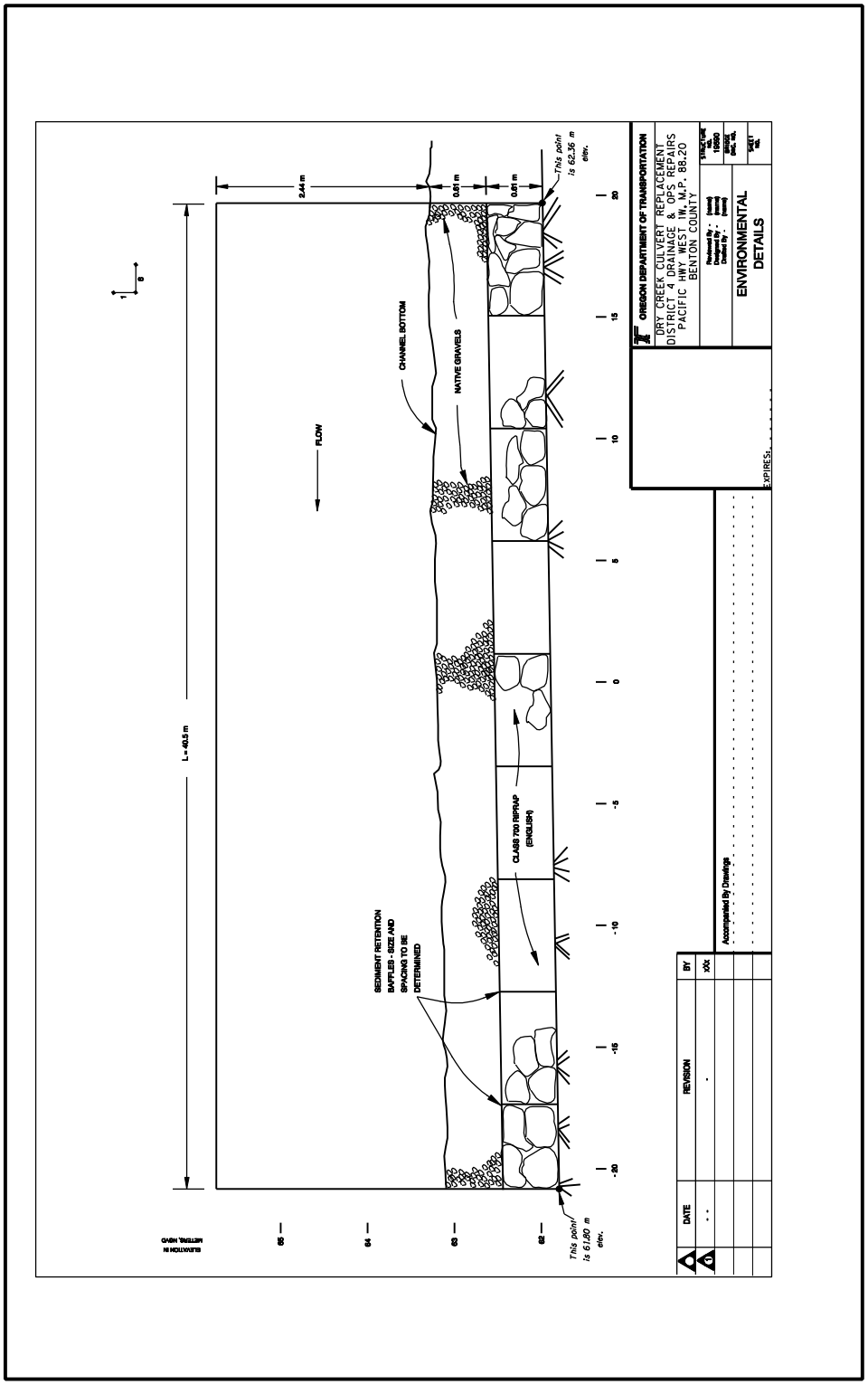


FIGURE 1: ELEVATION VIEW
OREGON DEPARTMENT OF TRANSPORTATION
DRY CREEK CULVERT REPLACEMENT
DISTRICT 4 DRAINAGE & OPS REPAIRS
PACIFIC HWY WEST (W, M.P. 88.20
BENTON COUNTY
HYDRAULIC DATA
DATE
REVISION
BY
Accompanied By Drawings
FIGURE 2: CHANNEL BOTTOM DETAILS
FIGURE 3: CROSS-SECTION OF REVETMENT...
EXPIRES:
NOTE: CULVERT CENTERLINE CROSSES ROADWAY CENTERLINE AT STA 8 51+44 WITH A 30° RIGHT HAND FORWARD SKEW
Approx. Barrel Length 117.5'
Wingwall, Northside only
Headwall, Concrete or Equiv.
Biotechnical Retaining Wall
Downstream Invert Elev. = 203.08'
Upstream Invert Elev. = 204.41'
Pipe Invert
Barrel Slope 0.01342 R/R
Design Channel
Flow
DISTANCE FROM ROAD CENTERLINE IN FEET
\\sc-salemfsv-1\hwye94g\ODOT_DATA\projects\Chapter 4\figures.dgn 3/12/2004 10:56:36 AM hwye94g


OREGON DEPARTMENT OF TRANSPORTATION
DRY CREEK CULVERT REPLACEMENT
DISTRICT 4 DRAINAGE & OPS REPAIRS
PACIFIC HWY WEST (1W), M.P. 88.20
BENTON COUNTY
ENVIRONMENTAL DETAILS
EXPIRES:
Accompanied By Drawings
DATE
REVISION
BY
L = 40.5 m
2.44 m
0.61 m
0.61 m
FLOW
CHANNEL BOTTOM
NATIVE GRAVELS
CLASS 700 RIPRAP (ENGLISH)
SEDIMENT RETENTION BAFFLES - SIZE AND SPACING TO BE DETERMINED
This point is 62.35 m elev.
This point is 61.80 m elev.
ELEVATION IN METRIC MSL
65
64
63
62
20
15
10
5
0
-5
-10
-15
-20
\\sc-salemrev-1\hwye94g\ODOT_DATA\projects\Chapter 4\figures.dgn 3/12/2004 10:52:26 AM hwye94g

# APPENDIX C – PAVEMENT DRAINAGE, STORM DRAINAGE, WATER QUALITY AND DETENTION BASIN DESIGN DOCUMENTATION FOR DEVELOPERS OR NON-ODOT PROJECTS

## Drainage Report

Drainage calculations, a drainage map, a site plan, construction drawings drawn to scale, and operation and maintenance manual must be submitted to the ODOT Region Technical Center for review and approval. Enough information should be included so the design can be independently verified.

## Drainage Calculations

The drainage calculations should be done using the same units as the drainage system plans, as much as practicable. This will help assure a timely review. The drainage calculations should contain the following information:

a. A narrative and tables as needed describing the characteristics of the contributing drainage basin prior to and after the proposed development, including, but not limited to, areas, soil types, vegetation, storage, runoff coefficients, and other runoff characteristics. A description of the changes to these characteristics due to the proposed development should also be included.
b. Calculations for the time-of-concentration, including slopes, lengths, hydraulic roughness, and other characteristics of the flow components.
c. A copy of the rainfall intensity-duration-recurrence interval curve.
d. The detention basin design calculations, printouts, and tables as per Chapter 12 if the simplified rational method is used, or the method described in Chapter 12 if the hydrograph and routing method is used. It should be clearly shown by calculation that the post-construction release rate from detention meets the requirements in Chapter 12.
e. Prepare pavement and storm drainage calculations, printouts and tables as per Chapter 13. Enough information should be submitted so the design can be independently verified.
f. Prepare water quality treatment calculations, printouts and tables as per Chapter 14 for improvements made with ODOT right-of-way. Enough information should be submitted so the design can be independently verified.

## Drainage Map, Site Plan, Construction Drawings and Operation and Maintenance Manual

The drainage map, site plan, construction drawings, and Operation and Maintenance Manual should include but not be limited to the following information:

a. The drainage basin and sub-basin boundaries with runoff coefficients and soil types if NRCS based methods are used (include on drainage map).
b. The flow path from the most remote point in the basin that is used to determine the time of concentration (include on drainage map).
c. The flow paths throughout the site, including directions of flows, points of concentration, and junctions (include on drainage map).
d. Buildings, landscaped areas, and impervious areas such as parking lots and sidewalks (include on drainage map, site plan, and construction drawings).
e. Contours of the site prior to development (include on drainage map and construction drawings).
f. Contours of the site after development (include on drainage map and construction drawings).
g. Details of the existing and proposed drainage systems including flow line elevations, sizes, materials, lengths, headwater depths for all pipes and ditches, and locations and rim elevations of all inlets and manholes (include on drainage map and construction drawings).
h. Details of the proposed system that includes the dimensions and bottom elevations of all facilities, and details of the primary and auxiliary outlets. The sizes, types, and elevations of all orifices should be shown (include on drainage map, site plan, and construction drawings).
i. System Operation and Maintenance Summary should be provided. Reference Section 4.6.6.

# Review Submittal Check List

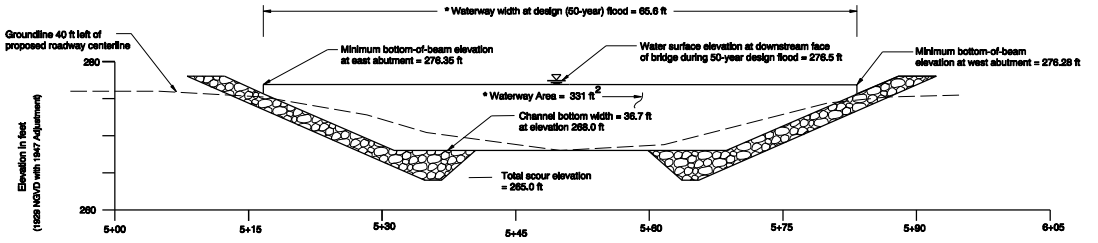
| | Reference Page or Appendix and/or Plan Sheet / Comments |
|---|---|
| ☐ **OVERVIEW** | |
| ☐ **Project Description** | |
| ☐ Location | |
| ☐ Highway name | |
| ☐ Highway number | |
| ☐ Highway Mile post | |
| ☐ County | |
| ☐ **Purpose of Report** | |
| ☐ Pavement drainage | |
| ☐ Storm drainage | |
| ☐ Detention | |
| ☐ Water Quality (ODOT Property Only) | |
| ☐ **BACKGROUND** | |
| ☐ **Existing drainage system** | |
| ☐ **Proposed drainage system** | |
| ☐ **PAVEMENT DRAINAGE CALCULATIONS** | |
| ☐ **Design narrative** | |
| ☐ **Design storm** | |
| ☐ **Runoff coefficients** | |
| ☐ **Drainage basin map (proposed)** | |
| ☐ **Time of concentration** | |
| ☐ **Peak flow rate** | |
| ☐ **Spread width of stormwater flow in gutter calculations** | |
| ☐ **Inlet capacity and spacing analysis** | |
| ☐ **STORM DRAINAGE CALCULATIONS** | |
| ☐ **Design narrative** | |
| ☐ **Design storm** | |
| ☐ **Runoff coefficients** | |
| ☐ **Drainage basin map (proposed)** | |
| ☐ **Time of concentration** | |
| ☐ **Peak flow rate** | |
| ☐ **Access structure analysis** | |
| ☐ **Pipe sizing analysis** | |
| ☐ **Hydraulic grade line analysis** | |
| ☐ **DETENTION CALCULATIONS** | |
| ☐ **Design narrative** | |
| ☐ **Design storm** | |
| ☐ **Runoff coefficients** | |
| ☐ **Drainage basin maps (existing and proposed)** | |
| ☐ **Time of concentration** | |
| ☐ **Pond storage volume (required and proposed)** | |
| ☐ **Peak flow rates (existing and proposed)** | |
| ☐ **Storm drain conveyance calculations (orifices, weirs, etc.)** | |
| ☐ **Outlet control structure release rates** | |
| ☐ **Auxiliary overflow capacity (100-year)** | |
| ☐ **Prepare calculations in units in plans** | |
| ☐ **System maintenance summary** | |

| | Reference Page or Appendix and/or Plan Sheet / Comments |
|---|---|
| ☐ **WATER QUALITY CALCULATIONS (ODOT PROPERTY ONLY)** | |
| ☐ **Design narrative** | |
| ☐ **Design storm** | |
| ☐ **Runoff coefficients** | |
| ☐ **Drainage basin map (delineate net new impervious area)** | |
| ☐ **Time of concentration** | |
| ☐ **Define water quality facility (pond, swale or water quality structure)** | |
| ☐ **Storm drain conveyance calculations** | |
| ☐ **Outlet control structure release rates (orifice, weirs, etc.)** | |
| ☐ **Pond storage volume (required and proposed)** | |
| ☐ **Peak flow rate (swale or water quality structure)** | |
| ☐ **Prepare calculations in units in plans** | |
| ☐ **Estimate when facility will be constructed** | |
| ☐ **System operation summary** | |
| ☐ **System maintenance summary** | |
| | |
| ☐ **DRAWINGS** | |
| ☐ **Overall Site plan** | |
| ☐ **Existing conditions plan** | |
| ☐ **Existing contours** | |
| ☐ **Existing drainage system** | |
| ☐ **Existing structures** | |
| ☐ **Existing waterways, wetlands, and ditches** | |
| ☐ **Proposed plan** | |
| ☐ **Proposed contours or spot elevations** | |
| ☐ **Proposed storm drainage system plan and details** | |
| ☐ **Proposed detention system plan and details** | |
| ☐ **Auxilliary outlet or overflow** | |
| ☐ **Screening provided to protect orifices** | |
| ☐ **Back-check calculations, plans and details for consistency** | |
| ☐ **Existing conditions (shaded)** | |
| ☐ **Landscape areas** | |
| ☐ **Impervious areas** | |
| ☐ **Maintenance access** | |
| ☐ **Outfall protection** | |

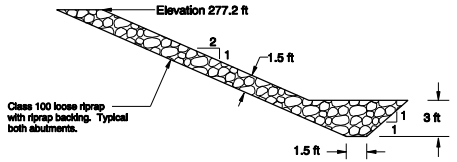
Appendix 4 – D Operation and Maintenance Manual Examples

- Water Quality Manhole
- Detention Tank
- Detention Pond
- Pond Swale Combo
- O&M Template Detention Facility
- O&M Template Water Quality Facility

# PLATE 4-1 EXAMPLE CROSS-SECTION OF PROPOSED BRIDGE




CROSS-SECTION VIEW OF DOWNSTREAM FACE OF BRIDGE




RIPRAP DETAIL

FIGURE 1: CROSS-SECTION OF PROPOSED BRIDGE

| | DATE | REVISION | BY |
|---|---|---|---|
| 1 | - - | - | xXx |
| | | | |
| | | | |
| | | | |

Accompanied By Drawings

OREGON DEPARTMENT OF TRANSPORTATION

Oak Creek Bridge
Pacific Highway - Lebanon (RR Tracks) M.P. 12.48
Corvallis - Lebanon Highway
Linn County

Reviewed By - (name)
Designed By - (name)
Drafted By - (name)

STRUCTURE NO. 18463
BRIDGE DWG. NO. ——
SHEET NO. ——

HYDRAULIC DATA

# PLATE 4-2: EXAMPLE CULVERT ELEVATION VIEW DRAWING

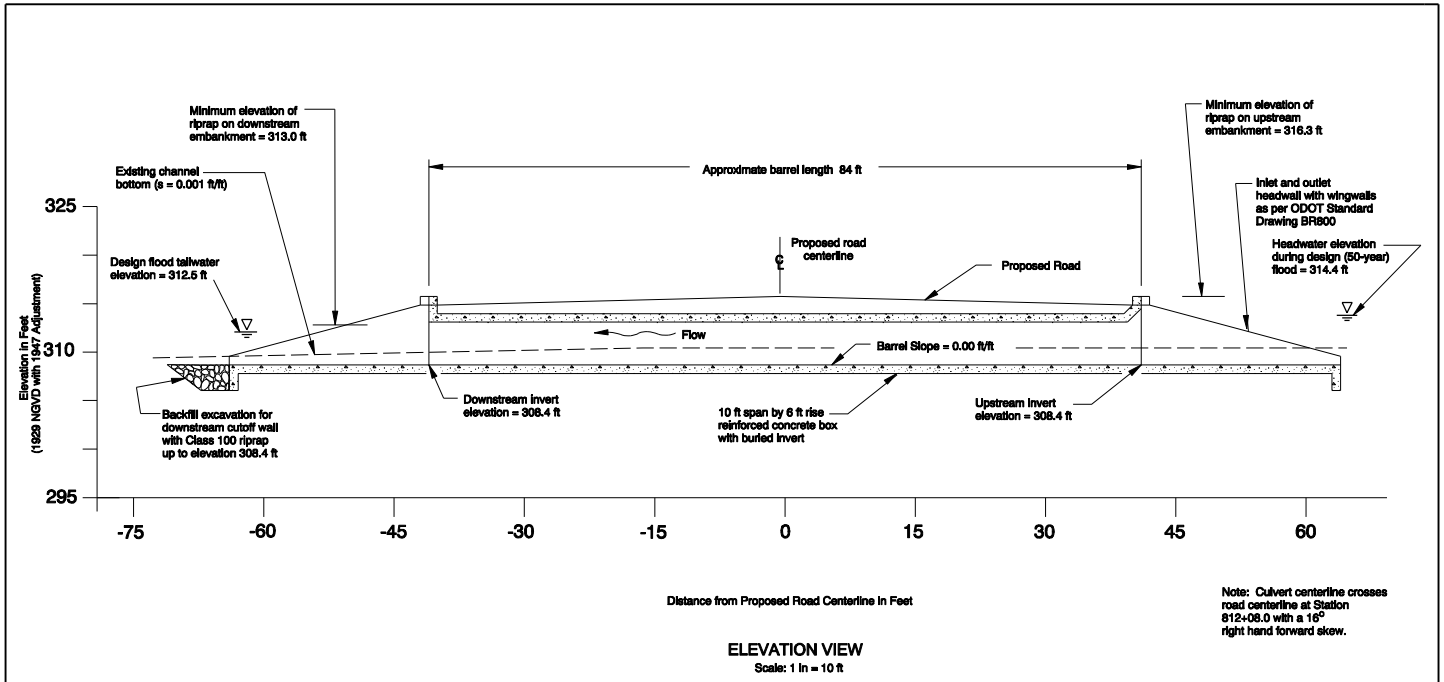



FIGURE 1: ELEVATION VIEW OF PROPOSED CROSS-CULVERT

| | DATE | REVISION | BY |
|---|---|---|---|
| 1 | - - | - | xXx |

Accompanied By Drawings

OREGON DEPARTMENT OF TRANSPORTATION

LITTLE OAK CREEK CULVERT
PACIFIC HIGHWAY - LEBANON (RR TRACKS) SECTION
CORVALLIS - LEBANON HIGHWAY, M.P. 15.40
LINN COUNTY

Reviewed By - (name)
Designed By - (name)
Drafted By - (name)

STRUCTURE NO. 18496
BRIDGE DWG. NO. -
SHEET NO. -

HYDRAULIC DATA

# CHAPTER 5

# PIPE MATERIALS

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## 5.1 Introduction

Many types of pipe materials are used in highway construction. The materials must be suitable for the site conditions so the facility will perform as intended. This chapter provides material selection guidance.

## 5.2 Policy and Practice

Agency policy for pipe material selection is to specify alternate materials where they can be used. Agency practice to implement this policy is in this chapter. This practice addresses most pipe applications. Additional material requirements or exceptions may occur for structures designed using ODOT Bridge Section policy and practice. Changes to alternate material practice and policy in the interim period between chapter updates will be addressed in ODOT Roadway Technical Bulletins. Guidelines and criteria for specific materials and applications are in the remainder of the chapter. Pipe material choices are based on all of the site conditions, and they must:

- satisfy project alternate pipe materials selection according to Title 23 Code of Federal Regulations (CFR) 635.411 (material or product selection)
- provide adequate hydraulic properties such as size and surface smoothness,
- withstand static forces caused by the weight of the pipe, the fluid in the pipe, and the weight of the fill over the pipe,
- withstand static and dynamic forces caused by wheel and track loads from traffic and construction equipment, pipe installation equipment and processes, pipe maintenance equipment and processes, and turbulent and unsteady flow within the pipe,
- withstand internal pressure to prevent the fluid from leaking out of the pipe into the surrounding bed materials,
- in the case of perforated or slotted pipe, to allow fluid from the surrounding bed material to enter the pipe,
- provide an adequate service life in relation to the design life of the facility,
- withstand corrosion caused by the fluids conveyed by the pipe,
- withstand corrosion caused by the surrounding soil if the pipe is in contact with the soil,
- withstand abrasion inside the pipe from solids carried by the flow,
- in certain applications, withstand combustion,
- provide desired fish passage characteristics, be non-toxic to the surrounding environment, and satisfy other environmental requirements,
- satisfy local government preferences, if required, and
- fulfill the need for experimental installations.

Each culvert, storm drain, or other installation shall be studied and suitable alternate materials allowed. The alternate materials must satisfy the requirements in the preceding list.

## 5.3 Pipe Material Abbreviations

There are many types of pipe materials, and the abbreviations for these names are listed in Table 5-1.

## 5.4 Definitions

Definitions and terms important to understanding pipe materials are listed in this section. Additional definitions are located in the manual glossary.

ASTM – Acronym for American Society for Testing and Materials (ASTM). ASTM is a non-profit organization that provides specifications covering the manufacture of pipes.

AWWA – Acronym for American Water Works Association. AWWA provides specifications covering the manufacture of pipes used in the water industry.

Dimension Ratio (DR) - Sidewall thickness to outside diameter ratio as defined in AWWA specifications.

Standard Dimension Ratio (SDR) - Sidewall thickness to outside diameter ratio as defined in ASTM specifications.

## 5.5 Pipe Materials

Numerous pipe materials are used in highway construction. More than one material can be used for most applications. The most commonly used pipe materials are discussed in this section. Additional requirements are in the Notes to Table 5-3.

### 5.5.1 Concrete Pipe

There are three types of concrete pipe allowed in the ODOT alternate materials policy; non-reinforced (NRCP), reinforced (RCP), or cast-in-place (CIPCP). Concrete pipes are used for culverts, storm drains, subsurface drains, and siphons. They are generally limited to non-pressure (gravity flow) applications.
NRCP and RCP are specified using AASHTO M 86 or M 170 classifications, respectively. The strength of the pipe increases as the class designation increases. Class 1 NRCP and Class I and II RCP lack structural strength and they are rarely used.

CIPCP is allowed as an alternate material. It can be cost effective at some locations. CIPCP and poured-in-place box culverts must be allowed to cure and reach adequate strength before they are subject to vibration or loading. In addition to strength related curing time considerations, CIPCP should be isolated from water in the surrounding habitat for at least 24 hours after placement. Allowance for curing time and isolation from surrounding water should be considered in the design.

CIPCP on Federal Aid projects must have steel reinforcement where subject to high fill heights or traffic live loads. This is also recommended design practice on other projects.

**Table 5-1 Pipe Material Abbreviations**

| | |
|---|---|
| CAP | Corrugated Aluminum Pipe |
| CAPA | Corrugated Aluminum Pipe-Arch* |
| CIPCP | Cast-in-Place Concrete Pipe |
| CPP | Corrugated Polypropylene Pipe |
| CPEP | Corrugated High Density Polyethylene Pipe |
| CSP | Corrugated Steel Pipe – Galvanized |
| CSP-Alzd. | Corrugated Steel Pipe - Aluminized |
| CSPA | Corrugated Steel Pipe-Arch – Galvanized* |
| CSPA-Alzd. | Corrugated Steel Pipe-Arch – Aluminized* |
| DIP | Ductile Iron Pipe |
| ISP | Iron or Steel Pipe - Galvanized |
| NRCP | Non-Reinforced Concrete Pipe |
| OHSR | Outward Helical Spiral Rib |
| PCAP | Perforated Corrugated Aluminum Pipe |
| PCP | Perforated Concrete Pipe |
| PCPEP | Perforated Corrugated High Density Polyethylene Pipe |
| PCSP | Perforated Corrugated Steel Pipe - Galvanized |
| PCSP-Alzd. | Perforated Corrugated Steel Pipe - Aluminized |
| PPVCP | Perforated Poly (Vinyl Chloride) Pipe |
| RCBC | Reinforced Concrete Box Culvert |
| RCP | Reinforced Concrete Pipe |
| SAP/OHSR | Smooth Aluminum Pipe with Outward Helical Spiral Rib |
| SSP/OHSR | Smooth Steel Pipe with Outward Helical Spiral Rib |
| SSP/OHSR-Alzd. | Smooth Steel Pipe with Outward Helical Spiral Rib - Aluminized |
| SWPEP-PR | Smooth Wall High Density Polyethylene Pipe - Pressure Rated |
| SWPVCP | Smooth Wall Polyvinyl Chloride Pipe |
| SWPVCP-PR | Smooth Wall Polyvinyl Chloride Pipe - Pressure Rated |
| SRPEP | Steel Reinforced High Density Polyethylene Pipe |

* also known as Arch-Pipe

### 5.5.2 Metal Pipe

Five types of corrugated metal pipe (CMP), are typically used in highway construction; iron or steel pipe (ISP), corrugated steel pipe (CSP), outward helically wound spiral rib pipe (OHSR), structural plate pipe, and ductile iron pipe (DIP).

ISP is typically small diameter galvanized pipe with threaded connections. It is often used for water distribution systems, irrigation systems, and small drains.

CMP is used for gravity flow (non-pressure) systems, such as culverts, storm drains, subsurface drains, slotted drains, and siphons. These pipes are available in several materials, such as aluminum, galvanized steel, or aluminized steel.

OHSR pipe is used for gravity flow culverts and storm drains. Unlike CMP, it has a smooth interior. This can provide added flow capacity in some applications. It is available in the same materials as CMP.

Structural plate pipe is composed of many corrugated metal plates that are bolted together. It is used for non-pressure applications such as culverts, equipment passes, and cattle passes. It is rarely used for other purposes. An advantage of structural plate is its ability to be transported in small sections or pieces to the jobsite and be bolted together in its final location. This can be very useful in areas with limited clearances for transport or construction. Structural plate pipes are available in the same materials as CMP.

DIP can be used for gravity or pressure flow applications. It has great strength and it is typically used where this strength is required. Pipes with minimal fill cover are the most common use in highway construction. DIP is rarely used for an entire drainage system because of its expense. It is used for the pipes where its strength properties are required, only. Less costly pipes are used for the remainder.

#### 5.5.2.1 CMP Wall Thicknesses

Corrugated metal pipe (CMP) wall thicknesses are specified to the nearest 0.001 inches. Standard wall thicknesses are listed in Table 5-2. Gage values are also listed. Gage was commonly used in the pipe industry, and it is listed on many older plans and specifications.

*Note: Not all pipe materials are available in all wall thicknesses. Thicknesses for common corrugated and smooth wall metal pipes are listed in the Fill Height Tables found in the Oregon Standard Drawings.*

**Table 5-2 Wall Thickness**

| Galvanized Iron and Steel | | Aluminum | |
|---|---|---|---|
| Wall Thickness in Inches | Gage | Wall Thickness in Inches | Gage |
| 0.064 | 16 | 0.060 | 16 |
| 0.079 | 14 | 0.075 | 14 |
| 0.109 | 12 | 0.105 | 12 |
| 0.138 | 10 | 0.135 | 10 |
| 0.168 | 8 | 0.164 | 8 |

Note: Dimensions applicable to uncoated or metallic coated pipes.

### 5.5.3 Plastic Pipe

Plastic pipes are available in a wide array of materials with many different properties. The most common types in highway construction are corrugated high density polyethylene pipe (CPEP), smooth (solid) wall polyethylene pipe (SWPEP), steel reinforced polyethylene pipe (SRPP), smooth inner wall poly (vinyl chloride) pipe (SWPVCP), and corrugated polypropylene (CPP) pipe.

#### 5.5.3.1 High Density Polyethylene Pipe (CPEP, SRPEP, and SWPEP)

CPEP is used for culverts and storm drains. It is suited for non-pressure applications. Perforated corrugated high density polyethylene pipe (PCPEP) is used for subsurface drains. The pipes are usually one of two configurations. Pipe conforming to AASHTO M 294 Type D specifications has a smooth inner liner, a smooth outer shell, and either spiral or helically wound ribs joining the liner and the shell. Pipe meeting AASHTO M 294 Type S has a smooth inner liner fused to a corrugated outer shell.

SRPEP is used for non-pressure (gravity flow) systems for storm drain and culvert systems with 1 foot or more of cover. The pipe and fittings must meet the requirements of:

- ASTM F 2562

SWPEP is used for non-pressure (gravity flow) systems for storm drain and culverts, as a pipe liner for culvert repair, and for irrigation systems. These are various diameter pipes capable of conveying pressure flow.

#### 5.5.3.2 Polyvinyl Chloride (PVC)

SWPVCP (inner smooth wall) is allowed as an alternate material for many applications. There are two main groups of SWPVCP; solid wall and profile wall.

SWPVCP is allowed for subsurface drain systems must meet the requirements of:

- ASTM D 2729 (4-inch perforated pipe is included in this specification)

SWPVCP is allowed for non-pressure (gravity flow) systems for sanitary sewer, storm drain, culvert, siphon, or irrigation systems with 2 feet or more of cover. The pipe and fittings must have a minimum pipe stiffness of 46 pounds per inch and meet the requirements of one of the following:

- ASTM D 3034 SDR 35
- ASTM F 679
- ASTM F 794 Series 46
- ASTM F 949 (46 psi stiffness)
- ASTM F 1760 Sewer Drain Series or Schedule 40 Series
- ASTM F 1803

Stronger SWPVCP is allowed for non-pressure sanitary sewer, storm drain, culvert, siphon, or irrigation systems with minimum cover depths of one foot or greater, but less than 2 feet. The pipe and fittings must meet the requirements of:

- ASTM D 3034 SDR 26
- ASTM F 794 Series 115
- ASTM F 949 (115 psi stiffness)
- AWWA C 900
- AWWA C 905 (maximum DR = 26)
  (DR = Sidewall thickness to inside diameter ratio as defined in AWWA specifications.)

SWPVCP is allowed for pressure flow systems with 2 feet or more of cover. The pipe and fittings must meet the requirements of:

- AWWA C 900
- AWWA C 905 (maximum DR = 32.5)

Stronger SWPVCP is allowed for pressure flow systems with a minimum cover depth of one foot or more, but less than 2 feet. The pipe and fittings must meet the requirements of:

- AWWA C 900
- AWWA C 905 (maximum DR = 26)

The pressure rating or pressure class selected will be dependent on the operating pressure expected in the pressure system. The maximum particle size shall not exceed ¾ inches for pressure pipe embedment material.

Use of stronger grades of SWPVCP may be justified for cover heights more than 40 feet. See SWPVCP Fill Height Table in Section 5.8. The stronger grades of SWPVCP are seldom used for an entire drainage system because of their cost. They are used for the pipes where their strength properties are needed, only. Less costly pipes are used for the remainder of the system.

#### 5.5.3.3 Polypropylene Pipe (CPP)

CPP is allowed as an alternate material. There are two main groups of CPP: double wall and triple wall.

Doublewall CPP is used for non-pressure (gravity flow) systems for sanitary sewer, storm drains, culverts, siphons, or irrigation systems with a minimum cover depth of one foot. The pipe must meet the requirements of:

- ASTM F 2736

Triplewall CPP is used for non-pressure (gravity flow) systems for sanitary sewers. The pipe must mee the requirements of:

- ASTM F 2764

## 5.6 Pipe Connections

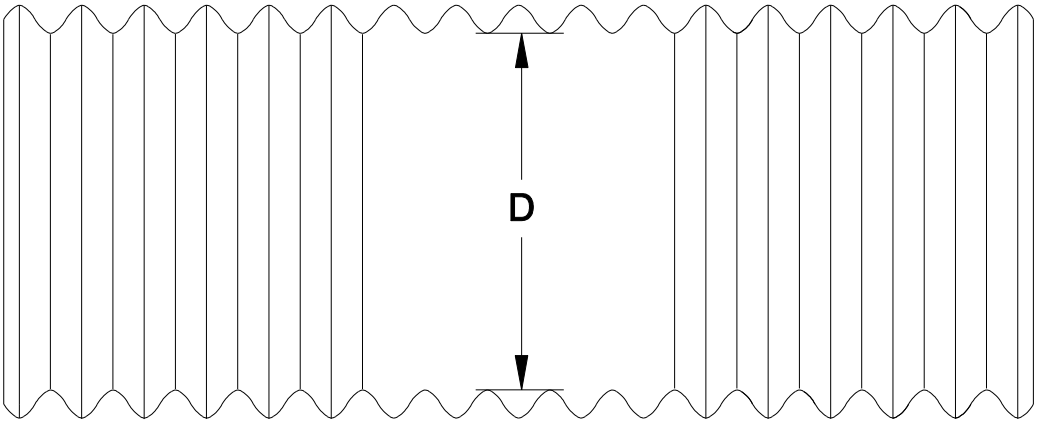
Pipe connections are an important aspect of hydraulic design. The connections must do an adequate job of keeping the fluids within the pipe and they must be functional as long or longer than the facility design life. General guidance is provided in this section. Detailed guidance is in the Oregon Standard Specifications for Construction, Oregon Standard Drawings, Oregon Special Provisions for Highway Construction, appropriate AASHTO, ASTM, and AWWA specifications, and the project plans.

**CMP Connections –** Corrugated metal pipes (CMP) or corrugated metal pipe-arches (CMPA) with either annular or helical corrugations are often used in highway applications. In order to design or select a connection, it is important to know the corrugation types on the pipes to be connected.

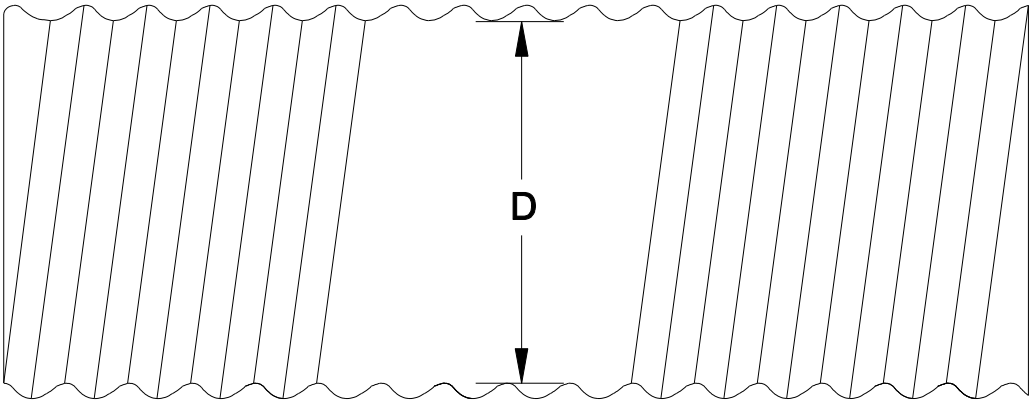
Annular corrugations are found on older CMP, and occasionally on new or replacement pipes. Annular corrugations are also found on the ends of many metal pipes with helical corrugations. Annular corrugations are perpendicular to the long axis of the pipe, as shown in Figure 5-1a. In other words, they wrap around the pipe in a manner similar to the segments on an earthworm or a millipede. Helical corrugations are found on new or replacement CMP and CMPA. Helical corrugations wrap around the pipe in a spiral manner similar to the threads of a screw or the coils of a spring, as shown in Figure 5-1b.

Coupling band connections can either be non-watertight or watertight. The non-watertight couplings are often called “dirt” or “dust” connections. ODOT requires watertight connections for siphons and other pressure systems, irrigation systems, sanitary sewers, and storm drains. The connections for siphons and sanitary sewers are pressure tested according to Oregon Standard Specifications for Construction or Oregon Special Provisions for Highway Construction. Designer discretion is used to decide if watertight joints are needed for other systems, such as culverts.

CMP coupling bands are shown in Oregon Standard Drawing RD326.




a) Annular Corrugations




b) Helical Corrugations

D = Pipe Diameter

**Figure 5-1 Annular and Helical Corrugations**

**Concrete Pipe Connections** – Concrete pipe connections are integral to the pipe. Several types are in common use, as shown in Figure 5-2. It is important to know the end and gasket type if a connection will be made to an existing pipe. The ends and gaskets of the new and old pipes should be compatible. Otherwise, a dissimilar pipe material joint is needed. Allowable methods

and materials for joining concrete pipes are described in the Oregon Standard Specifications for Construction.

Concrete pipe sections near culvert ends can separate if there is a loss of support from the underlying fill. Tie bars are often used to reinforce joints near the ends, as shown in Oregon Standard Drawing RD318.

**CPEP and SWPEP Connections** – Pipe connection requirements for corrugated and smooth wall high density polyethylene pipes are in American Society of Testing and Materials (ASTM) specifications and are to be an approved design and materials as per the current ODOT Qualified Products List.

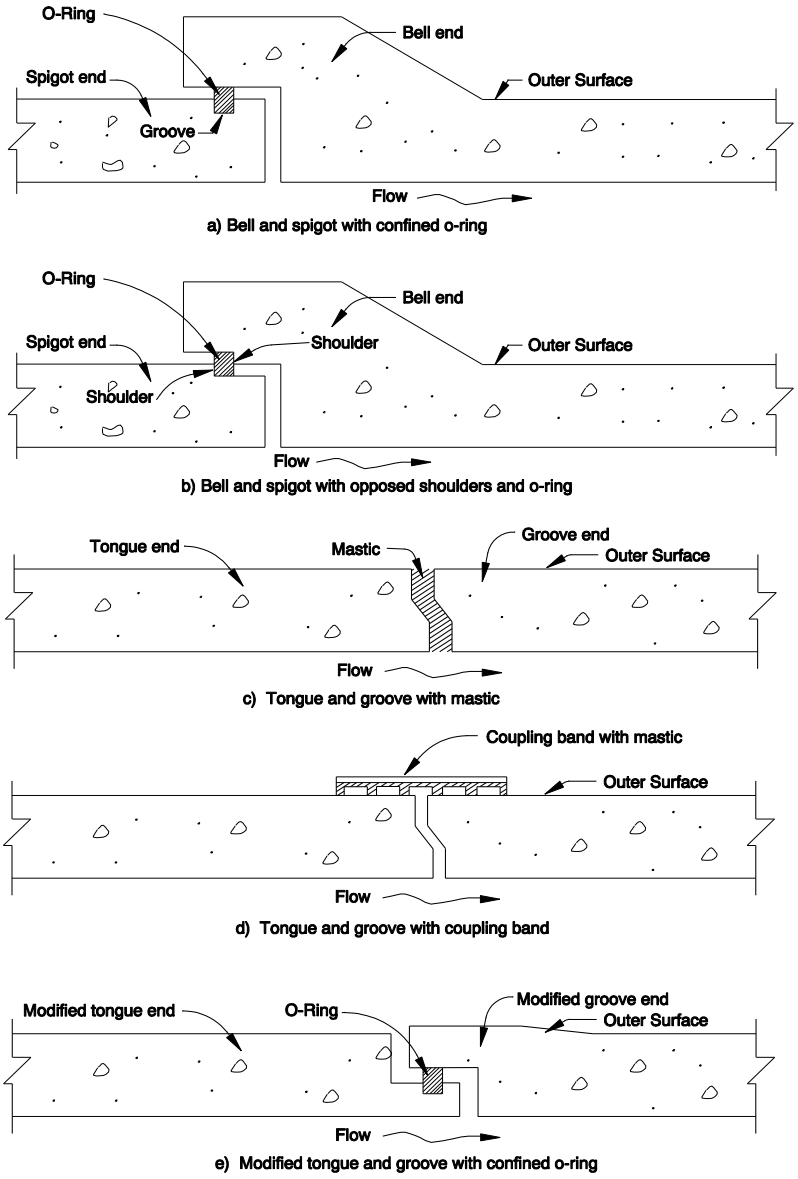
It is important to know the pipe corrugation type in order to design and select a connection. Type S high density polyethylene pipes have annular corrugations. Helical corrugations are not found on corrugated high density polyethylene pipe.

**SRPEP Connections** – Pipe connection requirements for corrugated and smooth wall high density polyethylene pipes are in American Society of Testing and Materials (ASTM) specifications and are to be an approved design and materials as per the current ODOT Qualified Products List.

**SWPVCP Connections –** Pipe connection requirements for poly (vinyl chloride) pipe with a smooth inner wall (SWPCCP) are in American Society of Testing and Materials (ASTM) or American Water Works Association (AWWA) specifications. The appropriate specification should be used for the selected pipe material. ODOT requires use of the stronger grades of SWPVCP. The fittings and connections for these pipes are also required to be of the same grade as the pipe, or stronger.

**CPP Connections** – Pipe connection requirements for corrugated and smooth wall high density polyethylene pipes are in American Society of Testing and Materials (ASTM) specifications and are to be an approved design and materials as per the current ODOT Qualified Products List.

**Dissimilar Materials or Damaged End Connections –** Dissimilar pipes such as metal and concrete, or others, can be connected using concrete closure collars, as described in the Oregon Standard Specifications. Reinforced concrete encasements can also be used as shown in Oregon Standard Drawing RD306. These connections can also connect new pipes to damaged ends on existing pipes. The damaged section is removed before the connection is made.




Notes: Flow directions shown. Many combinations of pipes and gaskets are used in addition to the examples shown. Concrete pipe joint methods and materials are listed in ODOT Standard Specifications.

**Figure 5-2 Concrete Pipe Connections**

## 5.7 Design Life

The design life is the length of time the pipe must provide adequate performance. The minimum design lives of pipes in various applications have been determined by the agency using factors

such as the importance of the installation, the inconvenience to the public if the facility is inoperable due to a pipe failure, the replacement costs, etc. Design lives are listed in Table 5-3.

Alternate pipe materials are also listed in Table 5-3. These materials have been approved for these applications provided they meet the applicable requirements listed in Section 5.2 and the requirements in the table notes. It is the designer's responsibility to evaluate each site and application and to determine the materials that will provide satisfactory performance.

*Note: This manual uses the term "storm drain" to describe a conduit used to convey storm runoff. The term "storm sewer" is used in other ODOT documents, such as bid item lists, standard specifications, standard drawings, etc. These terms can be considered to be interchangeable.*

**Table 5-3 Pipe Material Design Lives and Alternate Materials for Culverts, Storm Drains, Subsurface Drains, Slotted Drains, Irrigation Systems, and Siphons**

| Type of Installation | Location | Type of Facility | Design Life | Alternate Materials |
|---|---|---|---|---|
| Culverts (See Notes 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 13, 14, and 15) | Approach Roads and Outside of Travel Way and Shoulders or Curbs | All | 25 | CAP, CAPA, CIPCP, CPP, CPEP, CSP, CSP-Alzd., CSPA, CSPA-Alzd., DIP, NRCP, RCP, SAP/OHSR, SSP/OHSR, SSP/OHSR-Alzd., SWPVCP, SRPEP, SWPEP |
| | Cross-Culverts less than 72 inch span | Freeways, | 75 | CAP, CAPA, CIPCP, CPP,CPEP, CSP, CSP-Alzd., CSPA, CSPA-Alzd., DIP, NRCP, RCP, SAP/OHSR, SSP/OHSR, SSP/OHSR-Alzd., SWPVCP, SRPEP, SWPEP |
| | Cross-Culverts less than 72 inch span | Others | 50 | CAP, CAPA, CIPCP, CPP, CPEP, CSP, CSP-Alzd., CSPA, CSPA-Alzd., DIP, NRCP, RCP, SAP/OHSR, SSP/OHSR, SSP/OHSR-Alzd., SWPVCP, SWPEP |
| | Cross-Culverts more than or equal to 72 inch span | All | 75 | CAP, CAPA, CIPCP, CSP, CSP-Alzd., CSPA, CSPA-Alzd., RCBC, RCP, SAP/OHSR,SSP/OHSR, SSP/OHSR-Alzd. |
| Storm Drains (See Notes 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 1113, 14, and 15) | Within Travel Way and Shoulders or Between Curbs | All | 75 | CAP, CAPA, CIPCP, CPP, CPEP, CSP, CSP-Alzd., CSPA, CSPA-Alzd., DIP, NRCP, RCP, SAP/OHSR, SSP/OHSR, SSP/OHSR-Alzd., SWPVCP, SRPEP, SWPEP |
| | Outside of Travel Way And Shoulders or Curbs | All | 50 | CAP, CAPA, CIPCP, CPP, CPEP, CSP, CSP-Alzd., CSPA, CSPA-Alzd., DIP, NRCP, RCP, SAP/OSHR, SSP/OHSR, SSP/OHSR-Alzd., SWPVCP, SRPEP, SWPEP |

| Type of Installation | Location | Type of Facility | Design Life | Alternate Materials |
|---|---|---|---|---|
| Subsurface Drains (See Notes 1, 4, 5, 6, 7, 8, 9, and 11) | Longitudinal Outside of Travel Way | All | 25 | PCAP, PCP, PCSP, PCSP-Alzd., PPVCP, PCPEP |
| | Longitudinal or Cross Within Travel Way | Freeways, | 75 | PCAP, PCP, PCSP, PCSP-Alzd., PPVCP, SRPEP, PCPEP |
| | Longitudinal or Cross Within Travel Way | Other | 50 | PCAP, PCP, PCSP, PCSP-Alzd., PPVCP, SRPEP, PCPEP |
| Slotted Drains (See Notes 1, 3, 4, 5, 6, 7, and 12) | Within Travel Way and Shoulders or Between Curbs | All | 50 | CAP, CSP, CSP-Alzd. |
| | Outside of Travel Way and Shoulders or Curbs | All | 50 | CAP, CSP, CSP-Alzd. |
| Irrigation Systems (See Notes 1, 3, 5, 7, 8, and 14) | Within Travel Way and Shoulders or Between Curbs | Freeways | 75 | ISP, SWPEP-PR, SWPVCP-PR |
| | Within Travel Way and Shoulders or Between Curbs | Other | 50 | ISP, SWPEP-PR, SWPVCP-PR |
| | Outside of Travel Way and Shoulders or Curbs | All | 50 | ISP, SWPEP-PR, SWPVCP-PR |
| Siphons (See Notes 1, 3, 4, 5, 7, 10, 11, 13, 14 and 15) | Within ODOT Right-of-way | All | 75 | DIP, NRCP, RCP, SWPVCP, CPEP, SRPEP |

**Notes**

1. All alternate material selections must provide an adequate service life at the installation site. The service life must satisfy the applicable requirements listed in Section 5.2 of this chapter.

2. CIPCP is an acceptable experimental alternate material for a cross-culvert or storm drain within the travel way, if it is placed in a new construction area where it is not immediately subject to traffic or subsidence. The pipe shall reach design strength before it is subjected to traffic. Non-reinforced CIPCP shall not be used on Federal Aid projects where subject to highway live loads or under high fills.

3. The diameter and other hydraulic characteristics of a culvert, storm drain, slotted drain, irrigation, or siphon pipe are to be determined by a study of each application and each alternate material. The diameter and other characteristics such as the end treatments may be different for the various alternate materials, and these different features should be noted, as needed, on the Pipe Data Sheet. All drainage facilities are prepared by, or under the direct supervision of, a Professional Engineer registered in Oregon.

4. In addition to meeting the design life requirements of the general installation, the outfall of a corrugated metal pipe in brackish or salty waters shall be a corrugated steel pipe with a coating resistant to corrosion, an aluminized steel pipe, or an aluminum pipe. All of these outfall pipes should be surrounded by a clean granular backfill material.

5. The longer design life shall prevail for an installation in a location that is subject to more than one design life.

6. Metal pipes within 24 inches of a proposed lime treated soil or base shall use pH and resistivity results from tests on the in-situ soil and water to determine the appropriate material thickness for an adequate service life. Such a pipe shall receive a Type A (AASHTO M190) bituminous, Chevron Industrial Membrane (CIM), or epoxy coating to protect it from the lime in the soil or base treatment. The coating itself shall not be used as a factor in determining the service life. Lime treated aggregate in the asphalt concrete roadway pavement need not be considered when evaluating the need for additional material thickness or a coating.

7. For an installation in a location that may be affected by a cathodically protected facility, the Region Utility Specialist should be contacted to determine the facility's ownership, the extent of the field, and the financial responsibility for its mitigation, if required. The designer will then determine the required mitigation and see that action is taken for its implementation.

8. Plastic pipes such as CPP, CPEP, PCPEP, PPVCP, SWPEP-PR, SWPVCP, SWPVCP-PR or SRPEP shall not be used in an area with a higher than ordinary risk of fire. Examples are fuel yards and ditches that are cleared by burning.

9. Mitered ends are only allowed for CPP, CPEP, PCPEP, SWPVCP, and SWPVC-PR in locations where slope paving is required.

10. OHSR pipes are limited to applications where both ends of the pipe are enclosed in a structure (e.g. headwall, inlet, access hole, etc).

11. Sloped ends are not allowed on RCP with diameters greater than 54 inches, NRCP of all diameters, or PCP of all diameters.

12. Type A asphalt coating is required on a CAP or CSP-Alzd. slotted drain pipe.

13. DIP is used where structural requirements such as limited fill heights exclude the use of other materials.

14. Watertight joints shall be of approved design and materials (see current Qualified Products List).

15. NRCP Class 1 is not allowed in diameters less than 15 inches.

## 5.8 Service Life

The length of time a pipe must provide adequate performance is its design life, as discussed in the previous section. In comparison, the length of time a pipe is estimated to provide adequate performance before maintenance, repair, or replacement is its service life. The service life should be equal to or in excess of the design life, and the major factors that influence service life should be considered. Typically the major factors are:

- damage to the barrel caused by buoyancy failure (see Chapter 9),
- damage to the inlet caused by hydraulic forces or debris carried by the stream (see Chapter 9),
- distortion of the pipe caused by impact or static loads that are in excess of the structural capacity of the pipe, or incorrect camber at installation (see this chapter and Chapter 9),
- undermining of the pipe outlet due to scour (see Chapter 9 and Chapter 11),
- damage due to fire (plastic pipes are especially susceptible),
- damage from corrosion, both external corrosion caused by the material surrounding the pipe or internal corrosion caused by the liquids that flow through the pipe, and
- damage caused by abrasion from solids that are carried through the pipe by the flow.

Existing culverts are often extended with new materials when highways are realigned or widened. The existing culvert should be inspected to verify if it has a remaining service life at least as long as the applicable design life listed in Table 5-3. Replacing or rehabilitating the existing pipe should be considered if it will have an inadequate service life.

This section discusses the effects on the pipe service life due to fire, corrosion, and abrasion. Methods to prevent these effects from shortening the service life are also discussed.

### 5.8.1 Fire

Fire can occur almost anywhere on the highway system. There are many causes of these fires, such as forest, brush, or grass fires that enter the highway right-of-way, or spills of flammable liquids that ignite. ODOT Maintenance Personnel are often excellent sources of fire-related site information.

It is acceptable to use pipe materials that are susceptible to fire damage in most locations because the probability of a fire is relatively low. In some locations, however, there is a greater chance of fire. Examples are highways alongside fields that are burned to clear stubble, pipes adjacent to ditches that are cleared by burning, or pipes in areas where flammable liquids are stored or processed, such as fuel yards and refineries. Pipes or pipe coatings that are susceptible to fire damage should be avoided in these higher risk areas. Pipes or coatings that are susceptible to fire damage include plastic pipes of all types because the pipe itself can melt or burn, and bituminous or plastic coated pipes because the coating can burn or melt.

### 5.8.2 Corrosion

Corrosion can cause a pipe to deteriorate and shorten its service life. Corrosion is the destructive attack on a pipe by a chemical reaction with the materials surrounding the pipe. Corrosion problems can occur when metal pipes are used in locations where the surrounding materials have excessive acidity or alkalinity. The relative acidity of a substance is often expressed by its pH value. The pH scale ranges from 1 to 14, with 1 representing extreme acidity, 14 representing extreme alkalinity, and 7 representing a neutral substance. The closer the pH value is to 7, the less potential the substance has for causing corrosion.

Corrosion is an electrolytic process. As a result, it has the greatest potential of causing damage in soils that have a relatively high ability to pass electric current. The ability of a soil to convey current is expressed as its resistivity in ohms-cm, and a soil with a low resistivity has a greater ability to conduct electricity.

The corrosive potentials of soils are determined by laboratory testing on soil samples. This testing is needed for installations where pipes come into contact with the surrounding soils, either in the bedding of the pipe or in the conveyance of particles through the pipe. Therefore, soil samples should be taken near the upstream end of the proposed pipe.

Corrosion can also be caused by surrounding materials such as lime treated base, Portland cement concrete, and other materials. This corrosion can be prevented by isolating the pipe from its surroundings using coatings or other means. The notes to Table 5-4 discuss several critical situations and the needed coatings.

Corrosion can also be caused by excessive acidity in the fluids conveyed by the pipe. Water pH can vary considerably from drainage to drainage, and acidic water is not uncommon. It is good practice to test the pH of the stream where a culvert will be installed if water is available for testing.

The potential for corrosion can also be determined by inspecting existing pipes. The age of these pipes can be determined from the plans. The original thickness can be determined by measuring the thickness in an uncorroded area. Often the pipe gage is listed on the plans. This can be converted to thickness using Table 5-2. The thickness can be measured in a corroded area. The loss in thickness since installation can be divided by the pipe age to determine the thickness loss per year. This information is especially useful when estimating the wall thickness of a replacement pipe intended to provide an adequate service life.

The effects of mildly to moderately corrosive environments on pipe service life are shown in Table 5-4. The table also provides guidance on selecting corrosion countermeasures. The countermeasures include:

- selecting a pipe with an adequate wall thickness to prevent perforation from the corrosion,
- using a coated pipe, or
- selecting a pipe made from a more corrosion resistant material.

In some cases, specialized corrosion countermeasures may be needed that are not included in the table. This could occur if the environment is highly corrosive or the installation has a high value or a long design life, and conventional countermeasures cannot provide adequate protection. In these instances, the ODOT Senior Standards Engineer should be contacted. Specialized countermeasures may be needed such as:

- cathodic protection of the pipe, with either live current or the installation of sacrificial anodes such as zinc bars,
- installation of an oversize conduit with the intent that it will be relined at a later date after corrosion damage occurs, or
- use of corrosion resistant pipe materials such as stainless steel, etc.

The need for specialized protection can also occur if the fluids conveyed by the pipe contain de-icing salts or other chemicals, and local maintenance personnel should be contacted to see if salts or chemicals are used. Salt laden water is especially corrosive, and other de-icing chemicals are usually less corrosive or non-corrosive. The corrosion countermeasures for salt or chemical laden water are often site specific and they use specialized materials and methods. In these cases, the ODOT Senior Standards Engineer should be contacted for assistance.

## Table 5-4 Pipe Material Service Lives

| Materials | Location East or West of Cascades | Water and Soil pH | Soil Resistivity (ohm-cm) | Service life of 0.060 inch thick corrugated aluminum or 0.064 inch thick corrugated steel (years) |
|---|---|---|---|---|
| Galvanized Steel CSP, CSPA, PCSP, SSP/OHSR | East | 4.5—6.0 | 1500—2000 | 30 |
| | East | 6.0—7.0 | 1500—2000 | 35 |
| | East | 7.0—10.0 | 1500—2000 | 40 |
| | West | 4.5—6.0 | 1500—2000 | 15 |
| | West | 6.0—7.0 | 1500—2000 | 20 |
| | West | 7.0—10.0 | 1500—2000 | 25 |
| Aluminum CAP, CAPA, PCAP, SAP/OHSR | All Locations | 4.5—10.0 | more than 1500 | 75 |
| Aluminized Steel CSP-Alzd. CSPA-Alzd. PCSP-Alzd. SSP/OHSR-Alzd. | All Locations | 5—9 | more than 1500 | 75 |
| Concrete CIPCP, NRCP, PCP, RCP, RCBC | All Locations | 4.5—10.0 | more than 1500 | 75 + |
| Plastic CPP, CPEP, PPVCP, SWPEP-PR, SWPVC, SWPVC-PR SRPEP | All Locations | 4.5—10.0 | more than 1500 | 75 |

**Table 5-4, Contd. Pipe Material Service Lives**

**Notes**

1. For galvanized steel the service life will be modified for increased soil resistivity as follows:

| Resistivity | Factor |
|---|---|
| 2000 to less than 3000 | 1.2 |
| 3000 to less than 4000 | 1.4 |
| 4000 to less than 5000 | 1.6 |
| 5000 to less than 7000 | 1.8 |
| more than 7000 | 2.0 |

2. The service lives listed in the table are for 0.060 inch thick aluminum pipe or 0.064 inch thick steel pipe. Multiply the service life by the appropriate factor for different thicknesses. Not all pipe materials are available in all wall thicknesses. Wall thicknesses of common pipes are listed in the ODOT Fill Height Tables.

| Material | Wall Thickness (inches) | Material | Wall Thickness (inches) | Factor |
|---|---|---|---|---|
| Aluminum | 0.075 | Steel | 0.079 | 1.3 |
| Aluminum | 0.105 | Steel | 0.109 | 1.7 |
| Aluminum | 0.135 | Steel | 0.138 | 2.2 |
| Aluminum | 0.164 | Steel | 0.168 | 2.9 |

3. Bituminous coating (AASHTO M190) adds 10 years, and polymeric coating adds 50 years to the service life of metal pipe in all locations with Moderate or lower abrasion levels. The factors from Notes 1 and 2 are to be applied before adding this 10 or 50 years. Consult with the Oregon Department of Fish and Wildlife before using bituminous coated pipes in fish-bearing streams. *Note: Asphalt dipped pipes are no longer available from Oregon but can be obtained from adjoining states.*

4. A representative soil sample needs to be supplied for the project to select alternate pipe materials, wall thicknesses, and/or special coatings. Soil resistivities or pH readings outside of the indicated limits will require special design considerations. Consultation with the ODOT Senior Standards Engineer for appropriate materials is required for those extreme readings outside of the limits shown in Table 5-4. *Note: Additional soil samples may be needed for larger projects with multiple soil types.*

### 5.8.3 Abrasion

Abrasion damage can shorten pipe life. Abrasion is the wearing away of pipe material by water carrying sands, gravels, or rocks. Abrasion can damage or destroy the invert of the pipe and

cause a structural failure. As a result, the selection of pipe materials must consider the effects of abrasion, and if necessary, countermeasures should be used to assure that the service life of the pipe is as long or longer than its design life. Abrasion levels and the invert protection needed to resist abrasion are presented in Table 5-5.

Four abrasion levels are identified in the table. These descriptions are intended to provide general guidance, only, and not all of the listed criteria need to be present in order to justify a particular abrasion level. Other factors should also be considered, such as the abrasion history of pipes at or near the site that convey similar discharges and bed materials.

*Note: Invert abrasion can be reduced or eliminated by fish passage features. One example is a deeply buried invert, where the invert is covered during floods by stationary bed material. Another example is an invert covered by bed material retained in the pipe by sediment retention weirs. In these and similar cases, the degree of invert protection recommended in Table 5-5 may not be needed, and a lesser protection can be used.*

The selection of the abrasion level depends on many factors such as the slope of the pipe, the flow velocities in the conduit, and the sizes of the bed materials carried through the pipe. The 6-month discharge can be used to calculate flow velocities and the size of the bed material that will be transported through the pipe. This discharge is estimated as follows:

$$Q_{6\text{-Month}} = 0.64\ Q_2 \qquad \text{(Equation 5-1)}$$

Where:

$Q_{6\text{-Month}}$ = Flow expected to occur every year in cubic feet per second

$Q_2$ = Average annual flow in cubic feet per second. See Chapter 7 for calculation methods.

The maximum size of the sediment transported through the conduit can be determined by two methods. One method is to examine the channel bed material both upstream and downstream from the conduit, and to estimate the size range of the sediment by visual inspection or sampling and testing. This is the preferred method. See Chapter 6 for details.

The other method requires two steps. The first is to determine the mean depth and average velocity through a typical section of the channel during the 6-month discharge. The second step is to determine the maximum size of the bed materials that move through the pipe using Table 5-6.

The abrasion level is based on the size of the material that may move according to the table, or if smaller, the maximum size of the bed material in the channel. As an example, a stream can move rocks up to 2-1/2 inches in diameter according to the table. The streambed has a bottom of bedrock with banks of silt and clay, and the maximum size of the sediment observed in the

channel is approximately 0.040 inches. The maximum stone size used to determine the abrasion level is 0.040 inches because larger particles are not present.

**Table 5-5 Abrasion Levels and Countermeasures**
**(Adapted from the Washington Department of Transportation Hydraulics Manual)**

| Abrasion Level | General Site Characteristics | Recommended Invert Protection |
|---|---|---|
| Low | • Little or no bed load<br>• Slopes less than 1 percent Abrasive<br>• Velocities less than 3 feet per second | Generally, the protective treatments required for corrosion will provide adequate abrasion protection under these conditions. |
| Medium | • Minor bed loads of sands, silts, and clays<br>• Slopes 1 percent to 2 percent<br>• Velocities less than 6 feet per second | Generally, the protective treatments required for corrosion will provide adequate abrasion protection under these conditions.<br><br>An additional increment of wall thickness should be specified for metal pipes if existing metal pipes in the vicinity have abrasion damage. |
| High | • Moderate bed loads of sands and gravels, with stone size up to 3 inches<br>• Slopes 2 percent to 4 percent<br>• Velocities from 6 feet per second to 15 feet per second | Unprotected pipes or pipes with coatings intended to resist corrosion, only, will often have reduced life expectancies, sometimes lasting only a few years. Polymer coatings provide adequate abrasion protection.<br><br>Metal pipe thickness should be increased at least two increments, or the pipe invert should be paved with wire reinforced concrete. Reinforced concrete box culverts with an increased thickness of concrete between the surface of the bottom slab and the reinforcing bar are preferred over standard box culverts or reinforced concrete pipes. |
| Severe | • Heavy bed loads of sands, gravels, and rocks, with stone | Unprotected pipes or pipes with coatings intended to resist corrosion, only, will often |

| Abrasion Level | General Site Characteristics | Recommended Invert Protection |
|---|---|---|
| | sizes greater than3 inches | have extremely short life expectancies, sometimes lasting only a few months to a few years. |
| | • Slopes steeper than 4 percent | |
| | • Velocities greater than15 feet per second | Sacrificial metal plates, linings, or rails may need to be installed in the pipe or box invert to increase the service life. It is recommended the ODOT Geo-Environmental Section's Engineering and Assets Management Unit be contacted for additional guidance if this type of invert may be needed. A bridge or open-bottom culvert may be a more suitable choice.<br><br>Thermoplastic pipe often has better abrasion resistance than metal or concrete. However, it seldom can be adequately reinforced to provide additional invert protection and it is not recommended for this abrasion level. |

**Table 5-6 Bed Materials Moved by Various Flow Depths and Velocities (Adapted from material in the National Highway Institute "Highways in the River Environment" course and an earlier report by Keown and others.)**

| | | Approximate Nonscour Velocities (feet per second) | | | |
|---|---|---|---|---|---|
| | | Mean Flow Depth (feet) | | | |
| Bed Material | Grain Dimensions (inches) | 1.3 | 3.3 | 6.6 | 9.8 |
| Boulders | more than 10 | 15.1 | 16.7 | 19.0 | 20.3 |
| Large cobbles | 10 - 5 | 11.8 | 13.4 | 15.4 | 16.4 |
| Small cobbles | 5 - 2.5 | 7.5 | 8.9 | 10.2 | 11.2 |
| Very coarse gravel | 2.5 - 1.25 | 5.2 | 6.2 | 7.2 | 8.2 |
| Coarse gravel | 1.25 - 0.63 | 4.1 | 4.7 | 5.4 | 6.1 |
| Medium gravel | 0.63 - 0.31 | 3.3 | 3.7 | 4.1 | 4.6 |
| Fine gravel | 0.31 - 0.16 | 2.6 | 3.0 | 3.3 | 3.8 |
| Very fine gravel | 0.16 - 0.079 | 2.2 | 2.5 | 2.8 | 3.1 |
| Very coarse sand | 0.079 - 0.039 | 1.8 | 2.1 | 2.4 | 2.7 |
| Coarse sand | 0.039 - 0.020 | 1.5 | 1.8 | 2.1 | 2.3 |

| | | | | | |
|---|---|---|---|---|---|
| Medium sand | 0.020 - 0.010 | 1.2 | 1.5 | 1.8 | 2.0 |
| Fine sand | 0.010 - 0.005 | 0.98 | 1.3 | 1.6 | 1.8 |
| | | | | | |
| Compact cohesive soils: | | | | | |
| Heavy sandy loam | | 3.3 | 3.9 | 4.6 | 4.9 |
| Light sandy loam | | 3.1 | 3.9 | 4.6 | 4.9 |
| Loess soils in the conditions of finished settlement | | 2.6 | 3.3 | 3.9 | 4.3 |

Notes:

1. Bed materials may move if velocities are higher than the nonscour velocities.
2. Mean flow depth is calculated by dividing the cross-sectional area of the waterway by the top width of the water surface. If the waterway can be subdivided into a main channel and an overbank area, the mean flow depths of the channel and the overbank should be calculated separately. For example, if the size of moving material in the main channel is desired, the mean flow depth of the main channel is calculated by dividing the cross-sectional area of the main channel by the top width of the main channel.

## 5.9 Fill Heights

An essential aspect of pipe selection is the thickness of the cover over the pipe. This cover dissipates live loads from traffic, both during construction and after the facility is opened to the public. This distance is often called the "cover thickness" or "fill height." The fill height often influences the choice of pipe material or the shape or size of the pipe.

There are minimum and maximum fill heights for most types of pipe. The minimum fill height is measured between the top of the pipe and the surface of the subgrade. Pipes having less than the minimum cover thickness may be damaged or distorted by the wheel or track loads of traffic or construction equipment. Several methods can be used to protect pipes when minimum fill heights cannot be obtained, as follows.

1. Add additional fill cover over the pipe to give it adequate protection. Carefully remove the additional cover to restore the subgrade to the desired profile before placing the base aggregate and roadway surface.

2. Specify use of a stronger pipe in the critical location. This could be ductile iron pipe (DIP) or one of the stronger plastic pipes.

3. Surround the pipe with a strong bedding and pipe zone backfill material such as concrete.

4. Surround the pipe with a strong casing.

The maximum fill height is measured from the top of the pipe to the roadway pavement surface. Pipes with more than the maximum allowable fill height may be distorted or crushed by pressure from the surrounding earth. Methods 2, 3, or 4 used for minimum cover heights can also be used to prevent damage from excessive cover heights.

The allowable fill height ranges are listed in Oregon Standard Drawings RD380 through RD386 for corrugated metal and concrete pipes

The fill height ranges for precast and cast-in-place concrete box culverts are also in the Oregon Standard Drawings.

The cover heights for precast box culvert sizes not in the Standard Drawings are listed in Specifications M 259 and M 273 of the AASHTO "Standard Specifications for Transportation Materials" using "Interstate Live Load" conditions.

The fill heights for smooth inner wall polyvinyl chloride (PVC) pipes are listed in this section and in Oregon Standard Drawings RD388.

The fill heights for AASHTO M 294 Types D and S corrugated high density polyethylene pipes are listed in Oregon Standard Drawing RD390. Oregon Standard Drawing RD391 provides fill heights for ASTM F 2562 steel reinforced high density polyethylene pipes.

Manufacturer's recommended fill heights are used for products on the ODOT Qualified Products List that are not in Oregon Standard Drawings or AASHTO specifications. Shapes and materials not included in the Oregon Standard Drawing fill height tables, AASHTO, or the Qualified Products List should be analyzed for live load and earth pressures.

Fill heights less than or greater than the values in the listed references should not be used for pipes with diameters, equivalent diameters, or spans less than 72 inches unless approval is obtained from the ODOT Roadway Section Senior Standards Engineer. Fill heights less than or greater than the values in the listed references should not be used for larger pipes unless approval is obtained from the structural designer.

The influence of fill heights on pipe material selection is illustrated in the example shown on Plate 5-1.

### 5.9.1 Fill Heights for SWPVCP

ODOT alternate materials policy allows several grades of smooth inner wall polyvinyl chloride pipe (SWPVCP) for various applications and fill heights, as discussed in Subsection 5.4.3. The fill heights for SWPVCP are listed in Oregon Standard Drawing RD388 and in Table 5-7.

Fill heights are not listed for ASTM D 2729 SWPVCP. This pipe is used for subsurface drains, only. Fill heights are not a concern for typical applications where the pipe is enclosed in a drain trench constructed to ODOT specifications.

**Table 5-7 Fill Height Table for SWPVCP**

| Pipe | Minimum Recommended Fill Height in Feet | Maximum Recommended Fill Height in Feet |
|---|---|---|
| ASTM D 3034 SDR 35 *<br>ASTM F 679 (all pipes) *<br>ASTM F 758 Type PS 46<br>ASTM F 794 Series 46 *<br>ASTM F 949 (46 psi stiffness)<br>ASTM F 1760 Sewer-Drain Series or Schedule 40 Series<br>ASTM F 1803 (all pipes) | 2 | 40 |
| AWWA C 905 (DR = 32.5) | 2 | 41 |
| ASTM D3034 SDR 26<br>ASTM F 794 Series 115<br>ASTM F 949 (115 psi stiffness)<br>AWWA C 905 (DR = 26) | 1 | 46 |
| AWWA C 900 (DR = 25)<br>AWWA C 905 (DR = 25) | 1 | 48 |
| ASTM D 3034 SDR 23.5 | 1 | 50 |
| AWWA C 905 (DR = 21) | 1 | 61 |
| AWWA C 900 (DR = 18)<br>AWWA C 905 (DR = 18) | 1 | 69 |
| AWWA C 900 (DR = 14)<br>AWWA C 905 (DR = 14) | 1 | 109 |

Notes: Fill heights are based on HS 25-44 live load.
Minimum fill heights are vertical distance between top of pipe and surface of subgrade.
Maximum fill heights are vertical distance between top of pipe and surface of pavement.
See Standard Drawing RD388 for minimum and maximum recommended fill height.

**Table 5-7, Contd. Fill Height Table for SWPVCP**

| ASTM or AWWA Specification | Diameters Listed in Specification in Inches |
|---|---|
| ASTM D 3034 SDR 35 (46 psi stiffness) | 4, 6, 8, 10, 12, and 15 |
| ASTM D 3034 SDR 26 (115 psi stiffness) | 4, 6, 8, 10, 12, and 15 |
| ASTM D 3034 SDR 23.5 (153 psi stiffness) | 4 and 6 |
| ASTM F 679 all pipes (46 psi stiffness) | 18, 21, 24, 27, 30, 33, 36, 42, and 48 |
| ASTM F 758 Type PS 46 (46 psi stiffness) | 4, 6, and 8 |
| ASTM F 794 Series 46 (46 psi stiffness) | 4, 6, 8, 10, 12, 15, 18, 21, 24, 27, 30, 33, 36, 39, 42, 45, and 48 |
| ASTM F 794 Series 115 (115 psi stiffness) | 8, 10, 12, and 15 |
| ASTM F 949 (46 psi stiffness) | 4, 6, 8, 10, 12, 15, 18, 21, 24, 27, 30, and 36 |
| ASTM F 949 (115 psi stiffness) | 8, 10, 12, and 15 |
| ASTM F 1760 Sewer-Drain Series (46 psi stiffness) | 4, 6, 8, 10, 12, and 15 |
| ASTM F 1760 Schedule 40 Series (stiffness more than or equal to 50 psi) | 1.25, 1.5, 2, 3, 4, 6, 8, 10, and 12 |
| ASTM F 1803all pipes (46 psi stiffness) | 18, 21, 24, 27, 30, 33, 36, 39, 42, 45, 48, 54, and 60 |
| AWWA C 900 all pipes | 4, 6, 8, 10, and 12 |
| AWWA C 905 DR = 32.5 (57 psi stiffness) | 14, 16, 18, 20, 24, 30, 36, 42, and 48 |
| AWWA C 905 DR = 26 (115 psi stiffness) | 14, 16, 18, 20, 24, 30, and 36 |
| AWWA C 905 DR = 25 (129 psi stiffness) | 14, 16, 18, 20, 24, 30, 36, 42, and 48 |
| AWWA C 905 DR = 21 (224 psi stiffness) | 14, 16, 18, 20, 24, 30, and 36 |
| AWWA C 905 DR = 18 (364 psi stiffness) | 14, 16, 18, 20, and 24 |
| AWWA C 905 DR = 14 (815 psi stiffness) | 14, 16, and 18 |

DR = Sidewall thickness to outside diameter ratio as defined in AWWA specifications.
SDR = Sidewall thickness to outside diameter ratio as defined in ASTM specifications.

## 5.10 Pipe Material Selection Procedure

Procedures are outlined in this section for selecting alternate pipe materials. These instructions are general guidelines. Each application shall be studied and the pipe selections must satisfy all site conditions. The steps to select alternate pipe materials follow.

**Step 1 -** Obtain drainage studies, hydraulic studies, and location data for the project. A hydraulic study should be available for culverts or siphons greater than 48 inches diameter, equivalent diameter, or span. The drainage, hydraulic, or location information should provide the lengths, slopes, minimum sizes, and end treatments for the proposed pipes. Often these studies will also include comments or recommendations for pipe materials.

A stormwater report should also be available for storm drains. Often the report also lists the lengths, slopes, and minimum sizes of the pipes; and the locations, sizes, and types of other drainage features such as inlets and manholes. Recommendations or comments on pipe materials are often included.

A critical element of pipe selection is the Manning's internal roughness value used in the hydraulic design. It is important to verify the roughness value used in the design is applicable for the selected alternate materials. As an example, a hydraulic design assumed the pipes are smooth-walled, and it provided sizes and slopes for pipe materials such as concrete, smooth wall outward helical spiral rib, AASHTO 294 Type D or S corrugated high density polyethylene pipe, or smooth inner wall polyvinyl chloride pipe. The sizes and slopes from this design may not be applicable to rougher pipes such as corrugated metal. These rougher pipes may require larger sizes to provide adequate hydraulic capacities and need steeper slopes to produce the desired cleaning velocities.

In general, a hydraulic design based on the assumption that the pipes are corrugated will provide adequate sizes and slopes for corrugated pipes as well as smooth-walled pipes. In some cases the recommended sizes and slopes for the corrugated pipes may be larger and steeper than the minimum sizes and slopes needed for the smooth-walled pipes.

**Step 2 -** Determine the additional requirements that the pipe materials must satisfy, as listed in Section 5.2 of this chapter.

**Step 3 -** Determine if lime will be used in the subgrade or base, and if cathodically protected facilities are nearby. Determine if the outfall is into salty or brackish water.

**Step 4 -** Determine the design life, possible alternate materials, and special requirements based on Table 5-3. This is a preliminary estimate of the alternate materials. Not all of the materials listed in the table may be feasible. Some materials may not have adequate

structural capacity, abrasion or corrosion resistance, resistance to fire damage, or they may not satisfy environmental requirements or the preferences of local jurisdictions.

**Step 5 -** Obtain the results of the water pH tests and soil pH and resistivity tests. In addition, contact local maintenance personnel to see if the pipes will convey water containing deicing chemicals or salts. Contact local maintenance personnel to see if there is extraordinary fire danger in the area if plastic is an alternate material.

**Step 6 -** Determine the pipe materials that provide an adequate service life from Table 5-4. This table addresses the effects of corrosion. Coatings or increased wall thicknesses may be needed for some pipes.

**Step 7 -** For a culvert, determine the size of the bed materials in the channel. Visual examinations of the channel bed are sufficiently accurate for most applications. Perform a hydraulic analysis to estimate flow velocity if a visual inspection is not done. Estimate the size of the bed material that will move through the pipe during the 6-month discharge. Contact the hydraulics designer if assistance is needed.

For storm drain systems, underdrains, or siphons, determine the size of the bed materials the system may carry. In most cases the largest materials in the runoff are cinders, sands, or gravels, and the abrasion level is "Non Abrasive" or "Low Abrasive." The selection of the appropriate abrasion level should also consider the extent of the abrasion damage observed in the existing system.

**Step 8 -** Determine the types of abrasion protection that will be needed using Table 5-5. The thickness of some types of pipe may need to be increased, or other countermeasures may be needed, such as invert paving or the installation of settling basins to retain the abrasive sediments. Consider abrasion reduction due to fish passage features, if applicable.

**Step 9 -** Determine the range of fill heights over the pipe. Typically there will be two fill heights. One height will be the distance between the subgrade and the top of the pipe. This is the lowest fill height, and it must be adequate to prevent damage to the pipe from the wheel loads of both construction equipment and traffic. Culverts and storm drains are almost always installed on a slope, and the thinnest cover is over the top of the pipe, usually on the upstream end.

The other fill height is the distance between the pavement surface and the top of the pipe. This is the greatest fill height. It must be not be excessively large, in order to prevent the pipe from being damaged by the weight of the road embankment and traffic. A pipe on a slope usually has the thickest cover over the downstream end.

Fill heights are often shown graphically on the storm drain design profiles. Locating the pipes within the recommended range of fill heights is a critical design task. The hydraulic report or narrative may have important information on this subject.

**Step 10 -** Determine the pipe materials where the recommended range of fill heights encompasses the range of fill heights on the installation. The fill heights must be within the ranges on the appropriate Oregon Standard Drawing Fill Height Table or other appropriate reference. Fill heights on the project may require the wall thickness of some pipes to be increased, or some types or shapes of pipe may not be suitable.

**Step 11 -** List alternate materials meeting the preceding criteria on the Pipe Data Sheet. Make sure to list all needed information for each pipe, such as wall thickness, coating, end treatment, etc.

*Note: At this stage in the design, if the list of possible pipe materials includes a RCBC, the designer makes a cost comparison between the box and pipe alternatives. If the box is the most cost-effective material, a structural design is done and structural drawings are included with the roadway plans. Alternate materials are not listed on the Pipe Data Sheet. If one or more of the pipes are estimated to be the most cost-effective materials, all of the alternative pipe materials are listed on the Pipe Data Sheet and a structural design for a box is not included.*

### 5.10.1 Example - Selecting Alternate Materials for a Culvert

Determine alternate materials for a 115 foot long roadway cross-culvert on a non-interstate highway. The end treatment will be paved sloped ends at 1V: 3H slopes.

**Step 1 -** The Hydraulics Report says adequate hydraulic capacity can be provided by a 4-foot span by 3-foot rise RCBC, a 4-foot diameter circular concrete, plastic, or metal pipe, or a 4-foot 1-inch span by 2-foot 9-inch rise corrugated aluminum or steel pipe-arch.

**Step 2 -** Section 5.2 of this chapter is reviewed to see which requirements apply to this installation. The pipe materials must provide adequate hydraulic capacity, service life, corrosion and abrasion resistance, and structural capacity. There are no special requirements at this site for fish passage or local government preference.

**Step 3 -** The location data is reviewed to see if the subgrade or base is lime treated and if the outfall would be in salty or brackish water. There is no lime treatment and the outfall is into fresh water.

**Step 4 -** Table 5-3 is reviewed to determine the design life. The table notes were reviewed to see if there are special requirements or restrictions.

The design life is 50 years for this type of installation. Note 1 says that CIPCP can be used if it is not immediately subject to traffic. On this project it will be impractical to

restrict construction traffic from crossing the pipe immediately after it is placed. Therefore, CIPCP will not be specified. Note 10 says OHSR cannot be used unless the ends are enclosed in a structure. The slope paving is not a structure. OSHR will not be specified. Note 11 says NRCP cannot be used with slope ends. NRCP will not be specified.

At this stage in the design the possible alternate materials are CAP, CAPA, CPP, CPEP, SRPEP, CSP, CSPA, CSP-Alzd., CSPA - Alzd., RCBC, RCP, and SWPVCP.

**Step 5 -** The water pH test results are obtained. The water pH is 6.5, the soil pH is 7.5, and the soil resistivity is 1800 ohm-cm. Maintenance is contacted and asked about exceptional fire danger because plastic pipe is an alternative. There is no extraordinary danger of fire at this location.

**Step 6 -** Table 5-4 is reviewed to see which materials provide adequate service lives. This culvert is located west of the summit of the Cascade Mountains. Adequate service life can be provided by CAP or CAPA with a 0.060-inch wall thickness, CSP-Alzd. or CSPA-Alzd. with a 0.064-inch wall thickness, CPP, CPEP, SRPEP, RCP, or SWPVCP. Adequate corrosion resistance can be obtained by these pipes without additional countermeasures such as coatings or additional wall thickness.

An RCBC is listed as an alternative in the Hydraulics Report. The designer makes a cost-comparison between the RCBC and alternate pipe materials. Structural plans for a box culvert will be included if the RCBC is anticipated to be the most cost-effective material. Alternate pipe materials would not be listed. Conversely, alternate pipe materials would be listed if a pipe culvert is estimated to be the most cost-effective solution, and an RCBC would not be mentioned.

Calls are made to Oregon distributors and manufacturers. Precast sections of this small box are not available in the Pacific Northwest. The box would have to be cast-in-place on site or special forms would need to be made. Cast-in-place construction cannot be used because traffic cannot be kept off of the box until curing is completed and adequate strength is obtained. Fabricating forms for small precast sections are estimated to be costly. For these reasons the RCBC is not considered at this site, and alternate pipe materials will be listed. If available, the RCBC would provide an adequate 75+ year service life at this location.

CSP and a CSPA with a 0.064-inch wall thickness will provide a service life of only 20 years based on the table. This is too short. At least two methods can be used to give this galvanized corrugated steel pipe an adequate life.

Option 1: Specify a 0.064 inch wall thickness with a polymer coating. The service life will be increased by 50 years, based on Note 3 in the table. This will give the pipe a 70-year service life. This service life will be adequate.

Option 2: Increase the wall thickness four increments to 0.168 inches and not use a coating. This will increase the service life of the pipe by a factor of 2.9, as shown in Note 2. This 58-year service life will be adequate.

**Step 7-** Stones up to 3 inches in diameter are carried through the pipe, based on an inspection of the channel bed upstream and downstream from the existing culvert. These are the largest stones present in this reach of channel. The average velocity in the pipe during the 6-month discharge is 10 feet per second based on a hydraulic analysis.

**Step 8 -** Table 5-5 was reviewed to see if abrasion protection is needed. The abrasion level is "moderate" based on the table. Abrasion damage countermeasures are needed for all pipes except the CPP, CPEP, SRPEP, and SWPVCP. This protection can be at least a two increment increase in wall thickness for metal pipes, a polymer coating, or wire reinforced concrete paved invert, based on the table. The abrasion resistance will be obtained by increasing wall thickness or the polymer coating. A paved invert will not be used because of the installation difficulty in this small pipe.

At this stage in the design it is possible to select materials that will resist both corrosion and abrasion. The CAP, CAPA, CSP-Alzd. and CSPA-Alzd. alternatives will have a two increment wall thickness increase to prevent abrasion damage, to a wall thickness of 0.105 inches for the aluminum and 0.109 inches for the aluminized steel alternatives, or a polymer coating with a 0.060 and 0.064 thickness for aluminum and aluminized steel, respectively. The galvanized CSP and CSPA will have a four increment wall thickness increase to 0.168 inches to resist both corrosion and abrasion, or a polymer coating with a 0.060 and 0.064 thickness for aluminum and aluminized steel, respectively. The CPP, CPEP, SRPEP, RCP, and SWPVCP are adequate.

**Step 9 -** The fill heights for the round pipes were calculated from the roadway design data. The shallowest fill height will be under the upstream edge of the roadway between the subgrade and the top of the pipe. This height of 1.0 feet will occur during construction, and it will need to dissipate the wheel loads from construction equipment traveling over the roadway embankment fill. The greatest fill height will be 6.6 feet. It will be under the finished grade surface of the pavement at the downstream edge of the road.

The same roadway data was used to calculate the fill heights over the pipe-arch. The 2-foot 9-inch rise pipe-arch is not as tall as the 4-foot diameter pipe. The fill heights over the pipe-arch will range from 2.3 feet to 7.9 feet.

**Step 10 -** The Fill Height Tables were reviewed to determine the wall thickness needed to resist deformation. The round corrugated metal pipes were checked first. The fill height tables for the CAP, CSP, and CSP-Alzd. are in Oregon Standard Drawing RD380. The minimum thickness for CAP is 0.075 inches in 2 1/2in x 1/2in corrugations and

0.135 inches in 3in x 1in corrugations. The minimum thickness for CSP and CSP-Alzd. is 0.079 inches in either corrugation size.

The fill height tables for the CAPA, CSPA, and CSPA-Alzd. are in Oregon Standard Drawing RD382. The range of fill heights for the recommended 4-foot 1-inch span by 2-foot 9-inch rise pipe-arch is 2.25 to 7 feet. This range is for aluminum and steel with all wall thicknesses and 2 ½ inch x ½ inch corrugations. The 7.9 foot fill height at the downstream end exceeds the maximum allowable fill height of 7 feet. A similar pipe-arch is available in 3inch x 1inch corrugations with a fill height range of 1.5 to 12 feet. This will work under the project fills. It has a 3-foot 10-inch span by 3-foot rise, and it was verified this pipe will be hydraulically adequate.

The fill height table for SRPEP is in Oregon Standard Drawing RD391. The minimum and maximum fill heights are 1.0 foot and 30 feet, respectively. The 1.0 to 6.6 foot fill heights for this installation are within the recommended range. SRPEP can be used.

The fill height recommendations for SWPVCP are in Section 5.8 of this chapter and in Oregon Standard Drawing RD388. The 1.0 foot minimum cover depth is too great for many grades of SWPVCP. A pipe meeting AWWA C 905 (DR = 25) specifications will work with the project cover heights, and the appropriate size is listed in Table 5-7.

The fill height ranges for RCP are shown on Oregon Standard Drawing RD386. The minimum and maximum fill heights are 1.0 foot and 23 feet, respectively for Class IV pipe. The 1.0 to 6.6 foot fill heights for this installation are within the recommended range. RCP can be used.

At this stage, there are many alternate materials, and they will be listed on the Pipe Data Sheet.

1. 4-foot diameter uncoated circular corrugated metal pipes, 2 2/3 inch x ½ inch corrugations or 3 inch x 1 inch corrugations, in these wall thicknesses:

   - 0.105 inch for CAP (2 ½ inch x ½ inch corrugations, only),
   - 0.135 inch for CAP,
   - 0.168 inch for CSP, or
   - 0.109 inch for CSP-Alzd.

2. 4-foot diameter polymer coated circular corrugated metal pipes, 2 2/3 inch x ½ inch corrugations or 3 inch x 1 inch corrugations, in these wall thicknesses:

   - 0.075 inch for CAP (2 ½ inch x ½ inch corrugations, only),
   - 0.135 inch for CAP,
   - 0.079 inch for CSP or CSP-Alzd.

3. 3-foot 10-inch span by 3-foot rise uncoated corrugated metal pipe-arch in 3 inch x 1 inch corrugations, in these wall thicknesses:

   - 0.105 inch for CAP,
   - 0.168 inches for CSP, or
   - 0.109 inches for CSP-Alzd.

4. 3-foot 10-inch span by 3-foot rise polymer coated corrugated metal pipe-arch in 3 inch x 1 inch corrugations, in these wall thicknesses:

   - 0.075 inch for CAP,
   - 0.079 for CSP or CSP-Alzd.

5. 4-foot diameter SRPEP

6. 4-foot diameter SWPVCP meeting AWWA C 905 specifications with DR = 25

7. 4-foot diameter Class IV RCP

### 5.10.2 Example - Selecting Alternate Materials for a Storm Drain

Determine the alternate materials for a storm sewer. The manholes and inlets are described on Oregon Standard Drawings. The outlet has a headwall with wingwalls and it is not shown on a standard drawing. It is shown on a separate plan sheet. The system layout is shown on Plate 5-1.

**Step 1 -** The stormwater report results were obtained. Smooth-walled pipes were needed on the trunk line to get enough hydraulic capacity. They were used in the hydraulic model for the trunk line. Corrugated pipes were modeled for the laterals. As a result, the selected alternate pipe materials can be smooth-walled for the trunk line, and corrugated or smooth walled for the laterals.

**Step 2 -** Section 5.2 of this chapter was reviewed to see which requirements apply to this installation. The pipe materials must provide adequate hydraulic capacity, service life, corrosion and abrasion resistance, and structural capacity. The local jurisdiction reports that the roots from trees in the area grow into the cracks in NRCP and they eventually break the pipes apart. They prefer that concrete pipe be RCP. There are no additional requirements.

**Step 3 -** The location data was reviewed to see if the subgrade or base is lime treated or if the outfall would be in salty or brackish water. There is no lime treatment. The outfall is into brackish water.

**Step 4 -** Table 5-3 was reviewed to determine the design life. The notes in the table were reviewed to see if there are special requirements or restrictions.

The design life is 75 years because almost all of the pipes are between curbs, according to the table. Note 2 says that CIPCP can be used if it is not immediately subject to traffic. On this project it will be impractical to restrict both construction and vehicle traffic from crossing the pipe immediately after it is placed. CIPCP will not be specified. Note 4 says metal pipe should be aluminum, aluminized steel, bituminous coated steel, or polymer coated steel if it is in brackish water. The two segments of pipe near the outlet will list one or more of these materials as alternatives. Note 9 says CPP, CPEP and SWPCVP pipes cannot be used with mitered ends without slope paving. Note 10 says OHSR can be used if all ends are enclosed in structures. The pipe ends will be in either inlet boxes, manholes, or at a headwall. CPP, CPEP, SRPEP, SWPVCP, and OHSR can be used. CAP and CSP are allowed for laterals, only. They are not used for the trunk line because they are not smooth-walled.

At this stage in the design the possible alternate materials are a CAP, CPP, CPEP, SRPEP, CSP, CSP-Alzd., SAP/OHSR, SSP/OHSR, SSP/OHSR-Alzd., SWPVCP, and RCP.

**Step 5 -** The water pH test results were obtained. The water pH is 6.5, the soil pH is 7.5, and the soil resistivity is 1800 ohm-cm.

**Step 6 -** Table 5-4 was reviewed to see which materials provide an adequate service life. This culvert is located west of the summit of the Cascade Mountains. Adequate service life can be provided by CPP, CPEP, SRPEP, CAP and SAP/OHSR with a 0.060-inch wall thickness, CSP-Alzd. and SSP/OHSR-Alzd. with a 0.064-inch wall thickness, SWPVCP, or RCP. These pipes do not require additional corrosion countermeasures such as coatings or additional wall thickness, and all of them are suitable for the two pipe sections exposed to brackish water.

CSP and SSP/OHSR with a 0.064-inch wall thickness will provide a service life of only 20 years based on the table. This is too short. Two options will be considered.

Option 1: Increase the wall thickness several increments and use a bituminous coating. This will increase the service life to only 68 years. This is also too short.

Option 2: Use CSP and SSP/OHSR with a 0.079-inch wall thickness. This will increase pipe life 1.7 times more than 20 years, to 34 years. A polymer coating will provide an additional 50 years, for a total of 84 years. This longer service life is adequate. This coating will also protect the two downstream sections of the trunk line from corrosion due to brackish water.

**Step 7 -** The runoff carries small amounts of sand that is placed on icy sections of the road in freezing weather. No abrasion damage is observed in the existing pipes. The abrasion level is considered to be "Non Abrasive."

**Step 8 -** Table 5-5 was reviewed, and the measures necessary to prevent corrosion are considered sufficient protection from abrasion.

**Step 9 -** The fill heights were determined from the storm drain profile. The fill heights range from 1.0 feet (distance between top of upstream ends of several pipes and subgrade surface) to 8.5 feet (distance between top of downstream end of pipe near outfall and road surface).

**Step 10 -** The Fill Height Tables were reviewed to see which materials are within the range of recommended fill heights. CAP, SRPEP, SWPVCP meeting AWWA specifications, and RCP (Class IV) are the only alternate materials for several pipes that have low fill heights. CAP, CPP, CPEP, SRPEP, CSP, CSP-Alzd., SAP/OHSR, SSP/OHSR, SSP/OHSR-Alzd., SWPVCP, and RCP (Class III) are alternate materials for the rest of the pipes, as shown in Plate 5-1.

At this stage, the alternate materials have been selected, as follows.

1. CAP with a 0.060-inch wall thickness for all laterals.

2. SRPEP for all pipes.

3. SRPEP for all laterals.

4. CPP and CPEP for all pipes, except those with fill heights less than 2 feet

5. CPP and CPEP for all laterals, except those with fill heights less than 2 feet.

6. CSP with a 0.079-inch thickness for all laterals.

7. CSP-Alzd. with a 0.064 thickness for all laterals.

8. SAP/OHSR with a 0.060-inch wall thickness for all pipes, except those with fill heights less than 2 feet.

9. SSP/OHSR with a 0.079 inch wall thickness and polymer coating. This material can be used for all pipes, except those with fill heights less than 1.5 feet.

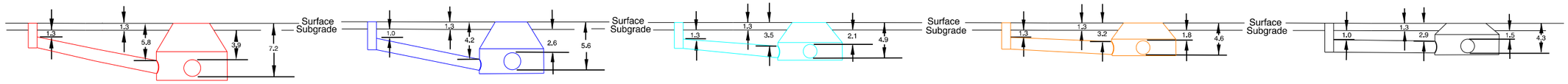
10. SSP/OHSR-Alzd. with a 0.064-inch wall thickness for all pipes, except those with fill heights less than 1.5 feet.

11. SWPVCP meeting ASTM F 679, ASTM F 794 Series 46, ASTM F 949 (46 psi stiffness), and ASTM F 1803 for 18, 21, and 24 inch diameter pipes with minimum cover depths of 2 feet or greater.

12. SWPVCP meeting AWWA C 905 DR = 26 for 18 inch diameter pipes with minimum cover depths less than 2 feet but equal to or more than 1 foot.

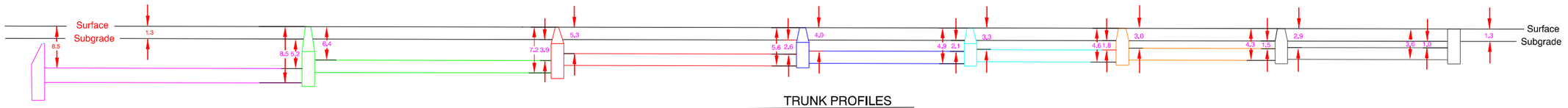
13. SWPVCP meeting ASTM D 3034 SDR 26, ASTM F 794 Series 115, ASTM F 949 (115 psi stiffness), and AWWA C 905 DR 26 for 15 inch diameter pipes with minimum cover depths less than 2 feet but equal to or more than one foot. Note: A 15-inch diameter pipe is not listed in AWWA C905. A 16-inch diameter pipe would be used.

14. RCP. Class IV will be an alternate where cover heights are less than 1.5 feet. Class III will be an alternate for pipes with more cover.



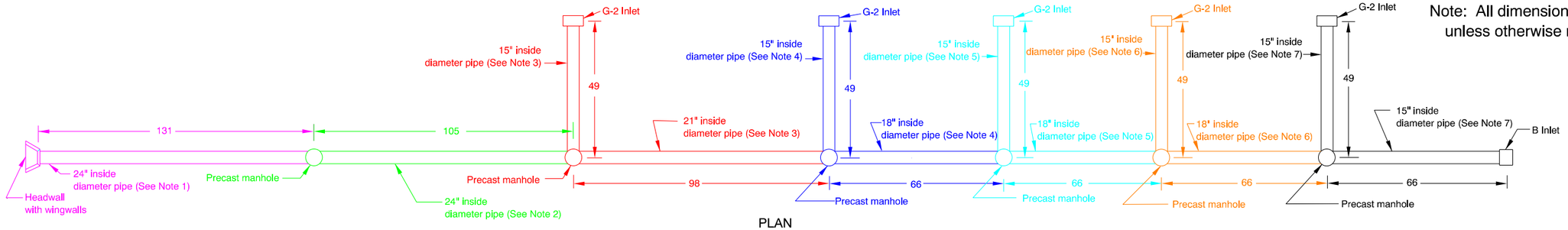

LATERAL PROFILES

No Scale




TRUNK PROFILES

No Scale Vertical Exaggeration




PLAN

No Scale

Note: All dimensions in feet unless otherwise noted.

INFORMATION IN NOTE 1

End treatment is headwall with wingwalls (shown on separate sheet)

Pipe length = 131 ft

Pipe diameter = 24"

Design cover (minimum cover over length of run) = 5.2 ft

Maximum cover over length of run = 8.5 ft

Alternate materials:
- CPP
- CPEP*
- SAP/OHSR
- SSP/OHSR - ALZD
- RCP (CLASS III)
- SWPVCP
- SRPEP

INFORMATION IN NOTE 2

Precast manhole on downstream end. Depth of manhole = 8.5 ft

Pipe length = 105 ft

Pipe diameter = 24"

Design cover (minimum cover over length of run) = 3.9 ft

Maximum cover over length of run = 6.4 ft

Alternate materials:
- CPP
- CPEP*
- SAP/OSHR
- SSP/OHSR - ALZD
- RCP (CLASS III)
- SWPVCP
- SRPEP

INFORMATION IN NOTE 3

Precast manhole on downstream end. Depth of manhole = 7.2 ft

Pipe length = 98 ft

Pipe diameter = 21"

Design cover (minimum cover over length of run) = 2.6 ft

Maximum cover over length of run = 5.3 ft

Alternate materials:
- CPP
- CPEP*
- SAP/OHSR
- SSP/OHSR - ALZD
- RCP (CLASS III)
- SWPVCP
- SRPEP

"G-2" INLET ON UPSTREAM END

Pipe length = 49 ft

Pipe diameter = 15"

Design cover (minimum cover over length of run) = 1.3 ft

Maximum cover over length of run = 5.8

Alternate materials:
- CAP
- CSP
- CSP-ALZD
- RCP (CLASS IV)
- SWPVCP **
- SRPEP

INFORMATION IN NOTE 4

Precast manhole on downstream end. Depth of manhole = 5.6 ft

Pipe length = 66 ft

Pipe diameter = 18"

Design cover (minimum cover over length of run) = 2.1 ft

Maximum cover over length of run = 4.0 ft

Alternate materials:
- CPP
- CPEP*
- SAP/OHSR
- SSP/OHSR - ALZD
- RCP (CLASS III)
- SWPVCP
- SRPEP

"G-2" INLET ON UPSTREAM END

Pipe length = 49 ft

Pipe diameter = 15"

Design cover (minimum cover over length of run) = 1.0 ft

Maximum cover over length of run = 4.2 ft

Alternate materials:
- CAP
- CSP
- CSP-ALZD
- RCP (CLASS IV)
- SWPVCP**
- SRPEP

INFORMATION IN NOTE 5

Precast manhole on downstream end. Depth of manhole = 4.9 ft

Pipe length = 66 ft

Pipe diameter = 18"

Design cover (minimum cover over length of run) = 1.8 ft

Maximum cover over length of run = 3.3 ft

Alternate materials:
- SSP/OHSR - ALZD
- RCP (CLASS III)
- SWPVCP**
- SRPEP

"G-2" INLET ON UPSTREAM END

Pipe length = 49 ft

Pipe diameter = 15"

Design cover (minimum cover over length of run) = 1.3 ft

Maximum cover over length of run = 3.5 ft

Alternate materials:
- CAP
- CSP
- CSP-ALZD
- RCP (CLASS IV)
- SWPVCP**
- SRPEP

INFORMATION IN NOTE 6

Precast manhole on downstream end. Depth of manhole = 4.6 ft

Pipe length = 66 ft

Pipe diameter = 18"

Design cover (minimum cover over length of run) = 1.5 ft

Maximum cover over length of run = 3.0 ft

Alternate materials:
- RCP (CLASS III)
- SSP/OHSR-ALZD
- SWPVCP**
- SRPEP

"G-2" INLET ON UPSTREAM END

Pipe length = 49 ft

Pipe diameter = 15"

Design cover (minimum cover over length of run) = 1.3 ft

Maximum cover over length of run = 3.2 ft

Alternate materials:
- CAP
- CSP
- CSP-ALZD
- RCP (CLASS IV)
- SWPVCP**
- SRPEP

INFORMATION IN NOTE 7

Precast manhole on downstream end. Depth of manhole = 4.3 ft

Pipe length = 66 ft

Pipe diameter = 15"

Design cover (minimum cover over length of run) = 1.0 ft

Maximum cover over length of run = 2.9 ft

Alternate materials:
- CAP
- CSP
- CSP-ALZD
- RCP (CLASS IV)
- SWPVCP**
- SRPEP

"G-2" INLET ON UPSTREAM END

Pipe length = 49 ft
Pipe diameter = 15"
Design cover (minimum cover over length of run) = 1.0 ft

Maximum cover over length of run = 2.9 ft

Alternate materials:
- CAP
- CSP
- CSP-ALZD
- RCP (CLASS IV)
- SWPVCP**
- SRPEP

NOTE: Pipe length is measured center of manhole to center of manhole
* AASHTO M 294 Type S or D, only.
** AWWA C 905, only.

PLATE 5-1
PLAN AND PROFILE FOR
STORM DRAIN EXAMPLE PROBLEM

# CHAPTER 6

# DATA COLLECTION

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## 6.1 Introduction

It is necessary to identify the types of data that will be required prior to conducting the engineering analysis. The importance of the project determines the level of effort needed for data collection. A comprehensive, accurate, and economical highway drainage design requires reliable and accurate data for its success. Failure to base the design on sufficient and appropriate data can lead to economic loss, potential loss of life and damage and temporary closure of the highway. Not all data discussed in this chapter will be needed for every project. Data collection for a specific project must be commensurate with the project scope and tailored to:

- site conditions;
- scope of the engineering analysis;
- environmental requirements;
- unique project requirements; and
- regulatory requirements.

In addition to the data in the preceding list, it may be necessary to collect information to assure and verify the design satisfies Oregon drainage law.

Uniform or standardized survey requirements for all projects may prove uneconomical or data deficient for a specific project. Special instructions outlining data requirements may have to be provided to the survey party by the designer for unique sites.

This chapter outlines the types of data that are normally required for drainage analysis and design, possible sources and other aspects of data collection. The following subjects are presented in this chapter:

- data collection sequence,
- sources and types of data,
- field reviews,
- drainage survey requirements, and
- data evaluation.

## 6.2 Data Collection Sequence

The data collection sequence for a typical ODOT highway improvement project with bridges and culverts is described in this section. Projects with storm drains, water quality systems, irrigation systems, etc. use a similar sequence and procedure. The data that is collected may be considerably different. Typical maintenance projects will not require all of these steps. The

remainder of this chapter discusses the data sources and the types of needed information. Field survey checklists are included in the appendices.
This sequence begins at the notification of project scoping, with the first visit to the site occurring during the initial scoping trip with the project team.

**Step 1 -** Obtain preliminary information. Detailed design information is not needed or available at this time. The intent is to collect the data for the initial scoping visit. Useful information for scoping includes the following.

- Plans of the existing road and the project prospectus. Preliminary road alignment and typical cross-section may have been developed. They should be obtained if available.
- Environmental documents that discuss hydraulic features. Copies are needed of both draft and issued documents.
- Planning documents and floodplain use regulations that discuss hydraulic features, obtain from the ODOT planner assigned to the project.
- Plans, maintenance history, and inspection reports for existing structures.
- Scour reports, flood data, and relevant information from the hydraulic files for existing structures.

**Step 2 -** Obtain preliminary information from outside sources. Some of this information may also be available from the ODOT contacts. Typical preliminary information includes the following.

- FEMA Flood Insurance Study and maps, from the FEMA website.
- Drainage Master Plans for projects within an Urban Growth Boundary.

**Step 3 -** Review all material obtained in the previous two steps. Note all items pertaining to hydraulic structures. Planning documents for urban projects often list allowable increases in flood elevations. Environmental documents will usually discuss proposed hydraulic structures. The statements in these documents will often affect hydraulic designs.

**Step 4 -** Attend the scoping visit with the project development team. Note and record hydraulic concerns. Collect data as needed to issue the Preliminary Hydraulic Recommendations shortly after the initial scoping visit. Mention to the team any hydraulic requirements based on the information collected in Steps 1 and 2.

An additional site visit is often needed to address concerns or to collect more information. The goal is to collect enough data for the preliminary recommendations. Detailed design information will be obtained later. The preliminary recommendations, including an example, are discussed in **Chapter 4**.

During the initial site visit look for roadway/waterway crossings that have the potential for fish passage issues. This could be crossings that passed fish historically, presently, or in the future if fish passage would be provided. The appropriate time to address the number and location of fish passage culverts is the preliminary stage of project development. Replacing existing small roadway cross-culverts with larger fish passage culverts can add considerable cost to a project. Preliminary hydraulic recommendations are not complete until the number and location of fish passage culverts are known and addressed in the recommendations.

*Note: An ODFW opinion is needed if there are potential fish passage culverts on the project. This opinion will provide the needed information. The ODFW representative visits the site prior to issuing the opinion. Experience shows it is good practice for the hydraulic designer to also attend this visit. This will give the designer a chance to provide input and address concerns.*

Look for proposed roadway embankments that would extend into water bodies, riparian areas, or wetlands. Although these are mainly geotechnical issues, considerable hydraulics assistance may be needed. The scour protection for these embankments, walls, etc. is a topic for the Preliminary Hydraulic Recommendations.

Look at existing culverts to see if they need to be replaced. Critical culverts that cannot be inspected visually can be looked at by video cameras. Requests for this service are made to the project team leader.

Consider each culvert from a construction, cost, and future maintenance viewpoint, whether to leave the existing pipe in-place, extend the pipe, rehabilitate the pipe, replace the pipe in-kind, or install a different size or type of pipe. Items to consider include the following.

- Will an existing culvert remaining in-place or extended provide an adequate service life?
- Is it more economical to rehabilitate than replace a culvert?
- Will the rehabilitated culvert meet all requirements?
- Will all culverts left in-place, rehabilitated, or extended meet fish passage requirements?
- Is trenchless installation more feasible than open-cut installation, or vice-versa?
- Will a new bridge be more economical than a culvert replacement?
- How will water be conveyed through the site during construction?
- How will traffic be conveyed through the site during construction?

Additional expertise will probably be needed to answer these questions. This preliminary stage in the project development is the appropriate time to address these issues.

Limited survey work may be needed in some instances to make preliminary recommendations. This is done only if the added information will be critical to the recommendations. The survey is usually done by level, rod, and tape, using an assumed datum. The survey results are used for the preliminary estimates only, and the more detailed and the more accurate project design survey is used for the facility design.

The sample recommendations in **Chapter 4** are an example of a limited preliminary survey. Roadway cross-sections and stream profiles were needed to verify if fish passage culverts could be installed without significantly excavating the streambeds or elevating the roadway. It was verified the culverts could be installed without road elevation or channel excavation. Knowing this was essential to the initial project development.

**Step 5 -** Request the hydraulic portion of the project design survey. The survey crew must get an accurate representation of the channel and overbank areas. Geotechnical exploration is almost always needed for hydraulic structure design, and this is the time to make the exploration request. It is good practice to meet on-site with the survey team leaders and the geotechnical exploration crew leader to discuss the data request. Many items should be marked in the field as well as described in the survey request. Ordinary high water elevations are an example.

*Note: The data collected during the design survey depends on the type, size, location, and quantity of hydraulic structures on the project. The project team needs to evaluate the preliminary hydraulics recommendations for each crossing and agree on the basic structure type (i.e. bridge, fish passage culvert, or culvert) to be considered during the detailed design. This should be done before the detailed hydraulic design survey is ordered.*

**Step 6 -** Verify that the roadway designer has ordered critical utility location information in areas of concern near hydraulic structures (e.g. pothole explorations to locate existing gas mains, storm drains, etc.).

**Step 7 -** Obtain information from internal and external sources to finish the hydraulic design, such as the following.

- Refined roadway alignment, cross-sections, and processed project and hydraulic survey data, including project vicinity map.

- Vertical control information to reference the project datum to other datums, such as the datum used by FEMA.
- Preliminary traffic staging plan.
- Up-to-date environmental documents and drafts.
- Up-to-date planning documents.
- Information to determine drainage areas, etc., from internal and external sources.
- Preliminary Foundations Report and geological exploration data.
- Floodplain maps and hydraulic computer models if needed, from FEMA.
- Utility location information.

*Note: Most of this information will also be used in subsequent structural designs.*

## 6.3 Data Sources with Website Addresses

Much of the data and information necessary for the design of highway drainage facilities is available from Federal Agencies involved in water resource activities. Other sources include cities and counties, local irrigation districts, and data developed and obtained by ODOT during the design of past highway projects. The following sections discuss potential sources of data and the type of data the source can provide.

### 6.3.1 External Data Sources

The following is a list of the principal external data sources and the type of data that is available. Much of the data is available on the Internet, and websites for the data sources are listed.

1. **U.S. Geological Survey (USGS)** Hydrologic data stations are maintained by the USGS at selected locations throughout Oregon and constitute the major water resources data network in the state for obtaining records on stream discharge and stage, reservoir and lake storage, ground-water levels, well and spring discharge, and the quality of surface and ground water. Every year some new stations are added and other stations are discontinued; thus, the USGS has both a current and a historical record of hydrologic data. The principal data obtained from the USGS is stream-flow data. The USGS also publishes topographic maps. The maps are commonly called 7.5-minute quadrangles and are plotted to a scale of 1:24,000. The maps are used to determine the drainage areas of watersheds. The USGS surface water data website address for Oregon is ***http://or.water.usgs.gov/*** and the mapping address is ***http://geography.usgs.gov/***

2. **Oregon Water Resources Department (OWRD)** Data available from OWRD includes data on water rights, groundwater data, data on wells, and surface water data. The OWRD website features a comprehensive listing of surface water gaging stations throughout the

state, both current and historical. Surface water data collected by the USGS, OWRD, and other agencies is available at the website ***www.wrd.state.or.us***

3. **Federal Emergency Management Agency (FEMA)** Many streams have been analyzed for local flood insurance studies (see **Chapter 2**). The flood insurance studies contain hydrologic data, water surface profiles, and floodplain maps of flood hazard areas. Even though these studies are a good source of data, their technical content should be reviewed prior to using the data. Many of the studies are outdated and/or will not reflect changes that may have occurred in the study reach since its initial publication. In these cases, additional data collection is necessary to supplement and update the FEMA hydraulic models used to develop the local flood insurance studies. FEMA studies and maps are available at ***www.fema.gov***

4. **Natural Resources Conservation Service (NRCS)** NRCS publishes soil surveys and soil maps. The soil data is needed when NRCS hydrologic methods are used to estimate runoff from a watershed. NCRS is also a source of many technical references pertaining to hydrology and hydraulic design. The NRCS website address is ***www.nrcs.usda.gov***

5. **U.S. Army Corps of Engineers (USACE)** USACE is the source of data and information on most of the major dams and reservoirs in the state. USACE is also a source for hydrology and water surface profiles for major rivers in Oregon, particularly the Columbia River and Willamette River. Both rivers are significantly regulated with a series of dams and reservoirs. The Corps is also a source of many technical references pertaining to hydrology, coastal protection, and hydraulic design.

   Oregon is part of three USACE districts. The majority of western and central Oregon is in the Portland District. Their website address is ***www.nwp.usace.army.mil*** The far eastern part of the state is in the Walla Walla District ***www.nww.usace.army.mil*** The Klamath River basin in southern Oregon is in the San Francisco District ***www.spn.usace.army.mil*** The national office of the USACE website address is ***www.usace.army.mil***

6. **National Oceanic and Atmospheric Administration (NOAA)** NOAA is the source of historical climatological data such as precipitation and temperature and is also the source for historical tide data. Climatological data is available from the National Weather Service (NWS) and the National Climatic Data Center (NCDC), which are both part of NOAA. Tide data is available from the National Ocean Service (NOS) Center for Operational Oceanographic Products and Services (CO-OPS), which is also part of NOAA. Tide data includes mean lower low water (MLLW), mean low water (MLW), mean high water (MHW), mean higher high water (MHHW), and extreme tide levels. NOAA also maintains surface water gaging stations throughout the state. The national NOAA website address is ***www.noaa.gov*** The NWS address is ***www.nws.noaa.gov*** The NCDC address is ***www.ncdc.noaa.gov*** The NOS CO-OPS address is ***www.co-ops.nos.noaa.gov***

7. **Oregon Climate Service (OCS)** The OCS is a source of historical climatological data for Oregon. The data available includes precipitation and temperature data. The website address is ***www.ocs.orst.edu***

8. **U.S. Bureau of Reclamation (USBR)** The USBR is a source of data for many dams and reservoirs associated with irrigation in Oregon. It is also a potential source of peak flow data and other design data for major irrigation systems such as the system of irrigation canals in the Klamath Falls area. The website address for Oregon (except the Klamath River basin) is ***www.usbr.gov/pn*** The address for the Klamath River basin is ***www.usbr.gov/mp***

9. **Local Irrigation Districts** Local irrigation districts are a source of peak flow data and other design and operational details for local irrigation systems.

10. **Cities and Counties** Cities and counties can often provide topographic maps, design data for local storm sewer systems, and drainage master plans. The drainage master plans usually include peak flow data for various storm recurrence intervals for existing land use conditions as well as future land use conditions. Cities and counties are also the source for local drainage ordinances that ODOT projects must satisfy.

### 6.3.2 Internal Data Sources

The following is a list of data sources within ODOT and type of data that is available. Website addresses for ODOT sources are included. Website addresses for other sources are listed in the previous subsection.

1. **Geo-Environmental Section's Engineering and Asset Management Unit** This information is the Engineering and Asset Unit files. Copies of this information are available to external personnel, such as consultants by visiting the Transportation Building, 355 Capitol Street N.E., Salem, Oregon. Some types of hydrologic data are available on the Geo-Environmental Section website. The address is ***www.oregon.gov/odot/hwy/geoenvironmental***

    a. Flood photographs. Most of the photographs that are available are photos of flood damage to the highway system during the December 1964 and January 1974 floods. Many oblique aerial photographs are available.

    b. Historic highwater marks. Historic highwater marks are available for several locations around the state.

    c. FEMA Flood Insurance Studies (FIS) for the cities and counties participating in the National Flood Insurance Program including the Flood Hazard Boundary Maps, Flood Boundary and Floodway Maps, and the Flood Insurance Rate Maps developed for each

FIS. The Hydraulics Unit has one copy of each available FIS and the maps that were produced for each study. FIS including maps are available on the internet at FEMA's website. It is recommended the FEMA website be searched for available material before the Engineering and Asset Management Unit is contacted.

d. USGS surface water records. The Engineering and Asset Management Unit has one copy of the annual water resources data report published by the USGS. The annual reports contain the data that is gathered at the active stream gages around the state. The data is also available on the Internet. It is recommended available material be retrieved from the USGS website before the Engineering and Asset Management Unit is contacted.

e. OWRD surface water records. The Engineering and Asset Management Unit has one copy of the surface water records published by OWRD from 1965 through 1978, and limited records after 1978. This data is also available on the OWRD website. It is recommended available material be obtained from the OWRD website before the Engineering and Asset Management Unit is contacted.

f. NOAA climatological data and tide data. The Engineering and Asset Management Unit receives the monthly NOAA publications *Climatological Data* and *Hourly Precipitation Data*. These publications summarize climatological data for the weather stations throughout Oregon. Climatological data and tide data is also available on the NOAA website. It is recommended the NOAA websites be searched for available material before the Engineering and Asset Management Unit is contacted.

g. Hydraulic project files. These files contain the data and calculations that were used to develop the hydraulic design for highway projects. These files were established in 1965. The files contain limited data from hydraulic studies prior to 1965. A Hydraulics Report will be in the file if one was prepared for the project. Typical data includes the topographic survey data required for the hydraulic design such as river cross-sections and longitudinal profiles of the channel bottom, site hydrology, backwater data for bridges and culverts, site photographs, flow velocities, flow depths, water surface profiles, and other miscellaneous hydraulic data. Project files include data for bridges, culverts, storm drains, drainage complaints, detention basins, and water quality facilities.

h. Rainfall intensity-duration-recurrence interval curves have been developed by the Engineering and Asset Management Unit for the entire state. These curves are available in **Chapter 7**.

i. Mean daily flow data for the entire state has been compiled by the Engineering and Asset Management Unit. This data is available at the Engineering and Asset Management Unit website.

*2.* **Geometronics Unit** The Geometronics Unit needs to be contacted for the availability of site-specific information discussed below. They are located at 200 Hawthorne Avenue S.E., B250 Salem, Oregon. Much of this information is listed, along with ordering instructions, at the Geometronics website *www.oregon.gov/odot/hwy/geometronics*

Aerial photography. Geometronics is the primary source of aerial photography for the highway system. They, along with the Engineering and Asset Management Unit, have aerial flood photography.

a. Maps. The unit also produces mosaics, planimetric maps, and contour maps.

b. Benchmarks. Geometronics maintains information on benchmarks.

3. **Bridge Section** The listed information is available to ODOT personnel by contacting the Bridge Section. Copies of this information are available to consultants and others by visiting ODOT Bridge Section Business Support, Transportation Building, 355 Capitol Street, N.E., Salem, Oregon.

*Note: Some information is available on a limited basis.*

a. Bridge maintenance and inspection files. These files contain information on the maintenance and inspection history of a bridge. Typical data and information includes past bridge inspection reports, underwater inspection reports, sounding data, periodic cross-sections of the bridge opening, performance of riprap at piers and abutments, and scour countermeasures constructed at the bridge site.

b. Bridge drawing files. These files contain the as-constructed drawings that show the details and dimensions of existing bridges. The drawings will usually show the subsurface soil exploration data (drill logs) that was obtained for the bridge foundation design. The files may also contain vicinity maps and drawings of box culverts and pipes that are 6 feet or larger in diameter. The vicinity maps are a source of historic channel cross-sections that can be used to evaluate the lateral and vertical stability of the channel.

c. Bridge construction records. The bridge construction records have copies of the daily diaries and weekly construction reports during the time the bridge foundations were constructed. These records have invaluable information about foundation types, the materials under and around the foundations, and footing bottom and pile tip elevations. This information is useful to determine the scour susceptibility of structures. Construction records for older bridges are either in the archives or on microfilm in the Bridge Section library.

d. Bridge scour files. These files contain the data and calculations that were used to estimate scour depths for the bridges that were analyzed as part of ODOT's bridge scour evaluation program. Typical data includes site hydrology, site photographs, river cross-sections, water-surface elevations, flow depths, and flow velocities.

e. Pile records. These records list the pile type, number, and tip elevations for piling supported foundations. Other useful information, such as the blow count and pile driving hammer size can be used to estimate the properties of the underlying materials. This information can be useful when predicting potential scour depths, and it is ordered from ODOT's Bridge Section Business Support.

4. **Roadway Section** Roadway drawing files. The roadway drawing files, often called "vertical" or "V" files, contain the as-constructed drawings for all highway projects exclusive of bridge drawings. The drawings are a source of data and information on the highway's horizontal and vertical alignment, typical roadway section, size and location of culverts, storm sewer details including pipe sizes and profiles as well as manhole and inlet locations, details of specially designed drainage appurtenances, and other miscellaneous details. Copies of these files are available from the ODOT Maps and Plans Center, Transportation Building, 355 Capitol Street, N.E., Salem, Oregon.

5. **Region Technical Center** Each of the five ODOT Region Technical Centers is a source of geotechnical information.

   a. Geotechnical exploration has been done for almost all significant hydraulic structures. Usually this is subsurface exploration using a drill to retrieve core samples. Geotechnical exploration records are available from the Region Technical Center.

   b. Bridge and large culvert preliminary and final foundation reports. These reports summarize the results of the subsurface exploration conducted for a bridge or large culvert. They also recommend the types of foundations suitable for either. The reports are the source of subsurface data that is used to identify the presence of erosion resistant rock that can limit the scour depth. The data is not only useful for the scour analyses of new bridges, but is also useful for scour analyses and scour countermeasure designs for existing bridges. Copies of these reports are available from the Region Technical Centers.

6. **Specifications Unit** Standard drawings and specifications for many aspects of highway construction are available from the Specifications Unit. All material is easily retrieved and printed from the Specifications Unit website

   ***http://www.oregon.gov/ODOT/HWY/SPECS/index.shtml***

a. Bridge Standard Drawings. The Bridge Standard Drawings include details and dimensions for bridge deck drains, box culverts and large culvert details, prestressed slabs, prestressed beams, and other miscellaneous items.

b. Roadway Standard Drawings. An entire section of the roadway standard drawings is devoted to drainage, and it includes dimensions and details for manholes and inlets, allowable fill height tables for pipes, sloped ends for pipes, and other miscellaneous drainage items.

## 6.4 Types of Data Needed

The designer must compile the data that are specific to the subject site. Often the data needed to conduct the hydraulic analysis will depend on the permit requirements, floodplain and environmental regulations. Therefore, the designer should be aware of these requirements. Following are the major types of data that may be required:

- floodplain use regulations (most counties require floodplain development permits),
- biological information,
- watershed characteristics,
- site characteristics, and
- hydrologic and meteorologic data.

Each of these types of data is discussed in more detail in the following sections.

### 6.4.1 Floodplain Use Regulations

Land use and development in floodplains is regulated near most large rivers, estuaries, and the ocean shore where these water bodies abut populated areas. These regulations often influence the type, size, and location of structures and roadway embankments within the floodplain. These regulations vary from jurisdiction to jurisdiction, and they are constantly being refined and updated. For this reason, it is recommended that current regulations be obtained for each project in the floodplain. These documents should be reviewed prior to the start of the design. Copies of these regulations can be requested from the local ODOT planner.

Floodplain use regulations require a hydraulic study for any work in a floodway, either temporary or permanent. These studies vary in complexity, from simple calculations and a one-page statement, to a comprehensive hydraulic study of the stream reach using hydraulic modeling.

A floodway boundary revision request is an example of a study requiring a hydraulic model. The model compares existing and proposed conditions using computer analysis. The input data describing existing conditions is ordered from FEMA, and it may take 6 months or more to get the information. As a result, if a floodway boundary revision request will be submitted, the existing model should be ordered from FEMA as soon as possible.

### 6.4.2 Environmental Requirements and Biological Information

Almost all hydraulic projects have biological impacts, and there are a myriad of federal, state, county and city regulations, standards, and criteria that apply to both the design and construction methods. As an example, state requirements are in legislative acts, Oregon Revised Statutes, Oregon Administrative Rules, and ODOT policy and criteria. These requirements vary considerably at different locations throughout the state, and they are constantly being revised.

The fish passage requirements are of utmost importance. They may significantly affect the scope of hydraulic work and the total project costs. An opinion from the Oregon Department of Fish and Wildlife is required for all highway stream crossings where fish pass, both currently and historically.

Documents such as Biological Assessments, Environmental Impact Statements, Biological Opinions, etc. may be in the process of being prepared, or are prepared, for the project. These documents almost always describe both the hydraulic structures and the construction activities along with their environmental impacts.

Biological information is also needed to design the project and temporary water management during construction. This information almost always includes at least the following:

- species affected,
- lifestages affected, and
- in-water work period.

Environmental requirements, biological information and recommendations, and draft and issued environmental documents, can be obtained from the ODFW, the regulatory agencies, and the ODOT Region Environmental Coordinator.

### 6.4.3 Watershed Characteristics

Watershed characteristics in conjunction with either hydrologic or meteorologic data are needed to estimate the site hydrology. The exception is when a stream gage having a sufficient period of record is located at the site and the gage data reflects current land use conditions. The principal watershed characteristics of interest are the drainage area, land use, basin slope, and soil type(s).

#### 6.4.3.1 Drainage Area

Several methods can be used to determine the drainage area. Topographic maps or auto-delineation using digital elevation model are often used to determine the size and boundaries of larger basins. Aerial photographs or field surveys are often used to determine the size and boundaries of smaller basins. Published references and existing reports can often provide data on drainage areas of all sizes. Often one or more methods are used in combination. It may be necessary to use topographic maps together with aerial photographs, a field review, and spot elevations checks with surveying instruments in order to determine the drainage area, particularly in watersheds that have little relief.

Care must be used when determining the size of the contributing drainage area. Any areas outside the physical boundaries of the drainage area that divert runoff into the drainage area being analyzed must be included in the total contributing drainage area. In addition, it must be determined if flow is diverted out of the basin before reaching the site. In Central Oregon where there has been relatively recent volcanic activity in geologic time, the hydrologic boundary or true drainage area is often uncertain because of groundwater exchange. There are also watersheds, particularly in Eastern Oregon, where a system of irrigation canals and ditches may divert water from one watershed into another. The irrigation systems are typically designed to pass the natural runoff, however, this should be field verified or verified with the local irrigation district.

**Topographic Maps -** The drainage area expressed in acres or square miles is usually determined by tracing the watershed divide on a topographic map with a planimeter. The USGS quadrangle maps are the most commonly used topographic maps and are the best maps available to determine the drainage areas of major rivers. Other sources of topographic maps include cities and counties and the ODOT Geometronics Unit. The topographic maps obtained from cities and counties are often used for urban drainage design such as storm drains and detention systems.

**Auto-Delineation Using a Digital Elevation Model –** Several methods to determine drainage areas can be used with the OWRD procedure for rural regression equations. One method is to delineate the watershed by automated procedures using elevations from a digital model. The user executes this option while logged onto the OWRD website. OWRD programs and data are used to determine the watershed properties.

**Aerial Photographs –** Aerial photographs can be used to determine drainage boundaries of smaller watersheds. Typically a magnifying stereoscope is placed over a pair of aerial photographs. The drainage area is plotted on paper or a map based on the three-dimensional image viewed in the stereoscope. Two overlapping photographs taken directly above the study site are needed to make the image. The Geometronics Unit has images from flights over almost all of the highway system and many other areas. It is helpful when ordering the photos to specify that multiple pictures are needed to provide a stereoscopic image.

**Field Surveys –** A direct field survey using conventional surveying instruments can also be used to determine the drainage area. The direct field survey may be needed to delineate the drainage area in very flat terrain or to verify the area determined with maps or aerial photographs. In flat watersheds or urban areas where the natural drainage pattern may have been changed, a field review may be necessary to delineate the drainage area. The best time to conduct a field review and observe the runoff pattern is when it is raining.

**Published References and Reports –** Published references and reports are a good source of drainage basin information. FEMA Flood Insurance Studies list the contributing areas of significant watersheds within the study area. Almost all of the bridges and large culverts on the state highway system have been the subject of one or more ODOT hydraulic studies, and the watershed areas are often listed in the reports or in the project files.

Drainage Master Plans have been prepared for almost all areas within urban growth boundaries. These plans show existing drainage boundaries, and they also show anticipated changes in drainage boundaries when the urban area develops. These drainage plans are often the best source of information about urban storm drain systems.

Drainage areas published in master plans should be used with caution. Sometimes the drainage area listed in the master plan is only the contributing area within the urban growth boundary, and there is additional contributing drainage area outside of the boundary. Watersheds adjacent to the urban growth boundary should be inspected to verify if the drainage area listed in the master drainage plan is the entire area.

#### 6.4.3.2 Land Use

Many of the techniques, materials, and references used to determine the drainage area can also be used to determine the land use. Information on existing land use and future urbanization trends may be obtained from:

- aerial photographs,
- zoning maps and drainage master plans,
- field review,
- NRCS soil surveys,
- USGS quadrangle maps, or
- a combination of the above techniques.

Existing land-use data for small watersheds can best be determined or verified from a field review. Changes discovered during the field review should be used to update information on maps and aerial photographs used to document the existing land use. The best source of

information on future land use is zoning maps and master plans obtained from municipal planning agencies.

#### 6.4.3.3 Basin Slope

A topographic map or a field survey is needed to determine the slope of the stream channel. The field survey is the usual technique to determine the local channel slope at a site where a drainage structure will be located, whereas topographic maps are used to determine the overall channel slope. The overall channel slope and channel length are needed to calculate the time of concentration or sub-basin flow times when using the Rational method or NRCS methods to estimate peak flows. The channel slope is also an input variable in some OWRD and USGS regression equations.

#### 6.4.3.4 Soil Type

Soil types are needed to estimate peak flows when using NRCS techniques. Soil types can be determined from the soil surveys and soil maps published by the NRCS.

### 6.4.4 Site Characteristics

A complete understanding of the physical nature of the natural channel or stream reach is of prime importance to a good hydraulic design, particularly at the site of interest. Any work being performed, proposed or completed that changes the hydraulic characteristics of a stream reach must be studied to determine its effect on the flow pattern and water surface profile. The principal site characteristics of interest are discussed in this section.

#### 6.4.4.1 Roughness Coefficients

Roughness coefficients, ordinarily in the form of Manning's n values, must be estimated for the entire limits of the stream reach being analyzed. A tabulation of Manning's n values with descriptions of their applications can be found in **Chapter 8**. Manning's n values are usually determined based on the designer's experience and judgment. The roughness coefficients can be estimated during a site visit or from photographs of the site.

The range of Manning's n values used in FEMA Flood Insurance Studies is often listed in the study report. Although these values are often a good reference, they should not be used for the stream to be analyzed unless it is verified they are appropriate.

### 6.4.4.2 Stream Profile and Water Surface Profile

A profile of the channel thalweg and a profile of the water surface at the time of the survey must be obtained in order to determine the slope of the channel. The profiles should extend a sufficient distance upstream and downstream from the project site to accurately determine the slope of the channel. For bridge or culvert crossings the profiles should extend upstream and downstream as shown in the following chart.

The channel slope can be determined from either the thalweg or the water surface profile. The water surface profile is usually a better indicator of the slope, as it will smooth out the irregularities of the thalweg profile. At sites where there is little or no flow at the time of the survey, an effective way to calculate the channel slope from the thalweg profile is to base the elevation change on the elevations of the thalweg at "zero-flow points." Zero-flow points are points in the profile that would control the pool elevations upstream of major riffles if there were no water flowing in the channel.

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**Stream Profile Chart**

| Bridge length "L" in feet * | Upstream and downstream profile distance from centerline of bridge, in feet |
|---|---|
| "L" less than 100 | 500 |
| 100 less than or equal to "L" less than 300 | 1000 |
| 300 less than or equal to "L" less than 500 | 2000 |
| 500 less than or equal to "L" less than 800 | ½ mile |
| 800 less than or equal to "L" | 1 mile |

| Culvert diameter "D" in feet * | Upstream and downstream profile distance from centerline of culvert in feet |
|---|---|
| "D" less than 6 | 300 to 500 |
| 6 less than or equal to "D" | 500 to 700 |

* greater of distance listed above or 20 times bankfull width per section 6.4.4.11 should be used.

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Additional information is also included in the profile notes. Stream gages should be shown on the profiles if they are relatively close. The elevation of the gage reading and the actual gage reading should be obtained in order to tie the gage datum to the survey datum. Ordinary high

water elevations, flow controls, and headcuts should also be noted on the profiles, as discussed in the remainder of this chapter.

#### 6.4.4.3 Stream Cross-Sections

Multiple cross sections are required to compute water surface profiles. Cross-sections are required at representative locations throughout a stream reach and at locations where changes occur in discharge, slope, shape, or roughness, at locations where levees begin or end and at bridges or control structures such as weirs. The cross-section must capture the channel and especially the channel surface below the water. The cross-section also must capture the floodplain and be able to contain the highest expected water surface elevation to be considered. Cross-sections should be placed at intervals that will divide a total reach into a series of subreaches each of which is as uniform in geometry and roughness as practical. Fairly uniform channels will require fewer cross-sections than those having many irregularities in size, shape, slope, or roughness. The sections should be located to enable proper evaluation of energy losses. Therefore, where abrupt changes occur such as at bridges, several cross-sections should be used to define the change. The cross-section should be perpendicular to the direction of flow. Note that perpendicular to the flow in the floodplain may differ from perpendicular to the flow in the channel in which case the section will have a "dogleg" and the direction of the section in floodplain will be different than in the channel.

#### 6.4.4.4 Structures and Roadway Data

The structure geometry and channel geometry of existing structures on the stream reach and near the site are needed to determine their capacities and effects on the stream flow. With bridges, needed data includes span lengths, deck width, deck profile, type of bridge rail (i.e., solid or flow through), bottom-of-beam elevations, type and number of piers, pier widths, and substructure orientation. The necessary culvert data includes size, inlet and outlet geometry, slope, end treatment, culvert material and flow-line profile. "As-built" highway construction plans may be available to obtain the required bridge and/or culvert data. Any structures, downstream or upstream, that may cause backwater or retard stream flow should be investigated.

Also, the manner in which existing structures have been functioning with regard to such items as scour, overtopping, debris and ice passage, fish passage, etc., should be noted. Photographs and high-water profiles or high-water marks of flood events at the structure and past flood scour data are invaluable in assessing the hydraulic performance of the existing facility. Maintenance personnel are often the best source of information on the hydraulic performance of existing structures and information on past floods.

The roadway embankment associated with existing structures usually obstructs the flow and in order to include the effect of the roadway, certain data on the roadway is needed. Needed data includes the profile, width, and alignment of the road. The profile of the road should reflect the

highest part of the roadway and should define the overtopping limits of the roadway if the roadway is subject to overtopping.

For projects where an existing structure will be replaced, data on the proposed structure and roadway are needed. The data requirements for proposed structures and the associated roadway is the same as for existing structures.

Existing culverts, storm drains, and water quality facilities within the project limits that will remain after the project is built should be inspected to determine their condition and remaining service life. These facilities may need to be rehabilitated or replaced. This is especially critical for culverts and storm drain that will be extended. Direct visual inspection and remote inspection by television are usually used. Culvert and Storm Drain Pipe inspection forms are shown in Appendices D and F, respectively.

#### 6.4.4.5 Acceptable Flood Elevations

Development and property use adjacent to the proposed site, both upstream and downstream, may determine acceptable flood elevations. Floor elevations of buildings or structures should be obtained. The presence of upstream development may limit the amount of backwater that the proposed crossing can create. In the absence of upstream development, acceptable flood levels may be based on freeboard or overtopping requirements of the highway. In these instances, the presence of downstream development may determine appropriate overflow points when an overtopping design of the highway is considered.

#### 6.4.4.6 Flood History

As mentioned in Subsection 6.4.4.4, the history of past floods and their effect on existing structures are invaluable in making flood hazard evaluation studies, and they provide needed information for sizing structures. Information may be obtained from newspaper accounts, local residents, maintenance personnel, apparent flood marks or other positive evidence of the elevation of historical floods. High water marks can be very useful when calibrating a hydraulic model. Changes in channel and watershed conditions since the occurrence of the flood must be evaluated in relating historical floods to present conditions.

Recorded flood data are available from federal agencies such as USACE, USGS, NRCS, FEMA, and USBR. Cities and counties may have flood data for structures under their jurisdiction. The ODOT Geo-Environmental Sections' Engineering and Asset Management Unit has flood photographs and high-water marks for the State highway system.

### 6.4.4.7 Debris and Ice

The quantity and size of debris and ice carried or available for transport by a stream during flood events should be investigated. The data obtained may determine the amount of freeboard necessary at the site. If possible, the times of occurrence of debris and ice in relation to the occurrence of flood peaks should be determined. The effect of backwater from debris and ice jams on recorded flood heights should be considered when using stream flow records. The records will usually state if the flood heights were affected by debris or ice. Data related to debris and ice considerations can be obtained by interviewing maintenance personnel. The potential for debris can be assessed during a site visit by noting the presence of debris being transported in the channel, the accumulation of debris against existing bridge piers, or the potential for debris based on the presence of trees growing along the channel banks or the adjacent floodplain.

### 6.4.4.8 Channel Stability

Channel stability is an important consideration relative to the stability of the structure and highway over time. A site inspection will usually reveal obvious channel instabilities such as active bank erosion, headcuts, channel degradation, channel migration and/or local scour problems at bridge piers and abutments. Typical data required to perform a stability analysis includes:

- maps,
- a series of historic aerial photographs,
- historic channel profiles and cross sections (structure opening cross-sections are surveyed periodically during the bridge inspection program),
- notes and photographs from field inspections,
- information on man's activities in the watershed, both recorded and obtained from personal interviews with ODOT maintenance personnel, landowners and others,
- sediment data including transport rate and gradation,
- observations from a visit to the site, and
- changes in stream hydrology and hydraulics over time.

More detailed information on the data requirements for evaluating and analyzing channel stability can be found in the following reference:

Lagasse, P.F., J.D. Schall, and E.V. Richardson, 2001, "Stream Stability at Highway Structures", Third Edition, Publication No. FHWA NHI 01-002, Hydraulic Engineering Circular No. 20, Federal Highway Administration, Washington, D.C.

### 6.4.4.9 Flow Controls

In addition to existing structures and roadways discussed in Section 6.4.4.4, other flow controls may exist upstream and/or downstream of the site. The controls must be identified in order to include their effects in the hydraulic analysis.

**Dams and Reservoirs -** Any existing dams or reservoirs near the site should be noted. Spillway elevations and design levels of operation should be obtained. These controls can affect water surface elevations and sediment transport at the project site. Upstream control of runoff in the watershed should also be noted. Conservation and/or flood control reservoirs in the watershed may effectively reduce peak discharges at the site and may also retain some of the watershed runoff. Capacities and operational designs for these features should be obtained. NRCS, USACE, USBR, Water Resources Dam Safety, consulting engineers and other reservoir sponsors often have complete reports concerning the operation and design of proposed or existing conservation and/or flood control reservoirs.

Installation of a nearby dam during the design life of a proposed structure can affect the crossing's hydraulic performance. If it appears a dam may be built, it should be noted on the location data. Upstream from a dam, the water surface elevations in the impounded reservoir can raise elevations and lower flow velocities in the contributing tributaries. This can lead to increased sediment buildup in the reservoir and at the mouths of the tributaries. Downstream from a dam, the peak discharges and sediment transport can be reduced due to storage of both water and sediment in the reservoir. This can cause a lowering of the channel bottom.

Removal of an upstream or downstream obstruction such as a dam or weir can also affect the hydraulic conditions at a crossing. It should be verified that nearby dams will remain in-place during the design life of the proposed structure. In recent years it is not uncommon for man-made dams or weirs to be removed from streams or rivers. Usually these are smaller dams that do not provide significant flood control. Removal of these dams can restore natural sediment transport to a stream, and result in a loss of sediment upstream from the dam and a corresponding increase in sediment downstream.

**Diversions –** Diversions upstream from a structure can affect the discharges through the site. Significant diversions upstream from the site should be noted. Any anticipated changes to the diversions should also be noted. Diversions can be artificial or natural.

Artificial diversions include flood control facilities, irrigation facilities, or undersized highway or railroad crossings. Flood control diversions can occasionally make an existing highway crossing obsolete or require a new crossing. Undersized culverts or bridges that have insufficient hydraulic capacity can divert flood flows. There may be an increase in peak flood flows downstream when these structures are replaced with hydraulically adequate or fish-passable crossings. The owners of the diversions are the best source for information about them.

Natural diversions include debris jams, mudflows, rock slides, or beaver dams. These diversions may be permanent or temporary. Diversions that divert flow onto developed property may be removed. Maintenance personnel and local officials are good sources for information about natural diversions and their possible removal.

Natural diversions also occur in rivers and streams on alluvial fans. This happens most often in the Columbia River Gorge and the slopes of Mount Hood. These streams periodically undergo natural diversions when a channel fills with sediment and the stream shifts to a new channel. In these areas all stream crossings on the fan are usually designed to convey the entire stream discharge, regardless of whether or not the stream is currently using the crossing. Also, some crossings need to be raised periodically when the channel fills with sediment. Region geologists and maintenance personnel are good sources of information about these sites.

**Downstream Water Bodies –** Downstream water bodies such as rivers, lakes, estuaries, or the ocean can often influence water surface elevations at highway crossings, as discussed in **Chapter 10**. These nearby water bodies should be described in the location notes. In some cases, the downstream water body needs to be included in the hydraulic model, and additional survey information will be needed, such as cross-sections or profiles. The hydraulic designer should be contacted to see if additional data is needed.

A crossing is sometimes located on a tributary just upstream from its confluence with a larger river or stream. The channel bottom of the larger waterway should be inspected downstream from the tributary mouth. Headcuts or channel degradation in larger waterway may progress upstream into the tributary and cause the channel bottom to drop at the crossing. This possibility, if it is likely to occur, should be mentioned in the hydraulic survey. Headcuts are discussed in **Chapter 9**.

**Geologic Controls –** An important aspect of hydraulic design is to predict future changes in channel alignment, both in the horizontal and vertical planes. Geologic controls such as rock ledges across the channel bottom can influence channel movement in the vertical plane. Hard rock outcroppings can influence lateral movement in the horizontal plane. These ledges and outcroppings should be noted if they are within the hydraulic survey limits.

#### 6.4.4.10 Bed Material

Bed material is the sediment mixture found in and on the streambed. Knowledge of the size and gradation of the bed material is necessary for most sediment transport analyses, including incipient motion analysis, armoring potential of the channel bed, sediment transport capacity, and scour calculations. The most common sediment characteristic needed is the median, $D_{50}$, sediment size. The size and gradation of the bed material can be determined with a sieve analysis of a bed sample or by using the pebble count method. The sieve analysis is appropriate

when the bed material size ranges from 0.002 – 1.25 inches. The pebble count method is used to obtain the size distribution of coarse bed materials (i.e., gravel and cobbles) that are too large to be sieved and is used when the bed material size ranges from 0.5 – 40 inches. A sieve analysis can be obtained by submitting a sample to the Materials Section. Appendix A explains several methods to determine bed material size and gradation.

#### 6.4.4.11 Ordinary High Water

The following definition was obtained from the Division of State Lands. "The Ordinary High Water (OHW) mark is a line on the bank or shore to which the high water ordinarily rises each year and is the waterward limit of upland vegetation and soil. This line is not established based on the level to which the water rises during major floods. It is generally recognizable by a visible change in the soil and vegetation." OHW is used by regulatory agencies to define the boundary of in-water work. Any work below the OHW elevation is considered to be in-water work and special measures must be taken to protect the waterway.

OHW corresponds to the elevation or stage where water just begins to overflow onto the floodplain for streams with well-defined floodplains. OHW will be contained in the channel and field stage indicators will need to be used to determine OHW for streams that are entrenched in the landform. Field stage indicators must also be used at sites where floodplains are not well developed. Following are field indicators of OHW. These are the same indicators geomorphologists use to determine bankfull stage. As many corroborating features as possible should be used to determine the OHW elevation.

- The elevation of the floodplain. This is the most reliable indicator of OHW.
- The lowest extent of woody vegetation in the bank or where aquatic vegetation changes to terrestrial vegetation. Willows can be misleading as willows may become established below OHW, especially during periods of drought or low flow. Certain mature species of birch, dogwood, and alder tend to consistently colonize and become established at levels very close to OHW.
- The elevation associated with the top of the highest depositional features (e.g., point bars, central bars within the active channel). This is typically considered the lowest elevation to be considered as OHW.
- A break in the slope of the banks and/or a change in the particle size distribution.
- The top of the zone of washed roots (exposed root hairs below an intact soil layer indicating exposure to erosive flow).
- Staining of rocks and lower limit of lichen or moss growth on rocks.
- Stains on bridge piers or culverts.
- Evidence of inundation features such as small benches.
- The elevation of flood deposited debris if corroborated by other indicators. High water marks are used to confirm OHW and should not be used as the primary evidence.

It is important to know the recent flood and/or drought history of the area to avoid being misled by false indicators such as colonization of riparian species below OHW during drought. A recent flood can give the impression that OHW is higher than it actually is.

The most common means to establish OHW is by the field marks previously discussed. These marks are used when OHW elevations are staked in the field by the biologist or hydraulics designer. The requirements for field assessment, flagging, and documentation of the OHW are outlined in ODOT Geo-Environmental Technical Bulletin GE09-07(B). A link to this bulletin is provided below:

ODOT Geo-Environmental Technical Bulletin GE09-07(B)

Occasionally OHW can be determined or verified by calculation. This is often the best method where field marks have been obliterated by a recent flood, or where there is an absence of appropriate field indicators. This is done by determining the 2-year event profile. This is the typical recurrence interval of the OHW. Stage-discharge relationships from nearby stream gages can also be used to calibrate or verify OHW elevations based on field marks.

Published data can also provide OHW elevations. As an example, OHW elevations for the Portland-Vancouver Harbor are available in a published study. These elevations are often acceptable to regulatory agencies. A critical aspect of using published elevations is to convert them to the project elevations while considering any differences in reference datums. The ODOT project elevation datum is often different than the published study elevation datum.

### 6.4.5 Hydrologic and Meteorologic Data

Hydrologic and meteorologic data includes historic streamflow and precipitation data for the watershed as well as low flow data. The data is needed to estimate the hydrology of a site using the methods presented in **Chapter 7**. The major source of stream flow data is the USGS. Stream flow data for Oregon is also available from the Oregon Water Resources Department (OWRD). The major source of meteorologic data is the National Weather Service (NWS) and the National Climatic Data Center. The Oregon Climate Service is also a source of meteorological data for Oregon.

## 6.5 Field Reviews

Field reviews should be made by the hydraulic designer in order for the designer to become familiar with the site. It also allows the designer to determine required cross-section locations and other survey requirements unique to the site. The most complete survey data cannot adequately depict all site conditions or substitute for personal inspection by someone

experienced in drainage design. Site and watershed characteristics that most often need to be confirmed or determined by field inspection are:

- selection of roughness coefficients,
- evaluation/observation of flow patterns, flow diversions, flow concentration, and likely flood patterns,
- watershed characteristics including land use and watershed boundaries,
- high-water marks or profiles,
- existing bridge/culvert dimensions and features (e.g., culvert size and material, culvert inlet and outlet geometry, etc.)
- drift/debris characteristics,
- location and length of roadway overtopping,
- evaluation of channel stability (e.g., signs of active bank erosion, observation of headcuts, potential for channel migration, etc.)
- location and existence of flow controls,
- location and size of overflow structures,
- size and limits of existing riprap protection,
- existence of pier and/or abutment scour problems and scour countermeasures,
- size of bed material (i.e., obtain bed sample for sieve analysis or conduct pebble count),
- existence of upstream and/or downstream structures that need to be included in the hydraulic analysis, and
- location of buildings or structures that may effect the allowable backwater elevation.

The visit to the project site should be made before final hydraulic design is undertaken. If possible, this should be combined with the scoping trip involving other disciplines involved in the project design (e.g., the roadway and structural designers, environmental specialists/coordinators, regulatory agencies, local officials, etc.). As much information as possible about the site should be gathered in the office prior to the site visit. The site visit is also an excellent time to meet onsite with maintenance personnel in order to get their input on the flood history and maintenance requirements unique to the site. It is also an opportunity to meet with the survey party chief responsible for the drainage survey to go over the survey data requirements.

Photographs should be taken. At a minimum, photos should be taken looking upstream and downstream from the site and along the contemplated highway centerline in both directions. Details of the streambed and banks should also be photographed plus structures in the vicinity both upstream and downstream. Close-up photographs complete with a scale or grid can be taken to facilitate estimates of the streambed gradation. Site photographs are invaluable if a lawsuit ever develops. The photographer should be identified on the photo submittal. They may need to provide information to legal counsel.

## 6.6 Drainage Survey Requirements

Complete and accurate survey information is necessary to develop a design that will best serve the requirements and characteristics of the site. Ideally, the individual in charge of the drainage survey should have a general knowledge of drainage design. The amount of survey data gathered shall be commensurate with the importance and cost of the proposed structure and the expected flood hazard. The hydraulics designer should meet onsite with the survey party chief to go over the required cross section locations and other unique data requirements of the site. This will ensure that the survey will be cost effective, gather the necessary data and avoid repeat visits to the site by the surveyors to gather additional data. Appendices B, C, and E outline the data requirements for bridges, culverts, and storm drains, respectively.

At some sites, photogrammetry may be effective in supplementing the topographical components of the ground survey. When using photogrammetry, a ground survey is required to provide data in areas obscured on the aerial photos (e.g., underwater and heavy vegetation).

## 6.7 Data Evaluation

Once the needed data have been collected, the next step is to compile it into a usable format. The designer must ascertain whether the data contains inconsistencies or other unexplained anomalies that might lead to erroneous calculations or results, draw all of the various pieces of collected information together, and fit them into a comprehensive and accurate representation of the hydrologic and hydraulic characteristics of the project site.

Experience, knowledge and judgment are important parts of data evaluation. Reliable data must be separated from that which is less reliable and historical data combined with that obtained from measurements. When combining topographical data from previous surveys and current surveys, the designer should verify that both surveys used the same vertical datum. Data should be evaluated for consistency and to identify any changes from established patterns. The review may include such items as previous studies and old plans for types and sources of data, how the data were used and any indications of accuracy and reliability. Historical data should be reviewed to determine whether significant changes have occurred in the watershed and whether these data can be used. Data acquired from the publications of established sources such as USGS can usually be considered as valid and accurate. Maps, aerial photographs, and land-use studies must be compared with one another and with the results of the field survey and any inconsistencies resolved.

### 6.7.1 Sensitivity Analysis

Sensitivity analysis may be used to evaluate data and establish the relative importance of specific data items to the final design. Sensitivity analysis consists of conducting a design with a range of values for specific data items. The effect on the final design can then be established. This is useful in determining what specific data items have major effects on the final design and the importance of possible data errors. Time and effort should then be spent on the more sensitive data items making sure these data are as accurate as possible. This does not mean that inaccurate data are accepted for less sensitive data items, but it allows prioritization of the data collection process given a limited budget and time allocation.

### 6.7.2 Data Accuracy

In any engineering computations, it is important to understand the limitations of accuracy of the computations based on the accuracy of the input data. In step-backwater computations utilizing HEC-RAS or WSPRO, there are several factors that have significant effects on the accuracy of the results — accuracy of the survey data, spacing between cross sections, correct establishment of upstream and downstream study limits, and selection of roughness coefficients.

Most field surveys of channel and floodplain cross sections are recorded to an accuracy of 0.1 foot. If the survey truly represents the cross sections of the reach of the stream being studied to a 0.1 foot accuracy, the greatest accuracy that would result from a step-backwater computation could be no more than 0.1 foot. Any results from channel and floodplain analysis expressed more precisely than 0.1 foot are simply due to the mathematics.

More precision is needed for survey data used to analyze and design roadway drainage, the structural components of stormwater treatment facilities, and other hydraulic structures. ODOT roadway and structural survey guidelines can be used in most applications. In situations the ODOT guidelines do not address, an accuracy of 0.01 feet is usually sufficient.

The accuracy of aerial survey technology for generating cross sectional coordinate data is governed by mapping industry standards. Cross sections obtained from contours of topographic maps developed by photogrammetric methods are generally not as accurate as those generated from field data collection methods. Aerial photography can supplement field survey cross sections. The use of aerial elevation survey technology permits additional coordinate points and cross sections to be obtained at small incremental cost, and the coordinate points may be formatted for direct input into commonly used water surface profile computer programs such as HEC-RAS and WSPRO.

For further information on determining the relationships between survey technology and accuracy employed for determining stream cross sectional geometry, degree of confidence in selecting Manning’s roughness coefficients, and the resulting accuracy of hydraulic

computations, refer to the USACE publication prepared for the Federal Highway Administration, “Accuracy of Computed Water Surface Profiles”, and Technical Paper No. 114, “Accuracy of Computed Water Surface Profiles Executive Summary”, December 1986. These publications also present methods of determining the upstream and downstream limits of data collection for a hydraulic study requiring a specified degree of accuracy. Computer software has been developed to perform the calculations for the various routines presented in these publications. “Preliminary Analysis System” (PAS) is available from the McTrans Center, University of Florida, Gainesville, FL.

# APPENDIX A
# STREAMBED MATERIAL ANALYSIS METHODS

Many methods are used to determine streambed material particle size and gradation. Several methods are discussed in this appendix.

**Sand Gauge Reference Card -** This method is best suited for sandy channel bottoms. The reference cards are small laminated plastic cards showing samples of different sand gradations. The samples are labeled, typically as coarse sand, medium sand, fine sand, etc. The card is placed on the streambed and the bed material is visually matched with a sample. The sand classification is considered to be the sample that best matches the sand.

**Laboratory Sieve Test -** This procedure is best suited for non-cohesive gravel, sand, and silt channel bottoms. Fine materials such as silt should not comprise more than ¼ of the sample. The method is to take a representative sample and perform American Association of State Highway and Transportation Officials (AASHTO) Test T-27 "Sieve Analysis of Fine and Coarse Aggregates." ODOT uses this method to test asphalt concrete aggregate, Portland cement concrete aggregate, and road base aggregate. Equipment to test samples with particle size gradations of 1-1/2 inch or less is fairly common throughout the state. A 33 pound sample, minimum, would be needed for a single test of bed material with a 1-1/2 inch maximum size. Smaller samples are used for finer materials. For example, an 11 pound sample (minimum) is adequate for bed material with a ¾ inch maximum size.

**Wire Screen Method –** This method is best suited for gravel channel bottoms. A wire screen with a square 1-inch mesh opening size is placed on the channel bottom. The gravel size is visually estimated using the mesh opening size as a reference. For example, a pebble that fills about ¼ of the mesh opening has a diameter of approximately ½ inch. Photographs of the screen over the bed material are a useful reference when discussing particle size and gradation.

**Pebble Count Method** – This is the best method for stream bottoms with larger stone sizes such as boulders, cobbles, gravels, etc. The pebble count method is a field method for determining the particle size distribution or gradation of streambed material. To determine the gradation, 100 particles are randomly selected for measurement. To avoid potential bias, the actual particle picked up for measurement must be selected on the "first blind touch", rather than seen and selectively picked up. The intermediate axis of the particle is measured. The measured particle sizes are then grouped together according to the sizes shown in Table 1. The pebble count data is then plotted on a log-normal graph as shown in the example. The $D_{50}$ size or other required particle size is selected from the graph.

**Table 1: Particle Class vs. Size**

| Particle Class | Size (inches) |
|---|---|
| Silt/Clay | Less than 0.002 |
| Fine Sand | 0.002 – 0.01 |
| Medium Sand | 0.01 – 0.02 |
| Coarse Sand | 0.02 – 0.04 |
| Very Coarse Sand | 0.04 – 0.08 |
| Fine Gravel | 0.08 – 0.32 |
| Medium Gravel | 0.32 – 0.63 |
| Coarse Gravel | 0.63 – 1.26 |
| Very Coarse Gravel | 1.26 – 2.51 |
| Small Cobbles | 2.51 – 5.0 |
| Large Cobbles | 5.0 – 10.1 |
| Small Boulders | 10.1 –20.2 |
| Medium Boulders | 20.2 – 40.3 |
| Large Boulders | 40.3 – 80.6 |
| Very Large Boulders | 80.6 – 161 |

Pebble Count Example

This example illustrates how to determine the particle size distribution or gradation of streambed material from a pebble count. Table 2 summarizes the raw data that was obtained by measuring the intermediate axis of 100 randomly selected particles in the field.

**Table 2: Pebble Count Data**

| Count No. | Size (inches) | Count No. | Size (inches) | Count No. | Size (inches) | Count No. | Size (inches) | Count No. | Size (inches) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 5 | 21 | 7/8 | 41 | 11/16 | 61 | 2 3/8 | 81 | 1 3/4 |
| 2 | 3 | 22 | 4 | 42 | 2 5/8 | 62 | 1 | 82 | 2 |
| 3 | 4 5/8 | 23 | 15/16 | 43 | 2 | 63 | 14 | 83 | 3 1/8 |
| 4 | 2 | 24 | 3 3/4 | 44 | 1 13/16 | 64 | 3/4 | 84 | 1 9/16 |
| 5 | 1 | 25 | 1 1/4 | 45 | 1 9/16 | 65 | 1 1/2 | 85 | 2 3/8 |
| 6 | 4 1/2 | 26 | 1 1/8 | 46 | 3 | 66 | 3 1/4 | 86 | 1 1/8 |
| 7 | 7 1/2 | 27 | 3 1/4 | 47 | 2 1/4 | 67 | 7/8 | 87 | 3 3/8 |
| 8 | 1 7/8 | 28 | 2 | 48 | 5/8 | 68 | 2 3/16 | 88 | 1 3/16 |
| 9 | 1 5/8 | 29 | 1 9/16 | 49 | 8 | 69 | 2 | 89 | 2 1/4 |
| 10 | 2 3/16 | 30 | 3 | 50 | 1 1/4 | 70 | 1 11/16 | 90 | 3/4 |
| 11 | 1 13/16 | 31 | 2 1/4 | 51 | 1 7/8 | 71 | 11/16 | 91 | 1 7/8 |
| 12 | 4 1/2 | 32 | 1 3/4 | 52 | 2 1/4 | 72 | 3 1/4 | 92 | 7/8 |
| 13 | 2 3/8 | 33 | 2 1/2 | 53 | 7/8 | 73 | 15/16 | 93 | 2 1/16 |
| 14 | 1 1/2 | 34 | 3 3/16 | 54 | 2 3/4 | 74 | 2 1/4 | 94 | 1 5/8 |
| 15 | 1 1/4 | 35 | 1 3/8 | 55 | 1 3/4 | 75 | 5/8 | 95 | 5 1/8 |
| 16 | 4 1/8 | 36 | 1 | 56 | 2 3/8 | 76 | 1 3/8 | 96 | 1 1/2 |
| 17 | 1 3/8 | 37 | 8 | 57 | 1 1/8 | 77 | 3 3/8 | 97 | 3 3/4 |
| 18 | 1 3/4 | 38 | 2 1/4 | 58 | 1 5/8 | 78 | 5/16 | 98 | 1 |
| 19 | 2 1/4 | 39 | 3/4 | 59 | 3 | 79 | 11/16 | 99 | 2 1/2 |
| 20 | 1 11/16 | 40 | 1 1/2 | 60 | 2 1/4 | 80 | 2 7/16 | 100 | 1 1/4 |

The raw data is then separated into the various particle classes in order to determine the particle size distribution shown in Table 3.

**Table 3: Particle Size Distribution**

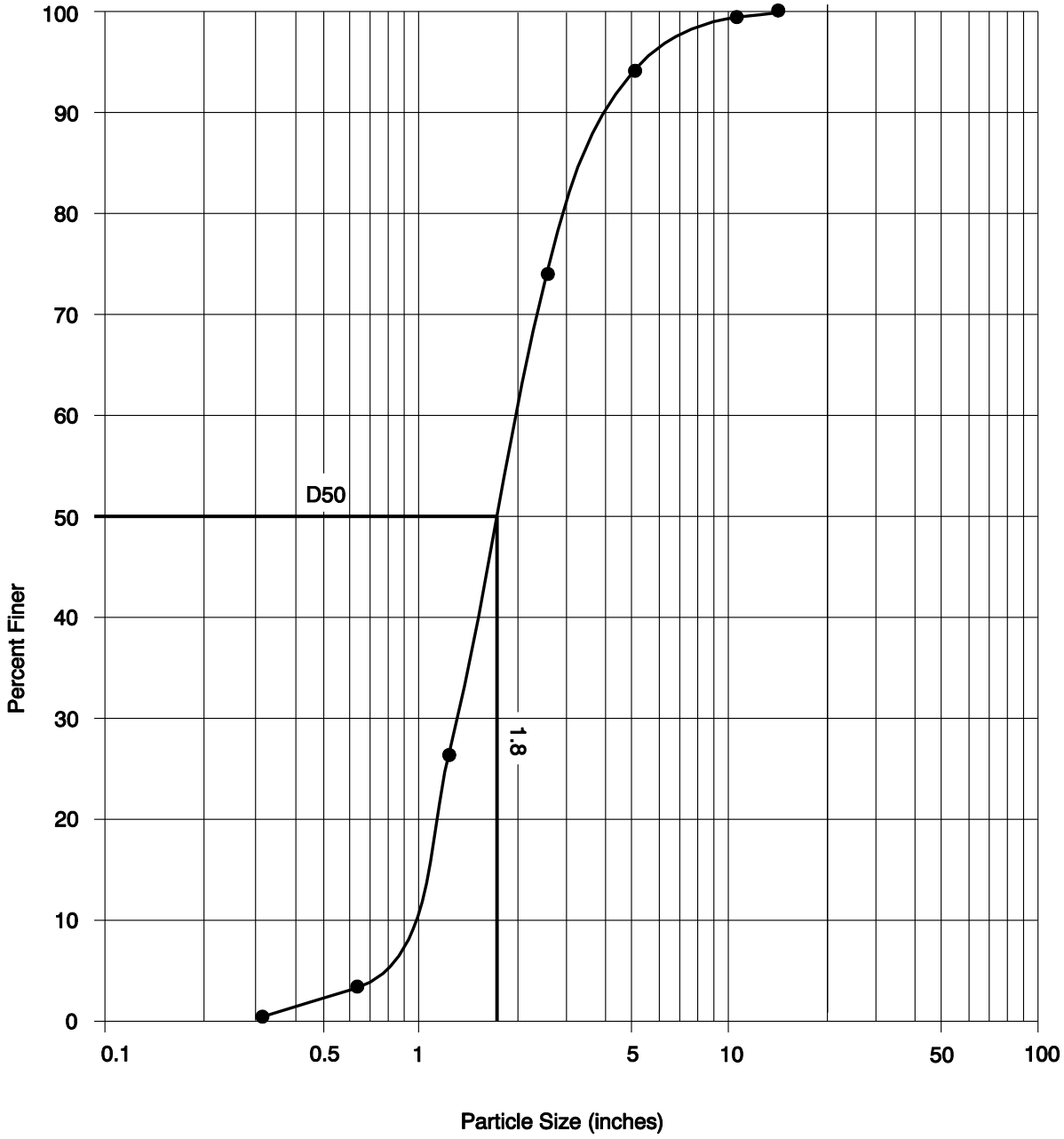
| Particle Class | Size (inches) | Count | Percent Total | Percent Cumulative |
|---|---|---|---|---|
| Silt/Clay | Less than 0.002 | 0 | 0 | |
| Fine Sand | 0.002 – 0.01 | 0 | 0 | |
| Medium Sand | 0.01 – 0.02 | 0 | 0 | |
| Coarse Sand | 0.02 – 0.04 | 0 | 0 | |
| Very Coarse Sand | 0.04 – 0.08 | 0 | 0 | |
| Fine Gravel | 0.08 – 0.32 | 1 | 1 | 1 |
| Medium Gravel | 0.32 – 0.63 | 2 | 2 | 3 |
| Coarse Gravel | 0.63 – 1.26 | 24 | 24 | 27 |
| Very Coarse Gravel | 1.26 – 2.51 | 47 | 47 | 74 |
| Small Cobbles | 2.51 – 5.0 | 20 | 20 | 94 |
| Large Cobbles | 5.0 – 10.1 | 5 | 5 | 99 |
| Small Boulders | 10.1 – 20.2 | 1 | 1 | 100 |
| Medium Boulders | 20.2 – 40.3 | 0 | 0 | |
| Large Boulders | 40.3 – 80.6 | 0 | 0 | |
| Very Large Boulders | 80.6 – 161 | 0 | 0 | |

The data listed in Table 3 is summarized in Table 4. A curve through the data points listed in Table 4 is shown in Figure 1. The $D_{50}$ or other particle sizes of interest can be obtained from Figure 1. For example the $D_{50}$ is 1.8 inches.

**Table 4: Particle Size vs. Percent Finer**

| Particle Size (inches) | Percent Finer |
|---|---|
| 14 | 100 |
| 10.1 | 99 |
| 5.0 | 94 |
| 2.51 | 74 |
| 1.26 | 27 |
| 0.63 | 3 |
| 0.32 | 1 |

**Figure 1: Stream Bed Gradation Curve**



# APPENDIX B
# TOPOGRAPHIC DATA REQUIRED FOR BRIDGE BACKWATER ANALYSIS

## 1.0 Introduction

This appendix discusses the field data that is required to conduct a bridge backwater analysis. This data is required for bridge replacements, bridge widening, and new bridge crossings. The hydraulics designer should meet onsite with the survey party chief to go over the required cross section locations and other unique data requirements of the site. This will ensure that the survey will be cost effective, gather the necessary data and avoid repeat visits to the site by the surveyors to gather additional data.

## 2.0 Digital Terrain Model

A method of obtaining hydraulic survey data using cross-sections and profiles is described. An alternative to surveying individual cross-sections is to develop a digital terrain model (DTM) for the project site. The structure survey guidelines in the ODOT Highway Design Manual are typically used for the DTM, and they are supplemented by additional hydraulic survey data as needed. Usually the DTM covers the project vicinity, and profiles and individual cross-sections are used beyond the DTM coverage. Care should be used to assure that the DTM and the remainder of the hydraulic survey are compatible. As an example, both surveys should use the same stream profile stationing and elevation datum.

The DTM should extend far enough upstream and downstream to include the bridge construction, detour structures, access roads, erosion control features, and temporary water management. Other features to be shown on a DTM are property and easement boundaries, trees and large bushes to be preserved, and utilities. Experience shows that a DTM extending from the nearest downstream cross-section through the approach section is usually enough. The nearest downstream section, full valley, and approach sections are all “cut” from the DTM. The water surface profiles and cross-sections beyond the DTM limits are surveyed individually.

The structures and irregular terrain at most water crossings are a challenge to accurately model with a DTM. It is important to construct the DTM based on available survey data and plot the vicinity map, cross-sections cut from the DTM, and the stream profiles. These drawings should be taken to the site and compared to the existing terrain. Often additional data points will be needed at critical locations, and the model will need to be adjusted so it represents actual conditions. These quality control steps should be completed before the model is submitted to the designers.

## 3.0 Required Field Data

### 3.1 General

a. Vertical Reference should be identified. All brass disks within project limits should be referenced and identified. Surveys in FEMA floodplains need to tie the reference marks used in the Flood Insurance Study to the project survey. The hydraulics designer can supply the locations and descriptions of the reference marks. The preferred vertical reference hierarchy is:

- known benchmark,
- assumed datum (as shown on plans), or
- other assumed datum.

b. Site Sketch should be prepared. The sketch should include north arrow, stream alignment, highway alignment, cross-section locations, horizontal control and turning angles of cross-sections, hydraulic structure(s), skew angle of each hydraulic structure(s) with respect to flow, buildings, scour holes at existing bridge piers, and other unique features at the site. The sketch should also show right-of-way and easement boundaries, utilities, and trees or bushes to be preserved.

c. Photographs. Each photograph should include a description of the item(s) shown. Record the name and address of the photographer and the date the photos were taken. Take photographs, either film or digital, of:

- Existing structure. This photo should show the waterway opening provided by the structure.
- Overflow structure(s). This photo should show the waterway opening provided by the structure(s).
- Channel upstream and downstream from structure. Take a photo looking upstream and a photo looking downstream while standing on the bridge or roadway.
- Overflow channels upstream and downstream from the overflow structure(s). Take a photo looking upstream and a photo looking downstream while standing on the overflow structure.
- Channel and floodplain upstream and downstream from structure at all cross-sections. Show features needed to estimate roughness, such as channel bottom, sides, vegetation, etc.
- Roadway overflow areas.
- Erosion or scour problem areas.
- Approaches to bridge. Take a photo looking ahead on line and a photo looking back on line from the bridge.
- Locations where floor elevations are obtained.
- Hydraulic controls, such as rock outcrops, rock ledges, weirs, dams, etc. (See chapter text.)

- Developed property in or adjacent to the waterway, such as pump intakes, retaining walls, fences, pedestrian bridges, structures, etc. These photos will record the condition of the facilities before the project is constructed. They may be valuable if there is future litigation.
- Ordinary High Water elevation marks. Usually these elevations are located and staked by the biologist or the hydraulic designer.

d. Describe significant debris accumulation near the bridge or in the channel within project limits.

e. Obtain floor elevations and locations of upstream houses and buildings.

## 3.2 Bridge/Highway

a. Roadway

- Obtain roadway profile; the profile should reflect the highest part of the roadway and also define the overtopping limits or the limits of the Full Valley Cross-Section (see subsection 3.3).

  At many sites, water will overtop at other places before it overtops the roadway centerline at the bridge. A profile is needed across the apex of the location where the water starts to overflow. An example is a bridge on a roadway with a constant grade. Overtopping occurs adjacent to the upstream face of the bridge, it flows down alongside the road to a nearby bridge several hundred feet away, then it crosses under the road in the nearby stream. A profile would be needed across the high point in the roadside ditch near the upstream face of the bridge where overtopping initially occurs.

b. Existing Bridge

- Obtain a minimum of three (3) deck elevations at locations corresponding with spot elevations on the plans. If no elevations are identified on the plans, obtain gutter elevations on each corner of the structure.
- Obtain top-of-footing and bottom-of-footing elevations if exposed.
- Define limits of existing scour holes with cross sections or sketches.
- Obtain cross-section of downstream and upstream bridge waterway opening. Obtain multiple waterway ground points, including underwater ground points, sufficient to accurately define the channel bottom. The cross-sections should include the bottom-of-beam elevation at each bent. Note that the bottom-of- beam elevation refers to the longitudinal beams and not the cross beams which support the longitudinal beams. The cross-sections should define the location of the abutments and the location and width of any intermediate bents. Also note the shape of the bents.

- Define the skew angle relative to the flow direction of the abutments and intermediate bents.
- Obtain the same data for overflow structures if any exist within the limits of the Full Valley Cross-Section.

### 3.3 Full Valley Cross-Sections

a. Located on the upstream and downstream side of the structure at the toe-of-slope and/or beyond the abutment wingwalls (if present). The intent is to capture what the natural channel looked like before the bridge and roadway embankments were constructed.

b. Oriented parallel to roadway alignment.

c. Skew angle relative to the flow direction should be the same as the bridge.

d. Obtain multiple channel ground points, including underwater ground points, sufficient to accurately define the channel bottom.

e. Width. The cross-section should extend across the full valley width, which includes the channel and adjacent floodplain.

f. The following guidelines describe the <u>minimum</u> cross-section widths that are usually adequate for the hydraulic analysis; however, there may be exceptions.

- For well-defined floodplain boundaries, obtain floodplain ground points high enough to extend to roadway overtopping elevation.

- For relatively flat and expansive floodplain, survey ten (10) bridge lengths (total maximum width, centered on bridge). If the floodplain is flat or at a constant grade beyond last surveyed point, indicate this on field notes.

### 3.4 Stream Cross-Sections

a. Width. Refer to subsection 3.3 e.

b. Should be taken perpendicular to the direction of flow. Note that perpendicular to the flow in the floodplain may differ from perpendicular to the flow in the channel in which case the section will have a “dogleg” and the direction of the section in floodplain will be different than in the channel. Left and right for the cross-sections are defined by looking downstream.

c. Should define limits of vegetation types. Note where vegetation changes from grass to brush, etc. This can be identified in the cross-section notes.

d. Should be taken at locations as shown in Figures 1 to 7. Additional downstream sections may be necessary to establish downstream controls that will influence the water surface profile. Highly irregular channels may require additional up and downstream cross sections. The cross-section locations shown in Figures 1 to 7 assume subcritical flow. Additional cross-sections are required upstream for supercritical flow.

## 3.5 Stream Profile

a. The profile should extend a sufficient distance upstream and downstream from the project site to accurately determine the slope of the channel.

b. Should extend upstream and downstream from the bridge centerline for the distances listed in the Stream Profile Chart (see Section 6.4.4.2). Additional profile length may be needed in some cases to satisfy subsection 3.5 a. Elevations should be located along the channel thalweg (lowest point in channel) and be taken every 50 feet and at significant channel bottom grade breaks. To orient the cross-sections looking downstream, stationing should run from upstream to downstream.

c. Should include stream bottom profile and water surface profile at time of survey.

d. Include past flood profiles if evidence from previous high water is apparent.

e. Include Ordinary High Water elevation marks. Usually these elevations are located and staked by the biologist or the hydraulic designer.

## 3.6 Miscellaneous Structures

a. Bridges within 500 feet or two bridge lengths (whichever is further) of project. Obtain one full valley section on downstream side of bridge, downstream waterway opening including bottom-of-beam elevation, and roadway profile.

b. Obtain elevations and dimensions of miscellaneous structures such as irrigation dams or weirs within 500 feet or two bridge lengths (whichever is further) of project.

## 3.7 Local Knowledge of Past Floods

a. Source of historic flood information is from local residents or maintenance personnel. If local knowledge is obtained, give name and phone number of contact, otherwise give name and phone number of people that can be contacted later.

b. Highest water elevation and date of occurrence. If a high-water elevation is available, the exact location and elevation of the high-water mark should be surveyed.

c. Is roadway overtopped? If so, obtain depth of water over road and length of roadway overtopped.

d. Any debris or ice problem?

e. Photos of past floods?

SITUATIONS

The following figures show typical situations encountered and location of cross-sections needed for the hydraulic analysis:

- Figure 1: Single Opening Stream Crossing With No Skew
- Figure 2: Single Opening Stream Crossing With Mild Skew
- Figure 3: Single Opening Stream Crossing With Large Skew
- Figure 4: Single Opening Stream Crossing With Skew and Curved Channel Alignment
- Figure 5: Multiple Openings
- Figure 6: Confluence of Two Tributaries
- Figure 7: Dual Bridges

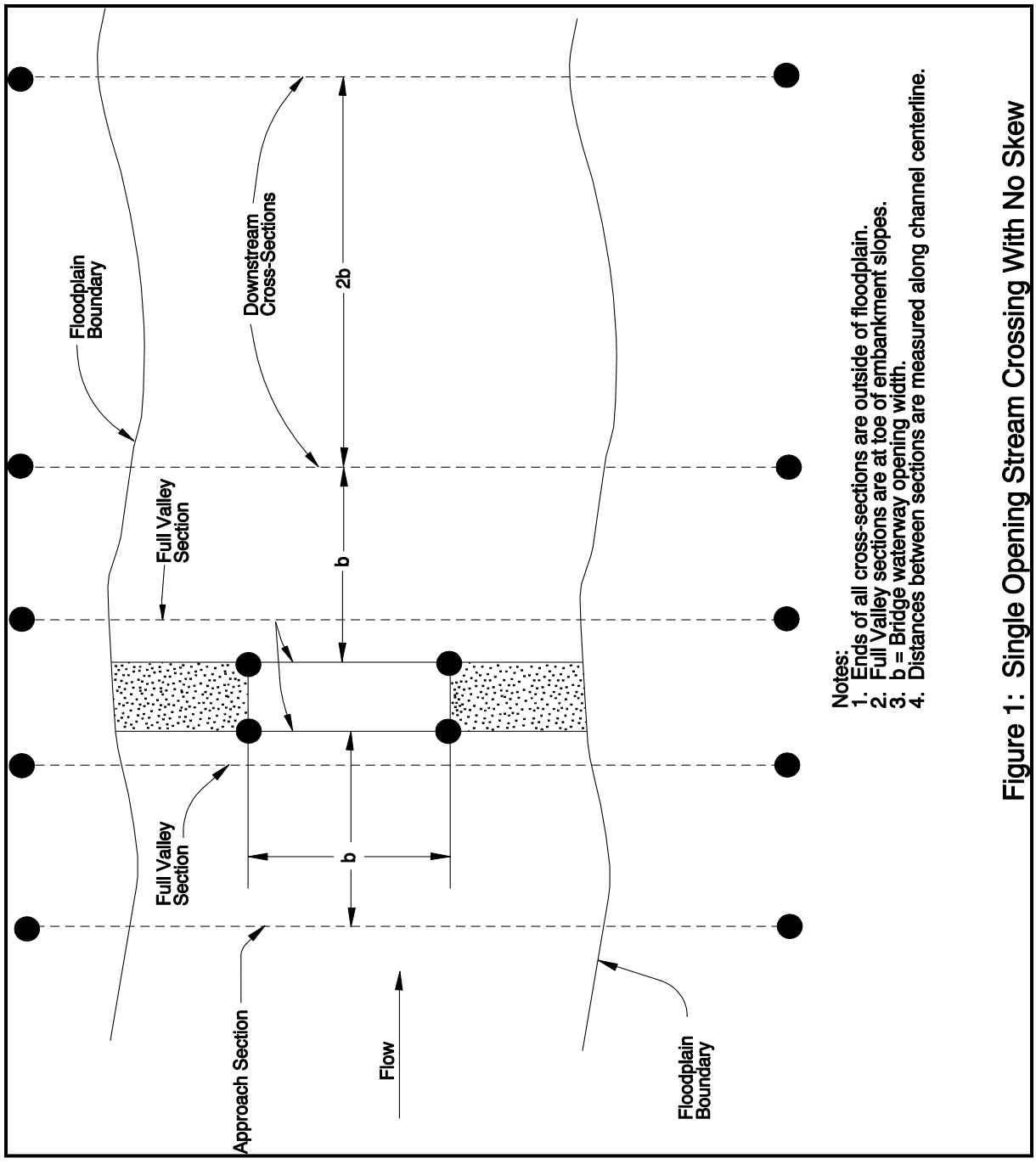



Figure 1: Single Opening Stream Crossing With No Skew

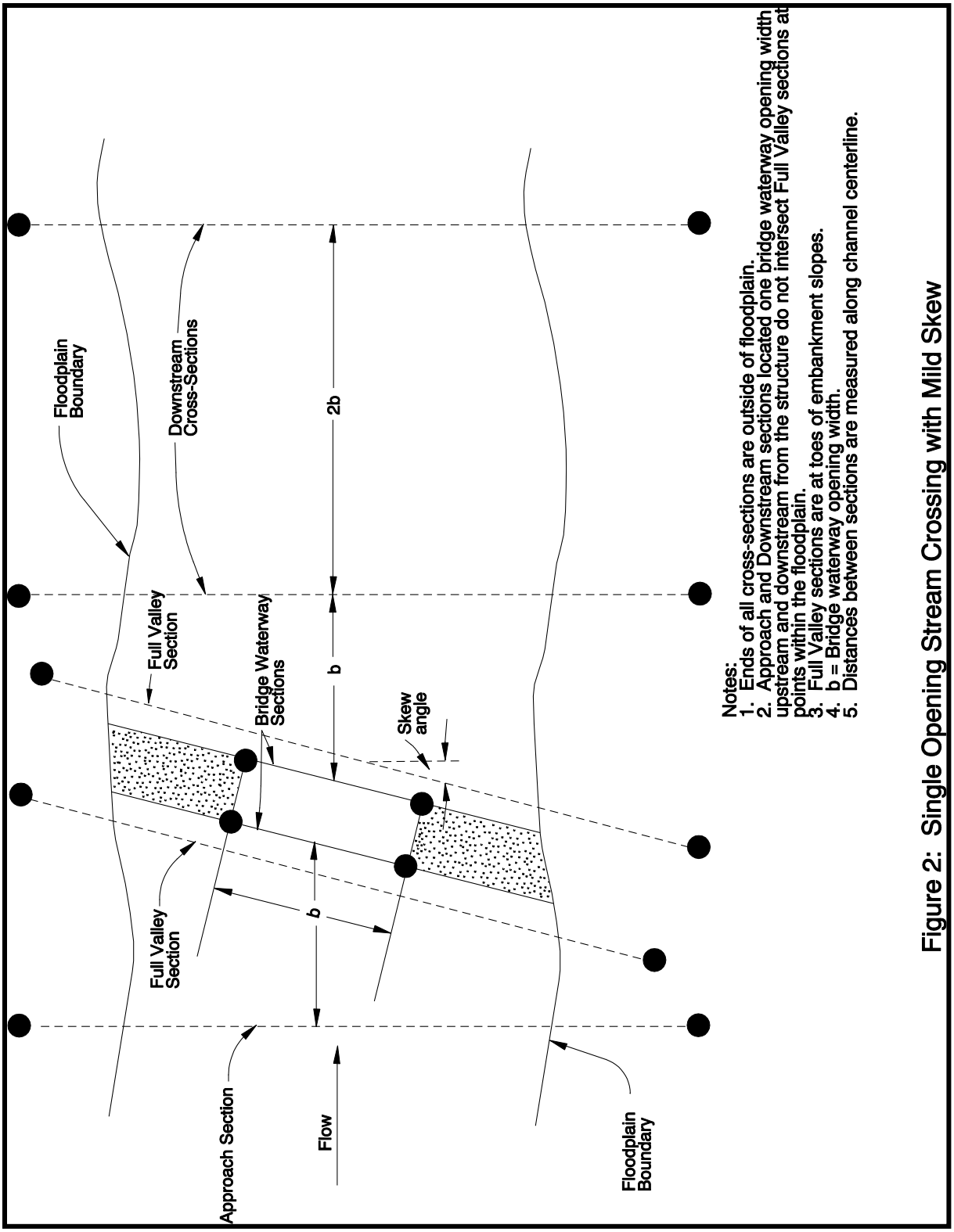



Figure 2: Single Opening Stream Crossing with Mild Skew

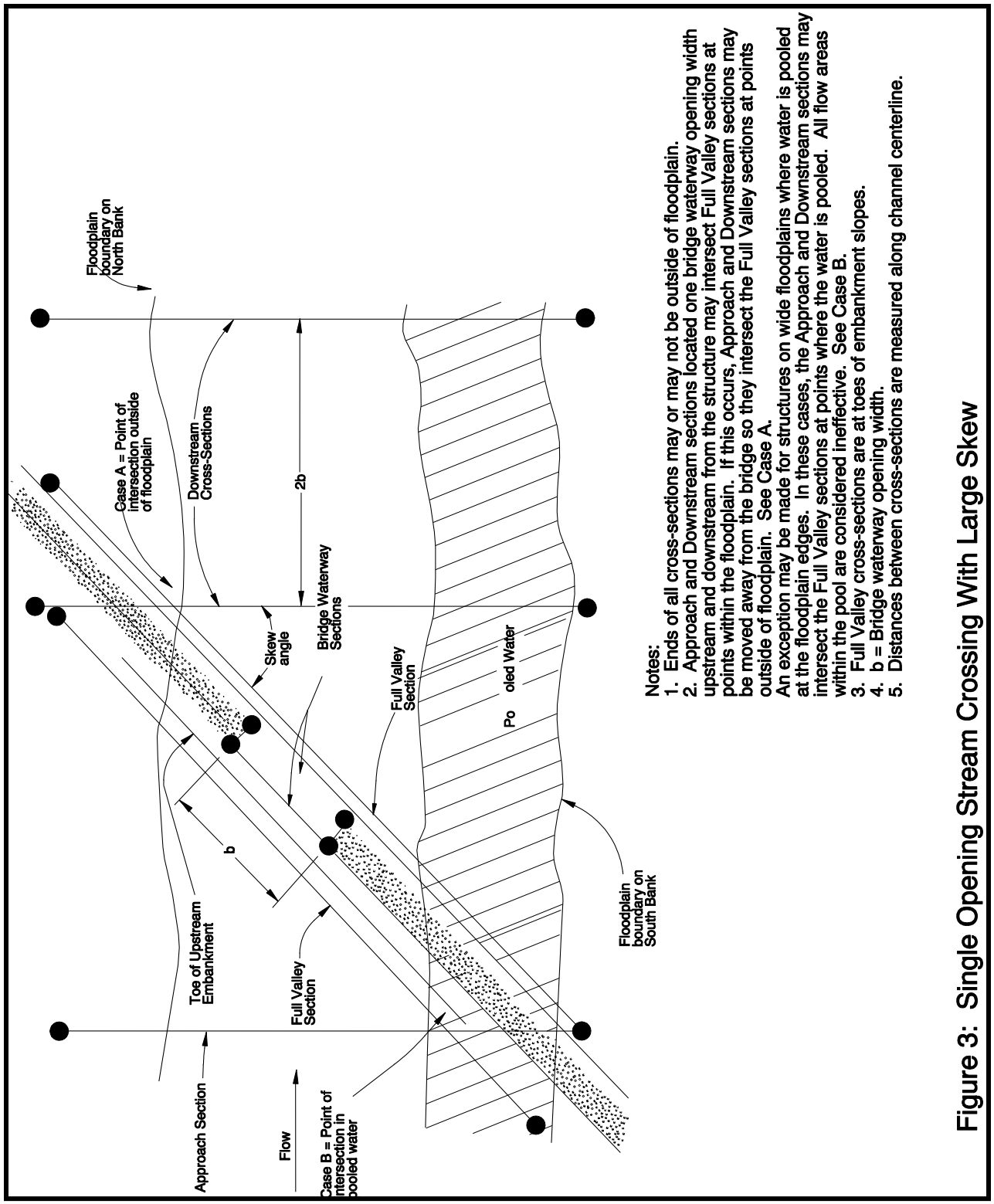



Figure 3: Single Opening Stream Crossing With Large Skew

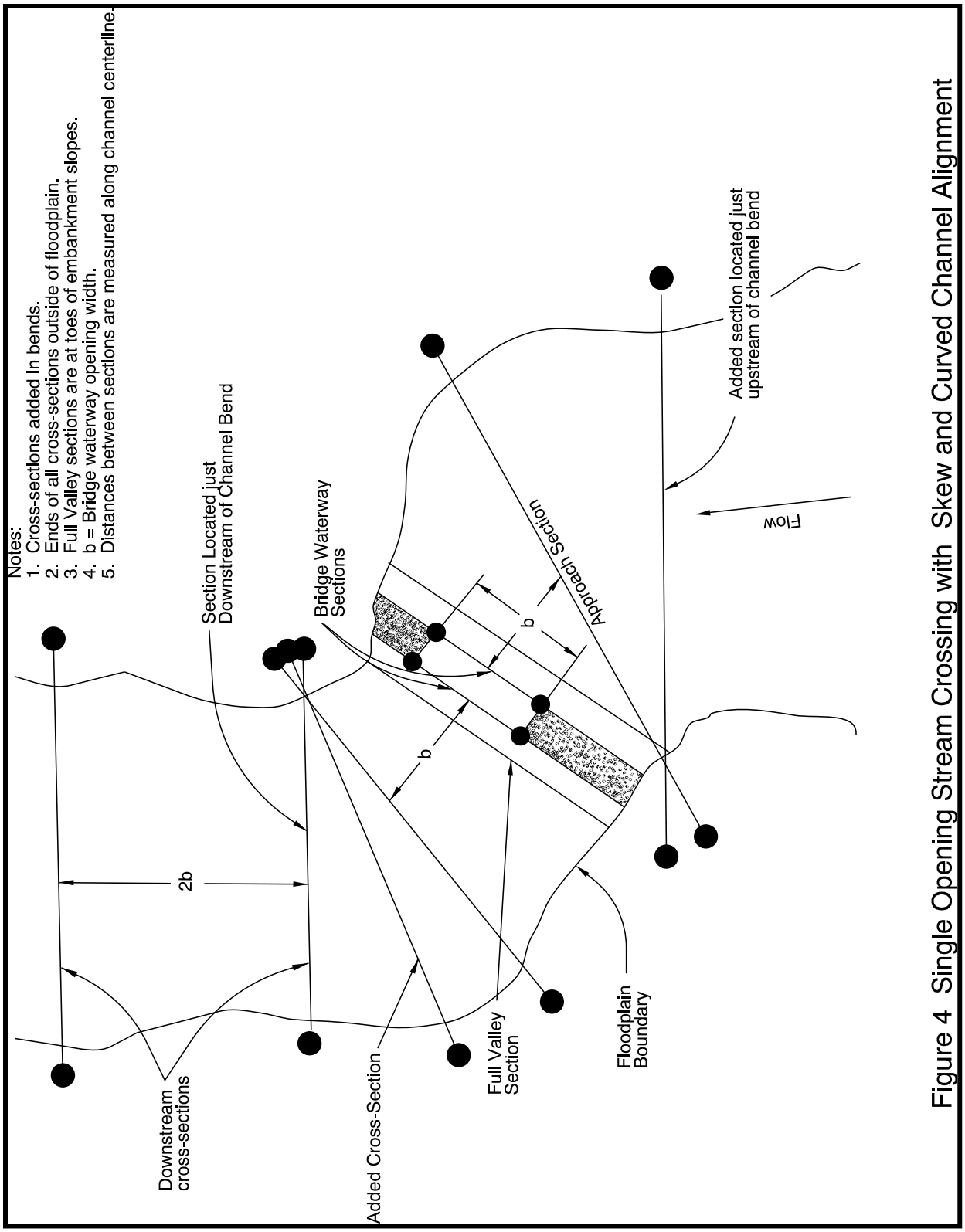



Figure 4 Single Opening Stream Crossing with Skew and Curved Channel Alignment

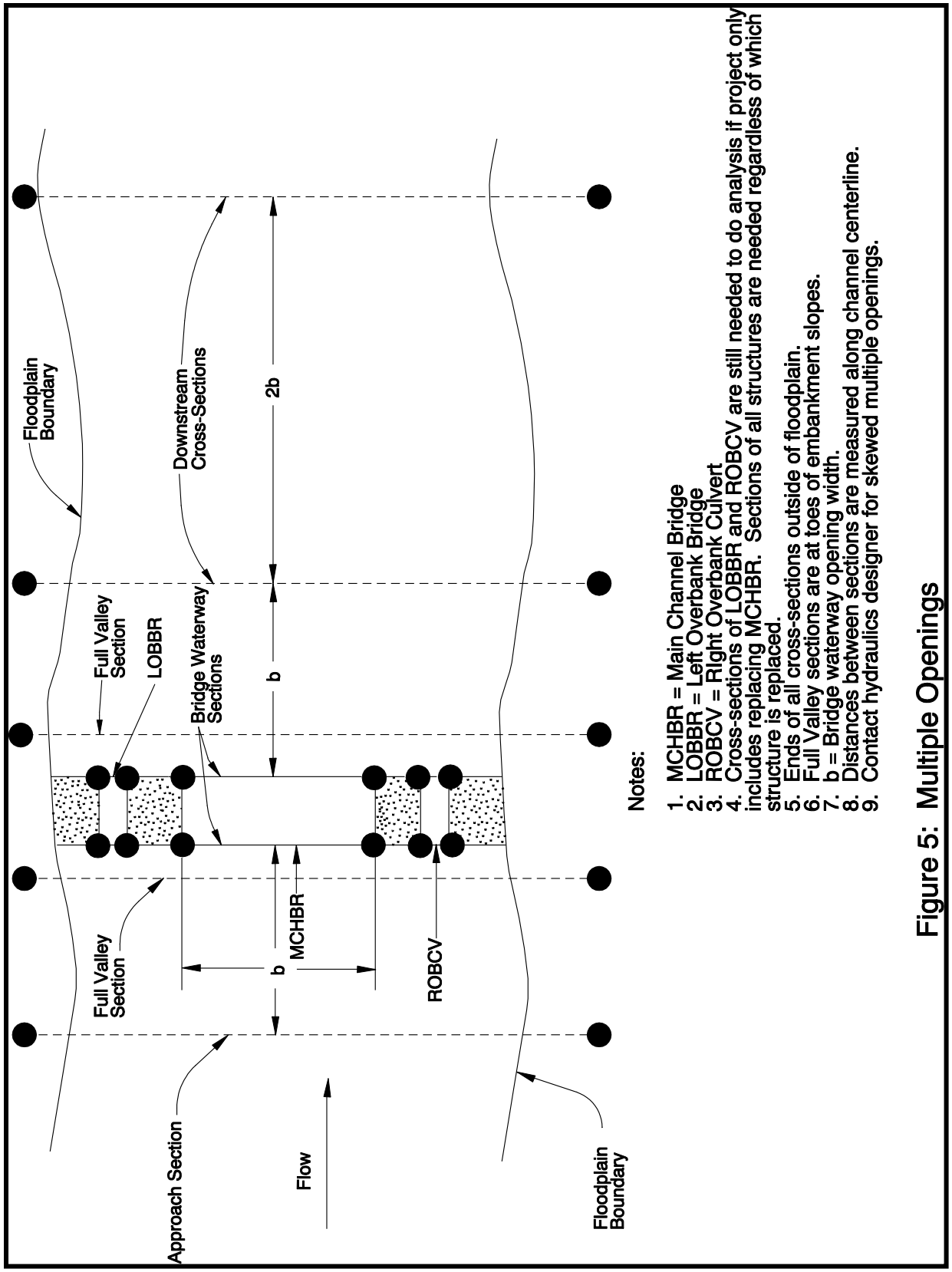



Figure 5: Multiple Openings

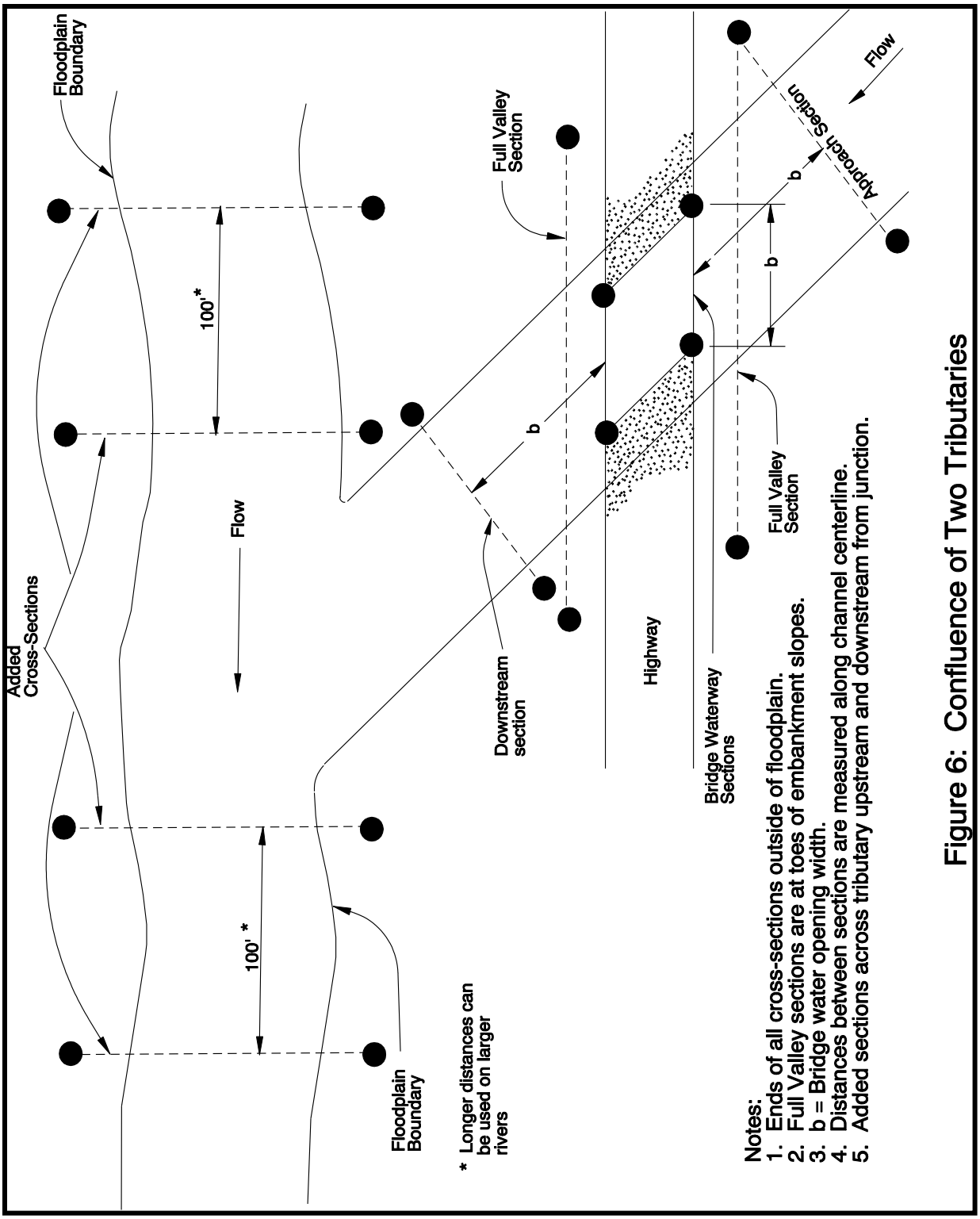



Figure 6: Confluence of Two Tributaries

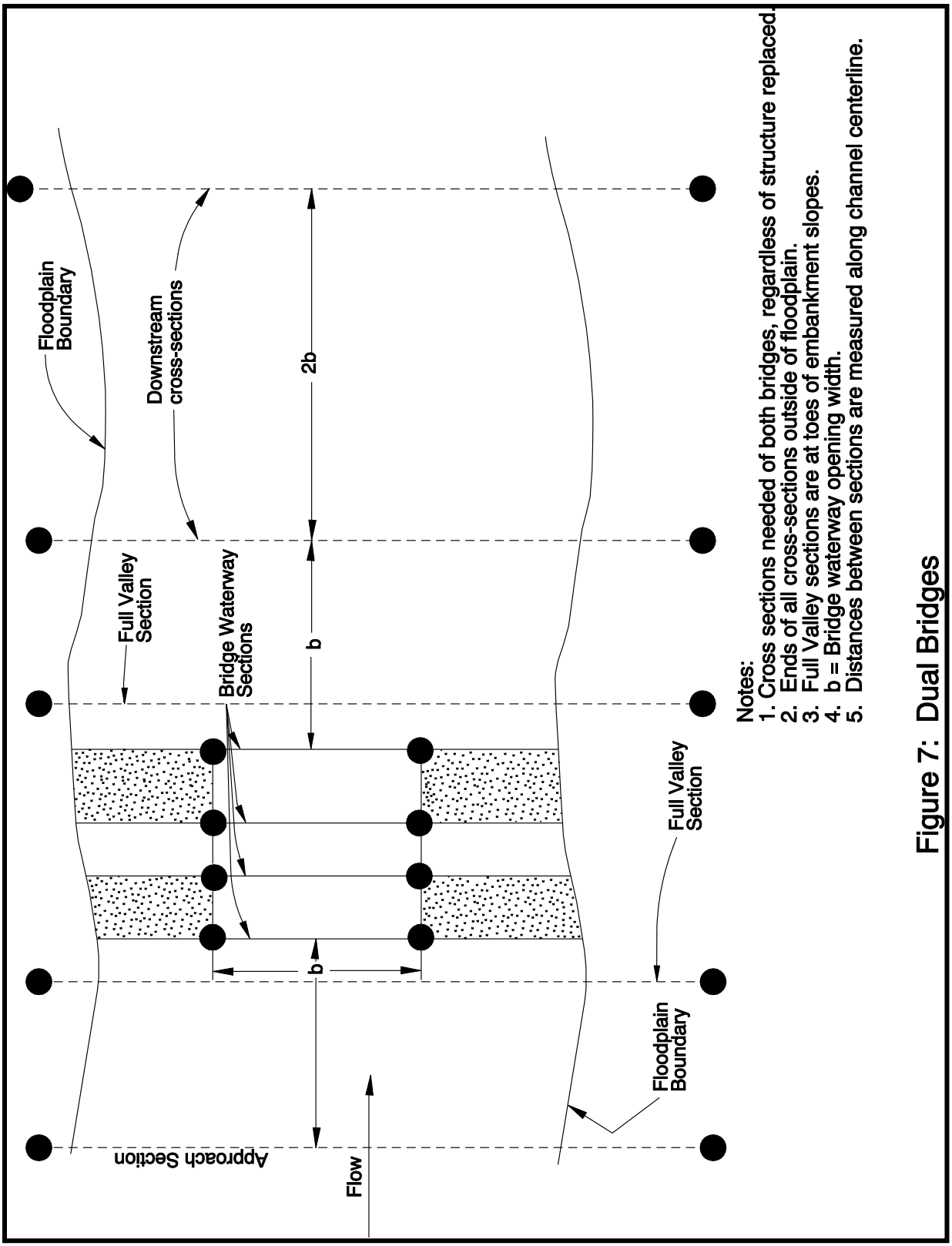



Figure 7: Dual Bridges

## HYDRAULIC SURVEY CHECK LIST

| Bridge No.: | | Checked By: | | Date: | |
|---|---|---|---|---|---|

GENERAL DATA

- ☐ **Vertical Reference Used**
  - ☐ **Known Benchmark**
    **or**
  - ☐ **Assumed Datum (as shown on plans)**
    **or**
  - ☐ **Assumed Datum**
- ☐ **Brass Disks**
- ☐ **FEMA Reference Marks**
- ☐ **Site Sketch**
  - ☐ North Arrow
  - ☐ Hydraulic Structures
  - ☐ Buildings
  - ☐ Scour Holes
  - ☐ Utilities
  - ☐ Stream Alignment
  - ☐ Cross-Section Locations
  - ☐ Horizontal and Vertical Controls
  - ☐ Turning Angles Between Cross-sections
  - ☐ Highway Alignment
  - ☐ Skew Angle (bridge deck to flow)
  - ☐ Right-of-Way and Easements
  - ☐ Trees and Bushes to be Preserved
- ☐ **Site Photos** ☐ **Name and Address of Photographer**
  - ☐ Existing Structure Waterway Opening
  - ☐ Upstream Channel (from bridge)
  - ☐ Downstream Channel (from bridge)
  - ☐ Upstream Floodplain (from bridge)
  - ☐ Downstream Floodplain (from bridge)
  - ☐ Approach to Bridge (ahead on line)
  - ☐ Approach to Bridge (back on line)
  - ☐ Overflow Structure(s) Waterway Opening
  - ☐ Overflow Upstream Channel (from structure)
  - ☐ Overflow Downstream Channel (from structure)
  - ☐ Overflow Upstream Floodplain (from structure)
  - ☐ Overflow Downstream Floodplain (from structure)
  - ☐ Roadway Overflow Areas
  - ☐ Erosion or Scour Problem Areas
  - ☐ Ordinary High Water marks (usually marks are staked)
- ☐ **Debris** accumulation near bridge or in channel within project limits
- ☐ **Floor Elevations**

**BRIDGE/HIGHWAY DATA**

- ☐ **Roadway Profile**
- ☐ **Deck Elevations** (3 min.)
- ☐ **Bridge Waterway**
  - ☐ **Downstream Face** ☐ **Upstream Face**
- ☐ **Bottom-of-Beam** elevation at each bent.
- ☐ **Top-of-Footing Elevations**
- ☐ **Cross Section Scour Holes**
- ☐ **Bent Information**
  - ☐ Location ☐ Shape ☐ Width ☐ Skew Angle (relative to flow)

**FULL VALLEY CROSS-SECTION**

- ☐ **Downstream Side** at toe-of-slope and/or beyond wingwalls
- ☐ **Parallel to Roadway Alignment**
- ☐ **Skew Angle** (relative to flow; should be same skew as Bridge)
- ☐ **Waterway Ground Points** sufficient to accurately define the channel bottom, including underwater ground points
- ☐ **Width**
  - ☐ Obtain Floodplain Ground Points high enough to extend to roadway overtopping elevation

    or
  - ☐ Ten (10) bridge lengths (total maximum width, centered on bridge)

**STREAM CROSS-SECTIONS**

- ☐ **Location**
  - ☐ APPR: One bridge length upstream of upstream face of bridge
  - ☐ EXIT: One bridge length downstream of the downstream face of the bridge
  - ☐ DOWN: Three bridge lengths downstream of the downstream face of the bridge
  - ☐ OTHER: Constrictions, controls, added upstream sections if supercritical flow
- ☐ **Width**
  - ☐ Obtain Floodplain Ground Points high enough to extend to roadway overtopping elevation

    or
  - ☐ Ten (10) bridge lengths (total maximum width, centered on bridge)
- ☐ **Perpendicular to Flow** ☐ **Oriented Facing Downstream**
- ☐ **Waterway Ground Points** (including underwater ground points)
- ☐ **Define Vegetation Limits**

**STREAM PROFILE**

- ☐ **Stationing Beginning Upstream** and increasing downstream
- ☐ **Thalweg Profile** (i.e., low point in channel)
- ☐ **Water Surface Profile**
- ☐ Extend **profile to distances as per Subsection 6.4.4.2**
- ☐ Elevations **every 50 feet** and at significant channel bottom grade breaks
- ☐ Elevations of **Ordinary High Water** marks

**MISCELLANEOUS STRUCTURES**

Obtain information as described in *Bridge/Highway Data and Full Valley Cross-Section for:*

- ☐ **Control/Irrigation/Weir Structures** within 500 feet or 2 Bridge Lengths (whichever is greater)
- ☐ **Overflow Structures** within limits of Full Valley cross-section

**LOCAL KNOWLEDGE**

☐ **Source**
- ☐ Name ☐ Date ☐ Telephone

☐ **High Water**
- ☐ Date ☐ Elevation/Location ☐ Historical Photos

☐ **Roadway Overtopping**
- ☐ Date ☐ Depth ☐ Length

☐ **Debris or Ice Problems**

# APPENDIX C
# TOPOGRAPHIC DATA REQUIRED FOR CULVERT ANALYSIS

## 1.0 Introduction

This appendix discusses the field data that is required to conduct a hydraulic analysis of a culvert. This data is required for culvert replacements, culvert extensions, and new culvert crossings. The hydraulics designer should meet onsite with the survey party chief to go over the required cross section locations and other unique data requirements of the site. This will ensure that the survey will be cost effective, gather the necessary data and avoid repeat visits to the site by the surveyors to gather additional data.

## 2.0 Digital Terrain Model

A method or obtaining hydraulic survey data using cross-sections and profiles is described. An alternative to surveying individual cross-sections is to develop a digital terrain model (DTM) for the project site. If a DTM is developed for the project, the limits of the DTM should extend far enough to allow the required cross-sections and profiles to be "cut" from the DTM.

Experience shows that the DTM should extend far enough upstream and downstream to include the culvert construction, detour structures, access roads, erosion control features, and temporary water management. The area occupied by the pipe jacking or ramming pit should also be included for these trenchless installations. The terrain model is extremely valuable or essential when these features are designed.

Other features that should be included are right-of-way and easement boundaries, utilities, and trees and bushes to be preserved. Guidance on making these models is in the "structure" survey guidelines in the ODOT Highway Design Manual. Often a DTM is made of the project site and it is supplemented by individual cross-sections and profiles outside of the DTM limits. Care should be exercised to assure the individual cross-sections and profiles are compatible with the DTM. As an example, both surveys should use the same stream profile stationing and elevation datum.

The terrain at most culvert crossings is irregular and difficult to accurately model with a DTM. It is important to construct the DTM based on the original survey, and to plot the vicinity map,

cross-sections cut from the DTM, and the stream profiles. These drawings should be taken to the site and compared to the existing terrain. Often additional data points will be needed at critical locations, and the model will need to be adjusted so it represents actual conditions. These quality control steps should be completed before the model is submitted to the designers.

*Note: These guidelines are for large culvert or fish passage culvert replacement, installation, or extension. Work on smaller culverts without fish passage concerns may require less survey data.*

## 3.0 Required Field Data

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### 3.1 General

a. Vertical Reference should be identified. All brass disks within project limits should be referenced and identified. Surveys in FEMA floodplains need to tie the reference marks used in the Flood Insurance Study to the project survey. The hydraulics designer can supply the locations and descriptions of the reference marks. The preferred vertical reference hierarchy is:

- known benchmark,
- assumed datum (as shown on plans), or
- other assumed datum.

b. Site Sketch should be prepared. The sketch should include north arrow, stream alignment, highway alignment, cross-section locations, horizontal control and turning angles of cross-sections, hydraulic structure(s), skew angle of each hydraulic structure(s) with respect to flow, buildings, and other unique features at the site. The sketch should show right-of-way and easement boundaries, utilities, and bushes and trees to be preserved.

c. Photographs. Each photograph should include a description of the item(s) shown. Record the name and address of the photographer and the date the photos were taken. Take photographs, either film or digital, of:

- Existing culvert. Take a photo of the inlet and outlet of the culvert.
- Overflow or structure(s). This photo should show the waterway opening provided by the structure(s). If the overflow structure is a culvert, take a photo of the inlet and outlet.

- Channel upstream and downstream from the culvert. Take a photo looking upstream and a photo looking downstream while standing on roadway. Other photos of the channel further upstream and downstream from the culvert are also helpful.
- Overflow channels upstream and downstream from the overflow structure(s). Take a photo looking upstream and a photo looking downstream while standing on the overflow structure.
- Floodplain upstream and downstream from the culvert.
- Roadway overflow areas.
- Erosion or scour problem areas.
- Take a photo looking ahead on line and a photo looking back on line from the culvert.
- Locations where floor elevations are obtained.
- Hydraulic controls such as rock outcrops, rock ledges, weirs, dams, etc. (See chapter text.)
- Developed property in or adjacent to the waterway, such as pump intakes, retaining walls, fences, pedestrian bridges, structures, etc. These photos will record the condition of the facilities before the project is constructed. They may be valuable if there is future litigation.
- Ordinary High Water elevation marks. Usually these elevations are located and staked by the biologist or the hydraulics designer.

d. Describe significant debris accumulation near the culvert or in the channel within project limits.

e. Obtain floor elevation and location of upstream houses and buildings.

### 3.2 Highway

a. Obtain a profile of the highway. The profile should reflect the highest part of the roadway. If the highway is subject to overtopping, the profile must also define the overtopping limits.

   Water will overtop at other locations than the road centerline at many sites. A profile is needed across the apex of the location where the water starts to overflow. An example is a culvert on a roadway with a constant grade. Overtopping occurs adjacent to the inlet, it flows down the roadside ditch to a sag several hundred feet away, then it crosses the road in the sag. A profile would be needed across the high point in the roadside ditch near the culvert inlet where the overtopping initially occurs.

b. Cross-section of the highway along the centerline of the culvert alignment. This section may be skewed relative to the highway centerline.

### 3.3 Existing Culvert

a. The following data is needed for projects that are replacing or extending an existing culvert. Most of this data may be available from existing roadway and/or bridge plans, but it should be field verified. If a culvert is proposed to replace an existing bridge, then the data requirements for the existing bridge described in Appendix D must be obtained.

b. Type of culvert (i.e., box culvert, circular, pipe-arch, etc.)

c. Culvert material (i.e., concrete, metal, or plastic)

d. Size

e. Length

f. End treatment (i.e., sloped, mitered, or projecting ends, headwall with wingwalls, etc.)

g. Top of barrel, water surface, invert, and ground surface elevations at the inlet and outlet. If the culvert is partially full of sediment, the flow line elevation of the culvert barrel and the elevation of the sediment must be obtained. If the culvert end is undermined, elevations are needed of the invert and the groundline under the invert.

h. The Culvert Inspection Form in Appendix D is required for all culverts within the project limits. Culverts in poor condition may need to be either replaced or rehabilitated.

### 3.4 Cross-Sections

a. Width. The cross-section should extend across the full valley width, which includes the channel and adjacent floodplain.

b. Obtain multiple channel ground points, including underwater ground points, sufficient to accurately define the channel bottom.

c. Should be taken perpendicular to the direction of flow. Note that perpendicular to the flow in the floodplain may differ from perpendicular to the flow in the channel in which case the section will have a “dogleg” and the direction of the section in floodplain will be different than in the channel. Left and right for the cross-sections are defined by looking downstream.

d. Should define limits of vegetation types. Note where vegetation changes from grass to brush, etc. This can be identified in the cross-section notes.

e. Should be taken at locations shown in Figure 1. If significant upstream ponding is likely, additional sections may be necessary to determine the storage capacity upstream of the culvert. Likewise, additional downstream sections may be necessary to establish downstream controls that will influence the tailwater elevation. Highly irregular channels may require additional up- and downstream cross sections.

### 3.5 Stream Profile

a. The profile should extend a sufficient distance upstream and downstream from the project site to accurately determine the slope of the channel.

b. Profile upstream and downstream from the culvert centerline the distances listed in 6.4.4.2. Additional profile length may be needed to satisfy 5a. Elevations should be located along the channel thalweg (lowest point in channel) and be taken every 50 feet and at significant channel bottom grade breaks.

c. Include the stream bottom profile and the water surface profile at the time of the survey.

d. Include past flood profiles if evidence from previous high water is apparent.

e. Include ordinary high water elevations. These elevations are usually staked in the field by the biologist or the hydraulic designer.

### 3.6 Miscellaneous Structures

a. Obtain elevations, dimensions, and profiles of other roadway crossings within 500 feet of project. Enough information needs to be obtained to include the crossing in the hydraulic model developed for the project.

b. Obtain elevations and dimensions of miscellaneous structures such as irrigation dams or weirs within 500 feet of project.

### 3.7 Local Knowledge of Past Floods

a. Source of historic flood information is from local residents or maintenance personnel. If local knowledge is obtained, give name and phone number of contact, otherwise give name and phone number of people that can be contacted later.

b. Highest water elevation and date of occurrence. If a high-water elevation is available, the exact location and elevation of the high-water mark should be surveyed.

c. Is roadway overtopped? If so, obtain depth of water over road and length of roadway overtopped.

d. Any debris or ice problem?

e. Photos of past floods?

f. Sediment problem?

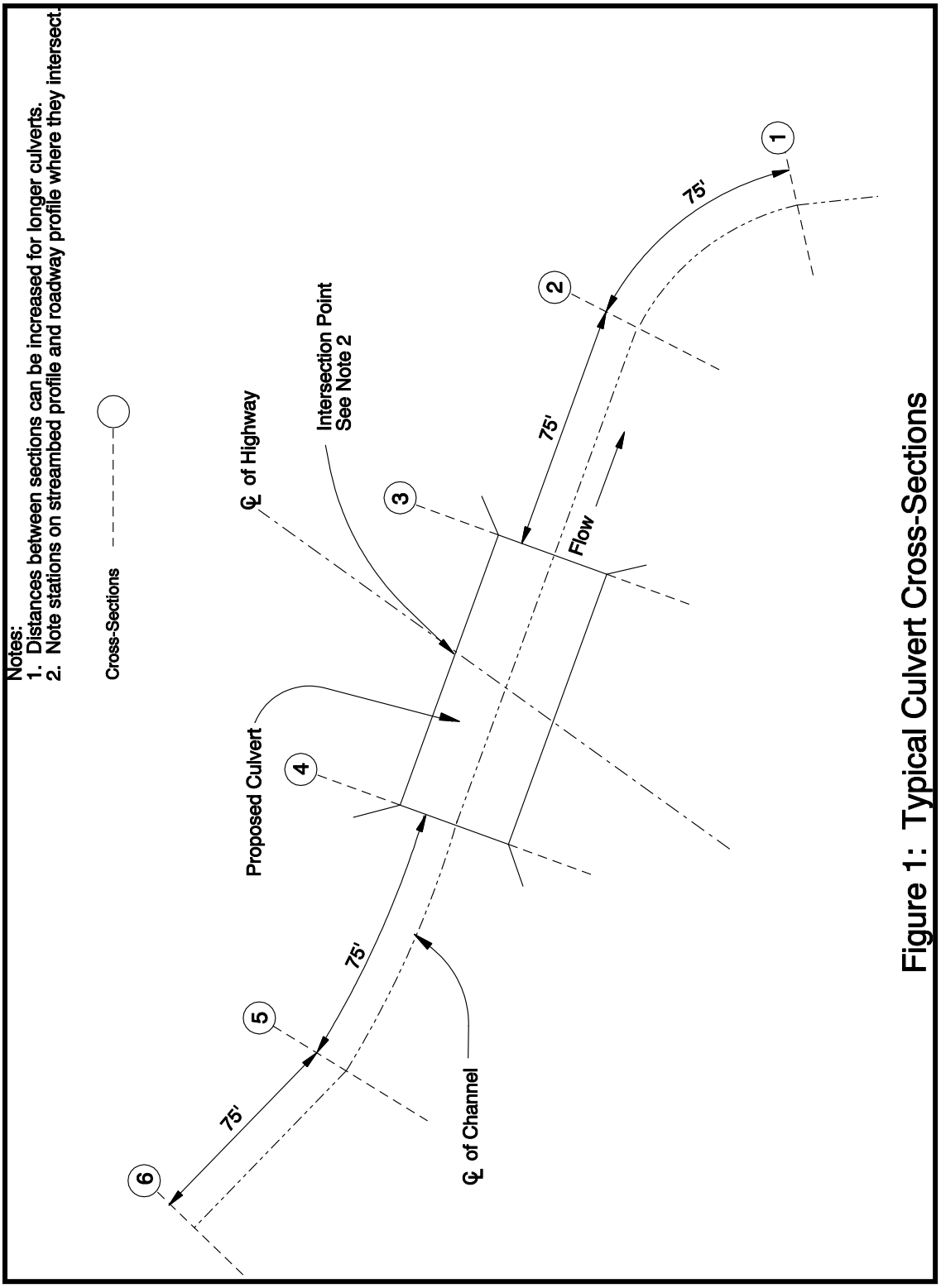



Figure 1: Typical Culvert Cross-Sections

# Oregon Department of Transportation

Culvert / Maintenance Inspection Form

| | | |
|---|---|---|
| Drainage Facility ID: | Inspection Date: | Inspection Interval (months): |
| | Inspector ID: | |
| | Collection Source: | Assessment Score: |

**Note:** Use ODOT's "Culvert Inspection & Inventory Handbook" when filling out this form. This handbook is designed solely to be a reference for ODOT's culvert inspections to ensure repeatable results and provide statewide uniformity and consistency.

## Site Information

| | | | | |
|---|---|---|---|---|
| Road ID: | Mile Point: | | Region | Latitude |
| Roadway ID: Add (1) / Non Add (2) | Mileage Type: Overlap Yes / No | Overlap Mileage: | District | Longitude |

## Secondary Location

| | | | |
|---|---|---|---|
| Road ID: | Roadway ID: | Mile Point: | Overlap Mileage: |

## Culvert Details

| | | | |
|---|---|---|---|
| Bridge Structure #: | Orientation: | Year Built: | |
| Inlet Side: Left / Right | Cross Public Approach Left<br>Pipe Left Public Approach Right<br>Pipe Right | Facility Usage: | |
| Skew: | | Inlet End Treatment | Inlet Protection Mat'l |
| Structure Width (ft.): | No. of Barrels: | Outlet End Treatment | Outlet Protection Mat'l |

## Profile - Embankment

| | |
|---|---|
| Height of Cover - Inlet (ft.) | Inlet Culvert Extension Beyond Fill (ft.): |
| Height of Cover - Outlet (ft.) | Outlet Culvert Extension Beyond Fill (ft.): |

## Photos Taken. List Order Taken:

Mile Point: _____ Outlet End: _____

Inlet End: _____ Outlet, Looking Inside End: _____

Inlet, Looking Inside: _____ Embankment Problem: _____

## Stream:

| | | | |
|---|---|---|---|
| Stream Name: | | Culvert: | Stream: |
| Outlet Pool Depth (ft.): | Drop to Outlet Pool (ft.): | Baffles Weirs | Fish Ladder Log Sills |
| Beaver Activity: | | Simulated Streambed | Roughened Channel Rock Weirs |
| | | Other: | Other: |

## Comments:

## Site Ratings

| Roadway | ----- Rating ----- | | | | |
|---|---|---|---|---|---|
| Rdwy Sag / Guardrail Dip | Good | Fair | Poor | Critical | No Rating |
| Rdwy Cracks / Patches | Good | Fair | Poor | Critical | No Rating |
| **Embankment** | **----- Rating -----** | | | | |
| Embankment Popouts | Good | Fair | Poor | Critical | No Rating |
| Seeps | Good | Fair | Poor | Critical | No Rating |

| Hydraulic | ----- Rating ----- | | | | |
|---|---|---|---|---|---|
| Bank Protection - Inlet | Good | Fair | Poor | Critical | No Rating |
| Bank Protection - Outlet | Good | Fair | Poor | Critical | No Rating |
| Scour - Inlet | Good | Fair | Poor | Critical | No Rating |
| Scour -Outlet | Good | Fair | Poor | Critical | No Rating |
| End Treatment Condition | Good | Fair | Poor | Critical | No Rating |
| Channel Alignment | Good | Fair | Poor | Critical | No Rating |

## Barrel A

| Barrel Span (in): | Barrel Rise (in): | Sediment Depth (in): | Barrel Shape: | Material: |
|---|---|---|---|---|
| | | Inlet = Outlet = | Circular Open Bottom Arch<br>Box Elliptical<br>Arch No Data | CMP Iron/Steel<br>Concrete Ductile<br>HPDE Smooth Clay<br>HPDE Corrugated Timber<br>PVC Smooth Fiberglass<br>Corrugated Alum |
| Barrel Length (ft.): | | | | |
| Rehab Date: | | Coating: Bituminous Concrete Polymer None | | |
| Rehab Type: | | | | |

**Ratings ...**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| General Barrel Condition | Good | Fair | Poor | Critical | No Rating | Out of Round | Good | Fair | Poor | Critical | No Rating |
| Blockage | Good | Fair | Poor | Critical | No Rating | Settlement | Good | Fair | Poor | Critical | No Rating |
| Cracking | Good | Fair | Poor | Critical | No Rating | Piping | Good | Fair | Poor | Critical | No Rating |
| Invert Damage | Good | Fair | Poor | Critical | No Rating | Drift | Good | Fair | Poor | Critical | No Rating |
| Open Joints | Good | Fair | Poor | Critical | No Rating | Vegetation Obstruction | Good | Fair | Poor | Critical | No Rating |

## Barrel B

| Barrel Span (in): | Barrel Rise (in): | Sediment Depth (in): | Barrel Shape: | Material: |
|---|---|---|---|---|
| | | Inlet = Outlet = | Circular Open Bottom Arch<br>Box Elliptical<br>Arch No Data | CMP Iron/Steel<br>Concrete Ductile<br>HPDE Smooth Clay<br>HPDE Corrugated Timber<br>PVC Smooth Fiberglass<br>Corrugated Alum |
| Barrel Length (ft.): | | | | |
| Rehab Date: | | Coating: Bituminous Concrete Polymer None | | |
| Rehab Type: | | | | |

**Ratings ...**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| General Barrel Condition | Good | Fair | Poor | Critical | No Rating | Out of Round | Good | Fair | Poor | Critical | No Rating |
| Blockage | Good | Fair | Poor | Critical | No Rating | Settlement | Good | Fair | Poor | Critical | No Rating |
| Cracking | Good | Fair | Poor | Critical | No Rating | Piping | Good | Fair | Poor | Critical | No Rating |
| Invert Damage | Good | Fair | Poor | Critical | No Rating | Drift | Good | Fair | Poor | Critical | No Rating |
| Open Joints | Good | Fair | Poor | Critical | No Rating | Vegetation Obstruction | Good | Fair | Poor | Critical | No Rating |

## Barrel C

| Barrel Span (in): | Barrel Rise (in): | Sediment Depth (in): | Barrel Shape: | Material: |
|---|---|---|---|---|
| | | Inlet = Outlet = | Circular Open Bottom Arch<br>Box Elliptical<br>Arch No Data | CMP Iron/Steel<br>Concrete Ductile<br>HPDE Smooth Clay<br>HPDE Corrugated Timber<br>PVC Smooth Fiberglass<br>Corrugated Alum |
| Barrel Length (ft.): | | | | |
| Rehab Date: | | Coating: Bituminous Concrete Polymer None | | |
| Rehab Type: | | | | |

**Ratings ...**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| General Barrel Condition | Good | Fair | Poor | Critical | No Rating | Out of Round | Good | Fair | Poor | Critical | No Rating |
| Blockage | Good | Fair | Poor | Critical | No Rating | Settlement | Good | Fair | Poor | Critical | No Rating |
| Cracking | Good | Fair | Poor | Critical | No Rating | Piping | Good | Fair | Poor | Critical | No Rating |
| Invert Damage | Good | Fair | Poor | Critical | No Rating | Drift | Good | Fair | Poor | Critical | No Rating |
| Open Joints | Good | Fair | Poor | Critical | No Rating | Vegetation Obstruction | Good | Fair | Poor | Critical | No Rating |

# APPENDIX E
# TOPOGRAPHIC DATA REQUIRED FOR STORMWATER ANALYSIS

## Introduction

This appendix discusses the field data that is required to conduct a hydraulic analysis of a storm drain system. This data is required for stormwater systems that are new, replacements, additions or modifications of pipes, inlets, manholes, storm water control, stormwater treatment and other storm drain facilities. The hydraulics designer should meet onsite with the survey party chief to go over the required limits of the survey beyond the edge of pavement or back of sidewalk and other unique data requirements of the site. This will ensure that the survey will be cost effective, gather the necessary data and avoid repeat visits to the site by the surveyors to gather additional data.

## Required Field Data

1. GENERAL

   a. Vertical Reference should be identified. All brass disks within project limits should be referenced and identified. Surveys in FEMA floodplains need to tie the reference marks used in the Flood Insurance Study to the project survey. The hydraulics designer can supply the locations and descriptions of the reference marks. The preferred vertical reference hierarchy is:

      - known benchmark,
      - assumed datum (as shown on plans), or
      - other assumed datum.

   b. Sketch should be prepared for each drainage structure (manholes, inlets, etc). The sketch should include north arrow, pipe alignment, benching details, internal apparatus configuration, internal dimensions, manhole rings, and manhole lid orientation to manhole base.

   c. Photographs. Each photograph should include a description of the item(s) shown. Record the name and address of the photographer and the date the photos were taken. Take photographs, either film or digital, of:

      - Existing pipe outfall. Take a photo of the outlet of the pipe and any energy dissipater structure or material.
      - Channel downstream from the pipe outfall. Take a photo looking upstream and a photo looking downstream while standing near the outfall.
      - Erosion or scour problem areas.

- Take a photo looking ahead on line and a photo looking back on line from the pipe outfall.
- Locations where floor elevations are obtained.
- Hydraulic controls such as rock outcrops, rock ledges, weirs, dams, valves, gates etc. (See chapter text.)
- Developed property in or adjacent to the waterway, such as pump intakes, retaining walls, fences, pedestrian bridges, structures, etc. These photos will record the condition of the facilities before the project is constructed. They may be valuable if there is future litigation.
- Ordinary High Water elevation marks. Usually these elevations are located and staked by individuals knowledgeable in stream geomorphology such as the project fisheries biologist or hydraulics designer. See **Chapter 6** for guidance.

d. Describe significant debris accumulation near the pipe outfall or in the channel within project limits.

e. Obtain floor elevation and location of upstream houses and buildings.

2. ROADWAY DRAINAGE FEATURES

a. The following data is needed for projects that are replacing or modifying an existing storm drain system. Most of this data may be available from existing roadway plans, but it should be field verified.
b. Obtain information on roadway drainage features. Roadway drainage features include roadside ditches, overside drains, curbs, curb & gutters, valley gutters, inlets (grates, curb openings, slotted drains, etc), manholes, and junction structures. The information should reflect the location, size, and elevation of each feature.
c. Collect data on all drainage features within the project R/W and features outside the project R/W that contribute stormwater runoff to the system being evaluated. This includes roof drains, area drains, parking lot drains, etc.
d. Inlet Information
   - Size of Grate/Opening
   - Type of Grate
     (1) Type 1: Bars in two directions; Bicycle Safe.
     (2) Type 2: Bars in one direction
   - Invert elevations of all pipes
   - Sump Description (Depth of sump below lowest flowline, depth of sediment in sump)
   - Material of inlet box if visible.
   - Internal Parts
     (1) Identify location of pipes protruding into inlets and distance of protrusion.
     (2) Diameter of Orifice Plate openings
     (3) Weir Height, width, thickness

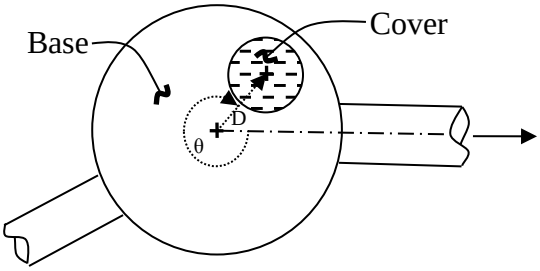
Table 1: Inlet Types

| I.D. | Configuration | Where Used | Standard Drawing No. |
|---|---|---|---|
| G-1 | Single Grate* | Small areas with no debris; recommended for two-lane highways. | RD364 |
| G-2 | Double Grate* | Where considerable water or debris is anticipated; recommended for multiple-lane highways | RD364 |
| G-2M | Single Grate | Areas without traffic, bicycles or pedestrians such as medians and ditches. | RD364 |
| G-2MA | Single Grate with Concrete Apron | Areas without traffic, bicycles or pedestrians such as medians and ditches having variable 1V:10H-1V:20H side slope. | RD364 |
| CG-1 | Single Grate* plus Curb Opening | Where debris or trash is anticipated and in sags; recommended for two-lane highways. | RD366 |
| CG-2 | Double Grate* plus Curb Opening | Where considerable water or debris is anticipated and in sags; recommended for multiple-lane highways | RD366 |
| CG-3 | Curb Opening only | Where longitudinal grade is relatively flat and bicycle traffic is anticipated; good debris passage capabilities. | RD372 |
| Curb Inlet Channel | Curb Opening Extension for attachment to CG-1 or CG-2 inlets. | Where longitudinal grade is relatively flat and bicycle traffic is anticipated; good debris passage capabilities. | RD366 |
| D | Single Grate* on Inclined Slope | Side drainages and at end of ditches where debris is anticipated; functions as a trash rack. | RD370 |
| Type 3 C.B. | Single Grate* | Where minimal debris or trash is anticipated and where pedestrians may be present. | RD378 |
| Area Drainage Basin or Field Inlet | Single Round Grate* with Concrete Apron. | Where No Traffic is anticipated. Medians, ditches, flat vegetated areas. | RD374 |
| Slotted Drain | CMP Slotted Drain | To collect shallow sheet flow or along narrow shoulders. Areas with minimal debris ; not recommended for sags. | RD328 |
| Trench Drain | Long Narrow Grates | To collect shallow sheet flow or along narrow shoulders. | N/A |
| Deck Drain Type A | Single Grate* Deck Drain. | Bridge deck drain | BR120 |

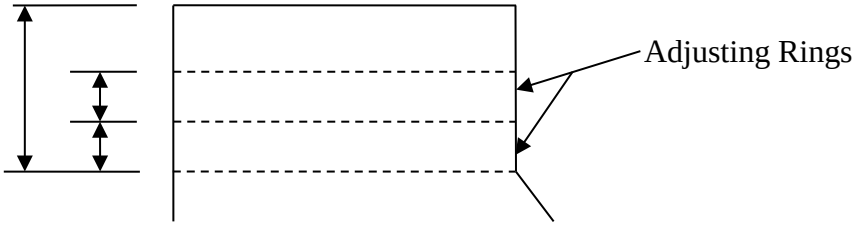
* Bicycle safe grate(s)

e. Manhole Information

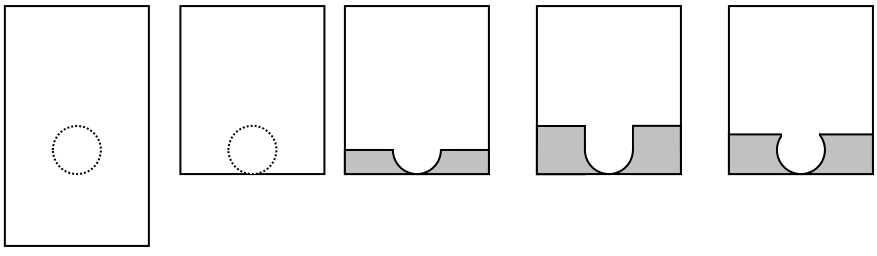
- Invert elevations of all pipes
- Center of Manhole Cover
- Center of MH Base
  During design and construction manholes are located and described by station of the center of the manhole base rather than the manhole cover.
  - Offset distance, D; and from center of manhole base to center of cover (± 0.2 ft).
  - Rotation angle, θ; from centerline of outlet pipe [largest diameter pipe if split flow manhole] measured clockwise to the center of manhole cover (± 10°).




- Material of manhole structure if visible.
- Inside Manhole Diameter
- Drop Manhole (Y/N); refer to standard drawing RD352.
- Measure number of adjusting rings on manhole riser and riser height.




- Benching: Describe type of benching in bottom of Manholes (see below).




- Sump Description (Depth of sump below lowest flowline, depth of sediment in sump)

- Internal Parts

(1) Identify location and of pipes protruding into manholes and distance of protrusion.
(2) Diameter of orifice openings
(3) Weir Height, width, thickness

3. STORM DRAIN SYSTEM PIPES

a. Pipe Condition: T.V. inspection should be used for projects that contain storm drain pipe. Storm drain pipes in poor condition may need to be replaced or rehabilitated. The Storm Drain Pipe Inspection Form in Appendix F can be used to evaluate the condition of a pipe.

b. Type of storm drain (i.e., rectangular (box), circular, pipe-arch, etc.)

c. Storm drain material (i.e., concrete, metal, or plastic)

d. Invert elevation on each end

e. Size

f. Length

g. End treatment (i.e., sloped, mitered, or projecting ends, headwall with wingwalls, etc.)

h. Pipe outfall and any energy dissipater structure or material.

4. STORM DRAIN PIPE PROFILE

a. The profile should extend a sufficient distance upstream and downstream from the project site to accurately determine the affects of any proposed modifications.

b. Profile 500-1000 feet upstream and downstream for storm drains smaller than 24- inches. Profile 1500-3000 feet upstream and downstream for storm drains 24 inches and larger. Adjust the profile length as necessary to satisfy 4a. Elevations should be located along the storm drain flowline (lowest point) and be taken at every structure or junction.

c. Report inaccessible flowlines due to appurtenances or obstructions.

5. CROSS-SECTIONS and STREAM PROFILE

   Cross-sections and a stream profile may be necessary if an analysis is required downstream of a pipe outfall to determine tailwater or downstream impacts. Any significant roadside ditches or channels may also require this data be collected. Refer to Appendix C.

6. LOCAL KNOWLEDGE OF PAST FLOODS

   a. Source of historic flood information is from local residents or maintenance personnel. If local knowledge is obtained, give name and phone number of contact, otherwise give name and phone number of people that can be contacted later.

   b. Highest water elevation and date of occurrence. If a high-water elevation is available, the exact location and elevation of the high-water mark should be surveyed.

   c. Is roadway flooded into traveled lanes? If so, obtain spread width of water into roadway and length of roadway flooding.

   d. Any debris or ice problem?

   e. Photos of past flooding?

   f. Sediment problem?

7. UTILITIES

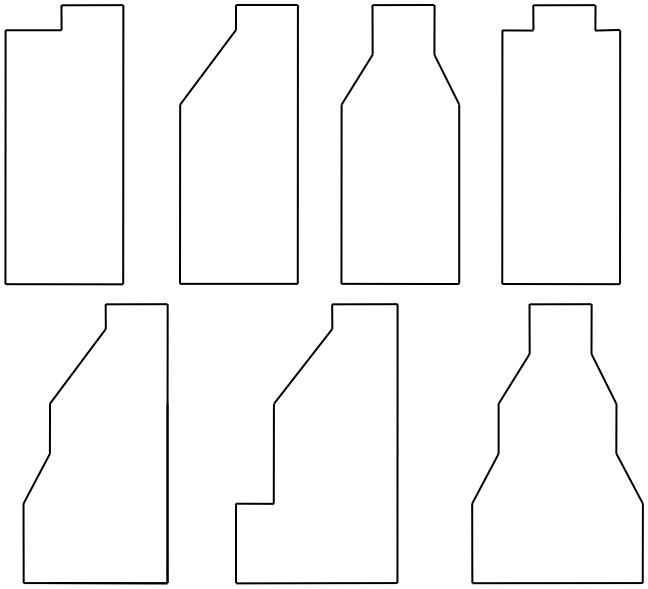
   The location, size, and elevation of all utilities should be obtained in areas where construction of new, modified, or replacement storm drainage facilities are anticipated. Utility conflicts discovered during construction can cause major project delays and cost overruns.

8. SPECIAL DATA

*This data would only be required if specifically requested by the designer.*

- Describe or show where flowline is measured
- Inside dimensions of Inlet Box
- Absolute confidence of flow direction and flowline elevation
- Material (Not visible from surface)
- Identify eccentricity of MH – Provide Sketch and internal dimensions.

Sample Sketch:



# APPENDIX F
# STORM DRAIN PIPE INSPECTION FORM

# STORM DRAIN PIPE INSPECTION FORM

| Hwy No. | Route No. | M.P. | Region | District | County | Date |
|---|---|---|---|---|---|---|
| | | | | | | |

| Pipe No. | Span (in) | Rise (in) | Length (ft) | Slope (%) | | |
|---|---|---|---|---|---|---|
| | | | | | | |

| **Shape** (Select one) | **Material** (Select one) | NOTE Here If Additional Recommendations or Comments have been Provided on Back of Form ☐ |
|---|---|---|
| Arch ☐<br>Box ☐<br>Circular ☐<br>Elliptical ☐ | Aluminum ☐ Timber ☐<br>Concrete ☐<br>Plastic ☐<br>Steel ☐ | |
| Other | Other | |

**(1 = Good for many years, 2 = Good for a few years, 3 = Needs immediate attention)**

| **Roadway over pipe** | **Rating** | **Remarks** |
|---|---|---|
| Pavement cracks or patches | | |
| Guardrail dips | | |
| Sags in roadway | | |
| Recent signs of high water | | |
| | | |

**Inlet / Outlet Protection**

| | | |
|---|---|---|
| Channel scour at Inlet / Outlet | | |
| Embankment erosion | | |
| Recent high water above crown | | |
| Misalignment with channel | | |
| Drift | | |
| Vegetation | | |
| Silt | | |

**Pipe Barrel**

| | | |
|---|---|---|
| Invert/wall damage (Corrosion) | | |
| Invert/wall damage (Abrasion) | | |
| Open joints (at pipes & structures) | | |
| Flow entering through joints | | |
| Headwall &/or Wingwall Damage | | |
| Out-of-round | | |
| Settlement | | |
| Blockage | | |
| Lateral pipe connection protrusions | | |
| Root Damage | | |
| Cracking | | |
| TOTAL POINTS* | | |
| **OVERALL CONDITION** | | |

| **Additional Data Needed** | **Distance (ft)** | **Comment** |
|---|---|---|
| Embankment height above pipe (U.S.) | | |
| Embankment height above pipe (D.S.) | | |
| Depth of water at outlet | | |
| Distance outlet invert to exist w.s. | | |

# CHAPTER 7

# HYDROLOGY

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Appendix H NRCS 24 Hour Precipitation Maps
Appendix I 2-Year 7-Consecutive Day Low Flows
Appendix J Mean Daily Exceedance Discharges
Appendix K Maximum Predicted Discharge Based on Station Records and Envelope Curve

## 7.1 Introduction

Hydrologic analysis is the most important step prior to the hydraulic design. It is needed to determine the rate of flow or volume of water the facility will need to convey or store. These discharges or volumes are essential to determine the type, size, location, and design of hydraulic facilities. In addition, hydrologic information is needed for many other purposes, such as supporting evidence during litigation.

## 7.2 Definitions

Definitions of many terms used in hydrology are included in the American Association of State Highway and Transportation Officials (AASHTO) glossary in this manual.

## 7.3 Hydrologic Study Types

Hydrologic studies for ODOT projects are usually one of five types:

- studies to determine peak discharge rates,
- studies to calculate peak discharge rates, flow volumes, and the relationship of flow rate and volume versus time,
- studies to calculate consecutive day low flows,
- studies to estimate mean daily exceedance flows, and
- studies to estimate the maximum discharge expected at a site based on flood records.

A hydrologic study to determine peak discharge rate estimates the greatest flow expected through the site or facility. Peak discharge rate is usually expressed as the flow in cubic feet per second (cfs) that is expected to be exceeded, on the average, only once in the specified recurrence interval. The recurrence interval is expressed in years. This peak flow is expected to occur during a moment, only. It is also called the "instantaneous peak flow." Instantaneous peak flows are often used in the design of stormwater drainage systems, bridges, culverts, energy dissipators, and channels.

A hydrologic study is often done to determine the peak discharge rate, the discharge rate versus time relationship, and the discharge volume. This study is often called a "hydrograph analysis" because a hydrograph is an integral part of the procedure. A hydrograph is a graph or table of the time distribution of a hydraulic characteristic at a specified location. Typically the hydraulic characteristic is shown on the Y-axis and time is shown on the X-axis. The area under the hydrograph curve is the discharge volume when discharge versus time is displayed. Hydrograph studies are often used to design water storage and treatment facilities.

Fish passage designs, temporary water management facility designs, and temporary construction require discharge estimates for a portion of the year. This part of the year is not the flood season, in almost all applications. Discharge estimates are calculated for the months the structures or facilities will be in place. Consecutive daily low flows, exceedance discharges, and maximum predicted discharges are used.

## 7.4 Hydrology in the Design Process

Hydrology is estimated early in the design process. It is good practice to use more than one method as a sensitivity or reasonableness check. The results from these various methods are used as discharges in the hydraulic modeling of the existing site or structure, such as a bridge or culvert. The hydraulic performance of the site or structure shown by the model is compared to the hydraulic history. The hydrology that produces the most reasonable results, based on site history, is selected for the subsequent design.

*Note: Verify the design hydrology is reasonable by comparing with at least two other applicable methods. This is especially important for critical structures. There are local micro-climates in many areas of Oregon that are not well represented by certain methods. This comparison is usually sufficient to detect these circumstances and aid in selecting the appropriate hydrology method.*

Multiple hydrology methods can also be used for storm drain system analysis and design. Typically the drainage area upstream from the system outfall is sufficiently large to allow several methods to be used. The results are compared to each other and to the site history. The discharge at the outfall is estimated. This is done before the system is analyzed in detail. The next step is to analyze the system in detail, and to adjust the model, as needed, to produce an outfall discharge similar to the value calculated in the previous step.

## 7.5 Supporting Data for Hydrologic Studies

Supporting data are needed to perform a hydrologic study. It is available from many sources, such as published references, calculation results, and drainage surveys. These topics are discussed in this chapter and **Chapter 6**.

It is important to obtain or request hydrologic data early in the design process. Some information, such as Flood Insurance Study hydraulic models, need to be ordered well in advance of when they will be needed. Data collection, such as watershed surveys, may take several weeks or months to complete. Considerable hydraulic information is routinely collected during ODOT surveys. Usually additional information will be needed. It is the designer's responsibility to obtain or request this additional information. A typical procedure to obtain supporting data follows.

**Step 1 -** Determine the type of facility to be designed. Various types of facilities require different kinds of hydrologic studies.

**Step 2 -** Determine the appropriate kind of hydrologic study for the facility. Usually the facility design needs either a study to determine peak flow rate or a hydrograph analysis. Fish passage facilities will need exceedance discharges and a consecutive day low flow analysis. Temporary water management for almost all facilities will require exceedance discharges. Critical features to be in place for a portion of the year may require an estimate of the maximum predicted discharge.

**Step 3 -** Determine suitable study methods. More than one method is used in many studies. The answers from the various methods are compared to each other and to the historic hydraulic performance of the site.

**Step 4 -** Review the procedures for each method and list the data requirements.

**Step 5 -** Obtain the supporting data.

As an example, a detention pond is to be designed. The facility design needs peak discharges, flow versus time relationships, and volume versus time relationships. The USGS Urban Regression Equations and the NRCS TR-55 method will be used to determine peak discharges. The NRCS TR-55 method will also be used to calculate the flow and volume versus time relationships.

The USGS procedure is reviewed. Drainage area characteristics are needed. They will be obtained from a USGS quadrangle map and a local Drainage Master Plan.

The TR-55 procedure is reviewed. It will need soil types and drainage path characteristics in addition to much of the information collected for the USGS method. Soil types will be obtained from the NRCS website. Drainage path characteristics will be obtained from a site visit.

## 7.6 Hydrologic Methods

Many hydrologic methods are available, and some of the more commonly used procedures are summarized in this section and presented in the chapter appendices.

Many of the methods presented in the appendices are available on computer programs. When using computer programs it is important to assure that regional and local data is inputted as much as possible. As an example, user-supplied rainfall intensity data for Oregon is preferred over the national rainfall intensity data included in some of the computer programs.

### 7.6.1 Outside Agency Hydrology

Communities and counties in Oregon adopt Drainage Master Plans to implement solutions to existing drainage problems and to assure that flooding will be minimized after land is developed. In addition, irrigation districts, drainage districts, power companies, and others have plans or procedures for their facility operation. These documents often include data that should be considered in the hydrologic study. This data is often called "outside agency hydrology." Outside agency hydrology almost always provides peak flow data, only.

There are many uses for outside agency hydrology. Its use can reduce the time used for design and save considerable expense. Use of outside agency hydrology may be required for hydraulic modeling to demonstrate the proposed hydraulic design conforms to local floodplain regulations or other standards.

Often the designer will make an independent hydrologic analysis to verify and supplement the outside agency hydrology. The designer should check the following before outside agency hydrology is used.

- Is the outside agency hydrology based on current data?
- Does the hydrology include the expected storm runoff?
- Have there been any changes in the drainage basin or facility since the current study, and do these changes make the outside agency hydrology invalid?
- Does the community or agency anticipate any changes to the current study?

### 7.6.2 Flood Insurance Studies

Flood Insurance Studies (FIS) for many Oregon waterways have been compiled for the Federal Emergency Management Agency (FEMA). The studies develop risk data to establish flood insurance rates and to assist communities in flood plain management. Detailed studies are made of flood prone areas using hydrologic and hydraulic analyses. These studies are used to determine the 10-year, 50-year, 100-year (base flood), and 500-year flood discharges, elevations, and water surface profiles; the 100-year and 500-year floodplain boundaries; and the 100-year floodway elevations and boundaries. The study results are adopted into many local ordinances. Peak discharges are listed in FIS. Hydrographic information, exceedances, or daily data are not included.

All ODOT projects in floodplains must be checked to see if they are in a floodway. The discharges and floodway boundaries in the FIS should be used in the analysis that shows that the project meets the floodway management ordinances if a project is in a floodway established by a detailed FIS. The FIS hydrology can also be used in the project design, except in the following circumstances.

- A credible independent analysis or community drainage master plan lists higher flows than the FIS. In these cases, the higher flows may be the more appropriate design values.

- Land use changes have occurred since the FIS that produce greater discharges than those listed in the FIS. In this instance, the higher discharges would be the design values.
- The project will be built to accommodate future flows that are higher than those listed in the FIS. In this case, the higher flows would be the design values.

A revision of the FIS hydrology can be requested if the FIS flows can be shown to be over or under estimated. The revised hydrology must be done by acceptable methods, and it may be used in the hydraulic study if the revision is approved by the appropriate regulatory agencies. Revision requests should only be undertaken after careful consideration. A revision request takes considerable resources and time to prepare, and an extended period is needed for review and approval. (Assume 1 year to 1.5 years to prepare, review, and obtain approval, if it is approved. Submitting a request to revise FEMA hydrology does not guarantee it will be approved.)

FIS hydrologic data is determined by examining the input or output from the applicable hydraulic model. These models are available from FEMA, and they use the U.S. Corps of Engineers Hydraulic Engineering Center - 2 (HEC-2) or the Hydraulic Engineering Center - River Analysis System (HEC-RAS) computer programs. FIS hydrologic data is also summarized in the FIS study text. The text summaries, however, do not always list all of the locations where the discharge rate changes in the hydraulic model. The designer should independently verify the following items when using FIS discharges.

- Is the hydrology realistic?
- Is the hydrology based on current data?
- Does the study reflect current basin conditions?
- Does the community anticipate any future changes to the current study?

### 7.6.3 Statistical Analysis of Stream Gage Data

Statistical analysis of stream gaging station records should be used if the gage is located on the stream and there are sufficient years of record. Twenty years are the minimum and at least 30 are preferred. The analysis results may need to be adjusted if the study site and the gage are at different locations on the stream, and the ratios of ungaged to gaged drainage areas are within specified limits. Formulae for these adjustments are in the appendices to this chapter, as follows:

- Western Oregon - Appendix B
- Eastern Oregon - Appendix C
- Arid Regions of Eastern Oregon - Appendix D

Statistical distributions based on gage data are periodically published by the USGS for many Oregon streams. The USGS Open-File Report 93-63 "Statistical Summaries of Streamflow in Oregon: Volume 2 - Annual Low and High Flow, and Instantaneous Peak Flow" lists statistical distributions. Distributions were also calculated during the development of the regional regression equations in Appendices B, C, and D, and they are listed in the publications.

The published statistical distributions are sufficiently accurate for most applications if the period of record used to develop the distribution includes all of the available streamflow data. If additional years of record are available, a new statistical analysis of the gage data should be made using the additional data.

The major source of stream gage records is from USGS, which collects flow data throughout the state. Gage data compiled by the USGS are published in an annual Oregon Water Resources Report. These reports are on file in the ODOT Geo-Environmental Section's Engineering and Asset Management Unit. USGS gage data is also available on the USGS website.

When a statistical analysis is desired and enough stream gage data is available, it should be analyzed using a log-Pearson Type III distribution. This method is appropriate, provided there are sufficient years of record. The log-Pearson Type III procedure in HEC-SSP or PEAKFQ provides peak flow estimates, only.

The Corps of Engineers HEC-SSP or USGS PEAKFQ programs use the statistical analysis method preferred by ODOT. The SSP procedure is described in the draft Corps of Engineers publication titled "HEC-SSP Statistical Software Package: User's Manual." The PEAKFQ method is described in the draft USGS publication titled "Users Manual for Program PEAKFQ, Annual Flood Frequency Analysis Using Bulletin 17B Guidelines." Typically, when ODOT uses this program:

- A weighted skew coefficient is used, and it is based on the station skew and the generalized skew coefficients. Generalized skew coefficients are shown on the map included with the March 1982 USGS Bulletin #17B titled "Guidelines for Determining Flood Flow Frequency." A skew map is provided in USGS Scientific Investigations Report 2005-5116 publication titled "Estimation of Peak Discharges for Rural, Unregulated Streams in Western Oregon" may be used for western Oregon, if desired.

- Flows are reported from the "COMPUTED CURVE" curve or table on the HEC-SSP or PEAKFQ output.

Hydrology based on a statistical analysis of gage data should be used with care. Many Oregon gaging stations have a relatively short period of record such as ten to twenty years. During the period when many of these stations were in operation, there was a major flood of 50 to 150-year recurrence interval, such as the 1964-65 flood, and a very dry year, such as 1978. These extreme events can distort a flow versus recurrence interval curve based on a short period of record. An unrealistic flood flow versus recurrence interval relationship can often be detected by a comparison with the statistical analyses in recent USGS publications of nearby gaging stations with longer periods of record. Another indicator of an unrealistic discharge versus recurrence interval relationship is how well the curve fits the plotted position of the gage data.

### 7.6.4 USGS Regression Equations

USGS regression equations are another method of determining flood flow versus recurrence intervals. They are often used when insufficient or no gage data is available, and they are applicable for these situations.

- A typical stream in the region used to develop the equations.
- Streams with no significant regulation or flow diversion upstream from the study site.
- Streams with drainage areas and other characteristics within the limits listed in the studies.

The rural equations calculate the 2-year through 100-year flows. The calculated flows are plotted on logarithmic probability paper and a best-fit line or curve is drawn through the data points to extrapolate the 500-year discharge. The urban equations provide the 2 through 500-year flows and extrapolation is not needed.

The USGS rural and urban regression equations are most often used to calculate peak discharges. The USGS has developed a method to calculate a runoff hydrograph for urbanized basins using the results of the USGS urban regression equations. This method is discussed in the Federal Highway Administration's Hydraulic Design Series No. 2 "Highway Hydrology." This procedure provides an alternative method to compare to the results of NRCS and Santa Barbara Urban hydrograph procedures.

#### 7.6.4.1 Regression Equations for Rural Western Oregon

Regression equations for western Oregon are in the USGS Scientific Investigations Report 2005-5116 "Estimation of Peak Discharges for Rural, Unregulated Streams in Western Oregon." A copy is in Appendix B. These regression equations are recommended for drainage basins with predominately rural land use.

#### 7.6.4.2 Regression Equations for Rural and Urban Eastern Oregon

Regression equations for eastern Oregon are in the USGS Open-File Report 82-4078 "Magnitude and Frequency of Floods in Eastern Oregon." A copy is in Appendix C. Rural and urban equations are included. These equations have been compiled into the USGS National Streamflow Statistics (NSS) Program. These equations are also included in the Hydrologic Modeling Module in the Federal Highway Administration's (FHWA's) Watershed Modeling System program.

#### 7.6.4.3 Regression Equations for Arid Regions of Rural Eastern Oregon

The 1994 USGS Open-File Report 93-419 "Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States" contains regression equations for a large part of eastern Oregon. The equations, limitations, and example problems are presented in Appendix D.

Regression equations in the USGS publication "Magnitude and Frequency of Floods in Eastern Oregon" can also be used for the area covered by the arid region equations. It is recommended that the discharge versus recurrence interval relationships be calculated by both methods in locations where either procedure can be used. The relationship that most closely matches the flood history of the site should be selected.

#### 7.6.4.4 Regression Equations for Urban Oregon

The nationwide regression equations described in the 1983 USGS Water-Supply Paper 2207 "Flood Characteristics of Urban Watersheds in the United States" by V.B. Sauer et al can be used to estimate peak discharges from urban watersheds. These equations are presented in Appendix E. The equations are used to determine:

- Quick, preliminary estimates for storm drain designs - especially outfalls.
- Checks of hydrology based on other methods. As an example, a check to verify the flows in a drainage master plan, or calculated by another method, are reasonable.
- Calculating peak discharges to use with the USGS urban hydrograph procedure.

This hydrologic method may not be appropriate in all urban situations. Land use development codes in urban areas may require other procedures. In addition, local drainage master plans often list the discharges to be used for proposed hydraulic structures.

### 7.6.5 Rational Method

One commonly used procedure for calculating peak flows from small drainages (200 acres or less) is the Rational Method. This method produces its most accurate results when used for runoff estimates from small drainages with large amounts of impervious area. Examples are housing developments, industrial areas, parking lots, etc. The Rational Method is included in Appendix F. There is also a Rational Method calculator available within the FHWA Hydraulics Toolbox program. It is also included in the Hydrologic Modeling Module in the FHWA Watershed Modeling System program. Input data such as runoff coefficients, rainfall information, etc., should be based on the procedures and information in the ODOT Hydraulics Manual when using this program.

### 7.6.6 Natural Resource Conservation Service Technical Release No. 55

The United States Natural Resources Conservation Service (NRCS), formerly the U.S. Soil Conservation Service (SCS), has developed Technical Release No. 55 (TR-55) method of determining peak flows and discharge versus time relationships for small urban and rural watersheds (watersheds with 24 hour or shorter times of concentration or areas of 25 square miles or less). The TR-55 method can also calculate the volumes of detention basins used to reduce peak flows. The method is presented in Appendix G. The appendix contains two NRCS publications. One publication, the "Oregon Engineering Handbook - Hydrology Guide" contains specific information about using the method in Oregon. The other publication, titled "Urban Hydrology for Small Watersheds" contains a detailed description of the method and its use.

The NRCS procedure can be done by hand calculations using the forms, tables, and graphs in the Appendix. It can also be done by computer using programs such as the NRCS TR-55 "Urban Hydrology for Small Watersheds" program.

The NRCS method requires much of the same basic data as the Rational Method, such as the drainage area, runoff factor, time of concentration, and rainfall characteristics. The NRCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. With the NRCS method, the direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the amount of rainfall to obtain the precipitation excess.

This method requires information about soil types from NRCS and SCS soil surveys. Requests to the NRCS for soil type information should include the outline of the drainage basin drawn on a United States Geological Survey 7-1/2 - minute quadrangle map. The NRCS soil survey data is available at the Oregon NRCS website. The address is http://www.or.nrcs.usda.gov/pnw_soil/or_data.html

The TR-55 method requires the 24-hour rainfall data for 2-year through 100-year recurrence intervals. This information is shown on the maps in Appendix H. It is also available on the National Weather Service Western Regional Climatic Center website at this address: http://www.wrcc.dri.edu/pcpnfreq.html

### 7.6.7 Other Hydrograph Methods

The TR-55 hydrograph method produces its best results when used for small to mid-sized basins. In addition, TR-55 cannot analyze some drainage features, such as diversions. Other methods are often used to calculate peak discharge rates and runoff hydrographs from larger or more hydraulically complex basins, such as the NRCS Technical Release No. 20, and the U.S. Corps of Engineers Hydraulic Engineering Center - Hydrologic Modeling System (HEC-HMS). These methods can also be used for smaller basins.

The Santa Barbara Urban Hydrograph procedure and other methods are often used for stormwater drainage, water quality, and water storage system design. These methods, as well as TR-20 and HEC-HMS are not included as appendices to this manual. Guidance in the chapter addressing specific subjects, such as stormwater drainage, storage, and water quality should be considered when using these procedures.

### 7.6.8 2-Year, 7-Consecutive Day Low Flows

A hydrologic analysis to determine consecutive day low flow estimates the lowest daily discharge expected for a period of consecutive days during an average year. ODOT fish passage design uses the two-year 7-consecutive day low flow. This is an estimate of the lowest daily discharge expected

during seven consecutive days during the driest time of an average. This low flow is less than the mean daily discharge – except when the discharge does not change during the day. Consecutive day low flows are used in fish passage design. A procedure for estimating this discharge is in Appendix I.

### 7.6.9 Mean Daily Exceedance Discharges

Mean daily exceedance discharges provide statistical information about flows expected each day throughout a typical year. The exceedance discharge is the mean daily discharge that is expected to be exceeded for a specified number of days during the subject month. Usually this is expressed as a percentage of the days in the month. As an example, the 5 percent exceedance discharge is the mean daily flow expected to be exceeded 5 percent of the days of the month, or approximately 1.5 days a month. Exceedance discharges are used in temporary water management and fish passage design. This method is discussed in detail in Appendix J.

The exceedance discharges are, at best, a rough estimate of the discharge expected on any given day. They do not consider the fluctuations that occur throughout the day. There is almost always a daily peak and daily low flow higher and lower than the mean, respectively. In addition, the exceedance discharges represent a typical month in a typical year. They do not consider that the actual time of construction may be in month or year that is a wetter or drier than normal. As a result, extra allowance should be made in the design to account for discharges higher and lower than the listed discharges.

### 7.6.10 Maximum Predicted Discharges

A hydrologic analysis to determine the maximum predicted discharge estimates the greatest mean daily discharge expected through the site for the specified month. This estimate is based on records from nearby gaging stations. This discharge is used to design critical temporary water management facilities or critical temporary construction features to be in place for a portion of the year. The procedure is in Appendix K.

### 7.6.11 Flows Estimated by Flood Elevation Data

Historic flows can be calculated using surveyed high water marks. To do this, a stage versus discharge curve is calculated for the stream cross-section at the location of the high water mark. This process is described in **Chapter 8**. The historic flow is the discharge corresponding to the stage (elevation) of the high water mark. Water surface profiles at various discharges are available in published flood studies of many larger streams and rivers. Often an approximate stage-discharge curve can be calculated from these profiles.

### 7.6.12 Flash Floods

Some areas of Eastern Oregon are subject to occasional summer thunderstorms with high intensity rainfall. Localized flash floods can result when this type of storm occurs in basins with low soil permeability. Many times these floods will discharge out of normally dry draws or canyons. The local ODOT Area Maintenance Manager and/or local residents should be contacted for information on the basin's history of flash floods for drainage basins (less than 15 square miles) in Eastern Oregon.

The hydrology should be calculated with and without considering flash floods if the basin is found to have a high flash flood potential or had past flash floods. The higher discharges should be used in the design.

The design discharge in cubic feet per second considering flash floods can be estimated by the following regression equation:

$$\text{Average Flash Flood} = 300A^{0.637} \qquad \text{(Equation 7-1)}$$

The check flood in cubic feet per second considering flash floods is:

$$\text{High Flash Flood} = 535A^{0.637} \qquad \text{(Equation 7-2)}$$

Where:

A = Basin area in square miles ($mi^2$)

### 7.6.13 Nearby Gage Data Method

The nearby gage data method estimates site hydrology using statistical analyses of measured discharges in similar nearby basins. This procedure can be used to calculate design hydrology, or it can be used to verify the reasonableness of design hydrology calculated by other methods. This method is most appropriate in climatic areas that are not well represented by regional regression equations. This method includes the following steps.

1) Locate one or more nearby gaged basins with similar hydrological characteristics.
2) Analyze the gage data with a log-Pearson Type III distribution using Bulletin 17B procedures. Use a weighted skew based on regional and station coefficients. Calculate discharges and yields at the desired recurrence intervals.

*Note: Published discharges can be used if they are weighted as discussed. Weighted peak discharge in USGS Scientific Investigations Report 2005-5116 "Estimation of Peak Discharges for Rural, Unregulated Streams in Western Oregon, or weighted discharges from the Oregon Water Resources Department website "Estimation of Peak Discharges" program are not recommended for this purpose. They are weighted using a different method.*

3) Plot the relationship between basin yield and drainage area. An individual plot is needed for each recurrence interval. Approximate the data trend for each plot using a line or curve.
4) Calculate yields for the study basin based on the curves and the study basin drainage area.
5) Calculate the study basin discharges for each recurrence interval. This is determined by multiplying the predicted yields by the study drainage basin area.

This procedure is used to calculate 2-year 7-consecutive day low flows in Appendix I.

# APPENDIX A - RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES

## 1.0 Derivation of Curves

The Rainfall Intensity - Duration - Recurrence Interval (I-D-R) curves were computed in accordance with the method described in the 1973 NOAA Atlas 2 titled "Precipitation-Frequency Atlas of the Western United States, Volume X-Oregon." A regional rainfall analysis was made by comparing rainfall data for 136 cities and areas, and thirteen zones were established that have similar rainfall intensities. A set of I-D-R curves were calculated for each zone and the curves represent average values for that zone.

## 2.0 Use of Curves

The first step is to determine the zone in which the project is located. This can be accomplished by either using the enclosed map that shows the various zones or the alphabetical listing of cities and their corresponding zones. When the proper zone is located, the I-D-R curves for that zone will provide the needed rainfall intensities. The designer must use engineering judgement in selecting the proper zone if the proposed project lies within two zones or is on or near a dividing line. The zone with larger intensity should be used if no personal knowledge of the area is available, as this gives the most conservative design.

*Note: Rainfall Zone Map and Rainfall Zone Map with Highway Milepoints are in pdf formats.*

**TABLE 1: INDEX TO RAINFALL INTENSITY-DURATION-RECURRENCE INTERVAL CURVES**

| City | Zone |
|---|---|
| -A- | |
| Albany | 7 |
| Alsea | 4 |
| Amity | 8 |
| Andrews | 10 |
| Arlington | 11 |
| Ashland | 5 |
| Astoria | 1 |
| Austin | 9 |
| -B- | |
| Bandon | 1 |
| Baker | 13 |
| Beatty | 13 |
| Beaverton | 7 |
| Bend | 10 |
| Blue River | 5 |
| Boardman | 13 |
| Boring | 8 |
| Brookings | 1 |
| Brownsville | 8 |
| Burns | 13 |
| Burnt Woods | 5 |
| -C- | |
| Canby | 8 |
| Canyonville | 5 |
| Cascade Locks | 4 |
| Cave Junction | 5 |
| Chemult | 10 |
| Chiloquin | 10 |
| Condon | 13 |
| Coos Bay | 3 |
| Coquille | 4 |
| Corvallis | 8 |
| Cottage Grove | 8 |
| -D- | |
| Dallas | 8 |
| Dayton | 7 |
| Detroit | 5 |
| Drain | 5 |
| -E- | |
| Elgin | 12 |
| Elkton | 4 |
| Enterprise | 10 |
| Estacada | 5 |
| Eugene | 7 |
| -F- | |
| Florence | 3 |
| Forest Grove | 8 |
| Fort Klamath | 9 |
| Fossil | 13 |
| -G- | |
| Gladstone | 8 |
| Glendale | 5 |
| Gold Beach | 1 |
| Grants Pass | 5 |
| Gresham | 8 |
| -H- | |
| Haines | 10 |
| Hebo | 2 |
| Helix | 13 |
| Heppner | 13 |
| Hermiston | 13 |
| Hillsboro | 8 |
| Hood River | 5 |
| Huntington | 13 |
| -I- | |
| Imnaha | 10 |
| Independence | 8 |

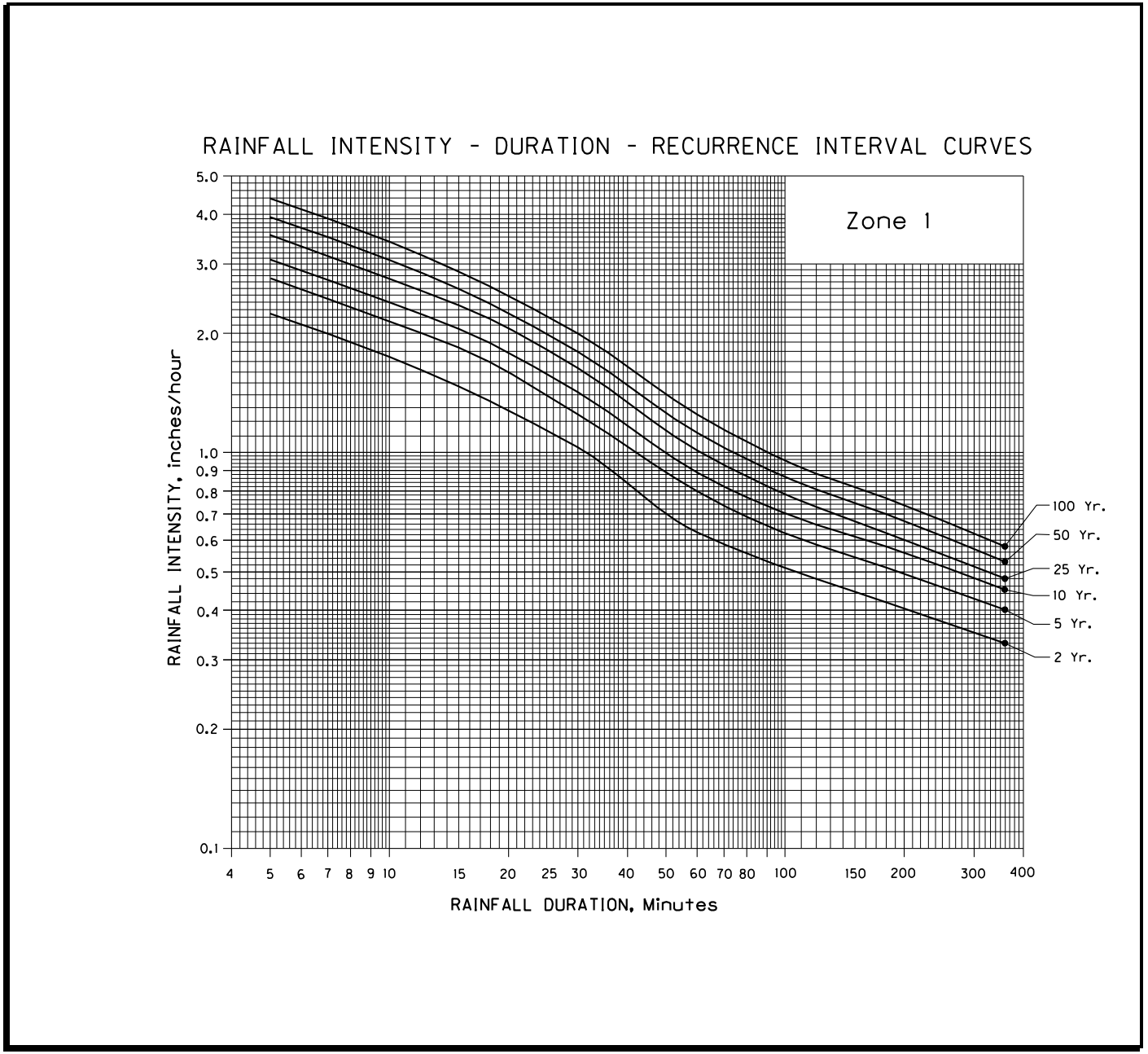
**TABLE 1: INDEX TO RAINFALL INTENSITY-DURATION-RECURRENCE INTERVAL CURVES**

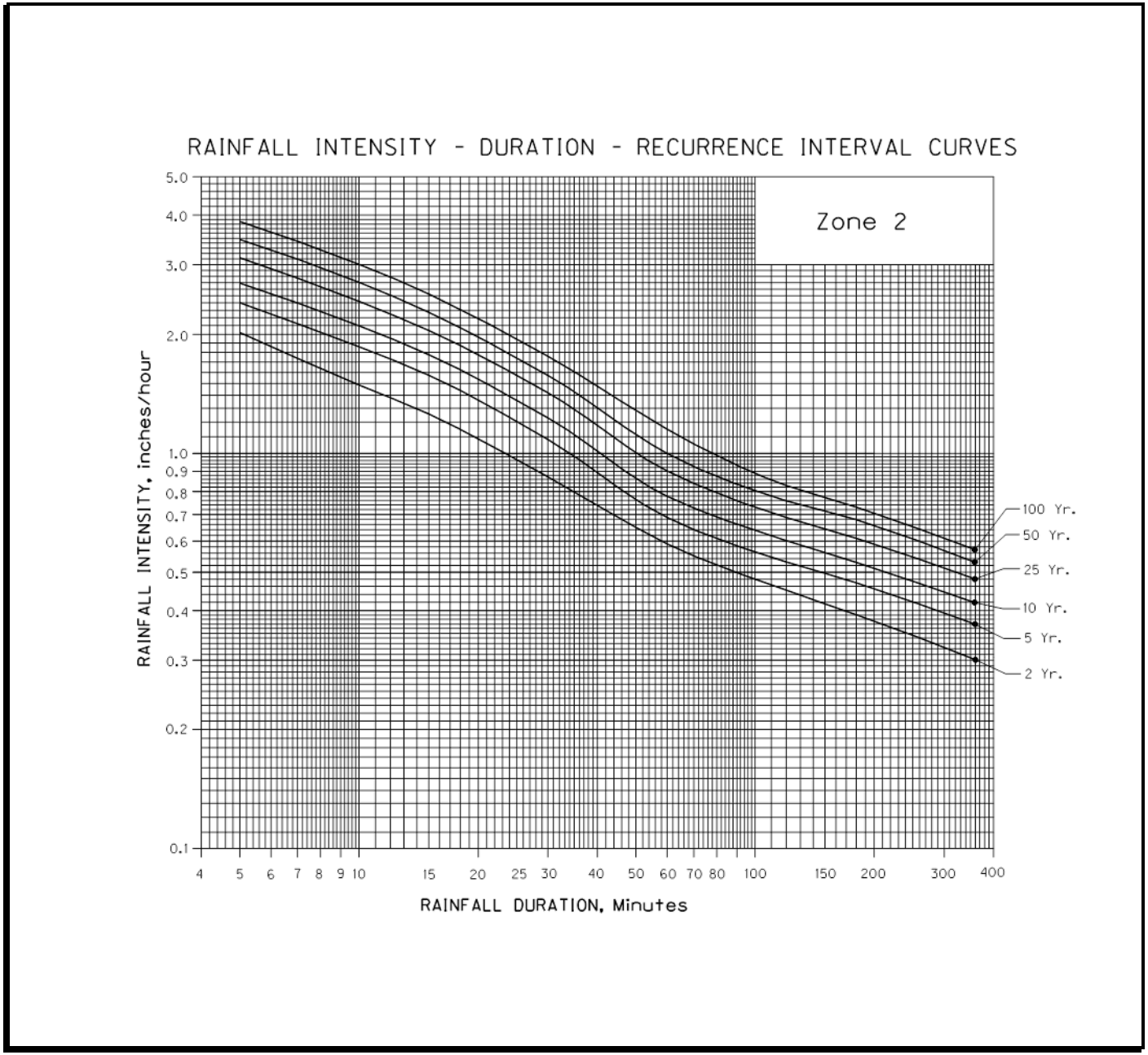
| City | Zone | City | Zone | City | Zone |
|---|---|---|---|---|---|
| -J- | | | | -R- | |
| | | Mitchell | 13 | | |
| John Day | 13 | Molalla | 8 | Rainer | 5 |
| Jordan Valley | 13 | Myrtle Point | 4 | Redmond | 13 |
| Junction City | 7 | | | Reedsport | 3 |
| Juntura | 13 | | | Rhododendron | 5 |
| | | -N- | | Rome | 13 |
| | | | | Roseburg | 6 |
| -K- | | Newberg | 7 | | |
| | | Newport | 2 | | |
| Kamela | 9 | North Bend | 3 | -S- | |
| Klamath Falls | 10 | | | | |
| | | | | St. Helens | 8 |
| | | -O- | | Salem | 7 |
| -L- | | | | Sandy | 5 |
| | | Oakridge | 5 | Scappoose | 8 |
| Lapine | 10 | Ontario | 13 | Scottsburg | 4 |
| Lake Oswego | 7 | Oregon City | 8 | Seaside | 1 |
| LaGrande | 10 | | | Seneca | 12 |
| Lakeview | 13 | | | Shady Cove | 6 |
| Lebanon | 7 | -P- | | Sheridan | 8 |
| Lincoln City | 2 | | | Sherwood | 7 |
| Lyons | 5 | Paisley | 13 | Silver Lake | 13 |
| | | Paulina | 12 | Silverton | 8 |
| | | Pendleton | 12 | Siskiyou Summit | 5 |
| -M- | | Philomath | 8 | Spray | 13 |
| | | Port Orford | 1 | Stanfield | 13 |
| Madras | 13 | Portland | 7 | Stayton | 8 |
| McMinnville | 8 | Powers | 1 | Steamboat | 5 |
| Medford | 6 | Prineville | 10 | Sumpter | 10 |
| Mill City | 5 | | | Sutherlin | 5 |
| Milton Freewater | 13 | | | Suttle Lake | 9 |
| Milwaukie | 7 | | | Sweet Home | 5 |

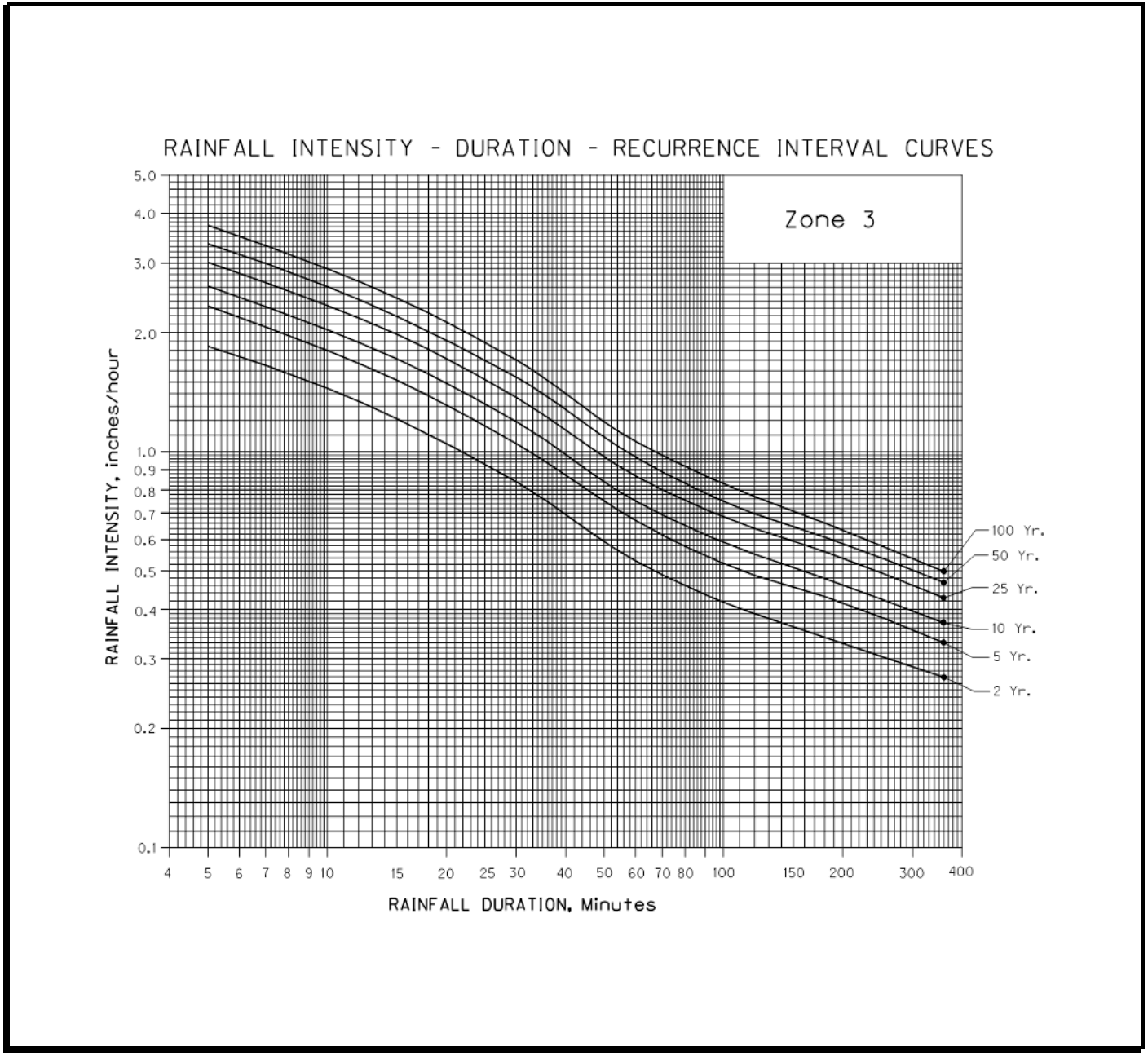
**TABLE 1: INDEX TO RAINFALL INTENSITY-DURATION-RECURRENCE INTERVAL CURVES**

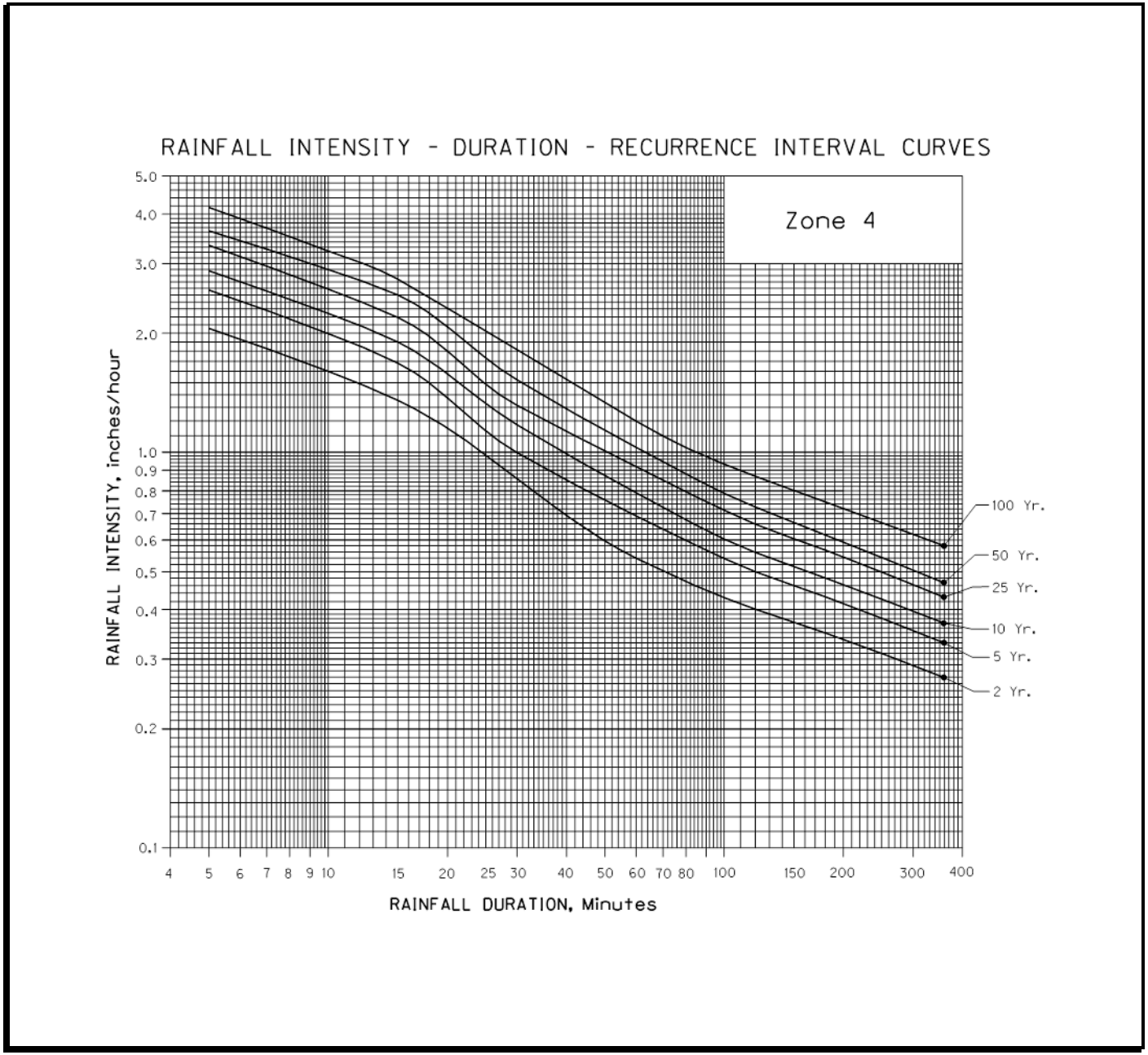
| City | Zone |
|---|---|
| -T- | |
| The Dalles | 11 |
| Tidewater | 4 |
| Tillamook | 2 |
| Tiller | 6 |
| Toledo | 2 |
| -U- | |
| Union | 13 |
| Union Creek | 3 |
| Umatilla | 13 |
| -V- | |
| Vale | 13 |
| Veneta | 5 |
| Vernonia | 8 |
| Vida | 5 |
| -W- | |
| Wagontire | 13 |
| Waldport | 2 |
| Wallowa | 10 |
| Willamina | 8 |
| Wilsonville | 7 |
| Woodburn | 7 |

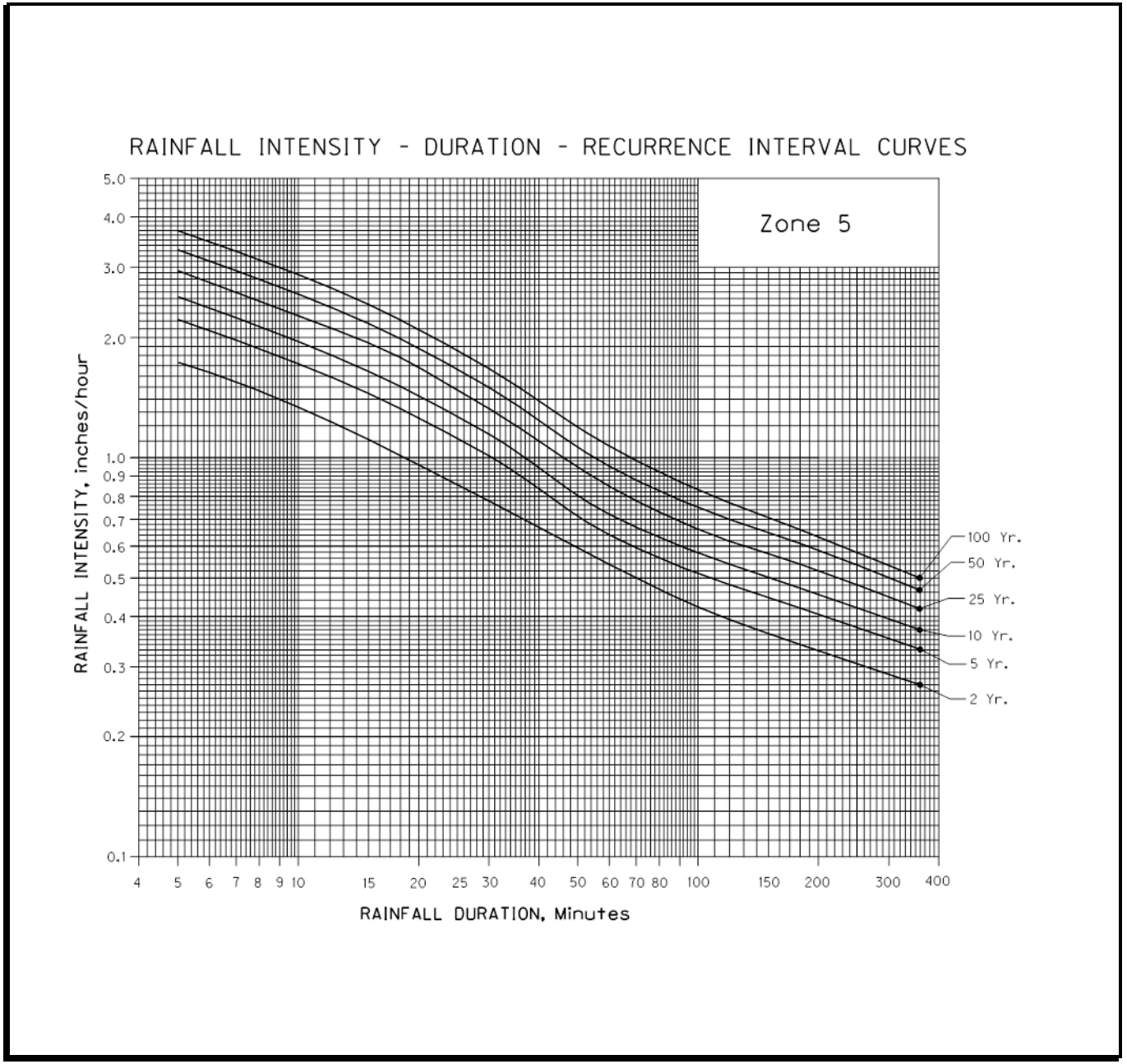
| City | Zone |
|---|---|
| -Y- | |
| Yachats | 2 |
| Yamhill | 8 |

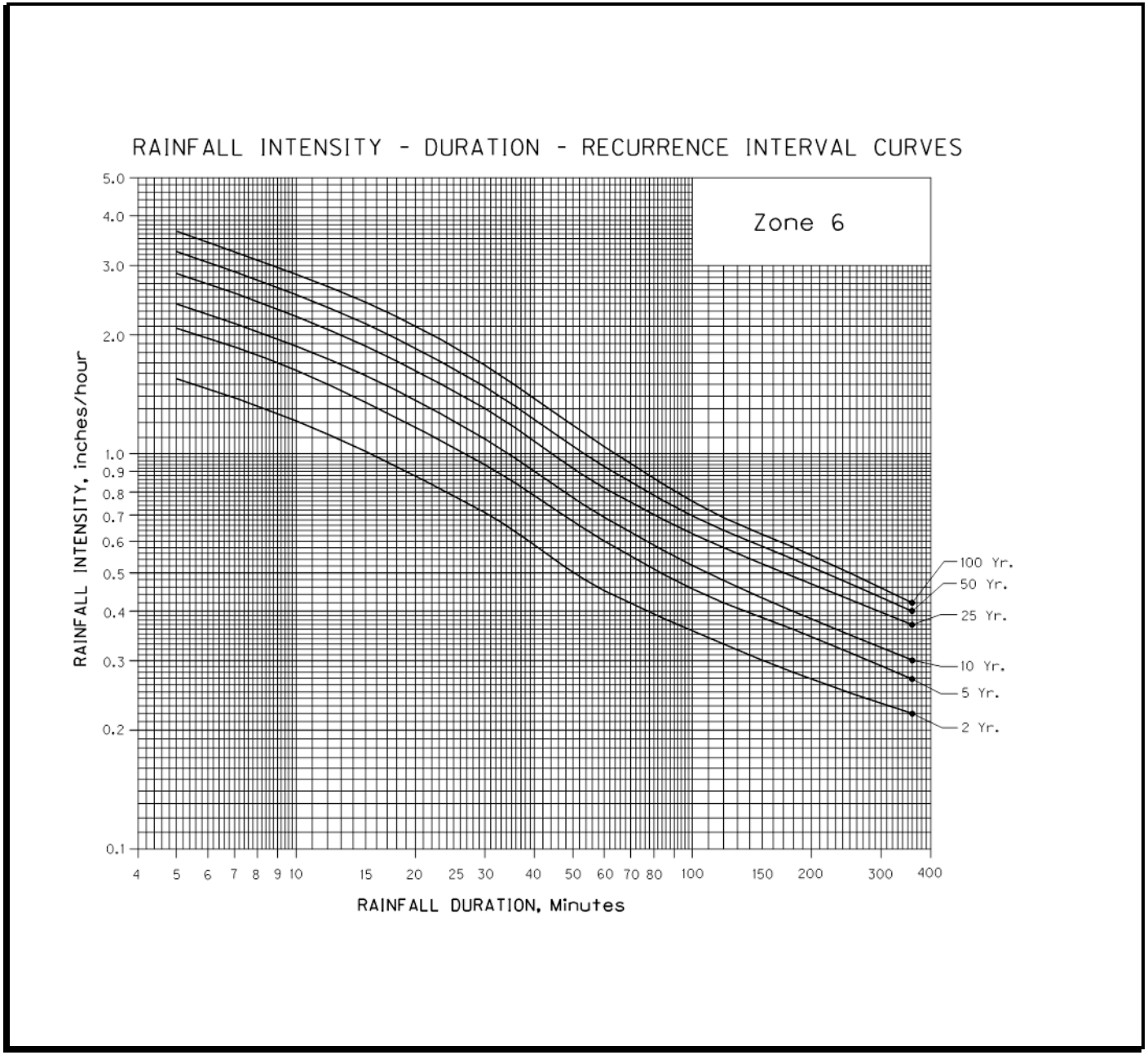

RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 1
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.

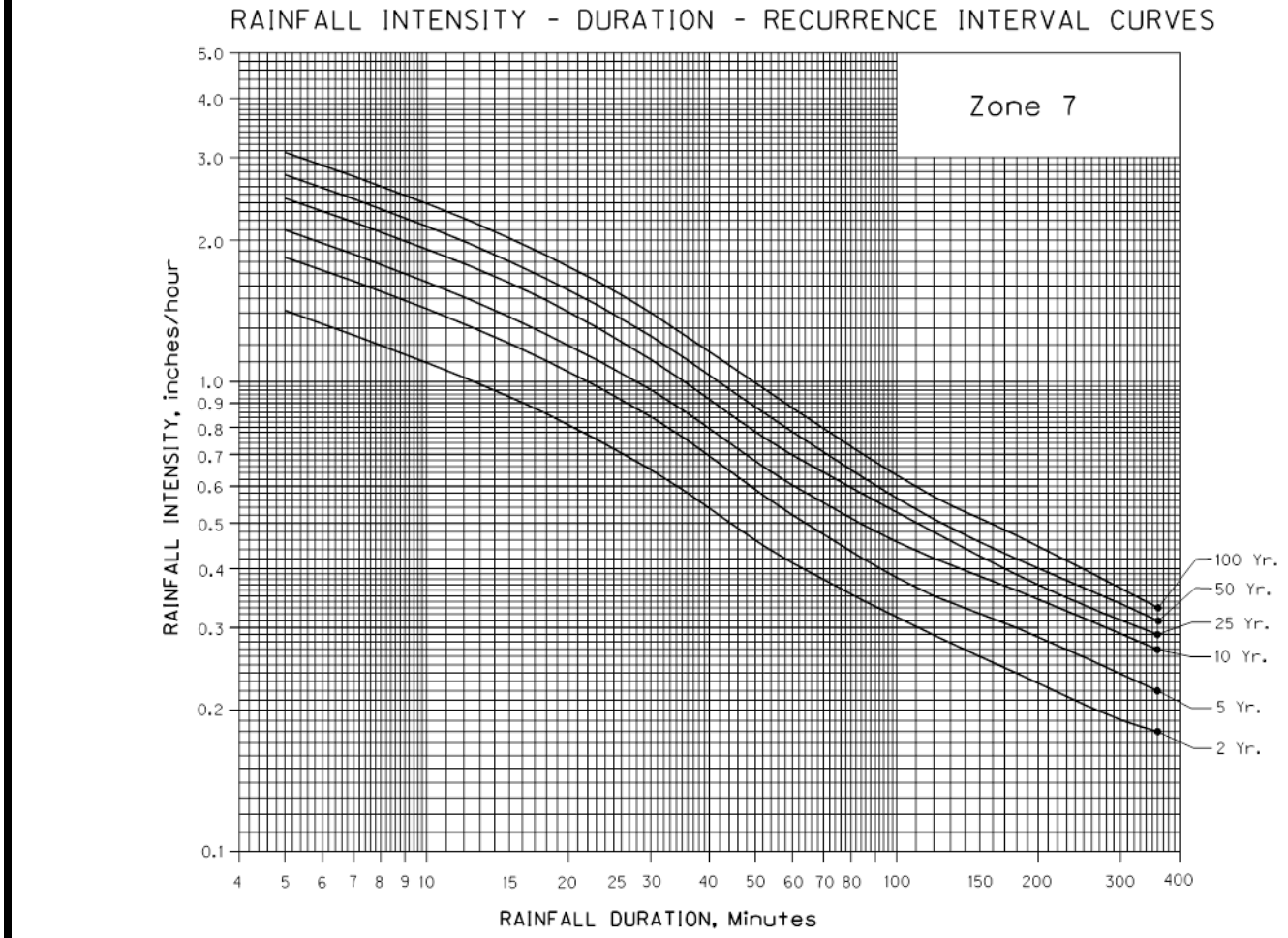

RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 2
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.

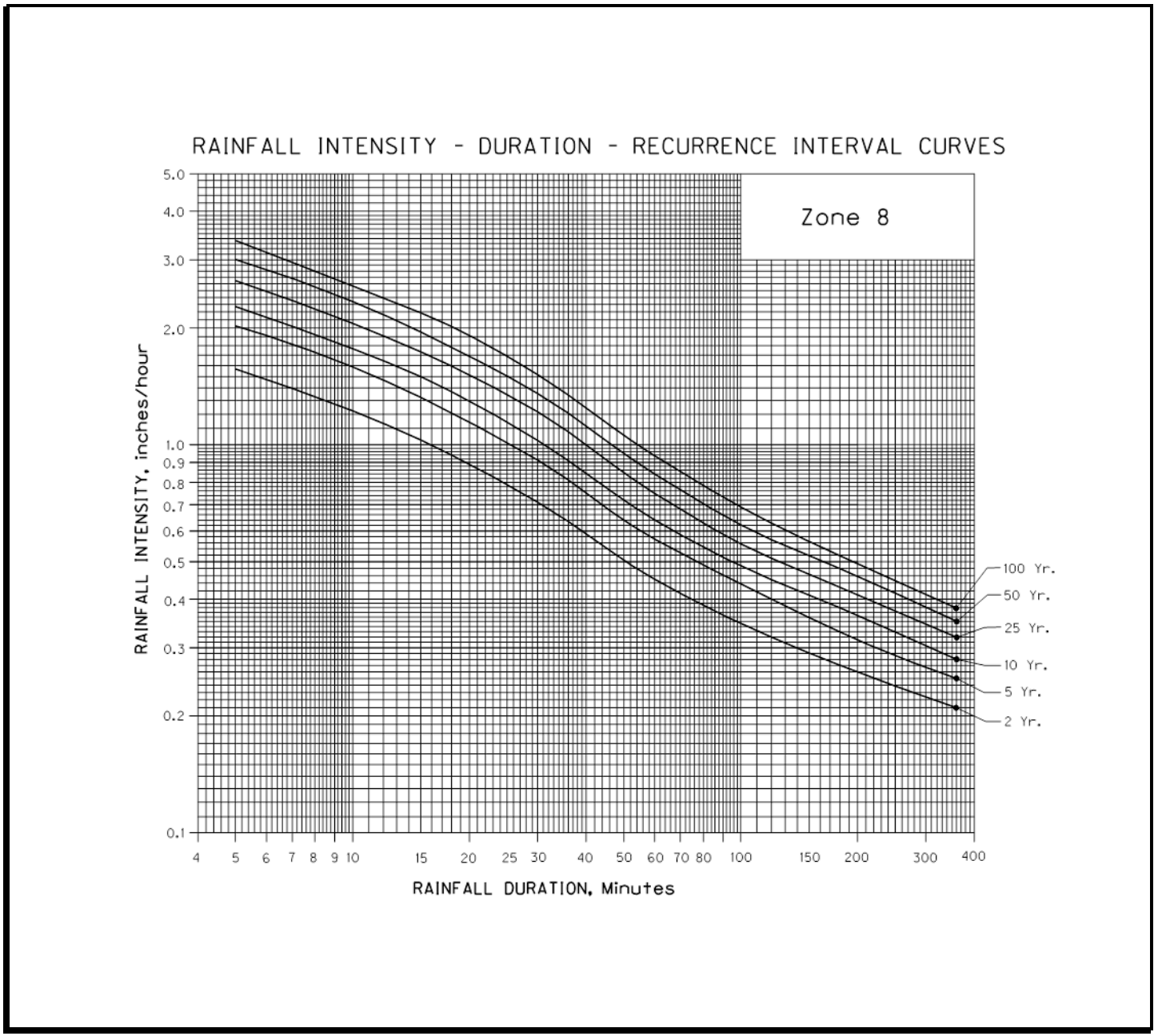

RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 3
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.


RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 4
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.


RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 5
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.


RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 6
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.


RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 7
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4 5 6 7 8 9 10 15 20 25 30 40 50 60 70 80 100 150 200 300 400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.


RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 8
RAINFALL INTENSITY, inches/hour
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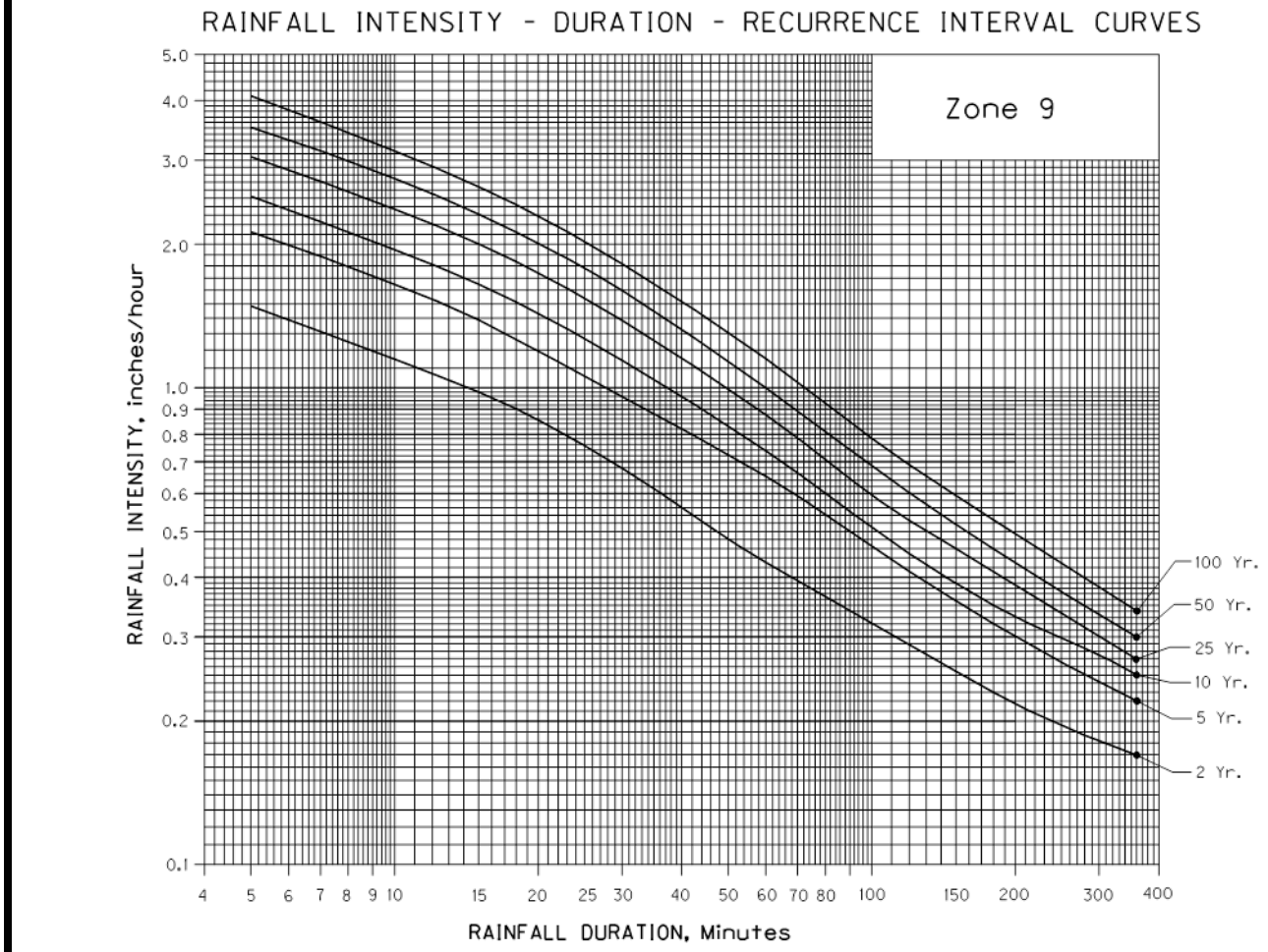
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4 5 6 7 8 9 10 15 20 25 30 40 50 60 70 80 100 150 200 300 400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.


RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 9
RAINFALL INTENSITY, inches/hour
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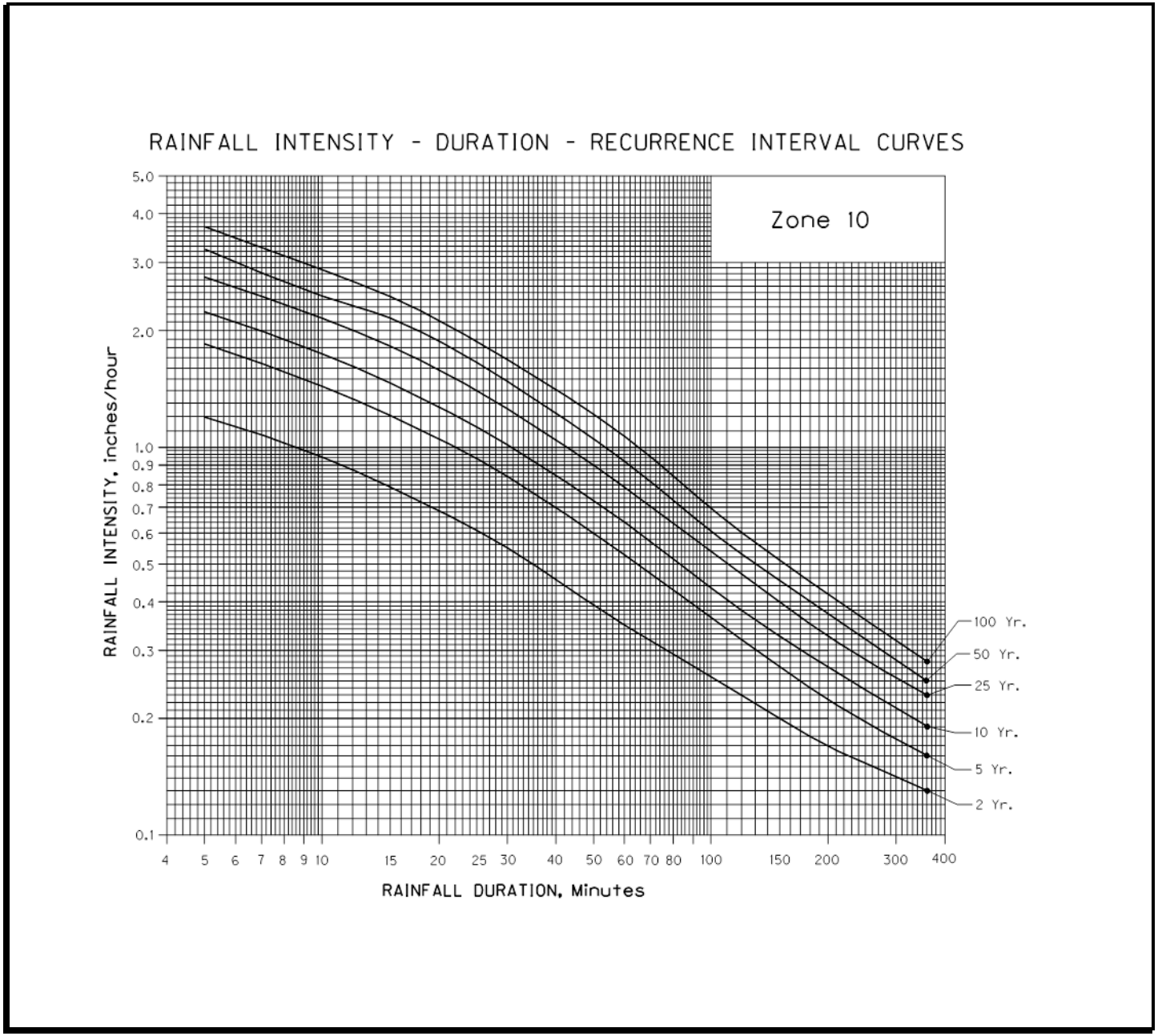
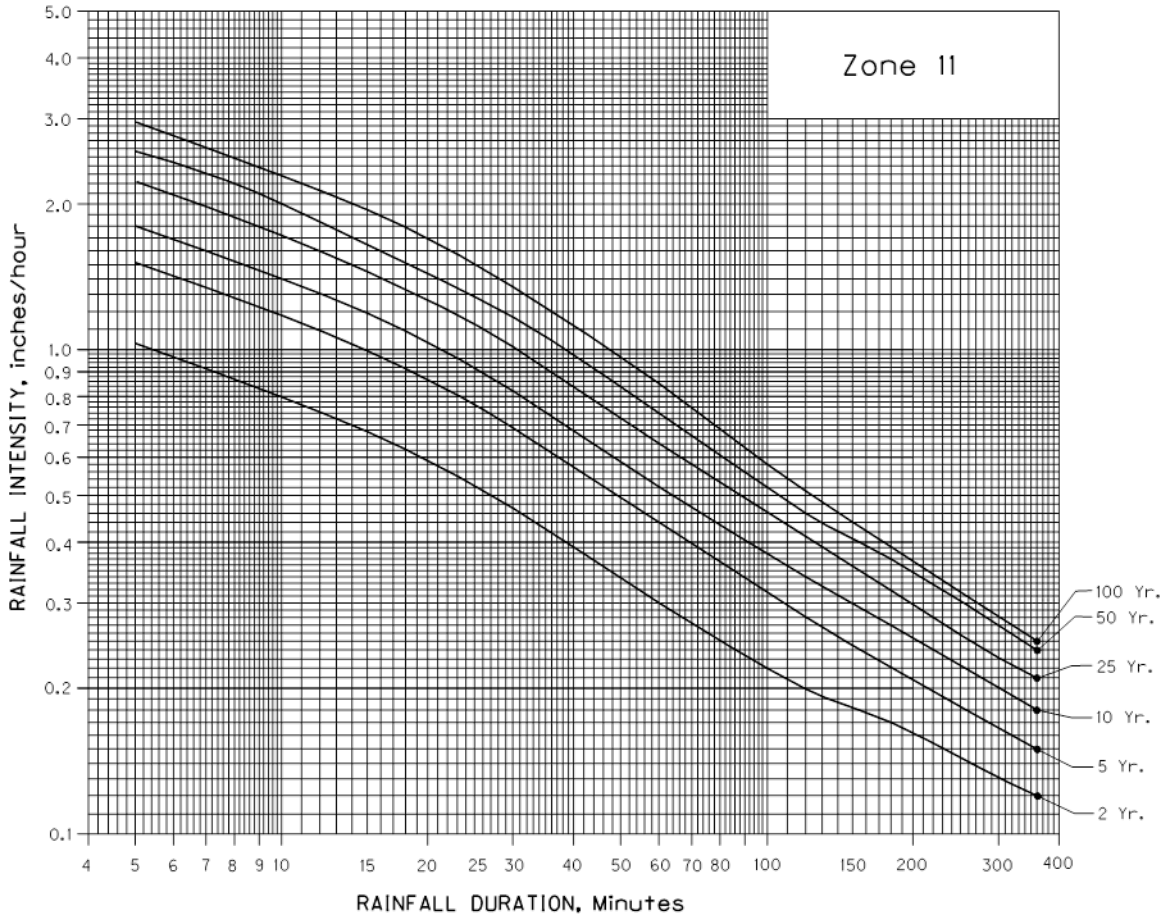
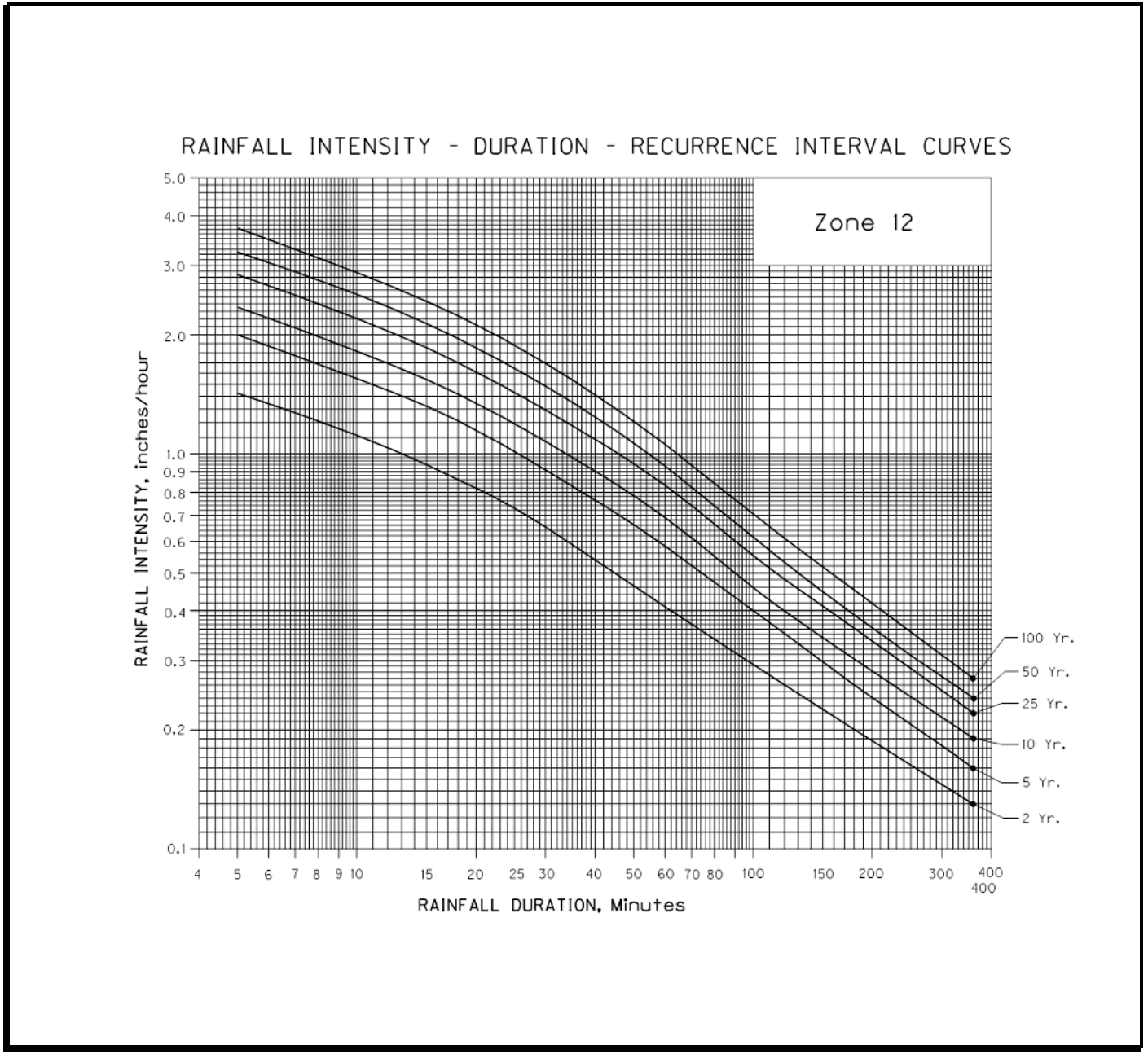
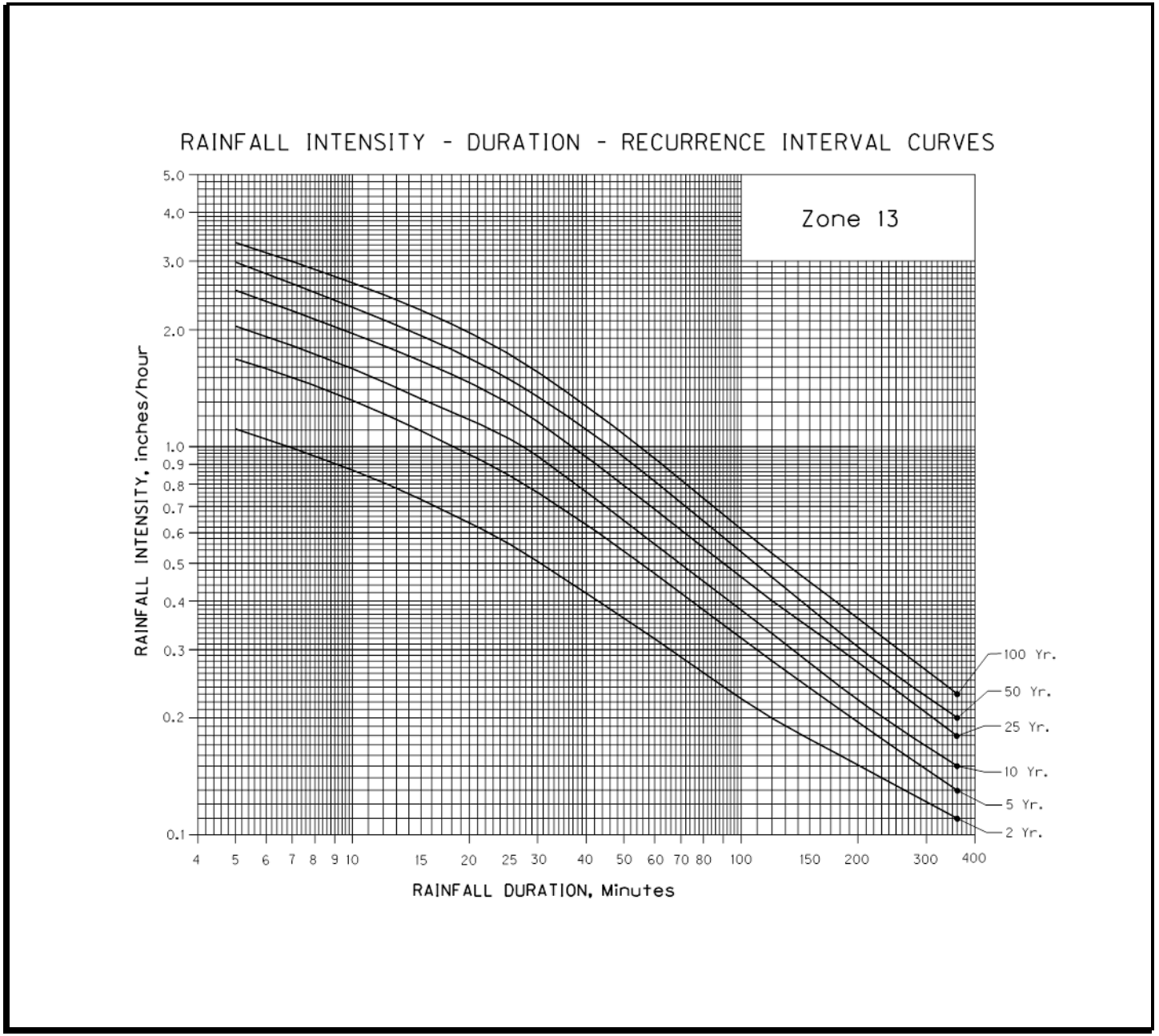
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.


RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 10
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.

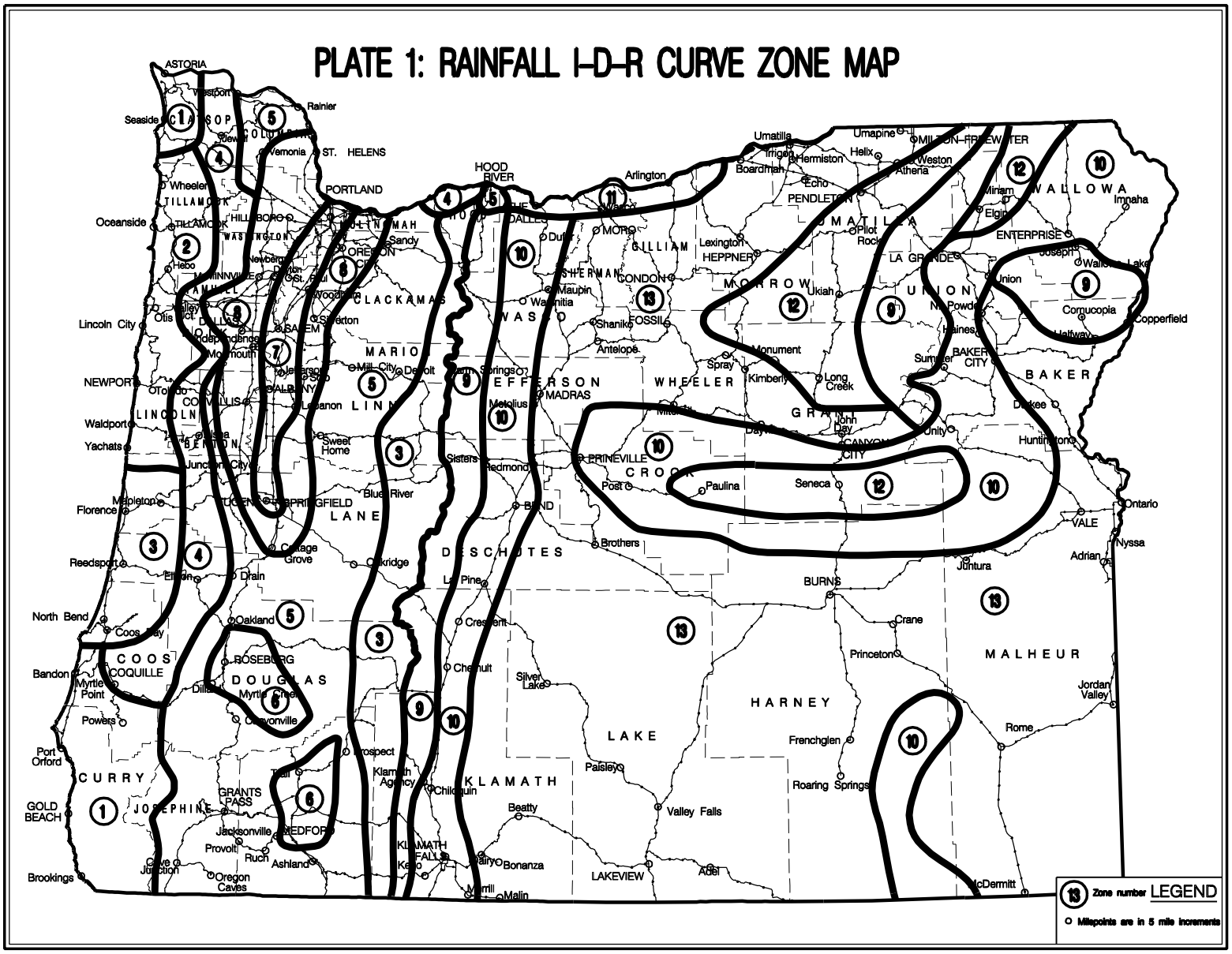
## RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES



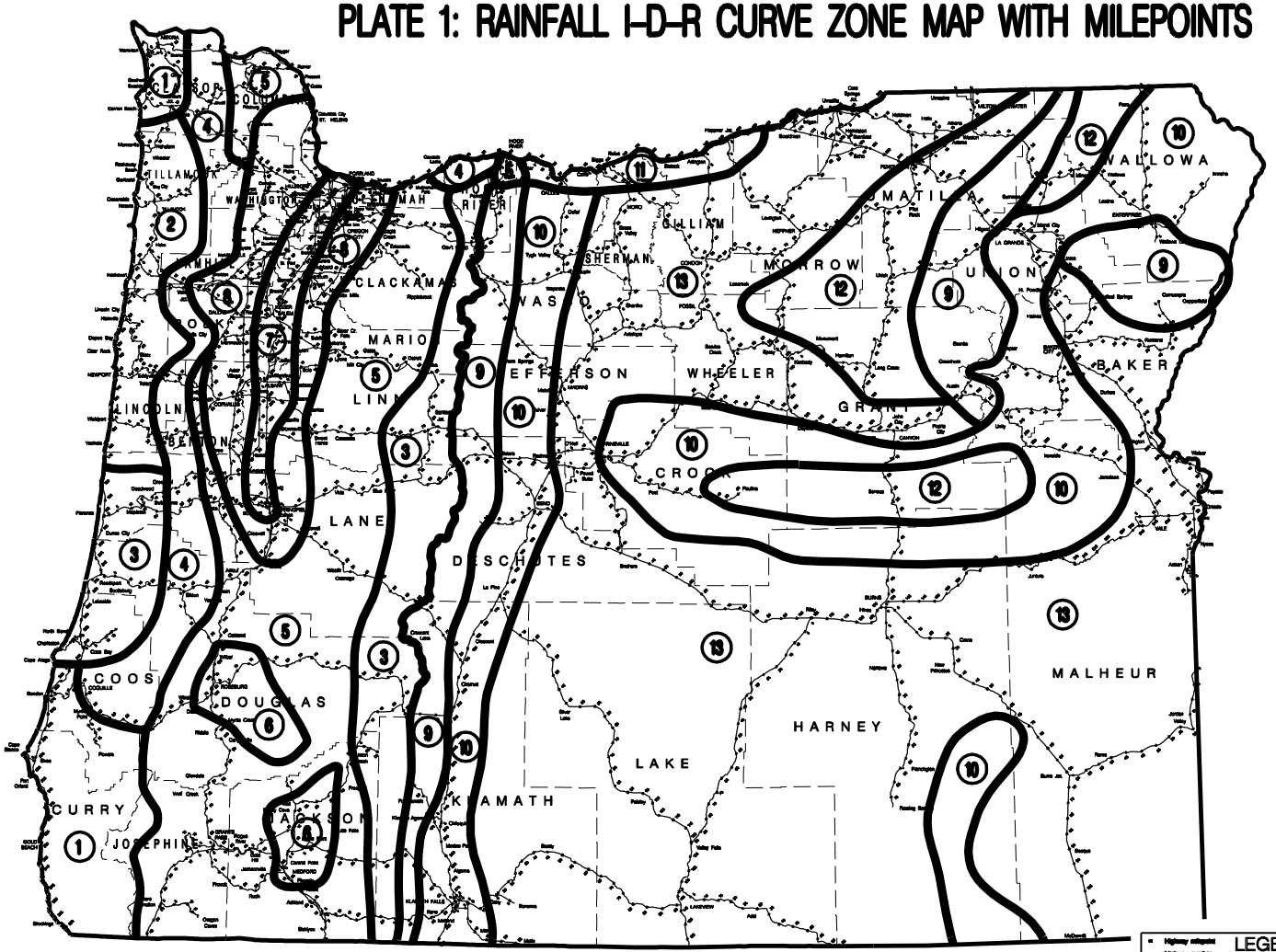

RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 12
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4 5 6 7 8 9 10 15 20 25 30 40 50 60 70 80 100 150 200 300 400
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.


RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 13
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.

# PLATE 1: RAINFALL I-D-R CURVE ZONE MAP


ASTORIA
Seaside
Rainier
Vernonia
ST. HELENS
PORTLAND
HOOD RIVER
Arlington
Umatilla
Umapine
Irrigon
Hermiston
Helix
Boardman
Echo
Weston
Athena
PENDLETON
Wheeler
TILLAMOOK
Oceanside
HILLSBORO
Sandy
MORO
GILLIAM
Lexington
HEPPNER
Pilot Rock
LA GRANDE
Elgin
Imnaha
ENTERPRISE
Joseph
Wallowa
Lake
Union
Cornucopia
Copperfield
Halfway
Haines
Baker City
Sumpter
Dixie
Huntington
Unity
John Day
CANYON CITY
Prairie City
Long Creek
Spray
Monument
Kimberly
Ukiah
Lincoln City
Otis
Depoe Bay
Newport
Toledo
Waldport
Yachats
Florence
Reedsport
Elkton
Drain
Oakridge
Cottage Grove
Sweet Home
Lebanon
Mill City
Detroit
Sisters
Redmond
Metolius
MADRAS
Shaniko
FOSSIL
Antelope
Maupin
Warm Springs
Wasco
Dufur
Mosier
The Dalles
Hood River
Gresham
Troutdale
Estacada
Molalla
Silverton
Woodburn
Mt. Angel
Scio
Lyons
Mehama
Gates
Idanha
Detroit
Blue River
Brothers
Post
Paulina
Seneca
PRINEVILLE
BEND
La Pine
Crescent
Chemult
Silver Lake
Chiloquin
Klamath Agency
Prospect
Trail
Butte Falls
Eagle Point
Shady Cove
Gold Hill
Rogue River
Grants Pass
Merlin
Galice
Wolf Creek
Glendale
Azalea
Canyonville
Myrtle Creek
Dillard
Roseburg
Oakland
Sutherlin
Drain
Elkton
Reedsport
Winchester Bay
Gardiner
Scottsburg
Coos Bay
North Bend
Bandon
Myrtle Point
Powers
Port Orford
Gold Beach
Brookings
Agness
Illahe
Marial
Kerby
Takilma
Cave Junction
Oregon Caves
Ruch
Provolt
Jacksonville
Medford
Ashland
Talent
Phoenix
Central Point
White City
Eagle Point
Shady Cove
Gold Hill
Rogue River
Grants Pass
Merlin
Galice
Wolf Creek
Glendale
Azalea
Canyonville
Myrtle Creek
Dillard
Roseburg
Oakland
Sutherlin
Drain
Elkton
Reedsport
Winchester Bay
Gardiner
Scottsburg
Coos Bay
North Bend
Bandon
Myrtle Point
Powers
Port Orford
Gold Beach
Brookings
Agness
Illahe
Marial
Kerby
Takilma
Cave Junction
Oregon Caves
Ruch
Provolt
Jacksonville
Medford
Ashland
Talent
Phoenix
Central Point
White City
Beatty
Bonanza
Malin
Merrill
Klamath Falls
Keno
Dairy
Chiloquin
Klamath Agency
Prospect
Trail
Butte Falls

# PLATE 1: RAINFALL I-D-R CURVE ZONE MAP WITH MILEPOINTS



# APPENDIX B - USGS REGRESSION EQUATIONS FOR RURAL WESTERN OREGON

This Appendix is the United States Geological Survey (USGS) Scientific Investigations Report 2005-5116 titled "Estimation of Peak Discharges for Rural, Unregulated Streams in Western Oregon". These procedures are suitable for rural areas.

*Note: It is highly recommended to independently verify the drainage area and GIS generated input coefficients are reasonable when utilizing Oregon Water Resources Department's interactive website utility that facilitates the use of these prediction equations.*

Other methods can be used to estimate runoff from developed areas. USGS nationwide urban regression equations can be used for developed areas as presented in Appendix E. Regression equations for specific urban areas in western Oregon are the subjects of the following reports:

Laenen, Antonius, USGS Water Resources Investigation Open-File Report 80-689, Storm Runoff as Related to Urbanization in the Portland, Oregon – Vancouver, Washington Area, (USGS: Portland, Oregon, 1980).

Laenen, Antonius, USGS Water-Resources Investigation 83-4143, Storm Runoff as Related to Urbanization Based on Data Collected in Salem and Portland, and Generalized for the Willamette Valley, Oregon (USGS: Portland, Oregon, 1983)

# APPENDIX C - USGS REGRESSION EQUATIONS FOR URBAN AND RURAL EASTERN OREGON

This Appendix is the United States Geological Survey (USGS) Open-File Report 82-4078 titled "Magnitude and Frequency of Floods in Eastern Oregon" published in 1983. USGS nationwide urban regression equations can also be used for urban areas.

*Note: Figure 1, Figure 2, and Figure 3 are in separate files.*

# APPENDIX D - USGS REGRESSION EQUATIONS FOR ARID REGIONS OF RURAL EASTERN OREGON

## 1.0 Introduction

Regression equations for arid regions of the United States are presented in the 1994 United States Geological Survey (USGS) Open-File Report 93-419 by Blakemore E. Thomas et al titled "Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States." The report presents regression equations and flood data for sixteen Flood Regions in ten western states. These guidelines address the two Flood Regions in Oregon, the Northwest Flood Region and the High Elevation Flood Region.

## 2.0 Flood Regions

The Northwest Flood Region is shown in Plate 1. Areas of this region at high altitudes are in the High Elevation Flood Region. The boundary between the two regions is based on the study site location and elevation, as shown in Figure 1. The Northwest Flood Region equations are used for study sites with elevations lower than the boundary elevation, and the High Elevation Flood Region equations are used for study sites with elevations higher than the boundary. A weighted average of the regression results from both regions is used if the study site is near the boundary. Peak flows within these flood regions can be estimated for gaged sites, sites near gages on the same stream, and ungaged sites.

*Note: The study site is the point on the stream where the discharge versus recurrence interval relationships are needed.*

## 3.0 Gaged Sites

Peak flow versus recurrence interval relationships at gaged sites can be estimated using the relationships in Table 1. An exception would be gaged sites where there are more recent records than Water Year 1986. In these cases, the entire period of record should be analyzed using the log-Pearson Type III method described in Chapter 7.

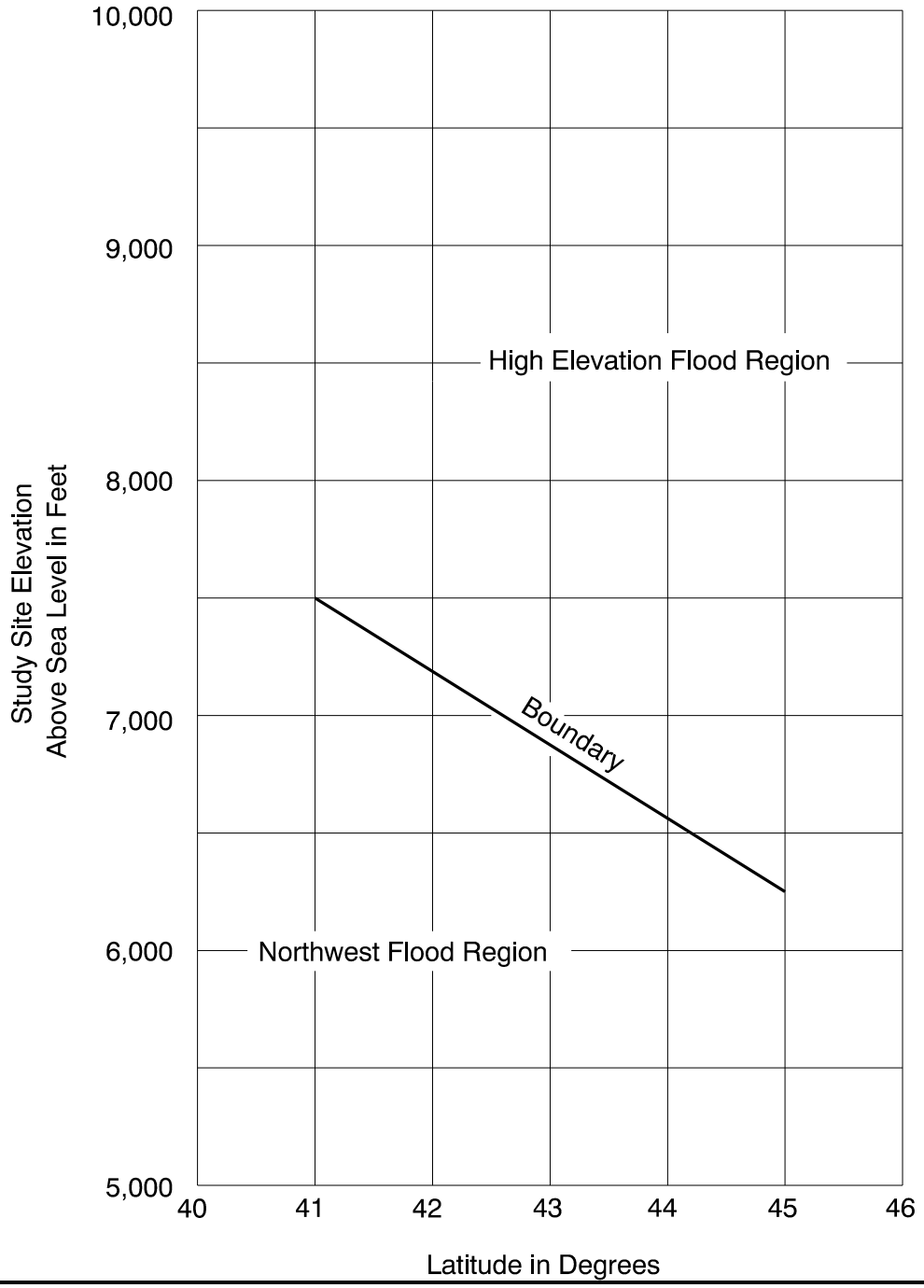



**Figure 1 Boundary of High Elevation Flood Region**

## 4.0 Sites Near Gaged Sites on Same Stream

Flood flow versus recurrence interval relationships can be estimated for ungaged study sites near gaged sites on the same stream. The ratio of the ungaged to gaged drainage areas should be between 0.5 and 1.5, and the two drainages should have similar characteristics which affect flood magnitudes, such as topography, geology, and vegetation. The following equation can be used:

$$Q_{T(u)} = Q_{T(g)} \left( \frac{A_u}{A_g} \right)^x \qquad \text{(Equation 1)}$$

Where:

$Q_{T(u)}$ = Peak discharge at the ungaged study site for T-year recurrence interval in cubic feet per second (cfs)

$Q_{T(g)}$ = Peak discharge at the gaged site for T-year recurrence interval in cubic feet per second

$A_u$ = Drainage area of ungaged site in square miles

$A_g$ = Drainage area of gaged site in square miles

x = Exponent for the flood region, as follows:

x = 0.7 for Northwest Flood Region

x = 0.8 for High Elevation Flood Region

## 5.0 Ungaged Sites

Peak flow versus recurrence interval relationships can be estimated by regression equations at ungaged study sites and sites on gaged streams with ungaged to gaged watershed ratios less than 0.5 or greater than 1.5. The equations for the Northwest Flood Region are:

| Average Recurrence interval, in years | Equation | standard error of prediction, in percent | Equivalent years of record |
|---|---|---|---|
| 2 | $Q = 13.1 (AREA^{0.713})$ | 72 | 0.96 |
| 5 | $Q = 22.4 (AREA^{0.723})$ | 66 | 1.80 |
| 10 | $Q = 55.7 (AREA^{0.727}) (ELEV/1,000)^{-0.353}$ | 61 | 3.07 |
| 25 | $Q = 84.7 (AREA^{0.737}) (ELEV/1,000)^{-0.438}$ | 61 | 4.64 |
| 50 | $Q = 113 (AREA^{0.746}) (ELEV/1,000)^{-0.511}$ | 64 | 5.47 |
| 100 | $Q = 148 (AREA^{0.752}) (ELEV/1,000)^{-0.584}$ | 68 | 6.05 |

The equations for the High Elevation Flood Region are:

| Recurrence interval, in years | Equation | Average standard error of prediction, in percent | Equivalent years of record |
|---|---|---|---|
| 2 | $Q = 0.124 (AREA^{0.845}) (PREC^{1.44})$ | 59 | 0.16 |
| 5 | $Q = 0.629 (AREA^{0.807}) (PREC^{1.12})$ | 52 | 0.62 |
| 10 | $Q = 1.43 (AREA^{0.786}) (PREC^{0.958})$ | 48 | 1.34 |
| 25 | $Q = 3.08 (AREA^{0.768}) (PREC^{0.811})$ | 46 | 2.50 |
| 50 | $Q = 4.75 (AREA^{0.758}) (PREC^{0.732})$ | 46 | 3.37 |
| 100 | $Q = 6.78 (AREA^{0.750}) (PREC^{0.668})$ | 46 | 4.19 |

Where the explanatory variables are determined as follows:

AREA = Drainage basin area upstream from the study site in square miles ($mi^2$)

ELEV = Mean drainage basin elevation in feet above sea level, determined by placing a transparent grid over the drainage basin drawn on the largest scale topographic map available. The elevations of a minimum of 20 equally spaced points are determined, and the average elevation is calculated. As many as 100 points may be needed for large basins.

*Note: ELEV is always higher than the study site elevation.*

PREC = Mean annual precipitation in inches determined by placing a grid over an isohyetal map of mean annual precipitation. The drainage area boundary is drawn on the map,

the mean annual precipitation is determined at each grid intersection, and the values are averaged for the basin. The mean annual precipitation map is shown in Plate 2.

## 6.0 Limits of Equations

The peak flow versus recurrence interval relationships are assumed to represent most streams in the arid region with 200 square miles or less drainage basin areas. Streams that may not be represented by the relationships are: streams with watershed areas more than 200 square miles, streams with significant upstream storage or diversion, streams in watersheds containing large areas of highly permeable rocks, and streams with less base flow than nearby gaged streams. The equations also may overestimate the discharges from streams with greater flood flow attenuation from overbank storage than nearby gaged streams. This overbank storage can often be significant on streams with small channels and large hydraulically rough floodplains.

The equations may overestimate flow in a stream that is part of a network of distributary flow channels, or a stream with a drainage basin having large areas of distributary flow. In these areas, discharge from a single channel flows into two or more distributary flow channels. This flow separation most often occurs on the relatively flat piedmont plains downslope from steeper mountain streams. These distributary flow channels often have an erratic flow pattern and they divide and join over wide areas of the alluvial plain. In these instances, the equations are most suited for determining discharges in the main stem channel upstream from the junction with the distributary channels.

The equations also may not represent streams with basin and climatic conditions outside the range of watershed characteristics used to develop the equations. The characteristics of the study watershed and the watersheds used to develop the equations can be compared using "clouds of common values." The clouds represent the ranges of the explanatory variables common to most of the watersheds used to develop the equations, as shown in Figure 2. It is likely that the regression equations represent flows from the watershed if the lines representing the explanatory variables intersect within the clouds. The regression equations should be used with caution if the lines intersect at a point outside of the clouds. This process is shown in the example.

Another comparison can be made to see if the predicted discharges are reasonable. It is to compare the 100-year discharge predicted by the equations to the 100-year flows from the watersheds used to develop the equations. The 100-year discharge versus drainage area relationships for the basins used to develop the equations are shown by dashed lines in Figure 3 for the Northwest Flood Region and Figure 4 for the High Elevation Flood Region. The regression equations should be used

with caution if the predicted 100-year discharge is significantly above or below the dashed lines.

The maximum historical discharge can be calculated at some study sites. It can be compared to the maximum recorded discharges from the basins used to develop the equations using Figures 3 and 4. The maximum peak discharges of record for the basins used to develop the equations are shown as dots on the figures. If the estimated historical peak discharge is significantly higher than the maximum recorded peak flows from basins of similar size, the reason should be investigated. Possible reasons can be deluges of water caused by the sudden failure of debris dams that have impounded water in the upstream basin, deluges caused by the failure of roadway embankments that have impounded water (usually due to plugged culverts), flash floods, or drainages with characteristics markedly different than the basins used to develop the equations.

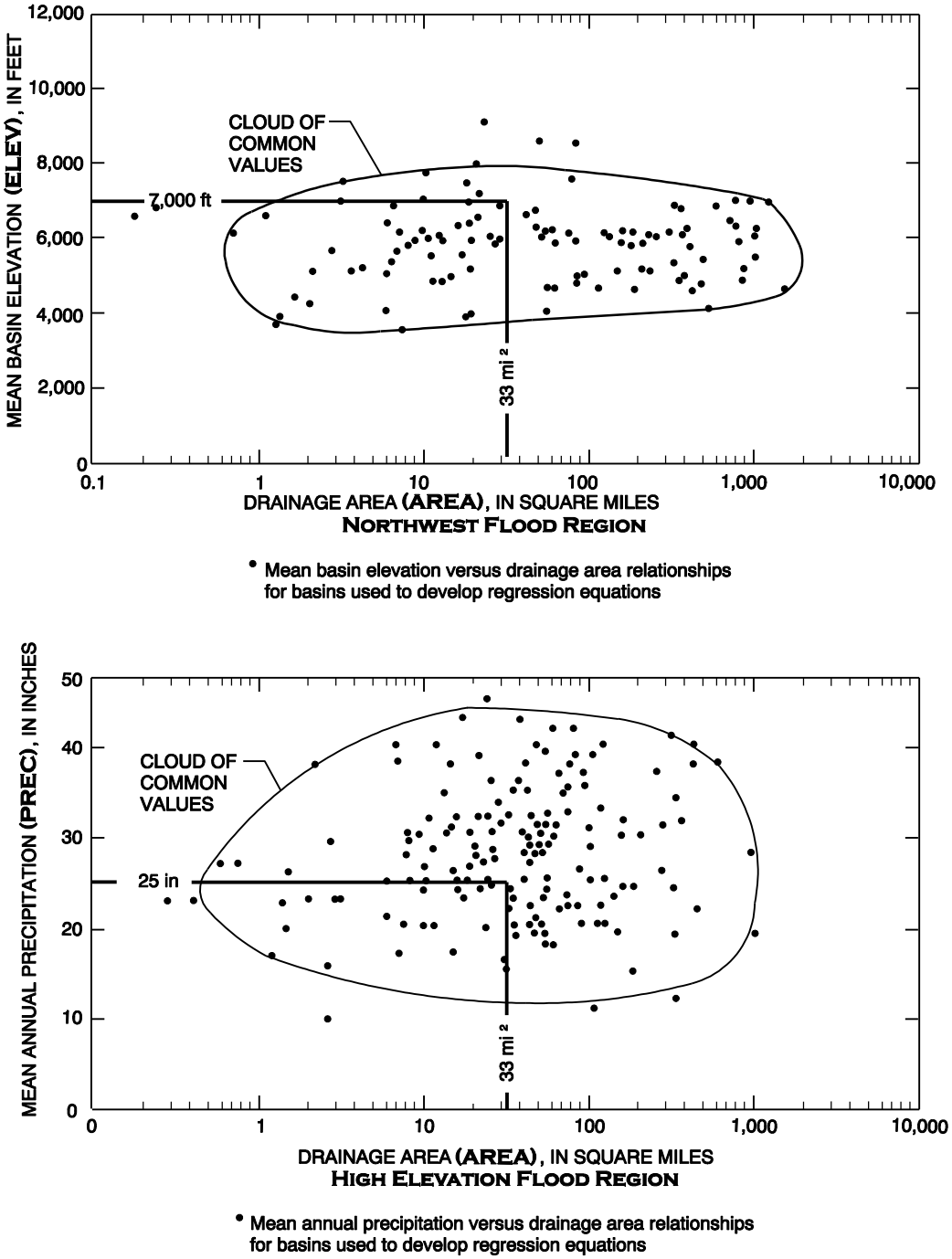



**Figure 2 Clouds of Common Values**

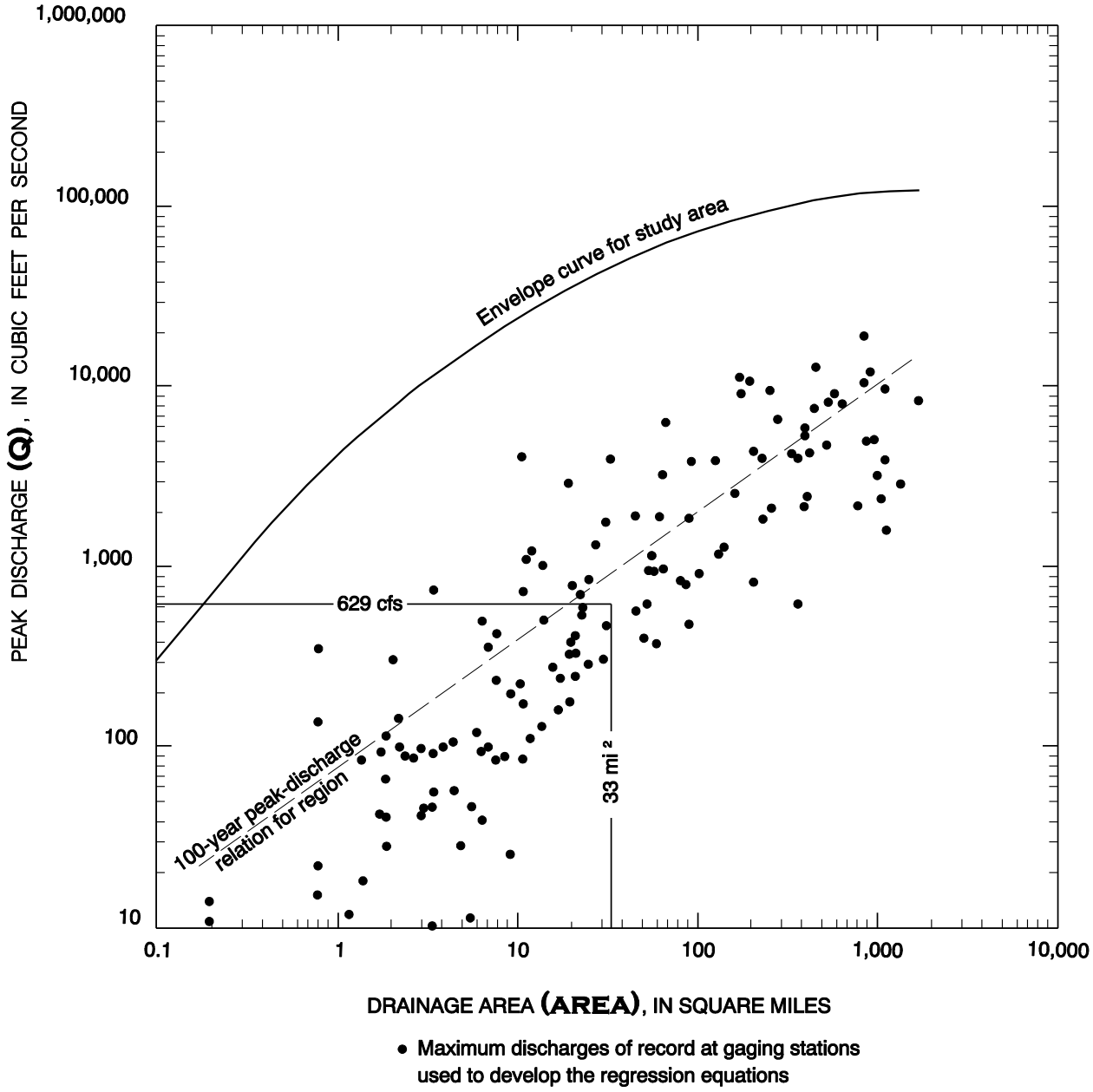



**Figure 3 Relationship Between Drainage Areas and 100-Year Peak Discharges, Maximum Peak Discharges of Record, and Maximum Potential Flood Flows in the Northwest Flood Region**

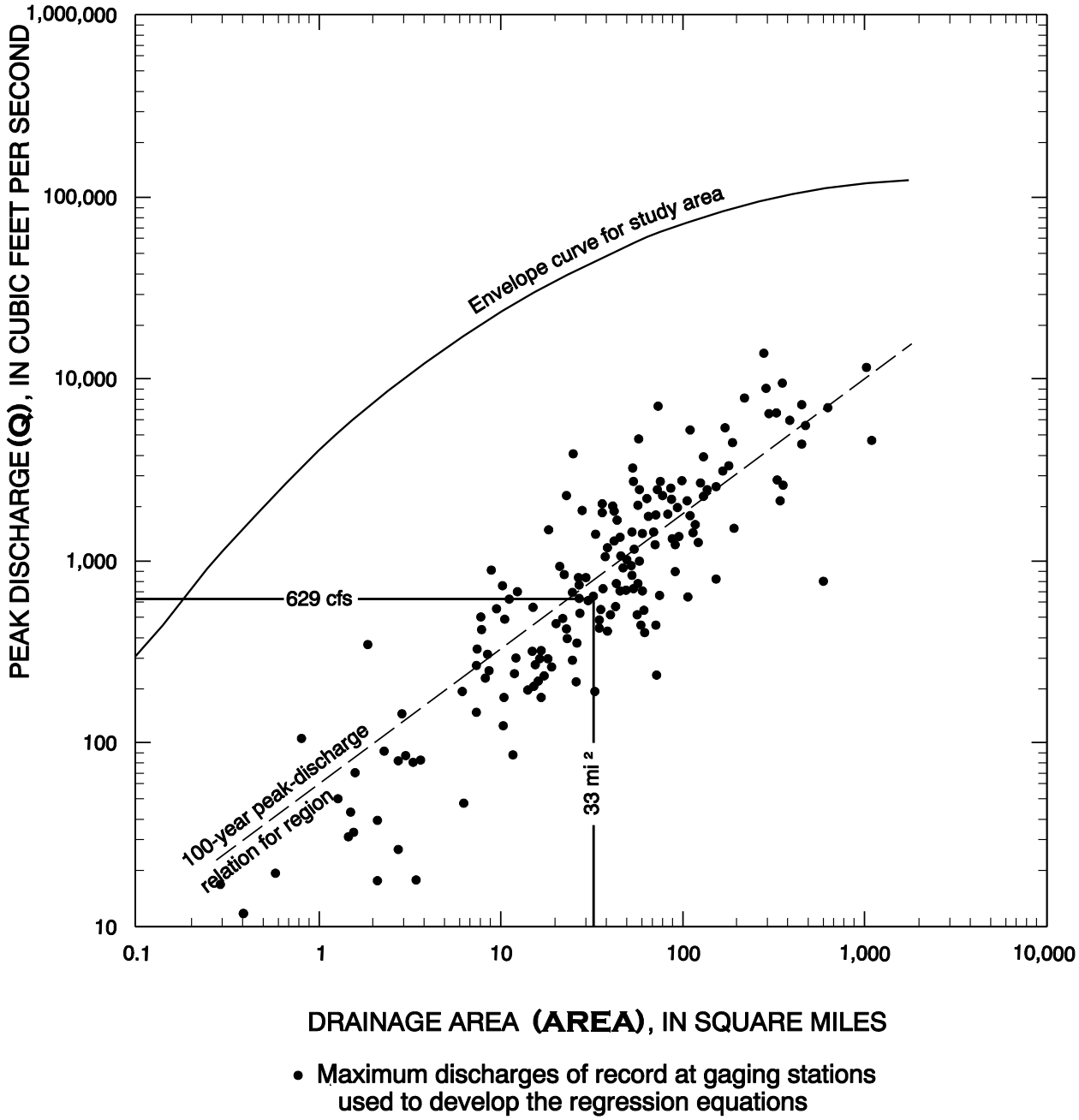



**Figure 4 Relationship Between Drainage Areas and 100-Year Peak Discharges, Maximum Peak Discharges of Record, and Maximum Potential Flood Flow in the High Elevation Flood Region**

A further comparison can be made between calculated discharges at the study site and discharges from the basins used to develop the equations. Envelope curves that enclose the maximum potential flood flows expected in the region are also shown in Figures 3 and 4. These curves are based on data from the gaged sites. The maximum potential flood flow for a basin of a given size is considerably larger than the 100-year flow. The predicted discharges may be erroneous and the cause should be investigated if the 100-year flow calculated by the regression equations for a basin is outside of the envelope curve. There may be an error in the calculations or the input to the equations.

## 7.0 Procedure

The procedure for using the equations is as follows:

**Step 1 -** Use the weighted peak flow versus recurrence interval values in Table 1 if the study site is a gaged site and the period of record does not extend later than Water Year 1986.

**Step 2 -** Analyze the gage data using a log-Pearson Type III distribution if the study site is a gaged location and there are records later than Water Year 1986. The procedure is outlined in Chapter 7.

**Step 3 -** Measure the study site drainage area, AREA or $A_u$, in square miles. Determine the gage drainage area (if there is a gage), $A_g$ in square miles. Determine the study site latitude and longitude in degrees. Determine the study site elevation (E) in feet.

**Step 4 -** Determine the ratio of the ungaged to gaged drainage areas if the study site is near a gaged site on the same stream. Use Equation 1 to determine the discharges from the ungaged basin if the area ratio is between 0.5 and 1.5.

It may be possible to analyze study sites on ungaged streams, or gaged streams with drainage area ratios less than 0.5 or greater than 1.5, with the regression equations. The remaining steps in this procedure apply to the regression equations.

**Step 5 -** Determine if the study site is within the boundaries of the Northwest or High Elevation Flood Regions using information from Step 3, Figure 1, and Plate 1.

**Step 6 -** Verify the site is suitable for a regression equation analysis. The equations may be appropriate if the drainage basin characteristics meet the listed requirements and the

explanatory variables are within the clouds of common values. The equation results should be used with extreme caution, or another hydrologic method should be used, if the basin does not meet the listed requirements or the variables are outside of the clouds.

**Step 6a -** Verify the stream has:

- a 200 square mile or less drainage area,
- no significant upstream flow detention in reservoirs,
- no significantly greater overbank storage than typical nearby gaged streams,
- no significant upstream flow diversion, and
- the stream is not part of a distributary flow pattern.

**Step 6b -** Determine the required explanatory variables as applicable, ELEV or PREC.

**Step 6c -** Verify the explanatory variables are within the appropriate clouds of common values shown on Figure 2.

**Step 7 -** Use the regression equations for the Northwest Flood Region if the study site elevation is 700 or more feet lower than the region boundary elevation. Use the High Elevation Flood Region equations if the study site elevation is equal to or higher than the region boundary elevation. Use a weighted discharge based on both the Northwest and the High Elevation Flood Regions if the study site elevation is within 700 feet below the boundary of the High Elevation Flood Region. The characteristics of the entire basin should be used in both computations. The weighted discharge should be computed as follows:

$$Q_{T(w)} = Q_{T(n)}\left[\frac{(B - E)}{700}\right] + Q_{T(h)}\left[1 - \left(\frac{B - E}{700}\right)\right] \qquad \text{(Equation 2)}$$

Where:

$Q_{T(w)}$ = Weighted discharge for T-year recurrence interval in cubic feet per second (cfs)

$Q_{T(n)}$ = Discharge from Northwest Flood Region equations for T-year recurrence interval in cubic feet a second

$Q_{T(h)}$ = Discharge from High Elevation Flood Region equations for T-year interval in cubic feet a second

B = Boundary elevation of the High Elevation Flood Region in feet

E = Elevation of the study site in feet

**Step 8 - C**heck to see if the predicted discharges are reasonable compared to the typical 100-year discharge versus drainage area relationship, peak discharges of record, and the maximum potential flood flows shown in Figures 3 or 4 for the applicable region.

## 8.0 Example Problem #1

---

The 50-year flow is needed for Bryson Brook. There is a gage on the stream near the study site.

**Steps 1 and 2 -** The peak flow versus recurrence interval relationship for the stream gage cannot be directly used. The gage is too far from the study site. These steps cannot be done.

**Step 3 -** The study site has a 64 square mile drainage area. AREA = $A_u$ = 64 square miles. The gage has a 58.0 square mile drainage area. $A_g$ = 58.0 square miles. The study site latitude is 45 degrees 10 minutes north, and the longitude is 118 degrees 53 minutes west. The study site elevation is 3,500 feet. E = 3,500 feet.

**Step 4 -** The ratio of ungaged to gaged areas is: $A_u/A_g$ = 64.0 / 58 = 1.1. The gaged flow can be adjusted to represent the discharge at the study site.

The study site, using Figure 1, is within the Northwest Flood Region based on its elevation, latitude, and longitude.

The exponent for the Northwest Flood Region is: x = 0.7

The 50-year discharge at the gage is: $Q_{50(g)}$ = 2,450 cubic feet per second.

Using Equation 1, the discharge at the study site, $Q_{50(u)}$ is:

$$Q_{50(u)} = 2{,}450 \times \left(\frac{64}{58.0}\right)^{0.7} = 2{,}620 \text{ cubic feet per second}$$

## 9.0 Example Problem #2

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The 50-year discharge is needed for Lenz Creek.

**Steps 1 and 2 -** These steps are not done. There is no gage on the stream.

**Step 3 -** The study site drainage area is plotted on a USGS 7-1/2 minute quadrangle map. The drainage area is: AREA = 33 square miles.

The study site latitude is 44 degrees 20 minutes north. The longitude is 118 degrees 30 minutes west.
The study site elevation is: E = 6,000 feet.

**Step 4 -** Not done. There is no gage on the stream.

**Step 5 -** The site is within the region covered by the arid region regression equations shown on Plate 1, based on the study site longitude and latitude. The boundary between the Northwest and High Elevation Flood Regions is 6,400 feet at latitude 44 degrees 20 minutes north, according to Figure 1. The study site elevation is 6,000 feet, so the site is within the Northwest Flood Region.

**Step 6a -** The basin is small enough to be represented by the equations because it is smaller than the maximum recommended size of 200 square miles. In the upstream watershed there are no storage reservoirs or flow diversions. The stream is not part of a distributary flow pattern. In addition, the overbank storage on this stream is not significantly greater than nearby gaged streams. As a result, the equations may be applicable.

**Step 6b -** A grid was laid out across the drainage basin boundaries drawn on the USGS quadrangle map, and the elevations of 23 grid intersection points are measured. These elevations are averaged to determine the mean basin elevation, ELEV.

ELEV = 7,000 feet

A grid is placed over the isohyetal map of mean annual precipitation in Plate 2 and rainfall is measured at each grid intersection. These rainfall values are averaged to determine the mean annual precipitation, PREC.

PREC = 25 inches

**Step 6c -** Lines corresponding to AREA and ELEV are plotted on the cloud of common values for the Northwest Flood Region. The intersection point is within the cloud, as shown in Figure 2. Lines corresponding to AREA and PREC are also plotted on the cloud for the High Elevation Flood Region. The intersection points of these lines are also within the cloud, as shown in Figure 2. As a result, the basin characteristics are within the range of explanatory variables applicable for the equations, and the equations may be appropriate for the drainage basin.

**Step 7 -** The 6,000 foot study site elevation is 400 feet lower than the High Elevation Flood Region boundary. A weighted discharge based on both the Northwest and High Elevation Flood Regions equations is needed because the study site is within 700 feet below the boundary. Using the Northwest Flood Region regression equation for the 50-year flow:

$$Q_{50}\ (113)\ (33^{0.746})\left(\frac{7000}{1000}\right)^{-0.511} = 568 \text{ cubic feet per second}$$

Using the High Elevation Flood Region equation:

$$Q_{50} = (4.75)\ (33^{0.758})\ (25^{0.732}) = 710 \text{ cubic feet per second}$$

The weighted discharge using Equation 2 is:

$$Q_{50(w)} = 568\left(\frac{6{,}400 - 6{,}000}{700}\right) + 710\ \left[1 - \left(\frac{6{,}400 - 6{,}000}{700}\right)\right] = 629 \text{ cubic feet per second}$$

**Step 8 -** Lines representing the weighted 50-year discharge and the drainage area are plotted on the relationship graphs for each flood region, as shown in Figures 3 and 4. The points where the lines intersect are fairly close to, and below, the dashed lines representing the 100-year discharge versus drainage area relationship. This is expected, because the 50-year discharge is usually lower than the 100-year discharge. The input data and calculations will need a careful review if the 50-year discharge intersection points are higher than the typical 100-year flow relationship lines.

The intersection points are well below the envelope curves. This is expected. The input data and calculations will likely be in error if the intersection points are near or above the envelope curves.

The drainage basin is suitable for the regression analysis and the calculations produce reasonable results. As a result, the estimated 50-year discharge in Lenz Creek at the study site is 629 cubic feet per second.

## Table 1 Weighted Peak Flow Versus Recurrence Interval Relationships for Gaged Sites in Oregon

| Station Number[1] | Station Name | Latitude In Decimal Degrees | Longitude In Decimal Degrees | Years of Record | Drainage Area In Square Miles | Mean Basin Elevation In Feet | Mean Annual Precipitation In Inches |
|---|---|---|---|---|---|---|---|
| 10366000 | Twentymile Creek near Adel, OR | 42.072 | 119.962 | 56 | 194.00 | 5,800 | 15.0 |
| 10370000 | Camas Creek near Lakeview, OR | 42.216 | 120.101 | 25 | 63.00 | 6,210 | 20.0 |
| 10371000 | Drake Creek near Adel, OR | 42.200 | 120.011 | 26 | 67.00 | 5,880 | 15.0 |
| 10371500 | Deep Creek above Adel, OR | 42.189 | 120.001 | 57 | 249.00 | 6,110 | 17.0 |
| 10378500 | Honey Creek near Plush, OR | 42.425 | 119.922 | 65 | 170.00 | 5,910 | 20.0 |
| 10384000 | Chewaucan River near Paisley, OR | 42.685 | 120.569 | 72 | 275.00 | 6,050 | 18.0 |
| 10390400 | Bridge Creek near Thompson Reservoir, OR | 43.025 | 121.200 | 16 | 10.60 | 6,170 | 25.0 |
| 10392300 | Silvies River near Seneca, OR | 44.175 | 119.214 | 15 | 18.40 | 5,530 | 30.0 |
| 10392800 | Crowsfoot Creek near Burns, OR | 43.899 | 119.497 | 14 | 8.50 | 5,790 | 25.0 |
| 10393500 | Silvies River near Burns, OR | 43.715 | 119.176 | 79 | 934.00 | 5,200 | 19.0 |
| 10393900 | Devine Canyon near Burns, OR | 43.772 | 119.004 | 17 | 4.96 | 5,410 | 15.0 |
| 10396000 | Donner und Blitzen River near Frenchglen, OR | 42.791 | 118.867 | 60 | 200.00 | 6,160 | 14.0 |
| 10397000 | Bridge Creek near Frenchglen, OR | 42.844 | 118.849 | 39 | 30.0 | 5,890 | 12.0 |
| 10403000 | Silver Creek near Riley, OR | 43.692 | 119.658 | 29 | 228.00 | 5,180 | 20.0 |
| 13178000 | Jordan Creek above LN Tree Creek near Jordan Valley, OR | 42.874 | 116.953 | 24 | 440.00 | 5,780 | 15.0 |
| 13182100 | Dago Gulch near Rockville, OR | 43.294 | 117.254 | 12 | 3.09 | 4,560 | 12.0 |
| 13182150 | Long Gulch near Rockville, OR | 43.321 | 117.195 | 10 | 1.38 | 5,030 | 12.0 |
| 13213900 | Malheur River Tributary near Drewsey, OR | 43.780 | 118.358 | 17 | 2.28 | 3,820 | 10.0 |
| 13214000 | Malheur River near Drewsey, OR | 43.785 | 118.331 | 61 | 910.00 | 4,900 | 16.3 |
| 13216500 | Near Fork Malheur River above Beulah Res. near Beulah, OR | 43.948 | 118.173 | 49 | 355.00 | 5,360 | 19.0 |
| 13227000 | Bully Creek near Vale, OR | 43.958 | 117.342 | 26 | 570.00 | 4,150 | 17.8 |
| 13228300 | Lytle Creek near Vale, OR | 43.957 | 117.226 | 13 | 6.46 | 2,700 | 10.0 |
| 13229400 | Lost Valley Creek Tributary near Ironside, OR | 44.314 | 117.903 | 12 | 1.86 | 4,050 | 10.0 |
| 14036800 | John Day River near Prairie City, OR | 44.319 | 118.557 | 14 | 17.40 | 6,320 | 28.0 |
| 14037500 | Strawberry Creek above Slide Creek near Prairie City, OR | 44.342 | 118.656 | 56 | 7.00 | 6,900 | 37.0 |
| 14038530 | John Day River near John Day, OR | 44.419 | 118.903 | 18 | 386.00 | 4,900 | 25.0 |
| 14038550 | East Fork Canyon Creek near Canyon City, OR | 44.246 | 118.911 | 15 | 24.80 | 5,780 | 25.0 |
| 14038600 | Vance Creek near Canyon City, OR | 44.289 | 118.978 | 14 | 6.54 | 5,060 | 20.0 |
| 14038750 | Beech Creek near Fox, OR | 44.568 | 119.108 | 12 | 1.94 | 5,190 | 20.0 |

Information in this table is from USGS Open-File Report 93-419.

Notes: [1]All stations in Northwest or High Elevation flood regions

## Table 1, Contd. Weighted Peak Flow Versus Recurrence Interval Relationships for Gaged Sites in Oregon

| Station Number[1] | Relation Characteristic[2] | | | | | Peak Discharge (cubic feet per second) For Indicated Recurrence Interval (years)[3] | | | | | | Maximum Peak Discharge of Record (cubic feet per second) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | L | H | D | O | U | 2 | 5 | 10 | 25 | 50 | 100 | |
| 10366000 | 1 | 0 | 1 | 0 | 0 | 1,450 | 2,960 | 4,310 | 6,460 | 8,450 | 10,800 | 3,670 |
| 10370000 | 0 | 1 | 0 | 0 | 0 | 464 | 796 | 1,070 | 1,510 | 1,910 | 2,370 | 3,190 |
| 10371000 | 0 | 0 | 0 | 0 | 0 | 474 | 1,220 | 1,980 | 3,340 | 4,720 | 6,490 | 6,210 |
| 10371500 | 0 | 0 | 0 | 0 | 0 | 1,270 | 2,530 | 3,590 | 5,220 | 6,640 | 8,250 | 9,420 |
| 10378500 | 0 | 0 | 1 | 0 | 0 | 466 | 1,190 | 1,920 | 3,180 | 4,400 | 5,880 | 11,000 |
| 10384000 | 1 | 1 | 0 | 0 | 0 | 952 | 1,570 | 2,100 | 2,880 | 3,570 | 4,360 | 6,490 |
| 10390400 | 0 | 0 | 0 | 0 | 0 | 66 | 111 | 150 | 208 | 257 | 312 | 218 |
| 10392300 | 1 | 0 | 0 | 0 | 0 | 74 | 113 | 148 | 200 | 241 | 283 | 152 |
| 10392800 | 0 | 0 | 1 | 0 | 0 | 50 | 76 | 97 | 127 | 150 | 174 | 88 |
| 10393500 | 1 | 0 | 0 | 0 | 0 | 1,350 | 2,240 | 2,970 | 4,020 | 4,900 | 5,840 | 4,960 |
| 10393900 | - | - | - | - | 1 | 41 | 71 | 98 | 132 | 157 | 184 | 28 |
| 10396000 | 0 | 0 | 1 | 0 | 0 | 1,370 | 2,140 | 2,630 | 3,230 | 3,660 | 4,100 | 4,270 |
| 10397000 | 1 | 0 | 0 | 0 | 0 | 109 | 178 | 231 | 302 | 358 | 414 | 301 |
| 10403000 | 1 | 0 | 1 | 0 | 0 | 612 | 1,030 | 1,340 | 1,770 | 2,110 | 2,460 | 1,810 |
| 13178000 | 0 | 1 | 0 | 0 | 0 | 1,900 | 2,930 | 3,640 | 4,650 | 5,480 | 6,350 | 7,530 |
| 13182100 | - | - | - | - | 1 | 29 | 51 | 74 | 100 | 121 | 143 | 46 |
| 13182150 | - | - | - | - | 1 | 16 | 28 | 40 | 53 | 63 | 73 | 18 |
| 13213900 | - | - | - | - | 1 | 24 | 41 | 63 | 86 | 105 | 126 | 100 |
| 13214000 | 1 | 0 | 0 | 0 | 0 | 2,070 | 3,870 | 5,360 | 7,590 | 9,510 | 11,600 | 12,000 |
| 13216500 | 1 | 0 | 0 | 0 | 0 | 938 | 1,490 | 1,930 | 2,550 | 3,070 | 3,620 | 3,970 |
| 13227000 | 0 | 0 | 0 | 0 | 0 | 848 | 2,320 | 3,830 | 6,290 | 8,560 | 11,200 | 8,980 |
| 13228300 | 0 | 0 | 0 | 0 | 0 | 97 | 178 | 247 | 343 | 429 | 527 | 497 |
| 13229400 | - | - | - | - | 1 | 20 | 35 | 53 | 73 | 88 | 104 | 41 |
| 14036800 | 0 | 0 | 1 | 0 | 0 | 74 | 122 | 160 | 213 | 254 | 294 | 155 |
| 14037500 | 1 | 1 | 0 | 0 | 0 | 90 | 133 | 164 | 211 | 251 | 295 | 354 |
| 14038530 | 1 | 0 | 0 | 0 | 0 | 1,690 | 2,700 | 3,470 | 4,590 | 5,570 | 6,640 | 5,830 |
| 14038550 | - | - | - | - | 1 | 129 | 228 | 310 | 419 | 506 | 594 | 285 |
| 14038600 | 0 | 0 | 0 | 0 | 0 | 20 | 35 | 51 | 75 | 94 | 114 | 39 |
| 14038750 | - | - | - | - | 1 | 21 | 36 | 50 | 67 | 80 | 93 | 28 |

Notes: [2]Relation characteristics:
L, low-discharge threshold used to compute station relation
H, high outlier detected in station record
D, relation fit through the plotted annual peak flows has the appearance of a dogleg or jump
O, station is regional outlier deleted from generalized least-squares regression analysis
U, relation was undefined
1, code applies
0, code does not apply
-, code is not applicable

[3]Peak discharges weighted using station and regional skews

## Table 1, Contd. Weighted Peak Flow Versus Recurrence Interval Relationships for Gaged Sites in Oregon

| Station Number[1] | Station Name | Latitude In Decimal Degrees | Longitude In Decimal Degrees | Years of Record | Drainage Area In Square Miles | Mean Basin Elevation In Feet | Mean Annual Precipitation In Inches |
|---|---|---|---|---|---|---|---|
| 14038900 | Fields Cr near Mount Vernon, OR | 44.393 | 119.307 | 13 | 17.50 | 5,310 | 22.0 |
| 14039200 | Venator Creek near Silvies, OR | 43.999 | 119.275 | 13 | 11.90 | 5,510 | 25.0 |
| 14040500 | John Day River at Picture Gorge near Dayville, OR | 44.521 | 119.625 | 59 | 1,680 | 4,580 | 22.0 |
| 14040700 | Whisky Creek near Mitchell, OR | 44.522 | 119.922 | 11 | 2.22 | 4,270 | 24.0 |
| 14040900 | Bruin Creek near Dale, OR | 44.897 | 118.793 | 12 | 4.63 | 5,220 | 25.0 |
| 14041500 | N. Fork John Day near Dale, OR | 44.999 | 118.940 | 29 | 5,200 | 5,450 | 27.0 |
| 14041900 | Line Cr near Lehman Springs, OR | 45.169 | 118.711 | 15 | 2.40 | 4,580 | 20.0 |
| 14042000 | Camas Cr near Lehman Springs, OR | 45.171 | 118.731 | 20 | 60.70 | 4,680 | 24.0 |
| 14042500 | Camas Creek near Ukiah, OR | 45.157 | 118.819 | 62 | 121.00 | 4,680 | 24.0 |
| 14043800 | Bridge Creek near Prairie City, OR | 44.542 | 118.540 | 15 | 6.93 | 5,350 | 30.0 |
| 14043850 | Cottonwood Cr near Galena, OR | 44.653 | 118.865 | 15 | 3.89 | 5,130 | 22.0 |
| 14043900 | Granite Creek near Dale, OR | 44.894 | 119.014 | 11 | 1.90 | 4,130 | 20.0 |
| 14044000 | MF John Day River at Ritter, OR | 44.889 | 119.140 | 56 | 515 | 4,800 | 23.0 |
| 14044100 | Paul Creek near Long Creek, OR | 44.724 | 119.132 | 11 | 3.5 | 4,490 | 18.0 |
| 14044500 | Fox Creek at Gorge near Fox, OR | 44.619 | 119.262 | 28 | 90.20 | 4,830 | 21.0 |
| 14046250 | Ives Canyon near Spray, OR | 44.860 | 119.714 | 12 | 2.73 | 3,460 | 18.0 |
| 14046300 | Big Service Creek near Service Creek, OR | 44.894 | 120.070 | 11 | 5.56 | 3,880 | 18.0 |
| 14046400 | Donnely Creek Tributary near Service Creek, OR | 44.772 | 120.003 | 18 | 1.85 | 2,880 | 16.0 |
| 14046900 | John Day Riv Trib near Clarno, OR | 44.906 | 120.568 | 21 | 1.36 | 3,730 | 15.0 |
| 14047350 | Rock Cr Trib near Hardman, OR | 45.078 | 119.569 | 14 | 6.25 | 4,100 | 20.0 |
| 14047450 | W Fk Dry Cr near Gooseberry, OR | 45.286 | 119.964 | 13 | 0.81 | 2,540 | 15.0 |
| 14077500 | N Fk Beaver Cr near Paulina, OR | 44.167 | 119.733 | 13 | 64.40 | 4,670 | 20.0 |
| 14077800 | Wolf Cr Tributary near Paulina, OR | 44.277 | 119.817 | 15 | 2.15 | 5,150 | 18.0 |
| 14078000 | Beaver Creek near Paulina, OR | 44.164 | 119.922 | 33 | 450 | 4,600 | 20.0 |
| 14078200 | Lizard Gulch Tributary near Hampton, OR | 43.589 | 119.983 | 16 | 19.60 | 5,000 | 15.0 |
| 14078400 | Lookout Creek near Post, OR | 44.311 | 120.240 | 14 | 7.53 | 5,670 | 25.0 |
| 14078500 | N Fork Crooked River above Deep Creek, OR | 44.333 | 120.083 | 11 | 159.00 | 5,130 | 21.0 |
| 14081800 | Ahalt Creek near Mitchell, OR | 44.433 | 120.351 | 22 | 2.28 | 5,130 | 25.0 |
| 14083000 | Ochoco Creek above Mill Creek near Prineville, OR | 44.308 | 120.644 | 13 | 200.00 | 4,654 | 21.0 |

Information in this table is from USGS Open-File Report 93-419.

Notes: [1]All stations in Northwest or High Elevation flood regions

## Table 1 Contd. Weighted Peak Flow Versus Recurrence Interval Relationships for Gaged Sites in Oregon

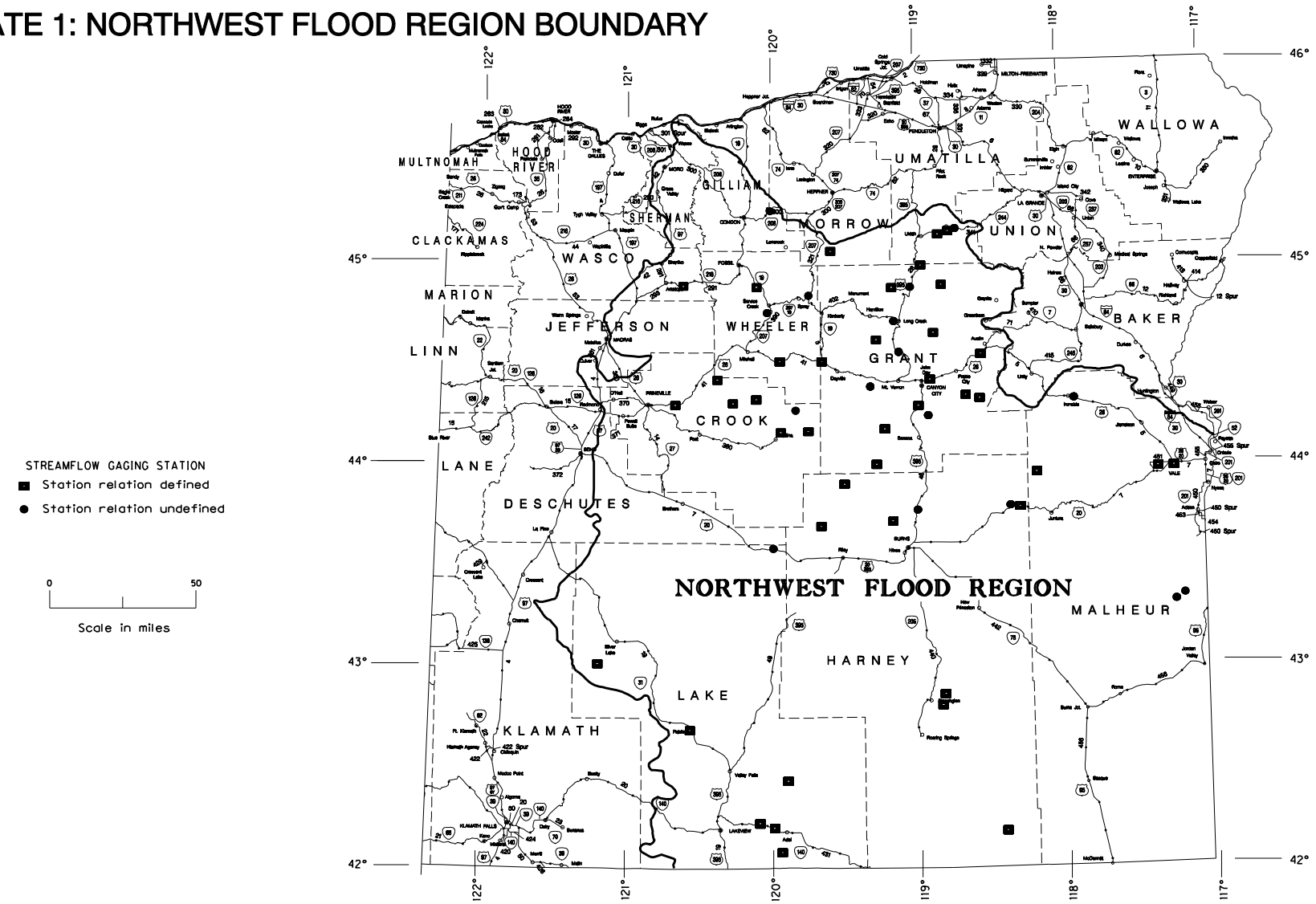
| Station Number[1] | Relation Characteristic[2] | | | | | Peak Discharge (cubic feet per second) For Indicated Recurrence Interval (years)[3] | | | | | | Maximum Peak Discharge of Record (cubic feet per second) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | L | H | D | O | U | 2 | 5 | 10 | 25 | 50 | 100 | |
| 14038900 | - | - | - | - | 1 | 101 | 177 | 248 | 336 | 407 | 480 | 240 |
| 14039200 | 1 | 1 | 0 | 0 | 0 | 57 | 80 | 103 | 138 | 165 | 194 | 108 |
| 14040500 | 0 | 0 | 1 | 0 | 0 | 2,820 | 4,590 | 5,950 | 7,850 | 9,410 | 11,000 | 8,170 |
| 14040700 | 0 | 0 | 1 | 0 | 0 | 33 | 70 | 103 | 150 | 192 | 240 | 143 |
| 14040900 | 1 | 0 | 0 | 0 | 0 | 34 | 47 | 60 | 79 | 94 | 109 | 57 |
| 14041500 | 0 | 0 | 0 | 0 | 0 | 3,020 | 4,360 | 5,170 | 6,190 | 6,980 | 7,770 | 8,170 |
| 14041900 | - | - | - | - | 1 | 24 | 42 | 62 | 83 | 100 | 118 | 90 |
| 14042000 | 0 | 0 | 0 | 0 | 0 | 614 | 947 | 1,200 | 1,540 | 1,840 | 2,170 | 1,880 |
| 14042500 | 0 | 0 | 0 | 0 | 0 | 1,050 | 1,580 | 1,970 | 2,490 | 2,920 | 3,370 | 3,840 |
| 14043800 | 0 | 0 | 0 | 0 | 0 | 40 | 62 | 82 | 109 | 131 | 155 | 98 |
| 14043850 | 0 | 0 | 0 | 0 | 0 | 47 | 67 | 82 | 103 | 120 | 137 | 98 |
| 14043900 | - | - | - | - | 1 | 21 | 36 | 54 | 73 | 88 | 105 | 66 |
| 14044000 | 0 | 0 | 0 | 0 | 0 | 1,640 | 2,420 | 2,970 | 3,700 | 4,280 | 4,870 | 4,730 |
| 14044100 | - | - | - | - | 1 | 32 | 55 | 81 | 110 | 134 | 158 | 56 |
| 14044500 | 1 | 0 | 0 | 0 | 0 | 407 | 785 | 1,120 | 1,630 | 2,090 | 2,620 | 1,860 |
| 14046250 | - | - | - | - | 1 | 27 | 46 | 75 | 103 | 127 | 153 | 86 |
| 14046300 | 0 | 0 | 0 | 1 | 0 | 7 | 17 | 33 | 58 | 77 | 97 | 11 |
| 14046400 | - | - | - | - | 1 | 20 | 35 | 60 | 84 | 104 | 127 | 42 |
| 14046900 | 0 | 0 | 0 | 0 | 0 | 30 | 47 | 60 | 76 | 88 | 101 | 83 |
| 14047350 | 1 | 0 | 1 | 0 | 0 | 56 | 88 | 114 | 148 | 175 | 203 | 117 |
| 14047450 | - | - | - | - | 1 | 11 | 19 | 34 | 48 | 60 | 73 | 134 |
| 14077500 | 0 | 0 | 1 | 0 | 0 | 587 | 741 | 844 | 989 | 1,120 | 1,240 | 955 |
| 14077800 | - | - | - | - | 1 | 23 | 39 | 54 | 73 | 87 | 101 | 300 |
| 14078000 | 0 | 1 | 1 | 0 | 0 | 1,360 | 2,700 | 3,950 | 5,980 | 7,910 | 10,200 | 12,800 |
| 14078200 | - | - | - | - | 1 | 109 | 193 | 275 | 375 | 457 | 542 | 177 |
| 14078400 | 1 | 0 | 1 | 0 | 0 | 49 | 74 | 94 | 121 | 142 | 164 | 85 |
| 14078500 | 0 | 0 | 1 | 0 | 0 | 1,320 | 1,650 | 1,870 | 2,180 | 2,460 | 2,770 | 2,500 |
| 14081800 | 1 | 1 | 0 | 0 | 0 | 38 | 56 | 70 | 89 | 105 | 122 | 122 |
| 14083000 | 1 | 0 | 0 | 0 | 0 | 371 | 600 | 834 | 1,190 | 1,490 | 1,790 | 821 |

Notes: [2]Relation characteristics:

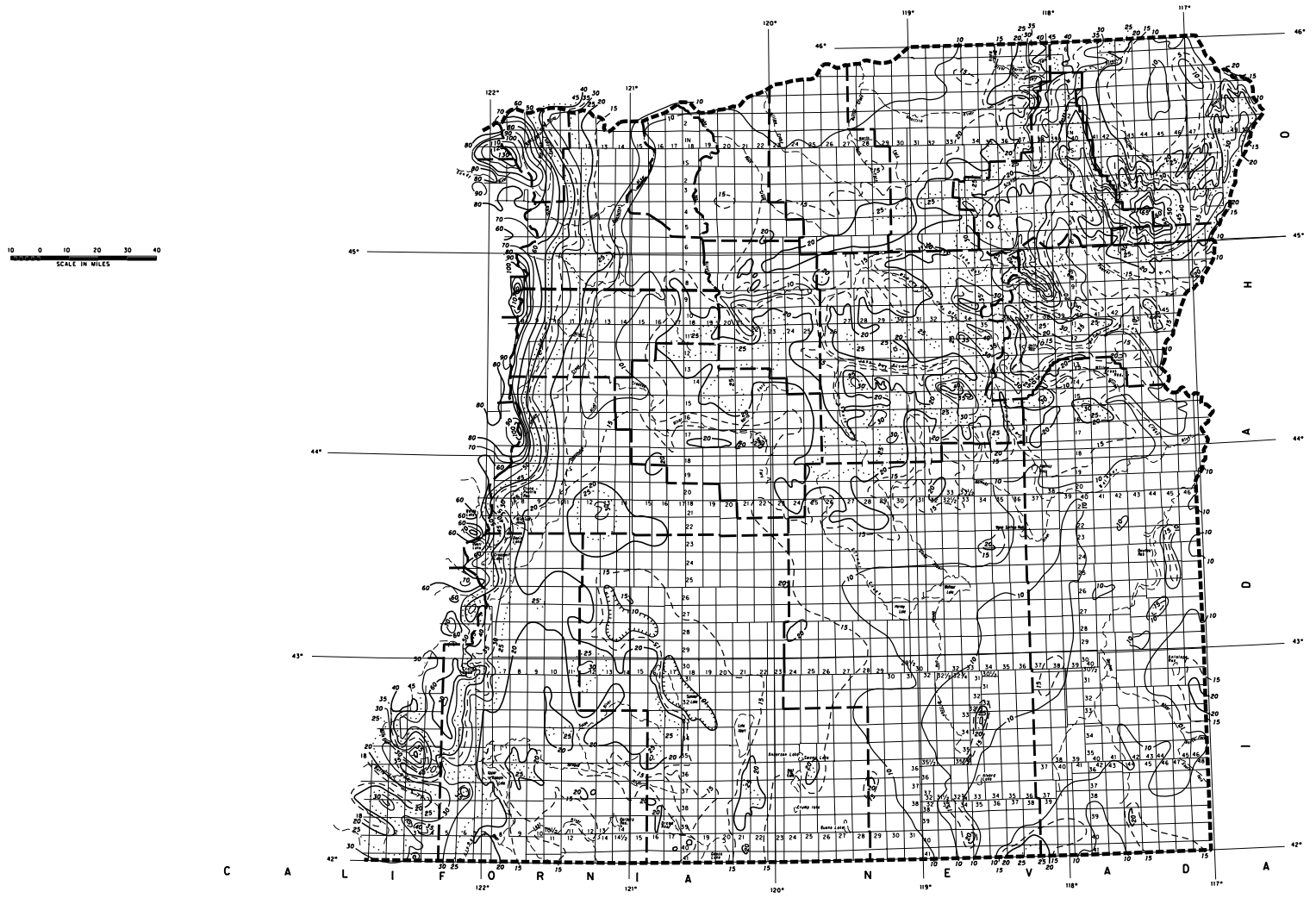
L, low-discharge threshold used to compute station relation
H, high outlier detected in station record
D, relation fit through the plotted annual peak flows has the appearance of a dogleg or jump
O, station is regional outlier deleted from generalized least-squares regression analysis
U, relation was undefined
1, code applies
0, code does not apply
-, code is not applicable

[3]Peak discharges weighted using station and regional skews

# PLATE 1: NORTHWEST FLOOD REGION BOUNDARY



# PLATE 2: MEAN ANNUAL PRECIPITATION IN EASTERN OREGON
## Isohyetal lines represent average annual rainfall in inches



# APPENDIX E – USGS NATIONWIDE URBAN REGRESSION EQUATIONS

## 1.0 Introduction

These nationwide regression equations are described in detail in the 1983 USGS Water-Supply Paper 2207 "Flood Characteristics of Urban Watersheds in the United States" by V.B. Sauer et al. They can be used to estimate peak discharges from urban watersheds.

The results of these urban regression equations should not be directly compared to the results of corresponding rural regression equations to determine the increased runoff caused by urbanization. To determine effects on discharge due to urbanization, it is recommended that both the undeveloped and developed watersheds be analyzed with the urban regression equations and the results of these two analyses be compared. This method is shown in the example.

Two versions of the USGS urban regression equations are presented. One version uses three input parameters. The first parameter is the drainage area, the second parameter accounts for alterations to the natural drainage system through use of a basin development factor, and the third parameter accounts for regional variations in runoff by use of an equivalent rural peak runoff rate. These three parameter equations are suitable for most applications.

The second version of the urban regression equations uses seven input parameters. Three of the parameters are the same as those listed for the three-parameter equations. The other four parameters are the slope of the main drainage channel, the rainfall intensity, the basin storage, and the percent of the basin area that is impervious.

The seven-parameter equations are recommended for calculating flows from urban drainages with significant basin storage. Basin storage is defined as the drainage area covered by lakes, reservoirs, ponds, swamps, and wetlands.

## 2.0 Equations

The equations and definitions of the equation variables are presented in Table 1. The equations are expected to produce results within the standard errors of regression in Table 1 if the input variables are within the ranges in the following list. Use of input values outside of these ranges may produce excessive errors in the discharge predictions.

- Drainage Area (A) for the urban basin should be between 0.2 and 100 square miles.

- The Main Channel Slope (SL) should not be less than 3.0 feet per mile or greater than 70 feet per mile (seven parameter equations, only). The maximum value of slope for use in equations is 70 feet per mile, although numerous watersheds used in the study (Sauer et al) had SL values up to 500 feet per mile.

- The 2-year - 2-hour Rainfall Intensity (RI2) should not be less than 0.2 inches or greater than 2.8 inches (seven parameter equations, only).

- The Basin Storage (ST) should not be more than 11 percent of the drainage area (seven parameter equations, only).

- The Basin Development Factor (BDF) should not be less than 0 or more than 12.

- The Impervious Surface Area (IA) should not be less than 3 percent or greater than 50 percent of the drainage area (seven parameter equation, only).

- The Drainage Area (A) and all other variables used to determine flows from the equivalent rural drainage basin must be within the ranges of values applicable to those equations.

## 3.0 Procedure

The analysis procedure is as follows:

**Step 1 -** Verify the input variables are within the acceptable ranges for both the rural and urban regression equations.

**Step 2 -** Use the USGS regression equations from **Chapter 7** Appendices B, C, or D to estimate the peak discharge from the equivalent rural drainage basin ($RQ_T$).

**Step 3 -** Determine the input variables SL, RI2, ST, and IA using the guidelines in Table 1. This step is needed when using the seven-parameter equations, only.

**Step 4 -** Determine basin development factor (BDF), as follows:

**a.** The first step is to divide the basin into upper, middle, and lower thirds on a drainage map. Each third should contain about one-third of the contributing drainage area. If two or more branches of a stream are in the same third, the stream sections should have approximately the same length. The stream lengths in different thirds, however, can be different. This is shown in Figure 1. As shown in the figure, the stream lengths in the

lower thirds are different than lengths in the middle thirds, and stream lengths in the middle thirds are different than stream lengths in the upper thirds. The boundaries between the thirds can be drawn by eye. Precise definition of the divisions is not necessary because the divisions have little effect on the final value of the BDF.

**b.** The second step is to evaluate and assign a code to the aspects of each third of the drainage area. The aspects are often determined by aerial photographs and verified by field inspection. Coding guidelines are as follows:

- **Channel Improvements -** If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channels), then a code of One (1) is assigned. To be considered prevalent, at least 50 percent of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a code of Zero (0) is assigned.

- **Channel Linings -** If more than 50 percent of the length of the main drainage and principal tributaries have been lined with an impervious material, such as concrete, then a code of One (1) is assigned to this aspect. If less than 50 percent of these channels are lined, then a code of Zero (0) is assigned. The presence of channel linings is a good indication that channel improvements have been performed and signifies a more highly developed drainage system. As a result, if the channel lining aspect is One the channel improvement aspect is usually also One.

- **Storm Drains -** Storm drains are enclosed drainage structures (usually pipes) frequently used on the secondary tributaries which receive drainage directly from streets or parking lots. Many of these drains empty into open channels. In some basins, however, they empty into channels enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a sub-basin consist of storm drains, a code of One (1) is assigned to this aspect; if less than 50 percent, then a code of Zero (0). Note that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, the aspects of channel improvements and channel linings would also be assigned a code of One.

- **Curb and Gutter -** If more than 50 percent of a sub-basin is urbanized, and if more than 50 percent of the streets and highways in the sub-basin are constructed with curbs and gutters, then a code of One (1) should be assigned to this aspect. Otherwise, it would receive a code of Zero (0). Urbanized land is defined as the area covered by industrial, residential, or commercial development. Drainage from curb and gutter streets often empties into storm drains. As a result, if the curb and gutter aspect is One the storm drains aspect is frequently also One.

## Table 1: USGS Nationwide Regression Equations for Urban Conditions

### a) Three Parameter Equations

| Peak Discharge Equations | Standard Error of Regression in Percent |
|---|---|
| $UQ_2 = 13.2A^{0.21} (13 - BDF)^{-0.43} RQ_2^{0.73}$ | ± 43 |
| $UQ_5 = 10.6A^{0.17} (13 - BDF)^{-0.39} RQ_5^{0.78}$ | ± 40 |
| $UQ_{10} = 9.51A^{0.16} (13 - BDF)^{-0.36} RQ_{10}^{0.79}$ | ± 41 |
| $UQ_{25} = 8.68A^{0.15} (13 - BDF)^{-0.34} RQ_{25}^{0.80}$ | ± 43 |
| $UQ_{50} = 8.04A^{0.15} (13 - BDF)^{-0.32} RQ_{50}^{0.81}$ | ± 44 |
| $UQ_{100} = 7.70A^{0.15} (13 - BDF)^{-0.32} RQ_{100}^{0.82}$ | ± 46 |
| $UQ_{500} = 7.47A^{0.16} (13 - BDF)^{-0.30} RQ_{500}^{0.82}$ | ± 52 |

### b) Seven Parameter Equations

| Peak Discharge Equations | Standard Error of Regression in Percent |
|---|---|
| $UQ_2 = 2.35A^{0.41} SL^{0.17} (RI2 + 3)^{2.04} (ST + 8)^{-0.65} (13 - BDF)^{-0.32} IA^{0.15} RQ_2^{0.47}$ | ± 38 |
| $UQ_5 = 2.70A^{0.35} SL^{0.16} (RI2 + 3)^{1.86} (ST + 8)^{-0.59} (13 - BDF)^{-0.31} IA^{0.11} RQ_5^{0.54}$ | ± 37 |
| $UQ_{10} = 2.99A^{0.32} SL^{0.15} (RI2 + 3)^{1.75} (ST + 8)^{-0.57} (13 - BDF)^{-0.30} IA^{0.09} RQ_{10}^{0.58}$ | ± 38 |
| $UQ_{25} = 2.78A^{0.31} SL^{0.15} (RI2 + 3)^{1.76} (ST + 8)^{-0.55} (13 - BDF)^{-0.29} IA^{0.07} RQ_{25}^{0.60}$ | ± 40 |
| $UQ_{50} = 2.67A^{0.29} SL^{0.15} (RI2 + 3)^{1.74} (ST + 8)^{-0.53} (13 - BDF)^{-0.28} IA^{0.06} RQ_{50}^{0.62}$ | ± 42 |
| $UQ_{100} = 2.50A^{0.29} SL^{0.15} (RI2 + 3)^{1.76} (ST + 8)^{-0.52} (13 - BDF)^{-0.28} IA^{0.06} RQ_{100}^{0.63}$ | ± 44 |
| $UQ_{500} = 2.27A^{0.29} SL^{0.16} (RI2 + 3)^{1.86} (ST + 8)^{-0.54} (13 - BDF)^{-0.27} IA^{0.05} RQ_{500}^{0.63}$ | ± 49 |

## Table 1, Contd. USGS Nationwide Regression Equations for Urban Conditions

General form of equations:

**3-parameter**: $UQ_T = C\ A^{b1}\ (13 - BDF)^{b2}\ RQ_T^{b3}$

**7-parameter**: $UQ_T = C\ A^{b1}\ SL^{b2}\ (RI2 + 3)^{b3}\ (ST + 8)^{b4}\ (13 - BDF)^{b5}\ IA^{b6}\ RQ_T^{b7}$

Where:

**$UQ_T$** = peak discharge from urban drainage basin in cubic feet per second for recurrence interval T.

**C** = regression constant.

**A** = contributing drainage area in square miles.

**SL** = main channel slope in feet per mile, measured between points which are 10 percent and 85 percent of the main channel length upstream from the study site (for sites where SL is greater than 70 feet per mile, use SL = 70 feet per mile in the equations).

**ST** = basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps, and wetlands (channel storage of a temporary nature, resulting from detention ponds or ponding upstream from the roadway embankments, is not included in the computation of ST).

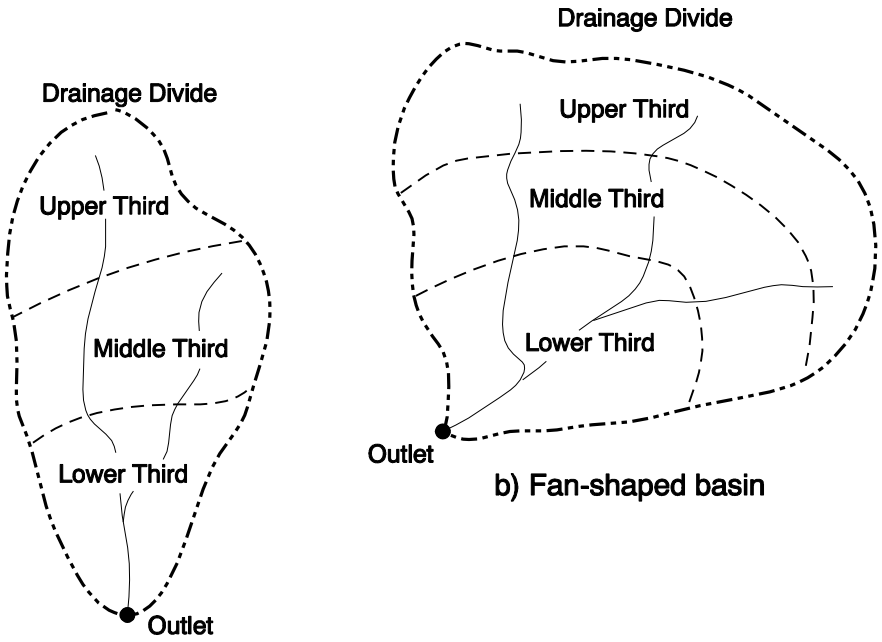
**RI2** = rainfall intensity in inches for the 2-year - 2-hour occurrence. Use the 2-year - 120-minute intensity based on the appropriate Intensity-Duration-Recurrence Interval charts in Chapter 7 Appendix A.

**BDF** = basin development factor, an index of the prevalence of the drainage aspects of (a) storm drains, (b) channel improvements, (c) impervious channel linings, (d) curb and gutter streets, and (e) percent of the subarea that is urbanized. A value of zero for BDF indicates the above drainage aspects are not prevalent, but does not necessarily mean the basin is nonurban. A value of 12 indicates full development of the drainage aspects throughout the basin.

**IA** = percentage of the drainage basin area occupied by impervious surfaces such as houses, buildings, streets, and parking lots.

**$RQ_T$** = peak discharge from equivalent rural drainage basin in cubic feet per second for recurrence interval T.

**b1, b2, b3, etc** = regression exponents.




Schematic of typical drainage basin shapes and subdivision into basin thirds. The basin thirds should have approximately equal areas. The lengths of the various branches of the stream channel within each third should be approximately equal. The total length of stream channel within a third does not have to be the same as the channel length within any other third.

**Figure 1 Basin Subdivision**

**c.** The third step is to add the aspects. The sum of the twelve codes is the BDF. The maximum value for a drainage basin is Twelve (12), and the minimum value is Zero (0). A Zero rating does not necessarily mean that the basin is unaffected by urbanization. A basin can be partially urbanized with some improvement of principal and secondary tributaries, and still have an assigned BDF of Zero. Likewise, a rating of Twelve does not necessarily mean the drainage basin is fully developed.

**Step 4 -** Enter the data calculated in the preceding steps into the regression equations listed in Table 1 and calculate the peak flows for the desired recurrence intervals.

## 4.0 Example Problem – USGS Urban Nationwide Regression Equations

A zoning change from mixed residential/commercial to industrial development is planned for a portion of the Rosalie Creek drainage basin in Helensburg. Pre-development and post-development peak discharges are needed. The city is in the Willamette Region of western Oregon.

**Step 1 -** The USGS regression equations for rural western Oregon were used to estimate the peak flows from an equivalent rural drainage basin. The Drainage Area (A) is 0.62 square miles based on measurement of a USGS quadrangle map. The 2-year 24-hour precipitation intensity (I) is 3.0 inches from Chapter 7 Appendix H.

These input variables are within the ranges of characteristics applicable for the Willamette Region equations listed in Chapter 7 Appendix B, and the drainage basin has little or no streamflow regulation or diversion. In addition, if urbanization is disregarded, the basin is typical of the drainage areas used to develop the rural regression equations. As a result, $RQ_T$ of the basin can be calculated by the rural regression equations, and the results are expected to be within the ranges of standard errors listed in the publications.

The area of the urban drainage basin is also within the recommended range of input values for the urban equations. Therefore, discharge estimates produced by the urban equations are expected to be within the ranges of standard errors listed in Table 1.

**Step 2 -** Peak flows for 2 through 100-year recurrence intervals were calculated using the Willamette Region rural equations. These discharges were plotted on logarithmic probability paper and the 500-year flow was determined by linear extrapolation. The peak flows are:

$RQ_2$ = 38 cubic feet per second (cfs)
$RQ_5$ = 56 cfs
$RQ_{10}$ = 70 cfs
$RQ_{25}$ = 90 cfs

$RQ_{50}$ = 105 cfs
$RQ_{100}$ = 122 cfs
$RQ_{500}$ = 165 cfs

**Step 3 -** The basin was subdivided into thirds on a map. The aspects were rated as follows:

- Lower third: existing and future land use is swamp with bordering parkland, and no existing or future drainage improvements are planned.

  Channel Improvement Aspect = 0
  Channel Lining Aspect = 0
  Storm Drain Aspect = 0
  Curb and Gutter Aspect = 0

- Middle third: existing use is light residential and commercial. Although over 50 percent of the area is urbanized, the drainage system is relatively undeveloped. There are curbs and gutters on many short side streets. Runoff from these gutters enters the ditches alongside the main roads. There are no storm drains. The future industrial area will have a fully developed drainage system including curbs, gutters, storm drains, channel improvements, and channel linings. Only the main stem of the creek will be left in a natural state.

  Channel Improvement Aspect = 0 (Existing) or 1 (Future)
  Channel Lining Aspect = 0 (Existing) or 1 (Future)
  Storm Drain Aspect = 0 (Existing) or 1 (Future)
  Curb and Gutter Aspect = 1 (Existing and Future)

- Upper third: existing and expected land use is residential. Most of the side streets have curbs and gutters that drain into roadside ditches alongside the main roads. There are no storm drains. The main stem of the creek and most principal tributaries are in a natural condition. No future improvements are planned.

  Channel Improvement Aspect = 0
  Channel Lining Aspect = 0
  Storm Drain Aspect = 0
  Curb and Gutter Aspect = 1

The basin development factor (BDF) is calculated by adding the aspects, as follows:

$BDF_{Existing} = 0 + 0 + 0 + 0 + 0 + 0 + 0 + 1 + 0 + 0 + 0 + 1 = 2$
$BDF_{Future} = 0 + 0 + 0 + 0 + 1 + 1 + 1 + 1 + 0 + 0 + 0 + 1 = 5$

**Step 4 -** The peak discharges were calculated for the existing basin as follows:

$UQ_2 = (13.2)\ (0.62^{0.21})\ (13 - 2)^{-0.43}\ (38^{0.73}) = 61$ cubic feet per second (cfs)
$UQ_5 = (10.6)\ (0.62^{0.17})\ (13 - 2)^{-0.39}\ (56^{0.78}) = 89$ cfs
$UQ_{10} = (9.51)\ (0.62^{0.16})\ (13 - 2)^{-0.36}\ (70^{0.79}) = 110$ cfs
$UQ_{25} = (8.68)\ (0.62^{0.15})\ (13 - 2)^{-0.34}\ (90^{0.80}) = 130$ cfs
$UQ_{50} = (8.04)\ (0.62^{0.15})\ (13 - 2)^{-0.32}\ (105^{0.81}) = 150$ cfs
$UQ_{100} = (7.70)\ (0.62^{0.15})\ (13 - 2)^{-0.32}\ (122^{0.82}) = 170$ cfs
$UQ_{500} = (7.47)\ (0.62^{0.16})\ (13 - 2)^{-0.30}\ (165^{0.82}) = 220$ cfs

The peak discharges for the developed basin were also calculated:

$UQ_2 = (13.2)\ (0.62^{0.21})\ (13 - 5)^{-0.43}\ (38^{0.73}) = 69$ cfs
$UQ_5 = (10.6)\ (0.62^{0.17})\ (13 - 5)^{-0.39}\ (56^{0.78}) = 100$ cfs
$UQ_{10} = (9.51)\ (0.62^{0.16})\ (13 - 5)^{-0.36}\ (70^{0.79}) = 120$ cfs
$UQ_{25} = (8.68)\ (0.62^{0.15})\ (13 - 5)^{-0.34}\ (90^{0.80}) = 150$ cfs
$UQ_{50} = (8.04)\ (0.62^{0.15})\ (13 - 5)^{-0.32}\ (105^{0.81}) = 170$ cfs
$UQ_{100} = (7.70)\ (0.62^{0.15})\ (13 - 5)^{-0.32}\ (122^{0.82}) = 190$ cfs
$UQ_{500} = (7.47)\ (0.62^{0.16})\ (13 - 5)^{-0.30}\ (165^{0.82}) = 240$ cfs

The peak flow versus recurrence interval curves before and after the anticipated development are shown in Figure 2. The changes in land use increase the peak discharges. This increase ranges from 8 cubic feet per second for the 2-year flood to 20 cubic feet per second for the 500-year flood.

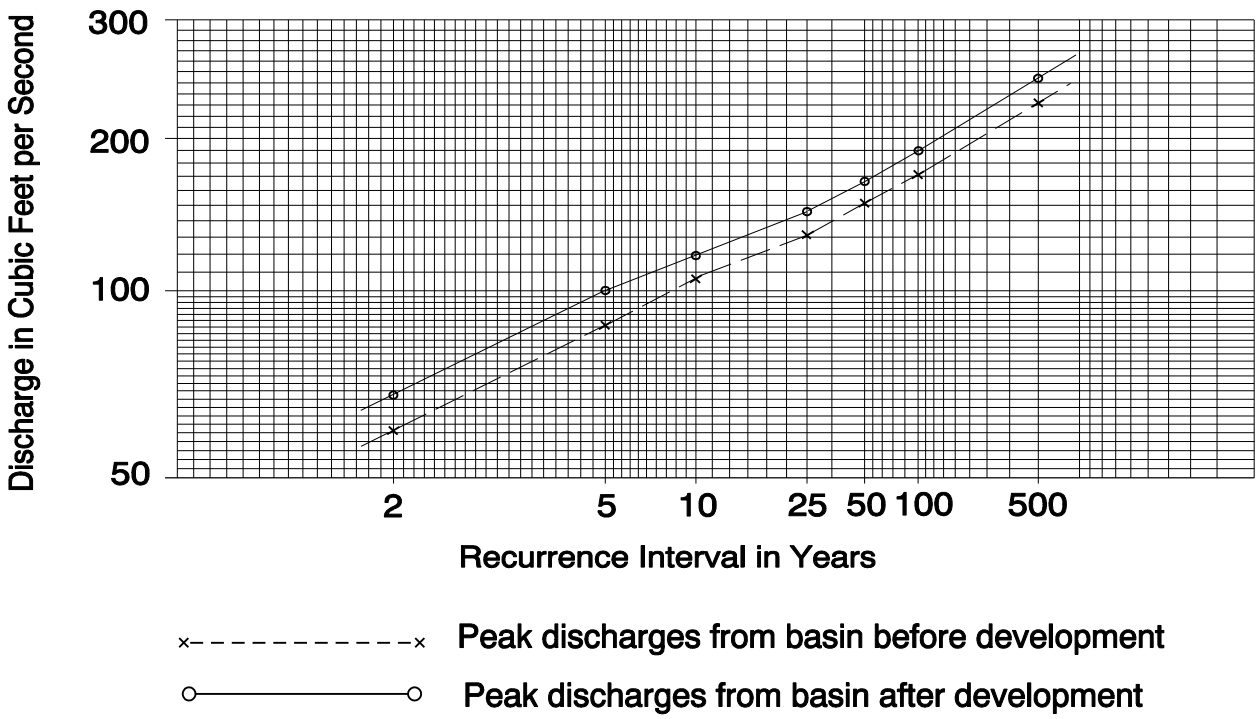



**Figure 2 Discharges from Rosalie Creek Urban Drainage Basin**

# APPENDIX F – RATIONAL METHOD

## 1.0 Introduction

One of the most commonly used procedures for calculating peak flows from small drainages less than 200 acres is the Rational Method. This method is most accurate for runoff estimates from small drainages with large amounts of impervious area. Examples are housing developments, industrial areas, parking lots, etc. The Rational Method is included in the HYDROLOGIC module of the FHWA's Watershed Modeling System computer software package and a Rational Method calculator is available within the FHWA Hydraulics Toolbox program. Input data such as runoff coefficients, rainfall information, etc., should be based on the procedures and information in the ODOT Hydraulics Manual when using this program.

## 2.0 Rational Equation

The Rational Method (or Rational Formula) is:

$$Q = C_f C i A \qquad \text{(Equation 1)}$$

Where:

- $Q$ = Peak flow in cubic feet per second (cfs)
- $C_f$ = Runoff coefficient adjustment factor to account for reduction of infiltration and other losses during high intensity storms
- $C$ = Runoff coefficient to reflect the ratio of rainfall to surface runoff
- $i$ = Rainfall intensity in inches per hour (in/hr)
- $A$ = Drainage area in acres (ac)

Limitations and assumptions in the Rational Method are as follows:

- The drainage area should not be larger than 200 acres.
- The peak flow is assumed to occur when the entire watershed is contributing runoff.
- The rainfall intensity is assumed to be uniform over a time duration equal to or greater than the time of concentration, $T_c$.
- The peak flow recurrence interval is assumed to be equal to the rainfall intensity recurrence interval. In other words, the 10-year rainfall intensity is assumed to produce the 10-year flood.

Detailed descriptions of the Rational Method input variables follow:

- **Runoff Coefficient "C" -** This variable represents the ratio of runoff to rainfall. It is the most difficult input variable to estimate. It represents the interaction of many complex factors, including the storage of water in surface depressions, infiltration, antecedent moisture, ground cover, ground slopes, and soil types. In reality, the coefficient may vary with respect to prior wetting and seasonal conditions. The use of average values has been adopted to simplify the determination of this coefficient. Table 1 lists runoff coefficients for various combinations of ground cover and slope. Where a drainage area is composed of subareas with different runoff coefficients, a composite coefficient for the total drainage area is computed by dividing the summation of the products of the subareas and their coefficients by the total area:

$$\text{Composite } C = \frac{\sum \left(C_{\text{Individual Areas}}\right)\left(A_{\text{Individual Areas}}\right)}{A_{\text{Total Area}}} \qquad \text{(Equation 2)}$$

  The impervious surface area is often a factor in stormwater storage and water quality treatment designs. Impervious surfaces have runoff coefficients greater than 0.80 based on Table 1. These are hard surfaces which either prevent or significantly retard the entry of water into the soil mantle. Common impervious surfaces include, but are not limited to, roof tops, walkways, patios, driveways, parking lots or other storage areas, concrete or asphalt paving, gravel roads, packed earthen materials, oiled, macadam, or other similar surfaces.

- **Runoff Coefficient Adjustment Factor "$C_f$" -** The coefficients in Table 1 are applicable for 10-years or less recurrence interval storms. Less frequent, higher intensity storms require adjusted runoff coefficients because infiltration and other losses have a proportionally smaller effect on runoff. Runoff coefficient adjustment factors ($C_f$) for storms of different recurrence intervals are listed in Table 2.

- **Rainfall Intensity "i" -** This variable indicates rainfall severity. Rainfall intensity is related to rainfall duration and design storm recurrence interval. Rainfall intensity at a duration equal to the time of concentration ($T_c$) is used to calculate the peak flow in the Rational Method. The rainfall intensity can be selected from the appropriate intensity-duration-recurrence interval (I-D-R) curve in Appendix A.

- **Area "A" -** The area is defined as the drainage surface area in acres, measured in a horizontal plane. The area is usually measured from plans or maps using a planimeter. The area includes all land enclosed by the surrounding drainage divides. In highway drainage design, this area will frequently include upland properties beyond the highway right-of-way.

**Table 1 Runoff Coefficients for the Rational Method**

| | FLAT | ROLLING | HILLY |
|---|---|---|---|
| Pavement & Roofs | **0.90** | **0.90** | **0.90** |
| Earth Shoulders | 0.50 | 0.50 | 0.50 |
| Drives & Walks | 0.75 | 0.80 | **0.85** |
| Gravel Pavement | **0.85** | **0.85** | **0.85** |
| City Business Areas | 0.80 | **0.85** | **0.85** |
| Apartment Dwelling Areas | 0.50 | 0.60 | 0.70 |
| Light Residential: 1 to 3 units/acre | 0.35 | 0.40 | 0.45 |
| Normal Residential: 3 to 6 units/acre | 0.50 | 0.55 | 0.60 |
| Dense Residential: 6 to 15 units/acre | 0.70 | 0.75 | 0.80 |
| Lawns | 0.17 | 0.22 | 0.35 |
| Grass Shoulders | 0.25 | 0.25 | 0.25 |
| Side Slopes, Earth | 0.60 | 0.60 | 0.60 |
| Side Slopes, Turf | 0.30 | 0.30 | 0.30 |
| Median Areas, Turf | 0.25 | 0.30 | 0.30 |
| Cultivated Land, Clay & Loam | 0.50 | 0.55 | 0.60 |
| Cultivated Land, Sand & Gravel | 0.25 | 0.30 | 0.35 |
| Industrial Areas, Light | 0.50 | 0.70 | 0.80 |
| Industrial Areas, Heavy | 0.60 | 0.80 | **0.90** |
| Parks & Cemeteries | 0.10 | 0.15 | 0.25 |
| Playgrounds | 0.20 | 0.25 | 0.30 |
| Woodland & Forests | 0.10 | 0.15 | 0.20 |
| Meadows & Pasture Land | 0.25 | 0.30 | 0.35 |
| Unimproved Areas | 0.10 | 0.20 | 0.30 |

*Note:*

- ***Impervious surfaces in bold***
- *Rolling = ground slope between 2 percent to 10 percent*
- *Hilly = ground slope greater than 10 percent*

**Table 2 Runoff Coefficient Adjustment Factors**

| RECURRENCE INTERVAL | RUNOFF COEFFICIENT ADJUSTMENT FACTOR |
|---|---|
| 10 years or less | 1.0 |
| 25 years | 1.1 |
| 50 years | 1.2 |
| 100 years | 1.25 |

- **Time of Concentration "$T_c$" -** The time of concentration ($T_c$), is defined as the time it takes for runoff to travel from the hydraulically most distant point in the watershed to the point of reference downstream. Most drainage paths consist of overland flow segments as well as channel flow segments. The overland flow component can be further divided into a sheet flow segment and a shallow concentrated flow segment. Urban drainage basins often will have one or more pipe flow segments. The travel time is computed for each flow segment and the time of concentration is equal to the sum of the individual travel times, as follows:

$$T_c = T_{osf} + T_{scf} + T_{ocf} + T_{pf} \qquad \text{(Equation 3)}$$

Where:

$T_c$ = Time of concentration in minutes (min.)
$T_{osf}$ = Travel time for the overland sheet flow segment in minutes (min.)
$T_{scf}$ = Travel time for the shallow concentrated flow segment in minutes (min.)
$T_{ocf}$ = Travel time for the open-channel flow segment(s) in minutes (min.)
$T_{pf}$ = Travel time for the pipe flow segment(s) in minutes (min.)

The drainage path used to determine the time of concentration need not include all of the listed segments. As an example, a roadway pavement bounded by curbs and drained by an inlet connected to a storm drain will have segments of overland sheet flow (pavement), open-channel flow (gutter), and pipe flow (storm drain). There is no shallow concentrated flow segment.

The travel times for the flow segments are determined as follows.

- **Overland Sheet Flow -** Overland sheet flow is shallow flow over a plane surface. It occurs in the furthest upstream segment of the drainage path, which is located immediately downstream from the drainage divide. The length of the overland sheet flow segment is the shorter of: the distance between the drainage divide and the upper end of a defined channel,

or a distance of 300 feet. The overland sheet flow velocity is usually slower than the velocities further downstream.

The kinematic wave equation can be used to estimate the time of concentration associated with overland sheet flow. The equation is shown below, and it is only applicable for travel distances equal to or less than 300 feet.

$$T_{osf} = \frac{0.93(L^{0.6}n^{0.6})}{(i^{0.4}S^{0.3})} \qquad \text{(Equation 4)}$$

Where:

$T_{osf}$ = Travel time for the overland sheet flow segment in minutes (min.)
L = Length of the overland sheet flow segment in feet (ft)
n = Manning's roughness coefficient (See Table 3)
i = Rainfall intensity in inches per hour (in/hr) See Appendix A.
S = The average slope of the overland area in feet per feet (ft/ft)

*Note: Calculating the time of concentration for overland sheet flow is an iterative or trial and error solution because both the flow time and the rainfall intensity are unknown. The procedure is illustrated in the Example.*

**Table 3 Manning's Roughness Coefficients for Overland Sheet Flow**

(Maximum Flow Depth = 1 inch)

| | |
|---|---|
| Pavement & Roofs | 0.014 |
| City Business Areas | 0.014 |
| Graveled Surfaces | 0.020 |
| Apartment Dwelling Areas | 0.050 |
| Industrial Areas | 0.050 |
| Urban Residential Areas (more than 6 units acre) | 0.080 |
| Meadows, Pastures & Range Land | 0.150 |
| Rural Residential Areas (more than 6 units acre) | 0.240 |
| Playgrounds, Light Turf | 0.240 |
| Parks & Cemeteries, Heavy Turf | 0.400 |
| Woodland & Forests | 0.400 |

- **Shallow Concentrated Flow -** Overland sheet flow often becomes either shallow concentrated flow or open-channel flow as it progresses down the drainage. It becomes

shallow concentrated flow if it enters a shallow or poorly defined channel such as a gully or rill. The average shallow concentrated flow velocity can be approximated using Figure 1, in which the velocity is a function of the watercourse slope and the surface type. The travel

$$T_{scf} = \frac{L}{60V} \qquad \text{(Equation 5)}$$

time for the shallow concentrated flow segment is calculated by the following formula:
Where:

$T_{scf}$ = Travel time for the shallow concentrated flow segment in minutes (min.)
L = Length of the shallow concentrated flow segment in feet (ft)
V = Average flow velocity in feet per second (ft/s), as shown in Figure 1.

*Note: This figure is from the 1972 "Soil Conservation Service Handbook".*

- **Open-Channel and Pipe Flow -** Overland sheet flow, or shallow concentrated flow, becomes open-channel flow when it enters a defined channel with known cross-sectional geometry, a channel visible on aerial photographs, or a channel indicated by a blue line on a USGS 7.5 - minute quadrangle map. Sheet or shallow concentrated flow becomes pipe flow when it enters a closed conduit such as a culvert or storm drain. The velocities in the segments of the drainage path with open-channel or pipe flow are determined by Manning's equation. Once the velocity is known, the travel time for the segment is calculated by dividing the segment length by the velocity. The procedure is similar to the method used with Equation 5 for shallow concentrated flow. Flows in open-channels, conduits and gutters are discussed in **Chapter 8**.

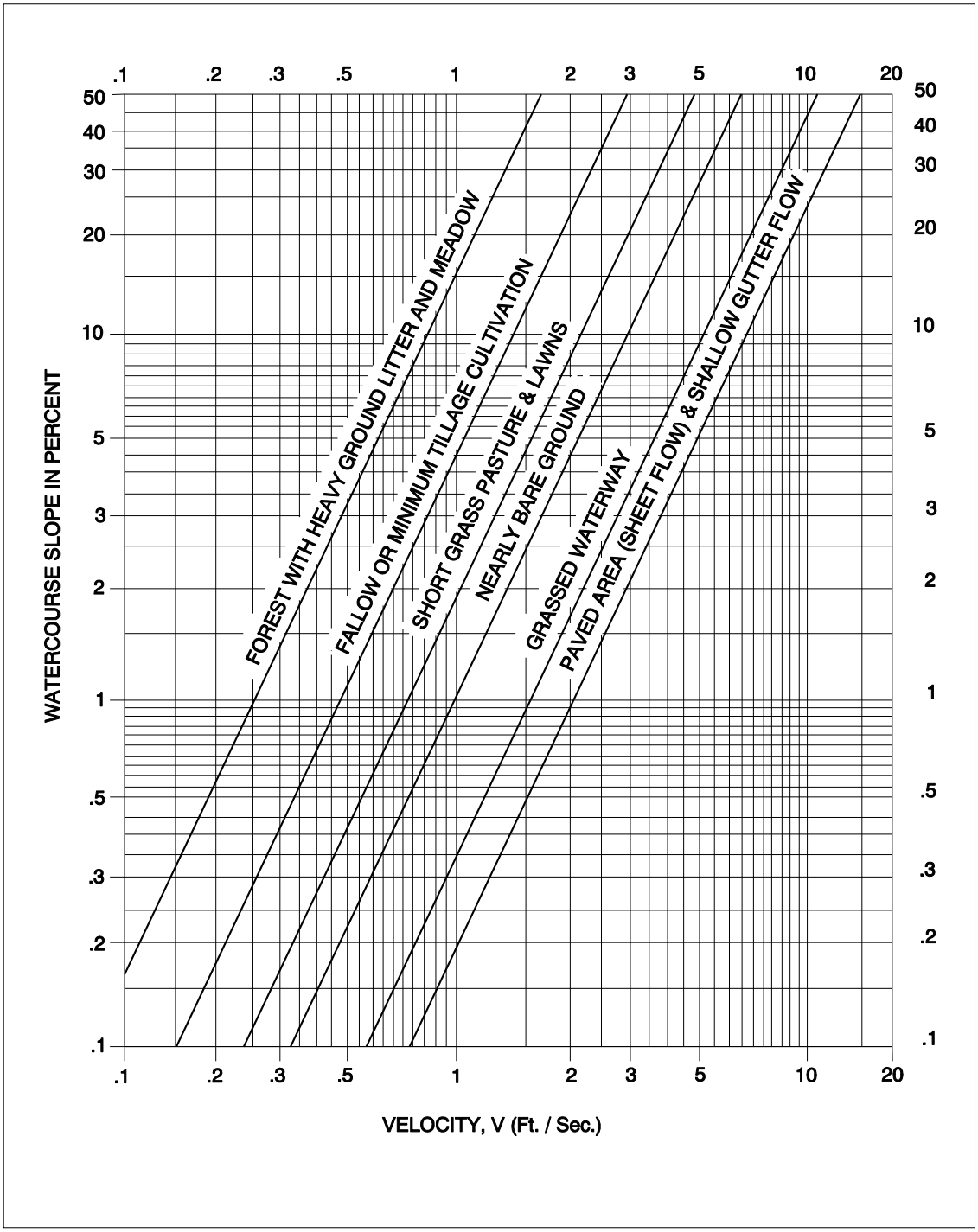



**Figure 1 Shallow Concentrated Flow Velocities**

## 3.0 Discharges at Junctions

The locations where tributary subbasin and basin drainage paths connect to each other are called flow junctions or junctions. An important part of a hydraulic analysis is to determine the peak discharges that will pass through these junctions. In many cases, it can be assumed that the peak discharge occurs when the entire upstream drainage area is contributing flow. This occurs when the storm duration is equal to the time of concentration of the tributary with the longest time of concentration, based on the assumptions used in the Rational Method.

The preceding assumption is not valid in some instances. Sometimes the peak discharge at a junction occurs when the storm duration corresponds to the time of concentration of the tributary with a shorter time of concentration. When this occurs, the entire drainage area of the tributary with the shorter time of concentration will be contributing flow, and only a portion of the drainage area with the longer time of concentration will be contributing runoff. The runoff from the more remote portions of the drainage basin with the longer time of concentration will not yet have arrived at the junction.

The designer should be aware that the peak flow at a junction can occur when the storm duration corresponds to the time of concentration of a tributary, and this possibility should be checked at critical junctions. **This situation is most likely at junctions where the flow from a large and impervious subbasin with a shorter time of concentration joins the discharge from an upstream subbasin that is mostly pervious with a longer time of concentration.** The following procedure can be used to determine discharges into junctions.

**Step 1 -** Calculate the times of concentration of the various tributaries upstream from the junction. For tributaries A, B ... n, these are $T_{cA}$, $T_{cB}$ ... $T_{cn}$. For the sake of organization, order the tributaries successively based on time of concentration. The tributary with the longest time of concentration is Tributary A, the tributary with the next longest time of concentration is Tributary B, etc.

**Step 2 -** Calculate the discharges from Tributaries A, B ….. n using the rainfall intensity corresponding to $T_{cA}$. This is $i_A$. The full drainage area of each tributary is contributing runoff. The formulae to calculate the discharges from each tributary, based on Equation 1, are:

$$Q_A = (C_f)\,(C_A)\,(i_A)\,(A_A)$$
$$Q_B = (C_f)\,(C_B)\,(i_A)\,(A_B)$$
$$Q_n = (C_f)\,(C_n)\,(i_A)\,(A_n)$$

The total flow into the junction is $Q_{Total}$ @ $T_{cA} = Q_A + Q_B + ... Q_n$

**Step 3 -** Recalculate the discharges from Tributaries A, B ... n using the rainfall intensity corresponding to $T_{cB}$. This is $i_B$. Only a portion of the drainage area of the tributary with the longer time of concentration will contribute runoff. In this case, it would be Tributary A. Use the ratio of the times of concentration to determine the portion that contributes runoff, as follows:

$$Q_A = (C_f)(C_A)(i_B)\left(\frac{T_{cB}}{T_{cA}}\right)(A_A)$$

The full drainage area of each tributary that has an equal or shorter time of concentration will contribute runoff. These are tributaries B ... n. The formulae to calculate the discharges are:

$$Q_B = (C_f)(C_B)(i_B)(A_B)$$
$$Q_n = (C_f)(C_n)(i_B)(A_n)$$

The total flow into the junction is $Q_{Total} @ T_{cB} = Q_A + Q_B + ... Q_n$

**Step 4 -** Recalculate the discharges from Tributaries A, B ... n using the rainfall intensity corresponding to $T_{cn}$. This is $i_n$. Only portions of the drainage areas of the tributaries with the longer times of concentration will contribute runoff. In this case, this would be Tributaries A and B. Use the ratios of the times of concentration to determine the portions that contribute runoff, as follows:

$$Q_A = (C_f)(C_A)(i_n)\left(\frac{T_{cn}}{T_{cA}}\right)(A_A)$$

$$Q_B = (C_f)(C_B)(i_n)\left(\frac{T_{cn}}{T_{cB}}\right)(A_B)$$

and

$$Q_n = (C_f)(C_n)(i_n)(A_n)$$

The total flow into the junction is $Q_{Total} @ T_{cn} = Q_A + Q_B + Q_n$

**Step 5 -** Compare the total discharges into the junction based on the different times of concentration. In this case this would be the results of Steps 2, 3, and 4. The highest total discharge governs, and it is to be used in the design or analysis.

## 4.0 Example Problem 1 - Rational Method

Several urban residential lots are the first drainage basin of a storm drain system. The 10-year time of concentration ($T_c$), and discharge (Q) are needed. The lots are small and it is assumed that runoff will be overland sheet flow. The project location data contains the following:

Basin area (A) = 1.24 acres
Length of overland flow (L) = 164 feet
Basin slope (S) = 0.02 feet/feet
Development density = 10 housing units per acre
Basin is in Imnaha

From Table 3: Manning's "n" for overland flow = 0.08 (value for an urban residential area)

From the I-D-R Curve Zone Map in Appendix A: Imnaha is in Zone 10

A time of concentration ($T_c$) is assumed, and the corresponding rainfall intensity (i) is determined from Zone 10 I-D-R curve for a 10-year intensity. Values from this curve are used in the kinematic wave equation (Equation 4) to calculate the $T_c$, and the calculated $T_c$ is compared to the assumed $T_c$. If the calculated and assumed $T_c$ values are the same, then the solution for $T_c$ has been found. If the calculated and assumed $T_c$ values are different, a new $T_c$ and corresponding i are used in Equation 4. This procedure is repeated until the calculated $T_c$ is in agreement with the assumed $T_c$, as follows:

**Iteration 1:** Assuming $T_c$ = 5 minutes, i = 2.2 inches per hour. Calculated $T_c$ = 10 minutes.

$$T_c = 0.93\frac{\left(164^{0.6}\right)\left(0.08^{0.6}\right)}{\left(2.2^{0.4}\right)\left(0.02^{0.3}\right)} = 10 \text{ minutes}$$

Assumed and calculated $T_c$ are not the same. Perform a second iteration.

**Iteration 2:** Assuming $T_c$ = 12 minutes, i = 1.6 inches per hour. Calculated $T_c$ = 12 minutes.

$$T_c = 0.93\frac{\left(164^{0.6}\right)\left(0.08^{0.6}\right)}{\left(1.6^{0.4}\right)\left(0.02^{0.3}\right)} = 12 \text{ minutes}$$

Since the assumed and calculated $T_c$ are in agreement, $T_c$ = 12 minutes and i = 1.6 inches per hour. Use of the I-D-R curve is shown in Figure 2.

From Table 2: $C_f$ = 1.0 for a 10-year recurrence interval.
From Table 1: C = 0.75 (rolling terrain, dense residential with 6 to 15 units per acre)

The discharge is calculated using the Rational Method (Equation 1) as follows:

$$Q_{10} = (1.0)\ (0.75)\ (1.6)\ (1.24) = 1.5 \text{ cubic feet per second}$$

## 5.0 Example Problem 2 - Rational Method

Drainage from the basin shown in Figure 3 flows to a highway culvert crossing. Peak flow is needed for a 50-year design storm using the Rational Method, $Q = C_f\ C\ i\ A$. The project is in Bend. A runoff coefficient adjustment factor ($C_f$) is needed because peak flow is to be determined for a 50-year storm. From Table 2, the adjustment factor is: $C_f = 1.2$

A composite runoff coefficient (C) is needed because the drainage basin contains subareas with different C values. Subarea C values are from Table 1, and the composite C value is calculated as follows:

| Description | "C" Value | Area (acres) | Impervious Area $C_iA_i$ (acres) |
|---|---|---|---|
| Rolling Forest | 0.15 | 3.2 | 0.5 |
| Flat Light Residential | 0.35 | 3.0 | 1.1 |
| Flat Pasture | 0.25 | 4.7 | 1.2 |
| | Total | 10.9 | 2.8 |

The composite runoff coefficient is calculated using Equation 2 as follows:

$$C = \frac{(0.5 + 1.1 + 1.2)}{10.9} = \frac{2.8}{10.9} = 0.26$$

The area (A) is also calculated during this step: A = 10.9 acres

The rainfall intensity (i) is needed. The Bend area is in Zone 10 according to the I-D-R Curve Index in Appendix A. The 50-year curve will be used. The time of concentration ($T_c$) must first be estimated to obtain the rainfall intensity. For this example, the drainage path used to determine the time of concentration is composed of two segments. The first segment is 300 feet long and it is assumed to be overland sheet flow. The remaining 900-foot long segment is assumed to be shallow concentrated flow.

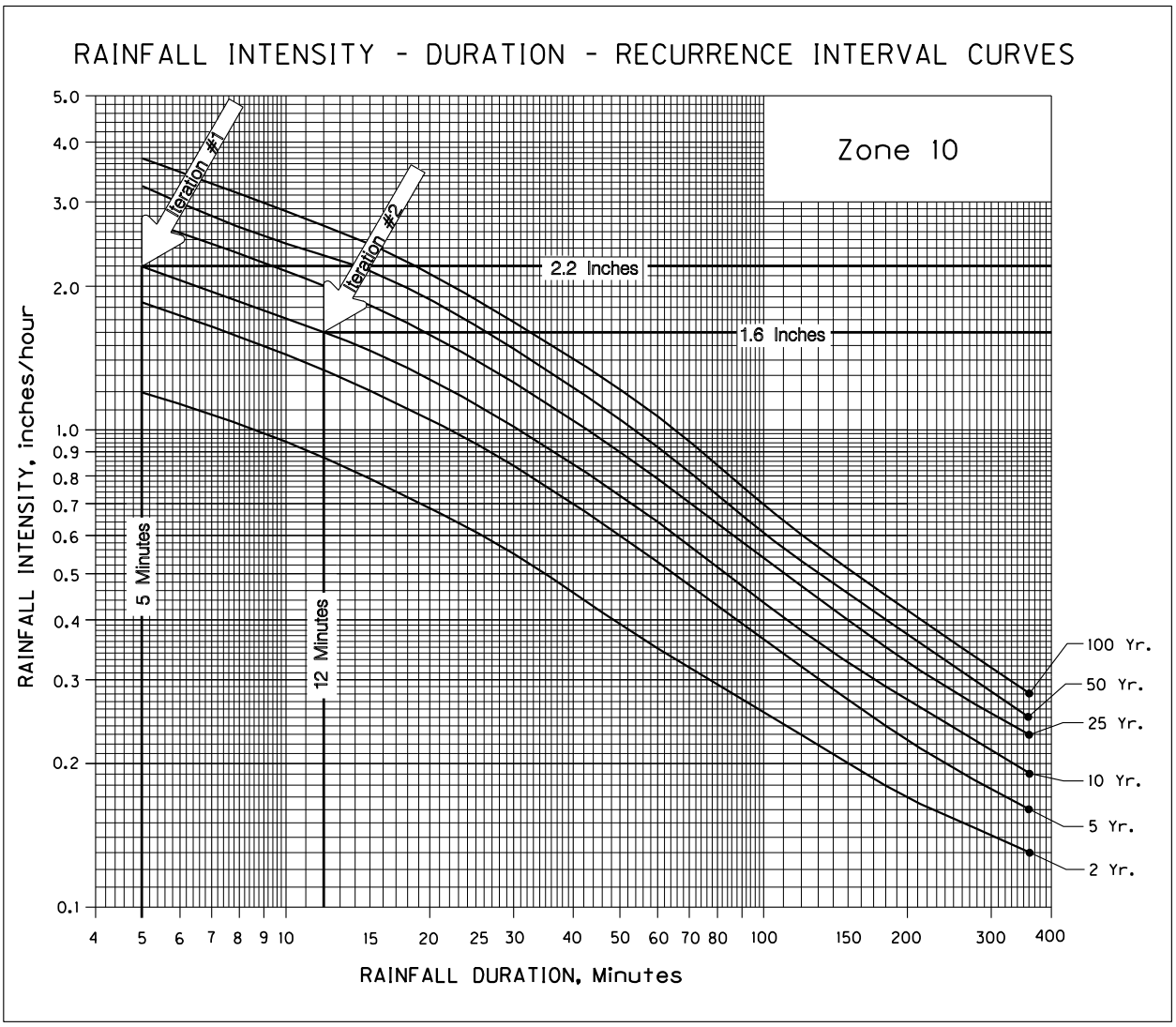



**Figure 2 Use of I-D-R Curve**

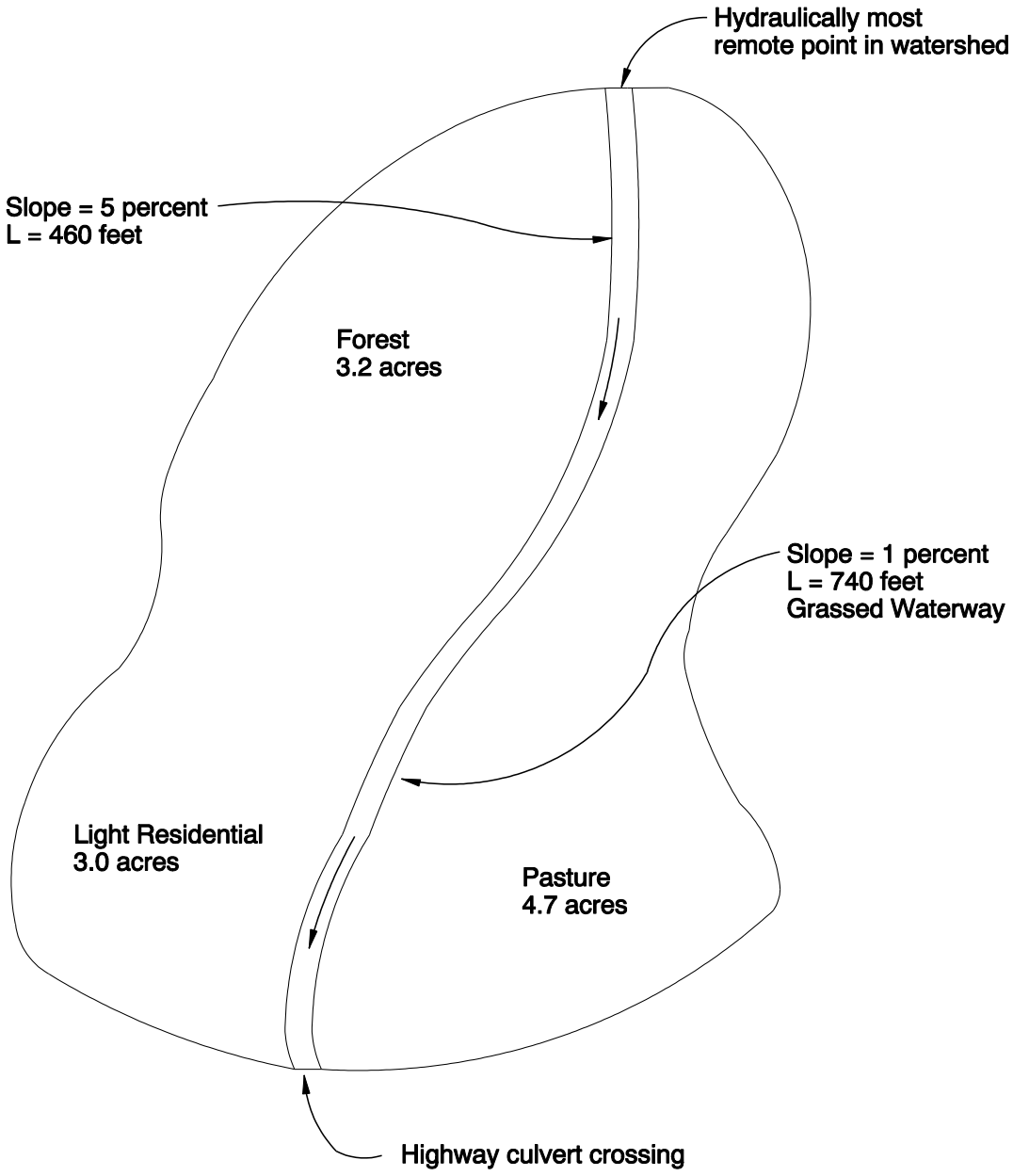



**Figure 3 Drainage Basin Near Bend, Oregon**

**Overland Sheet Flow Segment -** The travel time for the overland sheet flow segment is calculated as follows.

From Table 3: n = 0.40 (woodland and forest)

$$\text{From Equation 4: } T_{osf} = 0.93 \frac{\left(300^{0.6}\ 0.4^{0.6}\right)}{\left(i^{0.4}\ 0.05^{0.3}\right)} = \frac{40.4}{i^{0.4}}$$

From the 50-year Zone 10 I-D-R curve, and using a trial and error solution: $T_{osf}$ = 36 minutes

**Shallow Concentrated Flow Segment -** The travel time for the shallow concentrated flow segment is calculated as follows.

From the location data, 160 feet of the drainage path is over forested land with a 5 percent slope: L = 160 feet, and S = 5 percent

From Figure 1: V = 0.575 feet per second

From Equation 5, the travel time for shallow concentrated flow over forested land is:

$$T_{scf}\ (\text{forested land}) = \frac{160}{(60)\,(0.575)} = 5 \text{ minutes}$$

From the location data, the last 740 feet of the drainage path is a grassed waterway at a 1 percent slope: L = 740 feet, and S = 1 percent

From Figure 1: V = 1.5 feet per second

The travel time for shallow concentrated flow down the grassed waterway is:

$$T_{scf}\ (\text{grassed waterway}) = \frac{740}{(60)\,(1.5)} = 8 \text{ minutes}$$

From Equation 3, the time of concentration is: $T_c$ = 36 + 5 + 8 = 49 minutes

From the 50-year Zone 10 I-D-R curve using a rainfall duration that corresponds to the 49-minute time of concentration: i = 1.07 inches per hour

From Equation 1, the peak flow is calculated as follows:

$$Q_{50} = (1.2)\ (0.26)\ (1.07)\ (10.9) = 3.6 \text{ cubic feet per second}$$

# APPENDIX G - NRCS UNIT HYDROGRAPH METHOD

Two publications are included in this appendix, the "Oregon Engineering Handbook - Hydrology Guide" and Technical Release No. 55 "Urban Hydrology for Small Watersheds". Both of these publications are by the United States Department of Agriculture Natural Resources Conservation Service (NRCS), which was formerly the Soils Conservation Service (SCS).

The "Oregon Engineering Handbook" provides guidance for using the TR 55 method in Oregon and the "Urban Hydrology for Small Watersheds" publication provides detailed procedures for use of the method nationwide. Information such as the geographic boundaries for the rainfall distributions, the hydrologic soil groups, and the values of roughness coefficient "n" should be selected from the "Oregon Engineering Handbook."

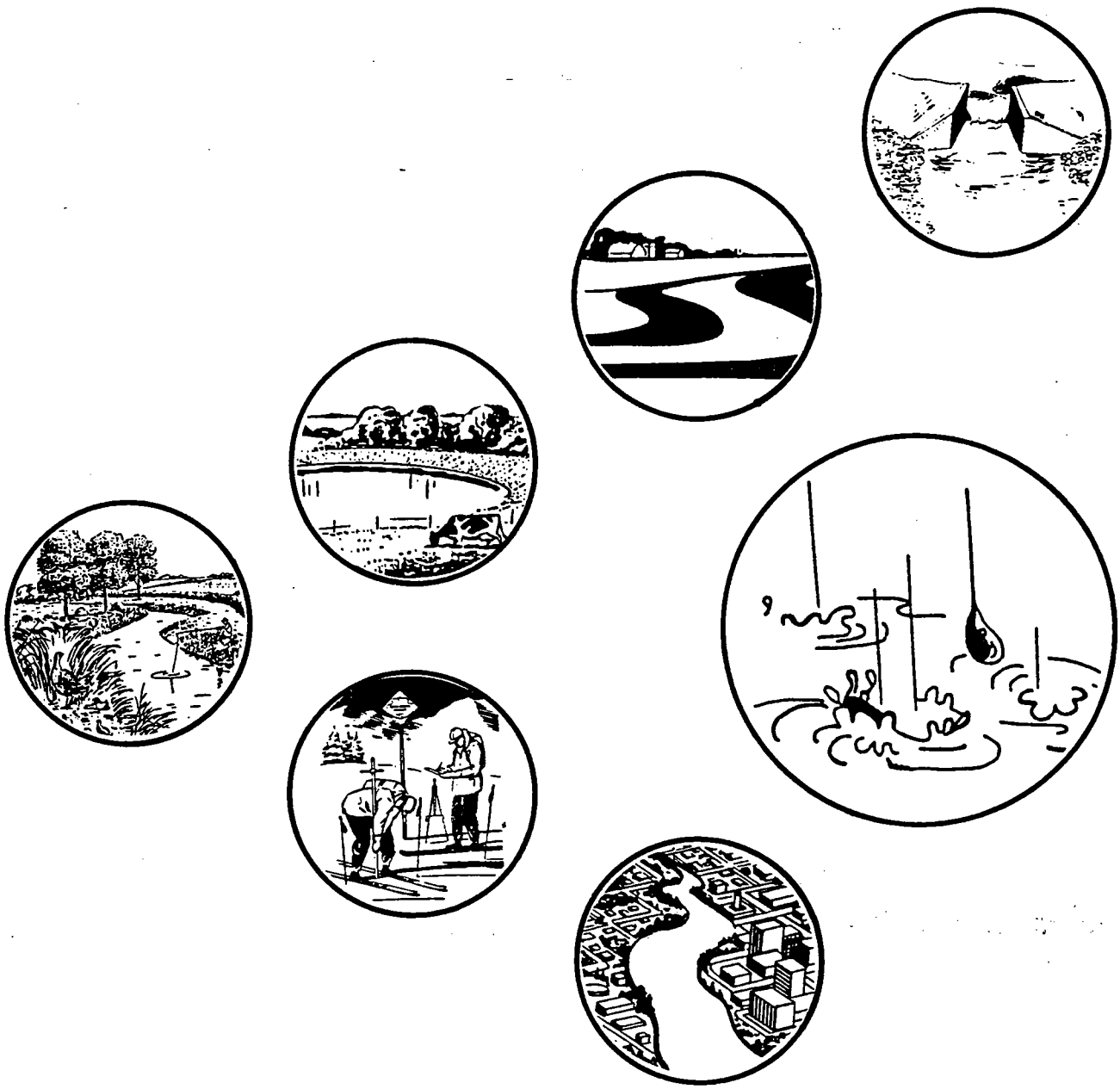
United States
Department of
Agriculture

**Soil
Conservation
Service**

Portland,
Oregon

# Oregon Engineering Handbook

## Hydrology Guide



# OREGON HYDROLOGY GUIDE

## FOREWORD

The Engineering Field Manual, Chapter 2, Estimating Runoff, presents a simplified method for determining storm runoff volume and peak rate of discharge. However, the method presented in this EFM chapter applies to watersheds with less than 2,000 acres. A simplified method for hydrology determination for watersheds over 2,000 acres did not exist until the June 1986 revision of Technical Release No. 55, Urban Hydrology for Small Watersheds, was issued. Although the title of TR-55 implies that it is primarily for urban hydrology, in actuality it applies to any watershed (urban and non-urban) within the limitations stated in the TR.

The purpose of the Oregon Hydrology Guide is to assist in applying the simplified hydrology procedures of TR-55 for developing peak flow and hydrograph estimates in Oregon. The guide accomplishes this by:

1. Providing instructions beyond those given in TR-55. This is done with detailed explanation, step-by-step procedures, and example solution. As mentioned in TR-55, the time of concentration (Tc) is a critical parameter. For this reason, the guide gives considerable emphasis to the determination of Tc. Chapters in the guide correspond to the chapters of TR-55.

2. Enclosing additional maps and tables specific to Oregon. Also included is information specific to Oregon needed to use the TR-55 computer programs.

The Oregon Hydrology Guide is to be used in conjunction with TR-55. No attempt was made to duplicate data already available in TR-55. For this reason, the guide should be filed with TR-55.

The procedure for estimating runoff, in Chapter 2 of the EFM, may continue to be used for watersheds less than 2,000 acres. Chapter 2 of the EFM, TR-55, TR-20, and streamgage analyses are accepted procedures for obtaining peak flows in Oregon.

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# CHAPTER 1

# INTRODUCTION

## Watershed and Sub-Area Drainage Areas

### Delineation

The drainage area of the watershed to be studied is outlined using a USGS topographic map, aerial photographs, county drainage or road maps, or other available map. A field observation of the terrain may be helpful in this delineation.

### Sub-Areas

One drainage outline is normally adequate for a watershed. Larger, or unique, watersheds may need to be divided into sub-areas. Watersheds which might need sub-division are: larger watersheds (over 2500 acres); watersheds with two or more major branches which are very different in watershed slope or shape; or watersheds which have a significant pond or swamp within the watershed.

The advantages of developing sub-areas are as follows: (1) results are more precise, (2) basic data is more easily developed, and (3) areas contributing most to the peak flow can be identified. Disadvantages of using sub-areas are as follows: (1) more data is required, (2) some required data may be difficult to obtain, (3) a hydrograph must be developed in addition to the peak flow, and (4) hydrographs may need to be routed to obtain a composite hydrograph.

### Measurement

The watershed or sub-area(s) is measured to determine its acreage. A planimeter, a dot grid, computation by average dimensions, or an electronic measuring device may be used to measure area.

The method used to measure the sub-area, and when to use sub-areas, is dependent on the precision necessary for the study. Generally, the more precise the data collection, the more accurate the answers. Structures in urban areas may require more intensive data collection procedures than those in rural areas.

# CHAPTER 2

# ESTIMATING RUNOFF

## Runoff Curve Number Procedure

The steps to determine the Runoff Curve Number (RCN) are (see Worksheet 2):

1. Identify the soil series found in the watershed area. Use published soil survey maps when available. If a soil map is not available, a soil scientist will need to determine the soil series.

2. Determine the Hydrologic Soil Group (HSG) from the list of soils in Appendix A of this guide, or from SCS Soils 5. Slope does not affect the HSG of a soil series. All soils of one HSG, regardless of slope, can be combined into one unit.

3. Determine the cover type, treatment, and hydrologic condition. Several combinations are listed on Table 2-2 of TR-55. Pick the combination which most closely fits the conditions in the watershed.

4. Read the applicable curve number for the cover description and HSG from Table 2-2 in TR-55.

5. If more than one soil or cover condition occurs in the watershed, determine the area in each combination of soil and cover. Area can be measured in either acres, square miles, or percent of the watershed.

6. Multiply the applicable curve number times the area.

7. Repeat Steps 1 through 6 for each soil-cover combination in the watershed.

8. Sum the areas and the products.

9. Compute the weighted curve number by the equation:

$$CN = \frac{\text{Total Product}}{\text{Total Area}}$$

10. Round off the curve number to the nearest whole number.

**Examples**

Example 2-1 - Runoff Curve Number

Develop RCN and Runoff for Watershed near Pendleton and 10-, 25-, 50-year frequency storms.

Sub-Area 1 Data:

| Soil | Cover, Treatment | Acres |
|---|---|---|
| Walla Walla, B slope- | Fallow Bare soil | 650 |
| Walla Walla, C slope- | Fallow Bare soil | 40 |
| Walla Walla, B slope- | Fallow w/crop residue, poor | 120 |
| Anderly, C slope- | Fallow Bare soil | 45 |
| Walla Walla, B slope- | Wheat straight row, poor | 650 |
| Walla Walla, C slope- | Wheat straight row, poor | 40 |
| Walla Walla, B slope- | Wheat contour w/residue, poor | 120 |
| Anderly, C slope- | Wheat straight row, poor | 45 |

Example 2-2 - Runoff Curve Number (RCN)

Develop RCN and Runoff for Sub-Area 2 of watershed near Pendleton for the 10-, 25-, and 50-year frequency storms.

Sub-Area 2 Data:

| Soil | Cover, Treatment | Acres |
|---|---|---|
| Walla Walla | Road, gravel | 7 |
| Anderly | Road, gravel | 1 |
| Walla Walla | Fallow, bare soil | 345 |
| Anderly | Fallow, bare soil | 30 |
| Walla Walla | Fallow, crop residence, good | 275 |
| Walla Walla | Wheat, straight row, poor | 344 |
| Anderly | Wheat, straight row, poor | 31 |
| Walla Walla | Wheat, contour & terraced, good | 275 |
| Walla Walla | Range, sagebrush, fair | 50 |
| Hermiston | Range, sagebrush, fair | 40 |

Project Example 2-1 By RSW Date 8-87

Location Uma. - Pendleton Checked IMU Date 8-87

Circle one: (Present) Developed

1. Runoff curve number (CN)

| Soil name and hydrologic group (HG APP-A) | Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio) (TR-55 Table 2-2) | CN 1/ Table 2-2 | CN 1/ Fig. 2-3 | CN 1/ Fig. 2-4 | Area ☒ acres ☐ $mi^2$ ☐ % | Product of CN x area |
|---|---|---|---|---|---|---|
| B-HSG Walla Walla | Fallow, bare soil | 86 | | | 690 | 59340 |
| B-HSG | Fallow, crop residue cover cover, poor condition | 85 | | | 120 | 10200 |
| B- | Wheat, straight row, poor condition | 76 | | | 690 | 52440 |
| B- | Wheat, contour, crop residue, poor condition | 73 | | | 120 | 8760 |
| C- Anderly | Fallow, bare soil | 91 | | | 45 | 4095 |
| C- | Wheat, straight row, poor condition | 84 | | | 45 | 3780 |
| | | | | | | |
| 1/ Use only one CN source per line. (TR-55) | | Totals = | | | 1710 | 138615 |

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{138{,}600}{1710} = 81.05$; Use CN = 81

2. Runoff

| | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|
| Frequency ..... yr | 10 | 25 | 50 |
| Rainfall, P (24-hour) ..... (HG APP-B) ..... in | 1.3 | 1.5 | 1.6 |
| Runoff, Q ..... in (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.) (TR-55) | .23 | .32 | .38 |

# Worksheet 2: Runoff curve number and runoff

Project Example 2-2 By RSW Date 8-87

Location Uma.-Pendleton Checked CUM Date 8-87

Circle one: (Present) Developed Sub-area 2

1. Runoff curve number (CN)

| Soil name and hydrologic group (HG APP-A) | Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio) (TR-55 Table 2-2) | CN 1/ Table 2-2 | CN 1/ Fig. 2-3 | CN 1/ Fig. 2-4 | Area ☒ acres ☐ mi² ☐ % | Product of CN x area |
|---|---|---|---|---|---|---|
| Walla Walla-B | Road - gravel | 85 | | | 7 | |
| | Fallow - bare soil | 86 | | | 345 | |
| Walla Walla-B | Fallow - crop residue, good | 83 | | | 275 | |
| Walla Walla-B | Wheat, straight row, poor | 76 | | | 344 | |
| Walla Walla-B | Wheat, contour & terrace, good | 70 | | | 275 | |
| Anderly-C | Fallow, bare soil | 91 | | | 30 | |
| | Wheat, str. row, good | 84 | | | 31 | |
| Anderly-C | Road - gravel | 89 | | | 1 | |
| Walla Walla-B Hermiston-B | Range, sagebrush, fair | 51 | | | 90 | |
| | | | | Totals = | 1398 | 108497 |

1/ Use only one CN source per line. (TR-55)

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{108500}{1398} = 77.61$; Use CN = 78

2. Runoff

| | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|
| Frequency ..... yr | 10 | 25 | 50 |
| Rainfall, P (24-hour) ..... (HG APP-B) ..... in | 1.3 | 1.5 | 1.6 |
| Runoff, Q ..... in (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.) (TR-55) | .16 | .24 | .29 |

# CHAPTER 3

# TIME OF CONCENTRATION AND TRAVEL TIME

## Intensity of Investigation Considerations

The intended use of the computed peak flows will determine the amount of effort that should be put into securing data for estimating time of concentration (Tc). A more detailed investigation is needed where it will be used for (a) detailed design, (b) complex/high hazard structures, (c) important planning decisions, or (d) economic justification. Less detail may be appropriate if it is to be used only for preliminary conclusions or where benefit-cost is not critical. A "minimum" detail would include measuring the travel distance from maps or aerial photographs and estimating average velocity from a general knowledge of the conditions in the area. The procedures in this chapter describe the steps needed to do a "high" detail investigation.

## Variability of Tc and Effect Upon the Hydrograph

The time of concentration influences the shape and peak of the runoff hydrograph. Changes in the Tc, due to changes in either the conditions of the water course or the land use, can cause changes in the hydrograph shape and its peak. The Tc will vary with the time of year and the conditions in the watershed. A channel with a lush growth will have a slower velocity than the same channel with dried grasses or as a bare channel. The flow across a frozen or a saturated field would be faster than under dry conditions. Variations in conditions can result in actual changes in Tc of 15% or more and a similar difference in peak flow.

Time of concentration is normally developed considering average flow conditions for the time of year when a problem is likely to occur. Where most of the damaging storms happen in the winter, such as in Western Oregon, the Tc should be developed for dormant conditions. If the damaging storms are resulting from thunderstorms, then summer conditions should be considered. Occasionally, it may be best to develop two "times of concentration" values, one for the dormant season and one for the growing season. Developed peak flows would be compared to determine the desired value to use. This would also require developing two runoff curve number values.

## Step Procedure

Step procedure for developing time of concentration is as follows. Worksheet 3 in TR-55 has been prepared to aid in developing the Time of Concentration.

1. Flow Path

   The first step in developing the Tc is to determine the flow path. This would normally be determined using a topographic map or an aerial photograph. Flow path is the path water would take as it flows from the headwaters of the watershed to the point of interest. For Tc,

path needed is the one that takes the longest travel time--usually the longest travel distance. Occasionally, it is not obvious whether a longer, moderately sloped path or a shorter, flat one would be the proper Tc path. When this is the case, both paths may need to be evaluated.

2. Reaches or Segments

Using the topographic map or aerial photograph, divide the flow path into reaches or segments, each with approximately a constant slope. Usually, reach breaks are also made where a channel starts and where intermittent flow becomes perennial. Reach breaks may be needed where there are major changes in channel roughness (n-value). A field examination will determine if changes in n-value are significant.

The number of reaches picked would also be related to the considerations for the detail discussed above. Three to five reaches normally would be adequate to define a small watershed. If more are needed, dividing the watershed into two areas should be considered.

Each segment is identified as a sheet flow, shallow concentrated flow, or open channel flow segment. The three groups are described in Chapter 3 of TR-55.

3. Segment Slope

When the reaches have been identified, the length (L) of the flow path in each reach is measured. Scaling distance off the topographic maps or photographs is usually adequate. The change in elevation ($\Delta h$) from one end of the reach to the other is also determined from maps or by field observation. Average slope of the reach is computed using the change in elevation divided by the length through the reach.

$$s = \frac{\Delta h}{L}$$

4. Sheet Flow Segment

Sheet flow segment is described on page 3-3 of TR-55. Develop Travel time (Tt) for this segment using equation 3-3. Data needed are: land surface description, flow length, land slope, and 2-year, 24-hour rainfall. Rainfall is available from a map in Appendix B. The other data is known from previous steps.

5. Shallow Concentrated Flow Segments

The segment(s) between sheet flow and a defined channel are computed using shallow concentrated flow as described on page 3-3. The curve for unpaved surface (figure 3-1) would normally be the condition encountered.

6. Open Channel Segments

The travel time (Tt) for the remaining reaches would be developed using channel hydraulics, normally Manning's formula. This is discussed on pages 3-3 and 3-4 of TR-55.

Channel hydraulics are determined as follows:

a. Check measurement of channel length (L) for the reach. Natural streams may have several curves and meanders, as well as overfalls and eddies. Tendencies are, therefore, to underestimate the length of channels and overestimate average slope through reaches.

b. Verify the vertical fall (h) through the reach. This can be taken from a topo map if one is available; otherwise, an approximate measure in the field is needed. Any overfalls should be subtracted from the total vertical drop since they would indicate a greater slope and, therefore, a greater velocity than actually occurs.

c. Recalculate the slope, as in step 3 above, if length or vertical fall have been revised.

$$s = \frac{\Delta h}{L}$$

d. Estimate the average n-value for the channel in the segment considering the vegetative conditions at the time a flood might occur. In the western portions of Oregon, most of the storms occur in November or December. Other locations may be affected by winter frozen soil conditions or by summer thunderstorms. The n-value to use may be picked from Table 3-2 following. The "Guide for Selecting Roughness coefficient 'n' Values for Channels," by Guy B. Faskin, SCS, and "Roughness Characteristics of Natural Channels," USGS WSP-1849, are additional good references. If the actual conditions are not similar to any on the table or more precision is desired, NEH-5 Hydraulics Supplement B has a procedure to develop a computed n-value.

e. Determine approximate trapezoidal channel dimensions for the reach. This is an average typical of the reach being computed. Measure or estimate the average bottom width (bw), depth (d), and top width (tw) of the channel for the reach being considered.

f. Calculate the flow area of the channel.

$$A = (d)\ \frac{(bw+tw)}{2}$$

g. Calculate the wetted perimeter (wp) and hydraulic radius (r).

$$wp = 2\left[\left(\frac{tw-bw}{2}\right)^2+d^2\right]^{\frac{1}{2}} + bw$$

$$r = \frac{A}{wp}$$

h. Determine the travel velocity (v) using Manning's formula.

$$v = \left(\frac{1.49}{n}\right)(r)^{0.67}\ (s)^{0.5} \qquad \text{[Eq 3-4]}$$

Table 3-2 Values of Roughness Coefficient "n"
Open Channels

| Type of Channel and Description | Minimum | Maximum |
|---|---|---|
| Lined Channels | | |
| Concrete | 0.011 | 0.020 |
| Riprap | 0.020 | 0.035 |
| Vegetative | 0.030 | 0.040 |
| Excavated | | |
| Earth, straight to winding | 0.020 | 0.040 |
| Earth bottom and rubble sides | 0.028 | 0.035 |
| Stony bottom and weeds on banks | 0.025 | 0.040 |
| Stony, smooth to jagged | 0.025 | 0.045 |
| Unmaintained | 0.050 | 0.140 |
| Natural Channels (minor streams, top width <100 ft.) | | |
| Clean, straight banks, few pools, weeds or stones | 0.025 | 0.040 |
| Winding, some pools, weeds and stones | 0.033 | 0.055 |
| Winding, stony sections, some weeds | 0.045 | 0.060 |
| Sluggish reaches, weedy, deep pools | 0.050 | 0.150 |
| Flood Plains | | |
| Pasture | 0.025 | 0.050 |
| Cultivated | 0.020 | 0.045 |
| Brush | | |
| Scattered brush, thick weed growth | 0.035 | 0.070 |
| Light brush and small trees | 0.035 | 0.080 |
| Medium to dense brush | 0.045 | 0.160 |
| Trees | | |
| Dense willows, full vegetation | 0.110 | 0.200 |
| Cleared land, tree stumps, sprouts | 0.030 | 0.080 |
| Heavy growth of timber, few down, little undergrowth | 0.080 | 0.120 |

Adapted from "Modern Sewer Design," 1980, American Iron and Steel Institute
Brater & King "Handbook of Hydraulics" 6th Edition, 1970
SCS NEH-5 Hydraulics

i. Determine travel time for segment in hours.

$$Tt = \frac{L}{(3600v)} \qquad \text{[Eq 3-1]}$$

7. Summation of Segments:

Determine time of concentration for entire sub-area by adding travel times of the segments including sheet flow and shallow concentrated flow and channel flow portions.

**Field Observations**

Much of the data needed to determine the time of concentration may require collection of information in the field. General observations are made in the field of conditions such as the soil-types, land cover and slopes, materials and vegetation in the channel, and the apparent degree of stability of the channel. Indications of debris flows as evidenced by deposition next to the channel and the size of deposited materials, may also be significant.

Other information obtained in the field includes: the shape and size of the channel (tw,bw,d), the average Manning's roughness coefficient (n-value), and a check of the channel slope using a hand level or a transit. These field data will be needed for all segments in each sub-area.

**Office Data Collection and Computations**

Many of the steps in determining Tc can be done in the office. Higher efficiency can be obtained by doing the map measurement and analysis in the office before going to the field. Computation and tabulation of travel times to develop time of concentration are office operations which normally will follow field data collection. Use worksheet 3 to tabulate and document time of concentration data. Tc sub-routine in the TR-55 computer program can aid in developing the data. A HP-41C program is also available which does the mathematics involved in worksheet 3. Cards and program description are available from the State Office.

## Examples

### Example 3-1 Time of Concentration

Compute Tc for sub-area 1 of watershed near Pendleton.

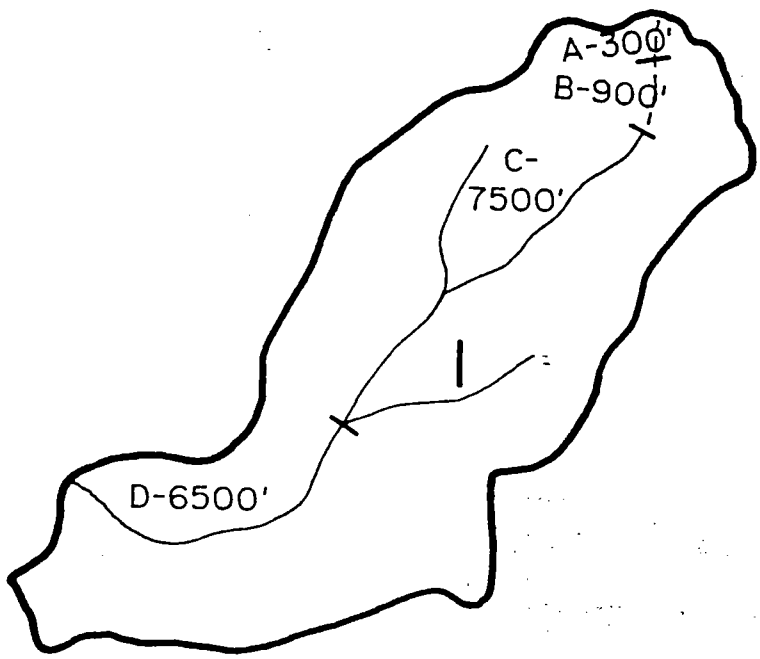
Sub-area 1 Data:

Segment A - Sheet flow on cultivated soil with residue cover >20%. Length is 300 feet with 10-foot difference in elevation.

Segment B - Shallow concentrated flow in cultivated field. Length is 900 feet, elevation change is 35 feet.

Segment C - Channel flow in vegetative draw (tall grasses) with average dimensions: d=1.0, bw=2.0, and tw=10.0.

Segment D - Channel flow in winding natural channel with some weeds and stones. Average channel dimensions are: d=4.0, bw=4.0, tw=10.0.



Project Example 3-1 By RSW Date 8-87

Location Uma. - Pendleton Checked IMU Date 8-87

Circle one: (Present) Developed Sub-area 1

Circle one: ($T_c$) $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | Segment ID | A | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) (TR-55) | | cult. >20% | | |
| 2. Manning's roughness coeff., n (table 3-1) | | 0.17 | | |
| 3. Flow length, L (total L ≤ 300 ft) | ft | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ | in | 1.0 | | |
| 5. Land slope, s 10/300 | ft/ft | .033 | | |
| 6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\,s^{0.4}}$ Compute $T_t$ | hr | 0.64 | + | = 0.64 |

| Shallow concentrated flow | Segment ID | B | | |
|---|---|---|---|---|
| 7. Surface description (paved or (unpaved)) | | unp. | | |
| 8. Flow length, L | ft | 900 | | |
| 9. Watercourse slope, s 35/900 | ft/ft | .039 | | |
| 10. Average velocity, V (figure 3-1) (TR-55) | ft/s | 3.18 | | |
| 11. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ | hr | 0.08 | + | = 0.08 |

| Channel flow | Segment ID | C | D | | |
|---|---|---|---|---|---|
| 12. Cross sectional flow area, a | $ft^2$ | 6.0 | 28.0 | | |
| 13. Wetted perimeter, $p_w$ | ft | 10.2 | 14.0 | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r | ft | .59 | 2.0 | | |
| 15. Channel slope, s | ft/ft | .010 | .006 | | |
| 16. Manning's roughness coeff., n (HG Table 3-2) | | .040 | .045 | | |
| 17. $V = \frac{1.49\,r^{2/3}\,s^{1/2}}{n}$ Compute V | ft/s | 2.62 | 4.07 | | |
| 18. Flow length, L | ft | 7500 | 6500 | | |
| 19. $T_t = \frac{L}{3600\,V}$ Compute $T_t$ | hr | 0.80 | + 0.44 | = 1.24 | |
| 20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) | hr | | | | 1.96 |

Example 3-2, Travel Time and Time of Concentration

Compute Tc and Tt for watershed near Pendleton, Sub-area 2

Sub-area 2 data:

Travel time -

Sub-area is downstream of sub-area 1 (see sketch). Main channel flows through the sub-area with average dimensions: d=4 feet, bw=5.0 feet, tw=12 feet. Channel is winding with stony sections and some weeds, generally in good condition. Flow distance is 9000 (7000 + 2000) feet with a 60-foot change in elevation.

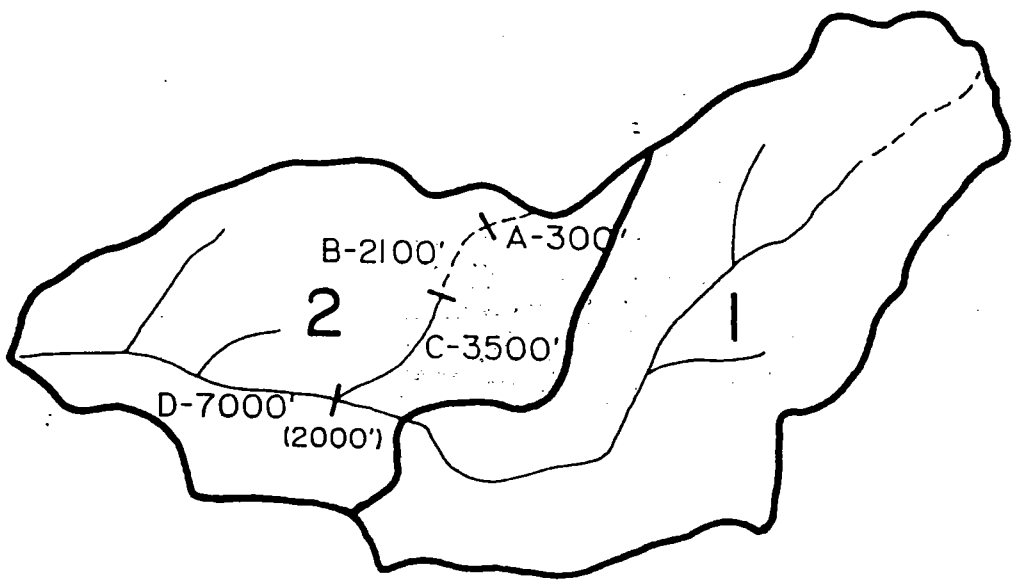
Time of Concentration -

Segment A - Flow over cultivated field with high residue. Length = 300 feet, Elevation Change = 44 feet.

Segment B - Shallow concentrated flow in crop field. Length = 2100 feet, Elevation Change = 44 feet.

Segment C - Channel flow through draw in scattered brush, heavy weeds. Average dimensions are: d=0.6 feet, bw=2.0 feet, tw=6.8 feet. Segment length=3500 feet and Elevation Change=70 feet.

Segment D - Flow in main channel described above with average dimensions: d=4 feet, bw=5.0 feet, tw=12 feet. Length=7000 feet.



# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project Example 3-2 By RSW Date 8-87

Location Uma.-Pendleton Checked IMU Date 8-87

Circle one: Present Developed Sub-area 2

Circle one: $T_c$ (T$_t$) through subarea For travel time

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| | | | | |
|---|---|---|---|---|
| **Sheet flow** (Applicable to $T_c$ only) | | Segment ID | | |
| 1. Surface description (table 3-1) ..... (TR-55) | | | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | | | |
| 3. Flow length, L (total L $\leq$ 300 ft) ..... | ft | | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | | | |
| 5. Land slope, s ..... | ft/ft | | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute $T_t$ ..... | hr | | + | = |
| **Shallow concentrated flow** | | Segment ID | -- | |
| 7. Surface description (paved or unpaved) ..... | | | | |
| 8. Flow length, L ..... | ft | | | |
| 9. Watercourse slope, s ..... | ft/ft | | | |
| 10. Average velocity, V (figure 3-1) ..... (TR-55) | ft/s | | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |
| **Channel flow** | | Segment ID | -- | |
| 12. Cross sectional flow area, a ..... | $ft^2$ | | 34.0 | |
| 13. Wetted perimeter, $p_w$ ..... | ft | | 15.6 | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ..... | ft | | 2.18 | |
| 15. Channel slope, s ..... | ft/ft | | .007 | |
| 16. Manning's roughness coeff., n (HG Table 3-2) ..... | | | .045 | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... | ft/s | | 4.66 | |
| 18. Flow length, L ..... | ft | | 9000 | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | 0.54 + -- | = .54 |
| 20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... | hr | | | .54 |

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project Example 3-2 By RSW Date 8-87

Location Uma.-Pendleton Checked ICU Date 8-87

Circle one: (Present) Developed — Sub-area 2

Circle one: ($T_c$) $T_t$ through subarea — for time of concentration

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| | | | | | |
|---|---|---|---|---|---|
| **Sheet flow** (Applicable to $T_c$ only) | | Segment ID | A | | |
| 1. Surface description (table 3-1) ..... (TR-55) | | | cult.-high res. | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | | 0.17 | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... | ft | | 300 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | | 1.0 | | |
| 5. Land slope, s ..... 6/300 | ft/ft | | .020 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute $T_t$ ..... | hr | | 0.78 | + | = .78 |
| **Shallow concentrated flow** | | Segment ID | B | | |
| 7. Surface description (paved or unpaved) ..... | | | unpave | | |
| 8. Flow length, L ..... | ft | | 2100 | | |
| 9. Watercourse slope, s ..... 44/2100 | ft/ft | | .021 | | |
| 10. Average velocity, V (figure 3-1) ..... (TR-55) | ft/s | | 2.3 | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | 0.25 | + | = .25 |
| **Channel flow** | | Segment ID | C | D | |
| 12. Cross sectional flow area, a ..... | $ft^2$ | | 2.64 | | see travel time comp's |
| 13. Wetted perimeter, $p_w$ ..... | ft | | 6.95 | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ..... | ft | | .38 | | |
| 15. Channel slope, s ..... 70/3500 | ft/ft | | .020 | | |
| 16. Manning's roughness coeff., n (HG Table 3-2) ..... | | | .045 | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... | ft/s | | 2.46 | 4.66 | |
| 18. Flow length, L ..... | ft | | 3500 | 7000 | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | 0.40 | + 0.42 | = .82 |
| 20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... | hr | | | | 1.85 |

# CHAPTER 4

# GRAPHICAL PEAK DISCHARGE METHOD

The Graphical Peak Discharge procedure is described in TR-55 Chapter 4. Follow the steps in worksheet 4 to obtain peak.

## Examples

### Example 4-1 Graphical Peak Discharge

Compute peak discharge for sub-area 1 of watershed near Pendleton for the 10-, 25-, and 50-year storms using Graphical procedure. Use data from Examples 2-1 and 3-1.

# Worksheet 4: Graphical Peak Discharge method

Project Example 4-1 By RSW Date 8-87

Location Uma.-Pendleton Checked ICU Date 8-87

Circle one: (Present) Developed Sub-area 1

1. Data:

Drainage area ..... $A_m$ = 2.67 $mi^2$ (acres/640)

Runoff curve number .... CN = 81 (From worksheet 2)

Time of concentration .. $T_c$ = 1.96 hr (From worksheet 3)

Rainfall distribution type = II (I, IA, II, III) (HG APP-B)

Pond and swamp areas spread throughout watershed ..... = --- percent of $A_m$ (____ acres or $mi^2$ covered)

| | | | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|---|---|
| 2. | Frequency ..... | yr | 10 | 25 | 50 |
| 3. | Rainfall, P (24-hour) ..... | in | 1.3 | 1.5 | 1.6 |
| 4. | Initial abstraction, $I_a$ ..... (Use CN with table 4-1.) (TR-55) | in | 0.469 | .469 | .469 |
| 5. | Compute $I_a/P$ ..... | | 0.36 | 0.31 | 0.29 |
| 6. | Unit peak discharge, $q_u$ ..... (Use $T_c$ and $I_a/P$ with exhibit 4-____) (TR-55) | csm/in | 167 | 184 | 190 |
| 7. | Runoff, Q ..... (From worksheet 2). | in | .23 | .32 | .38 |
| 8. | Pond and swamp adjustment factor, $F_p$ .... (Use percent pond and swamp area with table 4-2. Factor is 1.0 for zero percent pond and swamp area.) (TR-55) | | 1.0 | 1.0 | 1.0 |
| 9. | Peak discharge, $q_p$ ..... (Where $q_p = q_u A_m Q F_p$) | cfs | 103 | 157 | 193 |

# CHAPTER 5

## TABULAR HYDROGRAPH METHOD

The Tabular Peak Hydrograph procedure is described in Chapter 5 of TR-55. Follow the Steps on Worksheet 5a and 5b to develop peak hydrographs.

### Examples

Example 5-1 Tabular Peak Hydrograph

Compute the 25-year peak discharge at the downstream end of sub-area 2 of the watershed near Pendleton. Sub-area 2 data: DA = 1400 acres, RCN = 78. Use also data developed in Examples 2-1, 2-2, 3-1, and 3-2.

Solution:

Enter on worksheet 5a the drainage area (sq. mi.) time of concentration (Tc) and travel time (Tt). Times are obtained from worksheet 3. List the names of downstream subareas for each sub-area. For this example, the flow from sub-area 1 flows through sub-area 2; therefore, travel time summation to outlet is 0.54 hr. The rainfall, curve number, and runoff area read from worksheet 2. Multiply the area by the runoff to get AmQ. Initial abstraction is read directly from table 5-1 using the curve number, and Ia/P is calculated.

Next, determine for worksheet 5b the rounded values of Tc, Tt, and Ia/P to use in the hydrograph tables. In this example, a Tc of 2.0, an Ia/p of .30, and a Tt of .50 are the closest tabulated values to the computed figures. Identify the rainfall distribution for the area from the state map in appendix B. Pendleton is in a type II storm distribution area; therefore, exhibit 5-II, Tabular Hydrograph, will be applicable. Select the hydrograph times which need to be tabulated to cover the peak of each hydrograph. Using the proper combination of Tc, Ia/P, and Tt, identify the peak time for each sub-area. Sub-area 1 has a peak time of 14.0 hours, and sub-area 2 peaks at 13.6 hours. Hydrograph times of 13.0 through 14.6 were selected in this example to determine the shape of the hydrograph peak. For each time, read the unit discharge and multiply it by AmQ. For example, Sub-area 1, 13.0 hour, 37 times 0.83 equals 31 cfs; and for Sub-area 2, 13.6 hour, 185 times 0.50 equals 92 cfs.

Once all hydrographs are tabulated, sum the values for each time to get a composite hydrograph; i.e., at time 14.0, 142 plus 80 equals 222 cfs.

OR5-2

## Worksheet 5a: Basic watershed data

Project Example 5-1 Location Uma.-Pendleton By RSW Date 8-87

Circle one: (Present) Developed Sub-Areas 1&2 Frequency (yr) 25 Checked _____ Date _____

| Subarea name | Drainage area $A_m$ ($mi^2$) | Time of concentration $T_c$ (hr) | Travel time through subarea $T_t$ (hr) | Downstream subarea names | Travel time summation to outlet $\Sigma T_t$ (hr) | 24-hr Rainfall P (in) | Runoff curve number CN | Runoff Q (in) | $A_m Q$ ($mi^2$-in) | Initial abstraction $I_a$ (in) | $I_a/P$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2.67 | 1.96 | -- | 2 | 0.54 | 1.5 | 81 | .32 | 0.83 | .469 | .31 |
| 2 | 2.19 | 1.85 | 0.54 | -- | -- | 1.5 | 78 | .23 | 0.50 | .564 | .38 |
| | | | | | | | | | | | |
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From worksheet 3 | From worksheet 2 | From table 5-1 (TR-55)

(210-VI-ORHG, Sept. 1987)

## Worksheet 5b: Tabular hydrograph discharge summary

Project Example 5-1 Location Uma.-Pendleton By RSW Date 8-87

Circle one: (Present) Developed Sub-Areas 1&2 Frequency (yr) 25 Checked _____ Date _____

| Subarea name | Basic watershed data used 1/ | | | | Select and enter hydrograph times in hours from exhibit 5-II 2/ | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sub-area $T_c$ (hr) | $\Sigma T_t$ to outlet (hr) | $I_a/P$ | $A_m Q$ ($mi^2$-in) | | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | | | |
| | | | | | Discharges at selected hydrograph times 3/ -(cfs)- | | | | | | | | | | | |
| 1 | 2.0 | 0.50 | .30 | 0.83 | | 31 | 56 | 86 | 113 | 133 | 142 | 137 | 120 | | | |
| 2 | 2.0 | -- | .30 | 0.50 | | 58 | 74 | 84 | 92 | 85 | 80 | 66 | 55 | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
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| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| Composite hydrograph at outlet | | | | | | 89 | 130 | 170 | 205 | 218 | 222 | 203 | 175 | | | |

1/ Worksheet 5a. Rounded as needed for use with exhibit 5. (TR-55)
2/ Enter rainfall distribution type used. (HG APP-B)
3/ Hydrograph discharge for selected times is $A_m Q$ multiplied by tabular discharge from appropriate exhibit 5. (TR-55)

(210-VI-ORHG, Sept. 1987)

**Appendix A**

Hydrologic Soil Groups of the soil series used in Oregon. Use in place of Exhibit A-1 in TR-55.

## HYDROLOGIC GROUPS OF THE SOIL SERIES USED IN OREGON

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ABEGG | B | BERGSVIK | D | CARIS | C | CRUTCH | C | EDENBOWER | D |
| ABERT | B | BIEBER | D | CARLTON | C | CULBERTSON | B | EDSON | C |
| ABIQUA | B | BIGELOW | B | CARNEY | D | CULTUS | B | EIGHTLAR | D |
| ABIQUA, FLOODED | C | BILGER | D | CARRYBACK | C | CUMLEY | C | EILERTSEN | B |
| ACANOD | C | BINDLE | B | CASCADE | C | CUPOLA | B | ELLISFORDE | B |
| ACKER | B | BINS | B | CASSIDAY | C | CUPPER | B | ELLUM | C |
| ADKINS | B | BLACHLY | B | CATERL | B | CURANT | B | ELMORE | B |
| ADKINS, ALKALI | C | BLACKLOCK | D | CATHERINE | C | CURTIN | D | ELSIE | B |
| ADKINS, WET | C | BLALOCK | D | CATLOW | B | CURTIS CREEK | D | EMBAL | B |
| AGATE | D | BLAYDEN | D | CAZADERO | C | DABNEY | A | EMILY | B |
| AGENCY | C | BLUEJOINT | B | CENCOVE | B | DAMEWOOD | C | ENCINA | B |
| AHTANUM | D | BLY | B | CENTRAL POINT | B | DAMON | D | ENDERSBY | B |
| A'BEE | C | BOARDTREE | C | CHAMATE | B | DARBY | C | ENDICOTT | C |
| ALCOT | A | BOCKER | D | CHAMBERAM | B | DARKCANYON | C | ENKO | C |
| ALDING | D | BODELL | D | CHAPMAN | B | DAROW | C | ENOLA | B |
| ALGOMA | B/D | BOGUS | C | CHEHALEM | C | DAVEY | B | EPHRATA | B |
| ALICEL | B | BOHANNON | C | CHEHALIS | B | DAVEY, SANDY | A | ERA | B |
| ALOHA | C | BOILER | C | CHEHULPUM | D | SUBSTRATUM | | ESPIL | D |
| ALPHA | B | BONDRANCH | D | CHENOWETH | B | DAXTY | C | ESQUATZEL | B |
| ALS | A | BONNICK | A | CHERRYHILL | B | DAY | D | ETELKA | C |
| ALSEA | B | BOOMER | B | CHESNIMNUS | B | DAYTON | D | EVANS | B |
| ALSPAUGH | C | BOOTH | C | CHETCO | D | DAYVILLE | C | FALOMA | D |
| ALSTONY | B | BORAVALL | D | CHEVAL | C | DEATMAN | C | FANTZ | C |
| ALTHOUSE | B | BORGES | D | CHILCOTT | D | DEBENGER | C | FARMELL | B |
| ALVODEST | D | BORNSTEDT | C | CHILCOTT, BEDROCK | C | DECANTEL | D | FARMELL, CLAYEY | D |
| AMITY | D | BOROBEY | C | SUBSTRATUM | | DEE | C | SUBSTRATUM | |
| AMMON | B | BOSLAND | C | CHILOQUIN | D | DEFENBAUGH | B | FARVA | C |
| ANATONE | D | BOULDER LAKE | D | CHINCHALLO | D | DEGARMO | D | FELCHER | B |
| ANAWALT | D | BOULDROCK | B | CHISMORE | D | DEGNER | B | FELTHAM | B |
| ANDERLY | C | BOWLUS | B | CHITWOOD | D | DEHLINGER | B | FENDALL | C |
| ANDERS | C | BOYCE | D | CHOCK | D | DELENA | D | FERNHAVEN | B |
| ANGELPEAK | B | BOZE | B | CHOPTIE | D | DEMENT | B | FERNWOOD | B |
| ANUNDE | B | BRACE | C | CIRCLESAR | C | DEPOE | D | FERRELO | B |
| APPLEGATE | C | BRADER | D | CLACKAMAS | D | DEPPY | D | FERTALINE | D |
| APT | B | BRALLIER | D | CLAMP | D | DERALLO | B | FISHFIN | D |
| ARONAS | B | BRAND | D | CLATSOP | D | DERR | C | FIVEBIT | D |
| ARRITOLA | D | BRANNAN | B | CLAWSON | C | DERRINGER | C | FIVES | B |
| ARRON | D | BRAUN | C | CLIMAX | D | DESCHUTES | C | FLAGSTAFF | D |
| ASCAR | C | BRENNER | D | CLOQUATO | B | DESKAMP | C | FLANE | C |
| ASCHOFF | B | BRICKEL | C | COBURG | C | DESOLATION | B | FLOKE | D |
| ASTORIA | B | BRIDGECREEK | C | COKER | D | DESTER | C | FOEHLIN | B |
| ATERON | D | BRIDWELL | B | COLEMAN | C | DETER | C | FOPIANO | D |
| ATHENA | B | BRIGHTWOOD | B | COLESTINE | C | DICKERSON | D | FORDICE | B |
| ATRING | B | BROCK | D | COLLIER | A | DIGGER | C | FORDNEY | A |
| AUSMUS | D | BROCKMAN | C | CONCORD | D | DILANSON | D | FORDNEY, WET | C |
| AVERLANDE | D | BROGAN | B | CONDON | C | DILMAN | C | FORESTER | C |
| AWBRIG | D | BROWNLEE | B | CONLEY | C | DINZER | B | FORMADER | C |
| AYRES | D | BROWNSCOMBE | C | CONSER | D | DISABEL | C | FORSEER | C |
| BACONA | B | BRUNCAN | D | COPSEY | D | DIVERS | B | FORT ROCK | C |
| BAKEOVEN | D | BUCKCREEK | C | COQUILLE | D | DIXON | B | FRAILEY | B |
| BAKER | C | BUCKEYE | C | CORNELIUS | C | DIXONVILLE | C | FREEWATER | B |
| BALD | C | BUCKSHOT | B | CORNUTT | C | DOBBINS | C | FREEZENER | B |
| BALDER | D | BULL RUN | B | COTTRELL | C | DODES | B | FREN | B |
| BALDOCK | D | BULL RUN, HARDPAN | C | COUGHANOUR | C | DOGTOWN | B | FRITSLAND | B |
| BALDRIDGE | B | SUBSTRATUM | | COURT | B | DONICA | A | FROHMAN | C |
| BALM | D | BULLARDS | B | COURTNEY | D | DONICA, LOAMY | B | FUEGO | C |
| BANDON | C | BULLY | B | COURTROCK | B | SURFACE | | FURY | D |
| BANNING | C | BURBANK | A | COUSE | C | DONNING | D | FURY, DRAINED | C |
| BARCAVE | B | BURKE | C | COVE | D | DONNYBROOK | D | GACEY | D |
| BARHISKEY | A | BURLINGTON | A | COWSLY | C | DOWDE | B | GAPCOT | D |
| BARKLEY | C | BURNTRIVER | B | COYATA | C | DOYN | D | GARBUTT | B |
| BARLOW | B | BUTTON | D | CRABTREE | C | DREWS | B | GARDINER | A |
| BARNARD | C | BYBEE | D | CRACKERCREEK | B | DROVAL | C | GARDONE | A |
| BARRON | B | CALABAR | D | CRACKLER | B | DUART | C | GAULDY | B |
| BASHAW | D | CALDER | D | CRANKLER | B | DUBAKELLA | C | GEARHART | A |
| BATEMAN | B | CALDERWOOD | D | CRATER LAKE | B | DUFF | B | GEISEL | B |
| BAYSIDE | D | CALIMUS | B | CRIMS | D | DUFUR | B | GEM | C |
| BEDEN | D | CALOUSE | B | CROPLAND | D | DULANDY | C | GEM, STONY | D |
| BEDNER | C | CALZACORTA | D | CROOKED | D | DUMONT | B | GEPPERT | C |
| BEEKMAN | C | CAMAS | A | CROQUIS | D | DUPEE | C | GILISPIE | D |
| BEIRMAN | D | CAMPCREEK | C | CROWCAMP | D | DURKEE | C | GINGER | D |
| BELLPINE | C | CANDERLY | B | CRUISER | B | EAD | C | GINSER | C |
| BENHAM | C | CANEST | D | CRUME | B | EAGLECAP | B | GLASGOW | C |
| BENSLEY | B | CANTALA | B | CRUMP | D | EASTWELL | D | GLENEDEN | D |
| BENTILLA | C | CAPONA | C | CRUMP, DRAINED | C | ECOLA | C | GLOHM | C |

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUM, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| GOBLE | C | MORES | C | KUHL | D | MCCANN | B | NETARTS | B |
| GOODLOW | B | MORES, GRAVELLY | B | KWEO | A | MCCOIN | D | NEVADOR | B |
| GOODRICH | B | SUBSTRATUM | | LA GRANDE | C | MCCONNEL | B | NEVTAH | C |
| GOODWIN | B | MORNING | B | LABISH | D | MCCONNEL, FLOODED | A | NEWANNA | C |
| GOOLAWAY | C | MOSLEY | D | LABUCK | B | MCCULLY | C | NEWBERG | B |
| GOOSE LAKE | D | MOT LAKE | C | LADD | B | MCCURDY | C | NEWLANDS | B |
| GOSNEY | D | MOUSTAKE | C | LADERLY | C | MCDADE | C | NINEMILE | D |
| GRADON | C | MUSERLY | D | LADYCOMB | D | MCDUFF | C | NONPAREIL | D |
| GRANDE RONDE | D | MUOSPETH | C | LAIDLAW | C | MCEWEN | B | NORAD | B |
| GRASSVAL | D | MUFFLING | D | LAKEVIEW | C | MCGARR | C | NORTH POWDER | C |
| GRAYDEN | D | MUKILL | B | LAKI | B | MCGINNIS | C | NORTHRUP | C |
| GREENLEAF | B | MULLT | B | LAMBRING | B | MCKAY | C | NOTI | D |
| GREENSCOMBE | B | MUMMINGTON | C | LAMONTA | D | MCLOUGHLIN | B | NOTUS | C |
| GREGORY | D | MUNTROCK | B | LANGELLAIN | D | MCMEEN | C | NOUQUE | D |
| GPZLL | D | MURWAL | B | LANGLOIS | D | MCMILLE | B | NUSS | D |
| GRIBBLE | D | MUTCHINSON | C | LANGRELL | B | MCMULLIN | D | NYSSA | C |
| GRINDBROOK | C | MUTSON | B | LAPHAM | A | MCMURDIE | C | OAK GROVE | B |
| GUERIN | D | MYALL | C | LAPINE | A | MCNULL | C | OAKLAND | C |
| GURDANE | C | ICENE | D | LARMINE | D | MCNULTY | B | OATMAN | B |
| GUSTIN | D | ILLAHEE | B | LASTANCE | B | MEDA | B | OBRAY | D |
| GWIN | D | IMBLER | B | LATES | C | MEDCO | D | OCHOCO | C |
| GWIN, GRAVELLY | C | IMMIG | C | LATHER | D | MEDFORD | B | ODIN | C |
| GWINLY | D | INKLER | B | LATOURELL | B | MEHLHORN | C | OFFENBACHER | C |
| HACK | B | IRONDYKE | B | LAURELWOOD | B | MELBOURNE | B | OLAC | D |
| HACKWOOD | B | IRRIGON | C | LAWEN | B | MELBY | B | OLD CAMP | D |
| HAFLINGER | A | IZEE | C | LAYCOCK | B | MELO | C | OLEX | B |
| HAGER | D | JASON | D | LEGLER | B | MELLOWMOON | B | OLIPHANT | B |
| HALFWAY | D | JAYAR | C | LEMONEX | C | MEMALOOSE | C | OLOT | C |
| HALL RANCH | C | JEROME | D | LEMPIRA | B | MENBO | C | OLSON | D |
| HANKINS | C | JESSE CAMP | B | LERROW | C | MERLIN | D | OLYIC | B |
| HAPGOOD | B | JETT | B | LETHA | C | MERSHON | C | ONEONTA | B |
| HARANA | B | JIMBO | B | LETTIA | B | METOLIUS | B | ONTKO | D |
| HARDSCRABBLE | D | JOENEY | D | LICKSKILLET | D | MIKKALO | C | ONYX | B |
| HARLOW | D | JORY | B | LICKSKILLET, | C | MILBURY | C | OREANNA | B |
| HARRIMAN | B | JORY, STONY | C | GRAVELLY | | MILCAN | C | ORENEVA | C |
| HARRIMAN, WET | C | JOSEPHINE | B | LINSLAW | D | MILLICOMA | C | ORFORD | B |
| HARRINGTON | C | JOSSET | C | LINT | B | MINAM | B | OTOOLE | C |
| HARSLOW | C | JUMPOFF | C | LITHGOW | C | MINNIECE | D | OTWIN | C |
| HART | D | JUNTURA | D | LITTLE POLE | D | MOAG | D | OUTERKIRK | B |
| HARTIC | B | KAHLER | B | LOBERT | B | MOOKIN | C | OWYHEE | B |
| HATCHERY | C | KAMELA | C | LOCEY | C | MOODOC | C | OXBOW | C |
| HAZELAIR | D | KANID | B | LOCODA | D | MOE | B | OXLEY | C |
| HEADLEY | B | KANUTCHAN | D | LOFFTUS | C | MOLALLA | B | OXWALL | D |
| HEBO | D | KAUPPI | B | LOGDELL | B | MONDOVI | B | OZAMIS | D |
| HEBO, LOAMY | C | KEATING | C | LOLAK | D | MONTLID | C | PACKARD | B |
| SUBSTRATUM | | KEEL | C | LONELY | C | MOOLACK | A | PAIT | B |
| HECETA | D | KENNEWICK | B | LONERIDGE | C | MOREHOUSE | D | PALANUSH | C |
| HELMICK | D | KENUSKY | D | LOOKINGGLASS | C | MORFITT | B | PALOUSE | B |
| HELTER | B | KERBY | B | LOOKOUT | C | MORNINGSTAR | B | PANTHER | D |
| HELVETIA | C | KERRFIELD | D | LORELLA | D | MORROW | C | PARKDALE | B |
| HEMBRE | B | KETCHLY | B | LOSTBASIN | C | MOSIDA | B | PARMELE | C |
| HEMCROSS | B | KIESEL | C | LOSTINE | B | MOSIDA, CALCAREOUS | A | PATIO | C |
| HENLEY | C | KILCHIS | D | LOYLINE | C | MOUND | C | PAULINA | D |
| HENLINE | C | KILMEROUE | C | LOZA | D | MOYINA | D | PEARSOLL | D |
| HEPPSIE | D | KILOWAN | C | LUCE | C | MT. HOOD | B | PEAVINE | C |
| HERMISTON | B | KIMBERLY | B | LUCKIAMUTE | D | MUES | C | PEDIGO | C |
| HERSHAL | D | KINNEY | B | LURNICK | C | MULKEY | C | PEEL | C |
| HESSLAN | C | KINTON | C | MACKLYN | C | MULTNOMAH | B | PENGRA | C |
| HEZEL | B | KINZEL | B | MADELINE | D | MULTORPOR | A | PERDIN | C |
| HIBBARD | C | KIRK | D | MADRAS | C | MURNEN | B | PERNTY | D |
| HIGHCAMP | B | KIRKENDALL | C | MAKLAK | A | MURTIP | B | PERVINA | B |
| HIGHHORN | B | KLAMATH | D | MALABON | C | MUSTY | C | PHILOMATH | D |
| HILLSBORO | B | KLAWHOP | B | MALHEUR | C | NAGLE | B | PHOENIX | D |
| HISKEY | B | KLICKER | C | MALIN | C | NANSENE | B | PHYS | B |
| HOBIT | C | KLICKITAT | B | MANITA | C | NATAL | D | PIERSONTE | A |
| HOLCOMB | D | KLISTAN | B | MARACK | C | NATROY | D | PILCHUCK | C |
| HOLDERMAN | C | KLOOQUEH | B | MARCOLA | C | NECANICUM | B | PILCHUCK, | A |
| HOLLAND | B | KLOOTCHIE | B | MARSOEN | B | NEHALEM | B | PROTECTED | |
| HOLTLE | B | KNAPKE | B | MARTY | B | NEHALEM, FLOODED | C | PILOT ROCK | C |
| HOMESTAKE | C | KNAPPA | B | MASCAMP | D | NEKIA | C | PINEHURST | B |
| HONEYGROVE | B | KOEHLER | C | MASET | B | NEKOMA | B | PIT | D |
| HOOD | B | KOTZMAN | B | MAUPIN | C | NELSCOTT | C | PLUSH | B |
| HOODVIEW | B | KRACKLE | B | MAYGER | C | NEPALTO | A | POALL | C |
| HOOLY | C | KREBS | B | MCALPIN | C | NESKOWIN | C | PODEN | B |
| HOOPAL | D | KUBLI | D | MCBEE | C | NESTUCCA | D | POE | C |

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUM, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

## HYDROLOGIC GROUPS OF THE SOIL SERIES USED IN OREGON

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| POKEGEMA | B | SAHALIE | B | SPRINGWATER | C | VENA | C | WOODCOCK | B | | |
| POLLARD | C | SALANDER | B | STAMPEDE | D | VENATOR | C | WOODSEYE | D | | |
| POLLY | B | SALEM | B | STANFIELD | C | VENETA | D | WRENTHAM | C | | |
| PONINA | D | SALISBURY | C | STARBUCK | D | VERBOORT | D | WRIGHTMAN | C | | |
| POTAMUS | B | SALISBURY, HIGH | D | STARKEY | C | VERGAS | C | WUKSI | A | | |
| POUJADE | D | ELEVATION | | STATZ | D | VERMISA | D | WYEAST | D | | |
| POWDER | B | SALKUM | B | STAYTON | D | VERNONIA | B | WYETH | B | | |
| POWELL | C | SANTIAM | C | STEARNS | D | VIL | D | XANADU | B | | |
| POWLEY | D | SATURN | B | STEIGER | A | VILOT | C | YAINAX | B | | |
| POWWATKA | C | SAUM | B | STEIWER | C | VINING | C | YAKIMA | B | | |
| PRAG | C | SAUVIE | D | STICES | B | VIRTUE | C | YALLANI | B | | |
| PREACHER | B | SAUVIE, PROTECTED | B | STOCKEL | D | VOATS | B | YAMHILL | C | | |
| PRICE | B | SAWTELL | C | STOCKMOOR | C | VOLTAGE | B | YAMSAY | D | | |
| PRINEVILLE | C | SCAPONIA | B | STOVEPIPE | D | VONDERGREEN | C | YANCY | D | | |
| PRITCHARD | C | SCAREDMAN | B | STRAIGHT | C | VOORHIES | C | YAPOAH | B | | |
| PRONG | C | SCHAMP | C | STUKEL | D | WAGONTIRE | D | YAQUINA | D | | |
| PROSSER | C | SCHERRARD | D | STURGILL | D | WAHA | C | YAQUINA, DRAINED | C | | |
| PROVIG | C | SCHRIER | B | SUMPLEY | C | WAHKEENA | B | YARCO | D | | |
| PULS | D | SEARLES | C | SUTHERLIN | C | WANSTAL | D | YAWHEE | B | | |
| PYBURN | D | SEBASTIAN | D | SUVER | D | WAHTIGUP | B | YAWKEY | B | | |
| QUAFENG | C | SECRET CREEK | B | SVENSEN | B | WALDO | D | YELLOWSTONE | D | | |
| QUARTZVILLE | B | SEGUNDO | B | SWALER | D | WALDPORT | A | YONNA | D | | |
| QUARZ | C | SELMAC | D | SWALESILVER | D | WALLA WALLA | B | YOUTLKUE | D | | |
| QUATAMA | C | SEMIAHMOO | D | SWARTZ | D | WALLOWA | C | ZANGO | D | | |
| QUILLAMOOK | B | SERPENTANO | B | SYCAN | A | WALLUSKI | C | ZING | C | | |
| QUILLAYUTE | B | SEVENOAKS | A | TAHKENITCH | B | WALVAN | B | ZUMAN | D | | |
| QUINCY | A | SHANAHAN | B | TAKILMA | B | WAMIC | B | ZUMAN, PROTECTED | C/D | | |
| QUINTON | C | SHANO | B | TALAPUS | B | WANOGA | B | ZUMWALT | C | | |
| QUOSATANA | D | SHEFFLEIN | B | TALLOWBOX | C | WANSER | D | ZWAGG | C | | |
| RAFTON | D | SHERAR | C | TANDY | D | WAPATO | D | ZYGORE | B | | |
| RAIL | D | SHEVLIN | C | TATERPA | B | WAPINITIA | B | ZYZZUG | D | | |
| RAMO | C | SHIPPA | D | TATOUCHE | B | WARDSORO | A | | | | |
| RANDCORE | D | SHIVIGNY | B | TAUNTON | C | WARDEN | B | | | | |
| RASTUS | C | SHOAT | D | TEETERS | C | WARRENTON | D | | | | |
| RATTO | C | SHUKASH | A | TELEMON | D | WATAMA | C | | | | |
| RATTO, STONY | D | SIBANNAC | D | TEMPLETON | B | WATERBURY | D | | | | |
| RAZ | D | SIFTON | B | TERRABELLA | D | WATO | B | | | | |
| REALLIS | B | SILAS | B | TETHRICK | B | WAULD | C | | | | |
| REAVIS | B | SILAS, GRAVELLY | C | THADER | C | WAUNA | D | | | | |
| REDBELL | B | SUBSTRATUM | | THATUNA | C | WAUNA, PROTECTED | C | | | | |
| REDCLIFF | C | SILAS, WET | C | THISTLEBURN | B | WEBFOOT | C | | | | |
| REDMOND | C | SILCOX | B | TIMBERLY | B | WEDDERBURN | B | | | | |
| REDMOUNT | B | SILETZ | B | TISHAR | B | WELCH | D | | | | |
| REEDSPORT | C | SILVERTON | C | TOLANY | B | WESTBUTTE | C | | | | |
| REINECKE | B | SILVIES | D | TOLKE | B | WHETSTONE | C | | | | |
| REMOTE | B | SIMAS | C | TOLO | B | WHITEHORSE | B | | | | |
| RESTON | D | SINAMOX | B | TONOR | C | WHITESON | D | | | | |
| RHEA | B | SINKER | C | TOP | C | WHITING | B | | | | |
| RICCO | D | SISKIYOU | B | TOPPER | B | WHOBREY | C | | | | |
| RICKREALL | D | SISLEY | B | TOURNQUIST | B | WICKIUP | C | | | | |
| RIDLEY | C | SISTERS | A | TRASK | C | WILHOIT | B | | | | |
| RINEARSON | B | SKEDADDLE | D | TREHARNE | C | WILKINS | D | | | | |
| RITNER | C | SKELLOCK | B | TRUESDALE | C | WILLAKENZIE | C | | | | |
| RITTER | B | SKINNER | B | TUB | C | WILLAMETTE | B | | | | |
| RITZVILLE | B | SKIPANON | B | TULANA, DRAINED | B | WILLAMETTE, WET | C | | | | |
| ROBINETTE | B | SKOOKUM | C | TULANA, NONFLOODED | C | WILLANCH | D | | | | |
| ROBSON | D | SKOOKUMHOUSE | B | TUMALO | C | WILLIS | C | | | | |
| ROCKFORD | B | SKULLGULCH | C | TUMTUM | D | WILLOWDALE | B | | | | |
| ROCKLY | D | SKYLINE | D | TURBYFILL | B | WINBERRY | C | | | | |
| ROGUE | B | SKYMOR | D | TUTNI | B | WINCHESTER | A | | | | |
| ROLOFF | C | SLAYTON | D | TUTUILLA | C | WINCHUCK | C | | | | |
| RONDOWA | B | SLICKROCK | B | TYGH | C | WIND RIVER | B | | | | |
| ROSEBURG | B | SNAG | B | UKIAH | D | WINEMA | C | | | | |
| ROSEHAVEN | B | SNAKER | D | UMAPINE | D | WINGVILLE | D | | | | |
| ROUEN | C | SNELL | C | UMAPINE, DRAINED | C | WINLO | D | | | | |
| ROYAL | B | SNELLBY | C | UMATILLA | B | WINOM | D | | | | |
| ROYST | C | SNOW | B | UMIL | D | WINOPEE | B | | | | |
| RUCH | B | SNOWLIN | B | UMPCOOS | D | WINTLEY | B | | | | |
| RUCKLES | D | SOOSAP | C | UTLEY | B | WITHAM | D | | | | |
| RUCLICK | C | SORF | C | VALBY | C | WITZEL | D | | | | |
| RUDDLEY | D | SPANGENBURG | C | VALSETZ | C | WOLFER | B | | | | |
| RUGG | B | SPANGENBURG, | D | VAN HORN | B | WOLLENT | D | | | | |
| RUTAB | B | PONDED | | VANNOY | C | WOLOT | B | | | | |
| SAG | B | SPEAKER | C | VARCO | D | WOLVERINE | A | | | | |
| SAGEHILL | B | SPOFFORD | D | VEAZIE | B | WOODBURN | C | | | | |

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUM, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

## Appendix B - Synthetic Rainfall Distributions and Rainfall Data Sources

### Synthetic Rainfall Distributions

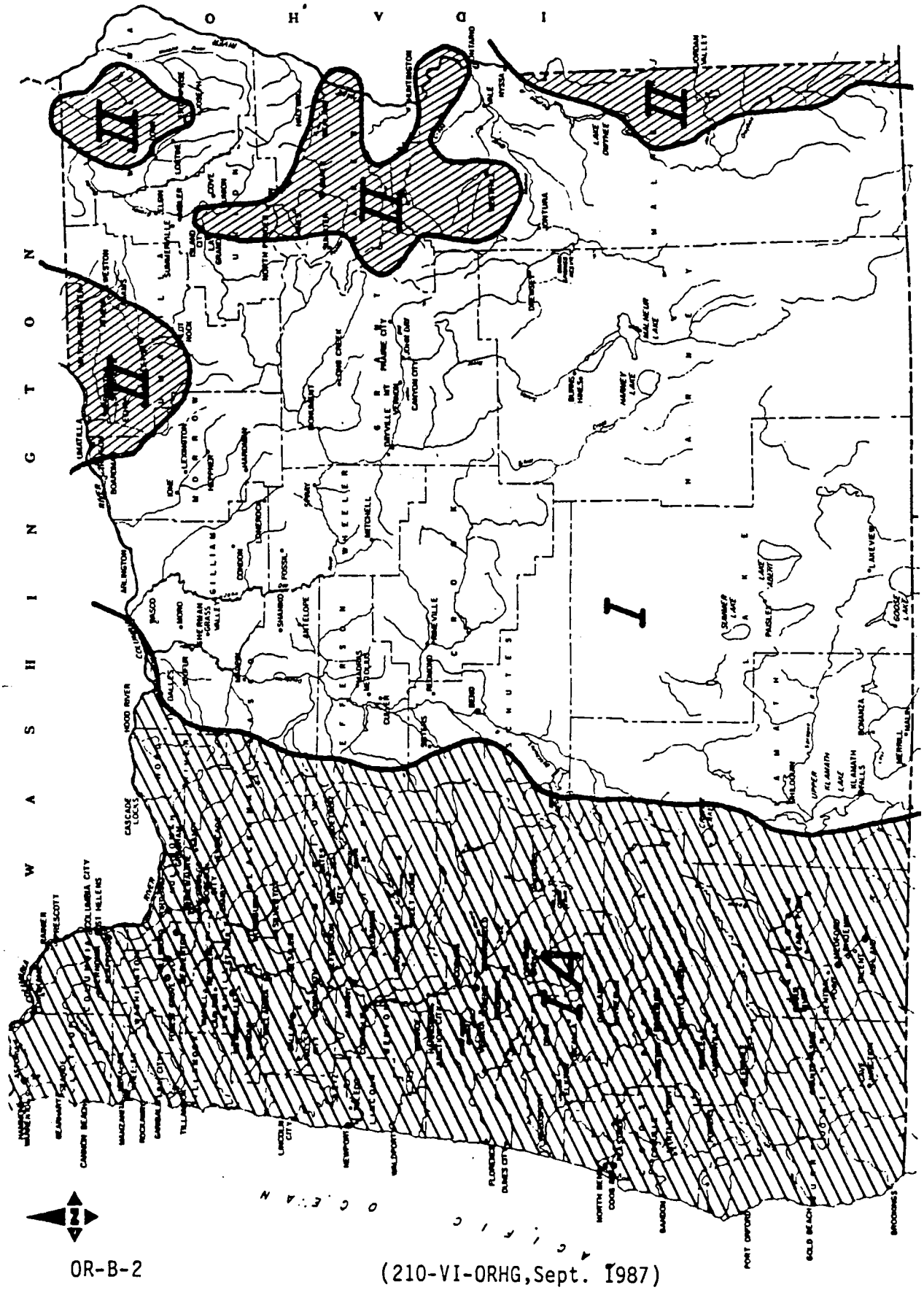
Storm distribution types I, IA, and II are to be used in Oregon. The following map shows the areas where each distribution type applies. Generally, the area from the east slopes of the Cascade Mountains to the Pacific Ocean would use type IA distribution. Type I is used for most of the area east of the Cascades with Type II being used in specific areas of extreme eastern and northeastern Oregon.

### Rainfall Data Sources

Precipitation records are collected and maintained by NOAA. Data is published monthly in Climatalogical Data (daily records) and Hourly Precipitation Data. These records are kept at the SCS State Office, at the WNTC-WSF, with the State Climatologist in Corvallis, and at many local libraries.

NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume X-Oregon, contains maps of the state with various storm durations and frequencies. The 24-hour storm duration maps have been reprinted and are included in this appendix.

NOAA Technical Report NWS 36, Water Available for Runoff ... for Selected Agricultural Regions in the Northwest United States, gives combined frequency of precipitation with water available from snowmelt for all or part of 11 counties in Northeast Oregon. Maps present Water Available for Runoff (WAR) for 1-, 5-, and 10-day durations and 2-, 10-, and 100-year frequencies. These may be used to supplement the precipitation maps. Copies of the NWS 36 report are available from the SCS Oregon State Conservation Engineer.




OR-B-2

(210-VI-ORHG, Sept. 1987)

## Appendix C - Computer Program

The TR-55 computer program is set up to help guide through the input of data and will solve the equations for computing RCN, Tc, and peak flow. Samples of the computer output of each routine are attached. These use the input data of the examples in this guide.

The program requests a county name as early input data. If the name entered agrees with one on the county rainfall table in the program, the rainfall-frequency data will be read by the program. A county rainfall table for Oregon is in the program; however, the large counties and varied rainfall patterns required that multiple locations in a county be identified. The following Table C-1 lists county-city combinations which are in the program.

The locations are generally in the agricultural areas of the counties. This assumes that usually peak flows will be wanted in these areas. If the location under study is not similar to one of the cities listed, rainfall data from the maps in Appendix B should be entered where needed.

The county-city entry must be entered just as it is listed for the program to recognize the location (i.e., Clackamas-Oregon Cy). The spacing and abbreviations need to be as listed. Upper or lower case letters are acceptable.

TABLE C-1 - TR-55 COMPUTER PROGRAM COUNTY RAINFALL DATA SITES

| AREA | COUNTY-CITY | AREA | COUNTY-CITY |
|---|---|---|---|
| 1 | BENTON-CORVALLIS | 1 | LINN-LEBANON |
| 1 | CLACKAMAS-OREGON CY | 1 | LINN-HALSEY |
| 1 | CLACKAMAS-SANDY | 1 | LINN-SWEET HOME |
| 1 | CLACKAMAS-MOLLALA | 1 | MARION-SALEM |
| 1 | CLATSOP-ASTORIA | 1 | MARION-MILL CITY |
| 1 | CLATSOP-CANNON BCH | 1 | MULTNOMAH-PORTLAND |
| 1 | COLUMBIA-CLATSKANIE | 1 | POLK-DALLAS |
| 1 | COLUMBIA-VERNONIA | 1 | POLK-FALLS CITY |
| 1 | COLUMBIA-SCAPPOOSE | 1 | TILLAMOOK-NEHALEM |
| 1 | LANE-FLORENCE | 1 | TILLAMOOK-TILLAMOOK |
| 1 | LANE-EUGENE | 1 | WASHINGTON-HILLSBORO |
| 1 | LANE-JUNCTION CITY | 1 | WASHINGTON-BANKS |
| 1 | LANE-OAKRIDGE | 1 | YAMHILL-MCMINNVILLE |
| 1 | LINCOLN-NEWPORT | 1 | YAMHILL-NEWBERG |
| 1 | LINCOLN-LINCOLN CY | | |

| AREA | COUNTY-CITY | AREA | COUNTY-CITY |
|---|---|---|---|
| 2 | COOS-COOS BAY | 2 | JACKSON-MEDFORD |
| 2 | COOS-MYRTLE POINT | 2 | JACKSON-BUTTE FALLS |
| 2 | COOS-POWERS | 2 | JEFFERSON-MADRAS |
| 2 | CROOK-PRINEVILLE | 2 | JOS.-GRANTS PASS |
| 2 | CROOK-CAMP CR JCT | 2 | JOS.-CAVE JUNCTION |
| 2 | CURRY-GOLD BEACH | 2 | KLAM.-KLAMATH FALLS |
| 2 | DESCHUTES-BEND | 2 | KLAM.-BONANZA |
| 2 | DESCHUTES-REDMOND | 2 | LAKE-LAKEVIEW |
| 2 | DOUGLAS-REEDSPORT | 2 | LAKE-PAISLEY |
| 2 | DOUGLAS-ELKTON | 2 | SHERMAN-MORO |
| 2 | DOUGLAS-ROSEBURG | 2 | WASCO-THE DALLES |
| 2 | HOOD R.-CASCADE LOC. | 2 | WASCO-MAUPIN |
| 2 | HOOD R.-HOOD RIVER | 2 | WASCO-ANTELOPE |

| AREA | COUNTY-CITY | AREA | COUNTY-CITY |
|---|---|---|---|
| 3 | BAKER-BAKER | 3 | MORROW-BOARDMAN |
| 3 | BAKER-SUMPTER | 3 | MORROW-HEPPNER |
| 3 | BAKER-RICHLAND | 3 | UMA.-HERMISTON |
| 3 | GILLIAM-ARLINGTON | 3 | UMA.-PENDLETON |
| 3 | GILLIAM-CONDON | 3 | UMA.-MILTON-FREE. |
| 3 | GRANT-MONUMENT | 3 | UNION-ELGIN |
| 3 | GRANT-JOHN DAY | 3 | UNION-LAGRANDE |
| 3 | HARNEY-DREWSEY | 3 | UNION-NORTH POWDER |
| 3 | HARNEY-BURNS | 3 | WALLOWA-ENTERPRISE |
| 3 | MAL.-ONTARIO | 3 | WALLOWA-WALLOWA |
| 3 | MAL.-JORDAN VALLEY | 3 | WHEELER-FOSSIL |



(210-VI-ORHG,Sept. 1987)

## Example 2-1

TR-55 CURVE NUMBER COMPUTATION VERSION 1.11

Project : Pendleton watershed User: rsw Date: 08-19-87
County : UMA.-PENDLETON State: OR Checked: ____ Date: ________
Subtitle: Examples- sub-area 1

| COVER DESCRIPTION | | | Hydrologic Soil Group A | B | C | D |
|---|---|---|---|---|---|---|
| | | | | Acres (CN) | | |
| CULTIVATED AGRICULTURAL LANDS | | | | | | |
| Fallow | Bare soil | ---- | - | 690(86) | 45(91) | - |
| | Crop residue (CR) | poor | - | 120(85) | - | - |
| Small grain | Straight row (SR) | poor | - | 690(76) | 45(84) | - |
| | C + Crop residue | poor | - | 120(73) | - | - |
| Total Area (by Hydrologic Soil Group) | | | | 1620 | 90 | |
| | | | | **** | **** | |

TOTAL DRAINAGE AREA: 1710 Acres WEIGHTED CURVE NUMBER:81*

* - Generated for use by GRAPHIC method

## Example 3-1

TR-55 Tc and Tt THRU SUBAREA COMPUTATION VERSION 1.11

Project : Pendleton watershed User: rsw Date: 08-19-87
County : UMA.-PENDLETON State: OR Checked: ____ Date: ________
Subtitle: Examples- sub-area 1

| Flow Type | 2 year rain | Length (ft) | Slope (ft/ft) | Surface code | n | Area (sq/ft) | Wp (ft) | Velocity (ft/sec) | Time (hr) |
|---|---|---|---|---|---|---|---|---|---|
| Sheet | 1.0 | 300 | .033 | d | | | | | 0.636 |
| Shallow Concent'd | | 900 | .039 | u | | | | | 0.078 |
| Open Channel | | 7500 | .010 | | .040 | 6.0 | 10.2 | | 0.797 |
| Open Channel | | 6500 | .006 | | .045 | 28.0 | 14.0 | | 0.443 |
| | | | | | | Time of Concentration = | | | 1.96* |
| | | | | | | | | | ***** |

--- Sheet Flow Surface Codes ---
A Smooth Surface
B Fallow (No Res.)
C Cultivated < 20 % Res.
D Cultivated > 20 % Res.
E Grass-Range, Short
F Grass, Dense
G Grass, Burmuda
H Woods, Light
I Woods, Dense

--- Shallow Concentrated --- Surface Codes ---
P Paved
U Unpaved

* - Generated for use by GRAPHIC method

## Example 2-2

TR-55 CURVE NUMBER COMPUTATION VERSION 1.11

Project : Pendleton Watershed
County : UMA.-PENDLETON State: OR
Subtitle: EXAMPLES sub-areas 1 & 2
Subarea : 2

User: rsw Date: 08-19-87
Checked: ____ Date: ________

| COVER DESCRIPTION | | | Hydrologic Soil Group A | B | C | D |
|---|---|---|---|---|---|---|
| | | | | Acres (CN) | | |
| FULLY DEVELOPED URBAN AREAS (Veg Estab.) | | | | | | |
| Streets and roads | | | | | | |
| Gravel (w/ right-of-way) | | | - | 7(85) | 1(89) | - |
| CULTIVATED AGRICULTURAL LANDS | | | | | | |
| Fallow | Bare soil | ---- | - | 345(86) | 30(91) | - |
| | Crop residue (CR) | good | - | 275(83) | - | - |
| Small grain | Straight row (SR) | poor | - | 344(76) | 31(84) | - |
| | Cont & terraces(C&T) | good | - | 275(70) | - | - |
| ARID AND SEMIARID RANGELANDS | | | | | | |
| Sagebrush (w/ grass understory) | | fair | - | 90(51) | - | - |
| Total Area (by Hydrologic Soil Group) | | | | 1336 | 62 | |
| | | | | ==== | ==== | |

SUBAREA: 2 TOTAL DRAINAGE AREA: 1398 Acres WEIGHTED CURVE NUMBER:78

## Example 3-2

TR-55 Tc and Tt THRU SUBAREA COMPUTATION VERSION 1.11

Project : Pendleton Watershed
County : UMA.-PENDLETON State: OR
Subtitle: EXAMPLES sub-areas 1 & 2

User: rsw Date: 08-19-87
Checked: ____ Date: ________

Subarea #2 - 2

| Flow Type | 2 year rain | Length (ft) | Slope (ft/ft) | Surface code | n | Area (sq/ft) | Wp (ft) | Velocity (ft/sec) | Time (hr) |
|---|---|---|---|---|---|---|---|---|---|
| Sheet | 1.0 | 300 | .020 | d | | | | | 0.778 |
| Shallow Concent'd | | 2100 | .021 | u | | | | | 0.249 |
| Open Channel | | 3500 | .020 | | .045 | 2.64 | 6.95 | | 0.396 |
| Open Channel | | 7000 | | | | | | 4.66 | 0.417 |
| | | | | | | | Time of Concentration = | | 1.84* |
| | | | | | | | | | ===== |
| Open Channel | | 9000 | .007 | | .045 | 34.0 | 15.6 | | 0.537 |
| | | | | | | | Travel Time = | | 0.54* |
| | | | | | | | | | ===== |

--- Sheet Flow Surface Codes ---
A Smooth Surface
B Fallow (No Res.)
C Cultivated < 20 % Res.
D Cultivated > 20 % Res.
E Grass-Range, Short
F Grass, Dense
G Grass, Burmuda
H Woods, Light
I Woods, Dense

--- Shallow Concentrated --- Surface Codes ---
P Paved
U Unpaved

* - Generated for use by TABULAR method

## Example 4-1

TR-55 GRAPHICAL DISCHARGE METHOD VERSION 1.11

Project : Pendleton watershed User: rsw Date: 08-19-87
County : UMA.-PENDLETON State: OR Checked: ____ Date: ________
Subtitle: Examples- sub-area 1

Data: Drainage Area : 1710 * Acres
Runoff Curve Number : 81 *
Time of Concentration: 1.96 * Hours
Rainfall Type : II
Pond and Swamp Area : NONE

| Storm Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|
| Frequency (yrs) | 2 | 5 | 10 | 25 | 50 | 100 | 0 |
| 24-Hr Rainfall (in) | 1 | 1.2 | 1.3 | 1.5 | 1.6 | 1.8 | 0 |
| Ia/P Ratio | 0.47 | 0.39 | 0.36 | 0.31 | 0.29 | 0.26 | 0.00 |
| Runoff (in) | 0.10 | 0.17 | 0.22 | 0.31 | 0.37 | 0.48 | 0.00 |
| Unit Peak Discharge (cfs/acre/in) | 0.196 | 0.244 | 0.260 | 0.286 | 0.295 | 0.306 | 0.000 |
| Pond and Swamp Factor 0.0% Ponds Used | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Peak Discharge (cfs) | 33 | 72 | 97 | 154 | 186 | 252 | 0 |

* - Value(s) provided from TR-55 system routines

## Example 5-1 (with Ia/P rounded)

TR-55 TABULAR DISCHARGE METHOD VERSION 1.11

Project : Pendleton Watershed User: rsw Date: 08-19-87
County : UMA.-PENDLETON State: OR Checked: ____ Date: ________
Subtitle: EXAMPLES sub-areas 1 & 2

Total watershed area: 4.854 sq mi Rainfall type: II Frequency: 25 years

| | Subareas 1 | 2 |
|---|---|---|
| Area(sq mi) | 2.67 | 2.18* |
| Rainfall(in) | 1.5 | 1.5 |
| Curve number | 81 | 78* |
| Runoff(in) | 0.31 | 0.23 |
| Tc (hrs) | 1.96 | 1.84* |
| (Used) | 2.00 | 2.00 |
| TimeToOutlet | 0.54* | 0.00 |
| (Used) | 0.50 | 0.00 |
| Ia/P | 0.31 | 0.38 |
| (Used) | 0.30 | 0.30 |

| Time (hr) | Total Flow | Subarea Contribution to Total Flow (cfs) 1 | 2 |
|---|---|---|---|
| 11.0 | 0 | 0 | 0 |
| 11.3 | 0 | 0 | 0 |
| 11.6 | 0 | 0 | 0 |
| 11.9 | 0 | 0 | 0 |
| 12.0 | 0 | 0 | 0 |
| 12.1 | 1 | 0 | 1 |
| 12.2 | 2 | 0 | 2 |
| 12.3 | 4 | 0 | 4 |
| 12.4 | 9 | 1 | 8 |
| 12.5 | 15 | 2 | 13 |
| 12.6 | 22 | 3 | 19 |
| 12.7 | 36 | 8 | 28 |
| 12.8 | 51 | 13 | 38 |
| 13.0 | 90 | 31 | 59 |
| 13.2 | 132 | 57 | 75 |
| 13.4 | 173 | 87 | 86 |
| 13.6 | 208 | 114 | 94P |
| 13.8 | 221 | 134 | 87 |
| 14.0 | 225P | 144P | 81 |
| 14.3 | 206 | 139 | 67 |
| 14.6 | 177 | 121 | 56 |
| 15.0 | 141 | 96 | 45 |
| 15.5 | 109 | 73 | 36 |
| 16.0 | 87 | 58 | 29 |
| 16.5 | 72 | 47 | 25 |
| 17.0 | 61 | 40 | 21 |
| 17.5 | 54 | 35 | 19 |
| 18.0 | 48 | 31 | 17 |
| 19.0 | 41 | 26 | 15 |
| 20.0 | 36 | 23 | 13 |
| 22.0 | 28 | 18 | 10 |
| 26.0 | 11 | 8 | 3 |

P - Peak Flow * - value(s) provided from TR-55 system routines

## Example 5-1 (with Ia/P as computed)

TR-55 TABULAR DISCHARGE METHOD VERSION 1.11

Project : Pendleton Watershed User: rsw Date: 08-19-87
County : UMA.-PENDLETON State: OR Checked: ____ Date: ________
Subtitle: EXAMPLES sub-areas 1 & 2

Total watershed area: 4.854 sq mi Rainfall type: II Frequency: 25 years

| | Subareas 1 | 2 |
|---|---|---|
| Area(sq mi) | 2.67 | 2.18* |
| Rainfall(in) | 1.5 | 1.5 |
| Curve number | 81 | 78* |
| Runoff(in) | 0.31 | 0.23 |
| Tc (hrs) | 1.96 | 1.84* |
| (Used) | 2.00 | 2.00 |
| TimeToOutlet | 0.54* | 0.00 |
| (Used) | 0.50 | 0.00 |
| Ia/P | 0.31 | 0.38 |

| Time (hr) | Total Flow | Subarea Contribution to Total Flow (cfs) 1 | 2 |
|---|---|---|---|
| 11.0 | 0 | 0 | 0 |
| 11.3 | 0 | 0 | 0 |
| 11.6 | 0 | 0 | 0 |
| 11.9 | 0 | 0 | 0 |
| 12.0 | 0 | 0 | 0 |
| 12.1 | 1 | 0 | 1 |
| 12.2 | 1 | 0 | 1 |
| 12.3 | 3 | 0 | 3 |
| 12.4 | 7 | 1 | 6 |
| 12.5 | 12 | 2 | 10 |
| 12.6 | 17 | 3 | 14 |
| 12.7 | 29 | 8 | 21 |
| 12.8 | 42 | 13 | 29 |
| 13.0 | 75 | 29 | 46 |
| 13.2 | 116 | 55 | 61 |
| 13.4 | 155 | 84 | 71 |
| 13.6 | 189 | 111 | 78P |
| 13.8 | 205 | 130 | 75 |
| 14.0 | 211P | 139P | 72 |
| 14.3 | 197 | 135 | 62 |
| 14.6 | 172 | 119 | 53 |
| 15.0 | 140 | 95 | 45 |
| 15.5 | 110 | 73 | 37 |
| 16.0 | 89 | 58 | 31 |
| 16.5 | 75 | 48 | 27 |
| 17.0 | 65 | 41 | 24 |
| 17.5 | 57 | 36 | 21 |
| 18.0 | 52 | 32 | 20 |
| 19.0 | 44 | 27 | 17 |
| 20.0 | 39 | 24 | 15 |
| 22.0 | 30 | 19 | 12 |
| 26.0 | 11 | 8 | 3 |

P - Peak Flow * - value(s) provided from TR-55 system routines

## Appendix D - Worksheets

Worksheets are included which can be reproduced for use with Chapters 2 through 5. Notes have been added to the worksheets referencing the data sources.

# Worksheet 2: Runoff curve number and runoff

Project _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed _____

## 1. Runoff curve number (CN)

| Soil name and hydrologic group (HG APP-A) | Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio) (TR-55 Table 2-2) | CN 1/ Table 2-2 | CN 1/ Fig. 2-3 | CN 1/ Fig. 2-4 | Area ☐ acres ☐ mi² ☐ % | Product of CN x area |
|---|---|---|---|---|---|---|
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |

1/ Use only one CN source per line. (TR-55) Totals =

CN (weighted) = $\frac{\text{total product}}{\text{total area}}$ = _____ = _____; Use CN = ☐

## 2. Runoff

| | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|
| Frequency ..... yr | | | |
| Rainfall, P (24-hour) ..... (HG APP-B) ..... in | | | |
| Runoff, Q ..... in (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.) (TR-55) | | | |

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed _____

Circle one: $T_c$ $T_t$ through subarea _____

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... (TR-55) | | | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | | | |
| 3. Flow length, L (total L ≤ 300 ft) ..... | ft | | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | | | |
| 5. Land slope, s ..... | ft/ft | | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute $T_t$ ..... | hr | | + | = |

| Shallow concentrated flow | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | | | |
| 8. Flow length, L ..... | ft | | | |
| 9. Watercourse slope, s ..... | ft/ft | | | |
| 10. Average velocity, V (figure 3-1) ..... (TR-55) | ft/s | | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |

| Channel flow | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... | $ft^2$ | | | |
| 13. Wetted perimeter, $p_w$ ..... | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ..... | ft | | | |
| 15. Channel slope, s ..... | ft/ft | | | |
| 16. Manning's roughness coeff., n (HG Table 3-2) ..... | | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... | ft/s | | | |
| 18. Flow length, L ..... | ft | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |

20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... hr

# Worksheet 4: Graphical Peak Discharge method

Project _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed _____

1. Data:

   Drainage area ..... $A_m$ = _____ $mi^2$ (acres/640)

   Runoff curve number .... CN = _____ (From worksheet 2)

   Time of concentration .. $T_c$ = _____ hr (From worksheet 3)

   Rainfall distribution type = _____ (I, IA, II, III) (HG APP-B)

   Pond and swamp areas spread throughout watershed ..... = _____ percent of $A_m$ (____ acres or $mi^2$ covered)

| | | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|---|
| 2. | Frequency ..... yr | | | |
| 3. | Rainfall, P (24-hour) ..... in | | | |
| 4. | Initial abstraction, $I_a$ ..... in (Use CN with table 4-1.) (TR-55) | | | |
| 5. | Compute $I_a/P$ ..... | | | |
| 6. | Unit peak discharge, $q_u$ ..... csm/in (Use $T_c$ and $I_a/P$ with exhibit 4-____) (TR-55) | | | |
| 7. | Runoff, Q ..... in (From worksheet 2). | | | |
| 8. | Pond and swamp adjustment factor, $F_p$ .... (Use percent pond and swamp area with table 4-2. Factor is 1.0 for zero percent pond and swamp area.). (TR-55) | | | |
| 9. | Peak discharge, $q_p$ ..... cfs (Where $q_p = q_u A_m Q F_p$) | | | |

## Worksheet 5a: Basic watershed data

Project _____ Location _____ By _____ Date _____

Circle one: Present Developed _____ Frequency (yr) _____ Checked _____ Date _____

| Subarea name | Drainage area $A_m$ ($mi^2$) | Time of concentration $T_c$ (hr) | Travel time through subarea $T_t$ (hr) | Downstream subarea names | Travel time summation to outlet $\Sigma T_t$ (hr) | 24-hr Rainfall P (in) | Runoff curve number CN | Runoff Q (in) | $A_m Q$ ($mi^2$-in) | Initial abstraction $I_a$ (in) | $I_a/P$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | |
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From worksheet 3 | From worksheet 2 | From table 5-1 (TR-55)

(210-VI-ORHG, Sept. 1987)

OR-D-6

# Worksheet 5b: Tabular hydrograph discharge summary

Project _____ Location _____ By _____ Date _____

Circle one: Present Developed _____ Frequency (yr) _____ Checked _____ Date _____

| Subarea name | Basic watershed data used 1/ | | | | Select and enter hydrograph times in hours from exhibit 5- 2/ | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sub-area $T_c$ (hr) | $\Sigma T_t$ to outlet (hr) | $I_a/P$ | $A_m Q$ ($mi^2$-in) | Discharges at selected hydrograph times 3/ -(cfs)- | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
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| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| Composite hydrograph at outlet | | | | | | | | | | | | | | | | |

1/ Worksheet 5a. Rounded as needed for use with exhibit 5. (TR-55)
2/ Enter rainfall distribution type used. (HG APP-B)
3/ Hydrograph discharge for selected times is $A_m Q$ multiplied by tabular discharge from appropriate exhibit 5. (TR-55)

(210-VI-ORHG,Sept. 1987)

United States
Department of
Agriculture

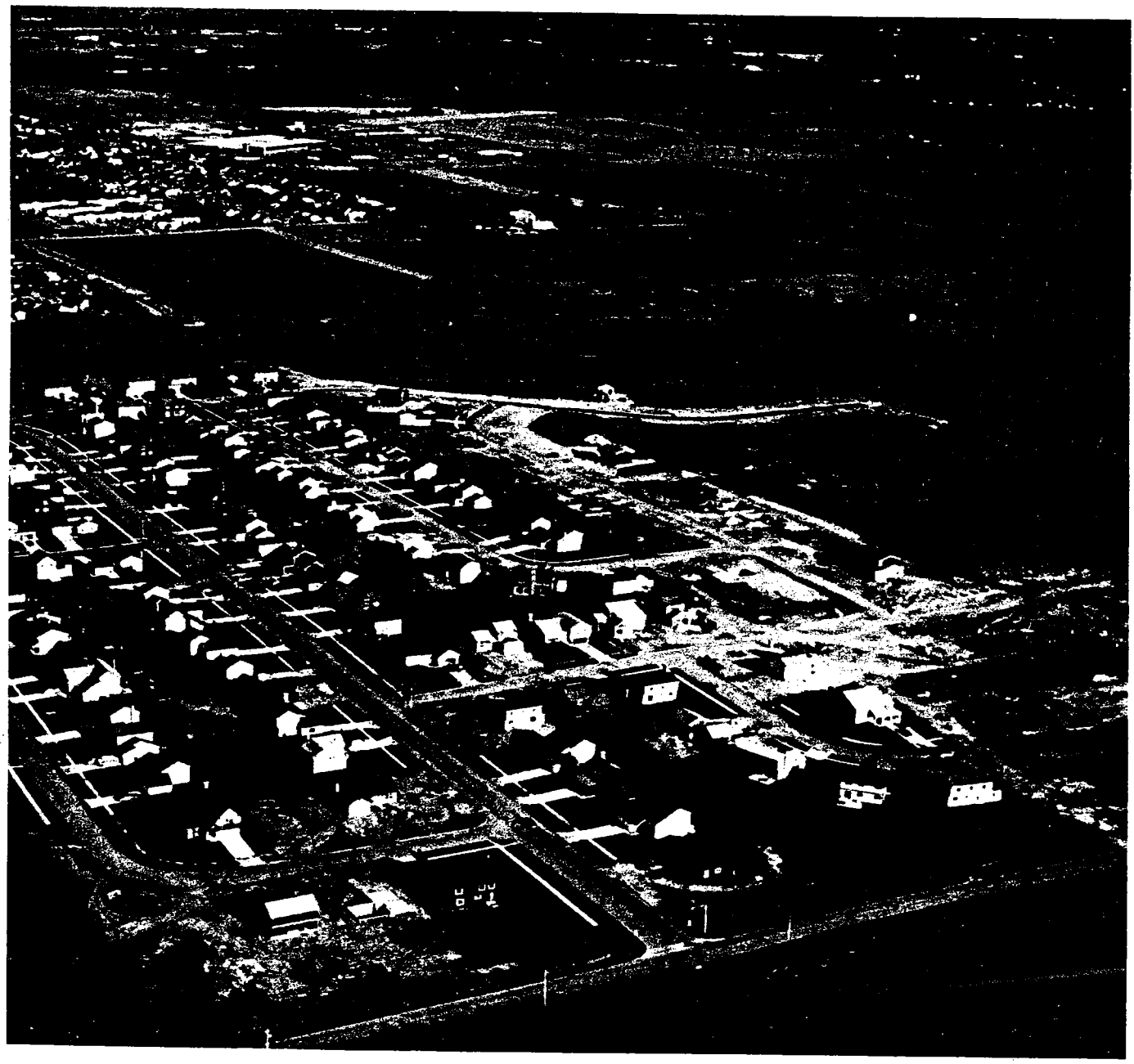
**Soil
Conservation
Service**

**Engineering
Division**

Technical
Release 55

June 1986

# Urban Hydrology for Small Watersheds



# Preface

Technical Release 55 (TR-55) presents simplified procedures to calculate storm runoff volume, peak rate of discharge, hydrographs, and storage volumes required for floodwater reservoirs. These procedures are applicable in small watersheds, especially urbanizing watersheds, in the United States. First issued by the Soil Conservation Service (SCS) in January 1975, TR-55 incorporates current SCS procedures. This revision includes results of recent research and other changes based on experience with use of the original edition.

The major revisions and additions are—

1. A flow chart for selecting the appropriate procedure;
2. Three additional rain distributions;
3. Expansion of the chapter on runoff curve numbers;
4. A procedure for calculating travel times of sheet flow;
5. Deletion of a chapter on peak discharges;
6. Modifications to the Graphical Peak Discharge method and Tabular Hydrograph method;
7. A new storage routing procedure;
8. Features of the TR-55 computer program; and
9. Worksheets.

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**Revised June 1986**

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## Metric conversions

The English system of units is used in this TR. To convert to the International System of units (metric), use the following factors:

| From English unit | To metric unit | Multiply by |
|---|---|---|
| Acre | Hectare | 0.405 |
| Square mile | Square kilometer | 2.59 |
| Cubic feet per second | Cubic meters per second | 0.0283 |
| Inch | Millimeter | 25.4 |
| Feet per second | Meters per second | 0.3048 |
| Acre-foot | Cubic meter | 1233.489 |
| Cubic foot | Cubic meter | 0.0283 |

Perform rounding operations as appropriate to indicate the same level of precision as that of the original measurement. For example:

1. A stream discharge is recorded in cubic feet per second with three significant digits.
2. Convert stream discharge to cubic meters per second by multiplying by 0.0283.
3. Round to enough significant digits so that, when converting back to cubic feet per second, you obtain the original value (step 1) with three significant digits.

## Definitions of symbols

| Symbol | Unit | Definition |
|---|---|---|
| a | $ft^2$ | Cross sectional flow area |
| $A_m$ | $mi^2$ | Drainage area |
| CN | | Runoff curve number |
| $CN_c$ | | Composite runoff curve number |
| $CN_p$ | | Pervious runoff curve number |
| $E_{max}$ | | Maximum stage |
| $F_p$ | | Pond and swamp adjustment factor |
| $H_w$ | ft | Head over weir crest |
| $I_a$ | in | Initial abstraction |
| L | ft | Flow length |
| $L_w$ | ft | Weir crest length |
| m | | Number of flow segments |
| n | | Manning's roughness coefficient |
| P | in | Rainfall |
| $P_{imp}$ | | Percent imperviousness |
| $P_2$ | in | Two-year frequency, 24-hour rainfall |
| $p_w$ | ft | Wetted perimeter |
| q | cfs | Hydrograph coordinate |
| $q_i$ | cfs | Peak inflow discharge |
| $q_o$ | cfs | Peak outflow discharge |
| $q_p$ | cfs | Peak discharge |
| $q_t$ | csm/in | Tabular hydrograph unit discharge |
| $q_u$ | csm/in | Unit peak discharge |
| Q | in | Runoff |
| r | ft | Hydraulic radius |
| R | | Ratio of unconnected impervious area to total impervious area |
| s | ft/ft | Slope of hydraulic grade line |
| S | in | Potential maximum retention after runoff begins |
| t | hr | Hydrograph time |
| $T_c$ | hr | Time of concentration |
| $T_p$ | hr | Time to peak |
| $T_t$ | hr | Travel time |
| V | ft/s | Average velocity |
| $V_r$ | acre-ft, $ft^3$, or water-shed-inch | Runoff volume |
| $V_s$ | acre-ft, $ft^3$, or water-shed-inch | Storage volume |

# Chapter 1: Introduction

The conversion of rural land to urban land usually increases erosion and the discharge and volume of storm runoff in a watershed. It also causes other problems that affect soil and water. As part of programs established to alleviate these problems, engineers increasingly must assess the probable effects of urban development, as well as design and implement measures that will minimize its adverse effects.

Technical Release 55 (TR-55) presents simplified procedures for estimating runoff and peak discharges in small watersheds. In selecting the appropriate procedure, consider the scope and complexity of the problem, the available data, and the acceptable level of error. While this TR gives special emphasis to urban and urbanizing watersheds, the procedures apply to any small watershed in which certain limitations are met.

## Effects of urban development

An urban or urbanizing watershed is one in which impervious surfaces cover or will soon cover a considerable area. Impervious surfaces include roads, sidewalks, parking lots, and buildings. Natural flow paths in the watershed may be replaced or supplemented by paved gutters, storm sewers, or other elements of artificial drainage.

Hydrologic studies to determine runoff and peak discharge should ideally be based on long-term stationary streamflow records for the area. Such records are seldom available for small drainage areas. Even where they are available, accurate statistical analysis of them is usually impossible because of the conversion of land to urban uses during the period of record. It therefore is necessary to estimate peak discharges with hydrologic models based on measurable watershed characteristics. Only through an understanding of these characteristics and experience in using these models can we make sound judgments on how to alter model parameters to reflect changing watershed conditions.

Urbanization changes a watershed's response to precipitation. The most common effects are reduced infiltration and decreased travel time, which significantly increase peak discharges and runoff. Runoff is determined primarily by the amount of precipitation and by infiltration characteristics related to soil type, soil moisture, antecedent rainfall, cover type, impervious surfaces, and surface retention. Travel time is determined primarily by slope, length of flow path, depth of flow, and roughness of flow surfaces. Peak discharges are based on the relationship of these parameters and on the total drainage area of the watershed, the location of the development, the effect of any flood control works or other natural or manmade storage, and the time distribution of rainfall during a given storm event.

The model described in TR-55 begins with a rainfall amount uniformly imposed on the watershed over a specified time distribution. Mass rainfall is converted to mass runoff by using a runoff curve number (CN). CN is based on soils, plant cover, amount of impervious areas, interception, and surface storage. Runoff is then transformed into a hydrograph by using unit hydrograph theory and routing procedures that depend on runoff travel time through segments of the watershed.

For a description of the hydrograph development method used by SCS, see chapter 16 of the SCS National Engineering Handbook, Section 4—Hydrology (NEH-4) (SCS 1985). The routing method (Modified Att-Kin) is explained in appendixes G and H of draft Technical Release 20 (TR-20) (SCS 1983).

## Rainfall

TR-55 includes four regional rainfall time distributions. See appendix B for a discussion of how these distributions were developed.

All four distributions are for a 24-hour period. This period was chosen because of the general availability of daily rainfall data that were used to estimate 24-hour rainfall amounts. The 24-hour duration spans most of the applications of TR-55.

One critical parameter in the model is time of concentration ($T_c$), which is the time it takes for runoff to travel to a point of interest from the hydraulically most distant point. Normally a rainfall duration equal to or greater than $T_c$ is used. Therefore, the rainfall distributions were designed to contain the intensity of any duration of rainfall for the frequency of the event chosen. That is, if the 10-year frequency, 24-hour rainfall is used, the most intense hour will approximate the 10-year, 1-hour rainfall volume.

## Runoff

To estimate runoff from storm rainfall, SCS uses the Runoff Curve Number (CN) method (see chapters 4 through 10 of NEH-4, SCS 1985). Determination of CN depends on the watershed's soil and cover conditions, which the model represents as hydrologic soil group, cover type, treatment, and hydrologic condition. Chapter 2 of this TR discusses the effect of urban development on CN and explains how to use CN to estimate runoff.

## Time parameters

Chapter 3 describes a method for estimating the parameters used to distribute the runoff into a hydrograph. The method is based on velocities of flow through segments of the watershed. Two major parameters are time of concentration ($T_c$) and travel time of flow through the segments ($T_t$). These and the other parameters used are the same as those used in accepted hydraulic analyses of open channels.

Many methods are empirically derived from actual runoff hydrographs and watershed characteristics. The method in chapter 3 was chosen because it is basic; however, other methods may be used.

## Peak discharge and hydrographs

Chapter 4 describes a method for approximating peak rates of discharge, and chapter 5 describes a method for obtaining or routing hydrographs. Both methods were derived from hydrographs prepared by procedures outlined in chapter 16 of NEH-4 (SCS 1985). The computations were made with a computerized SCS hydrologic model, TR-20 (SCS 1983).

The methods in chapters 4 and 5 should be used in accordance with specific guidelines. If basic data are improperly prepared or adjustments not properly used, errors will result.

## Storage effects

Chapter 6 outlines procedures to account for the effect of detention-type storage. It provides a shortcut method to estimate temporary flood storage based on hydrologic data developed from the Graphical Peak Discharge or Tabular Hydrograph methods.

By increasing runoff and decreasing travel times, urbanization can be expected to increase downstream peak discharges. Chapter 6 discusses how flood detention can modify the hydrograph so that, ideally, downstream peak discharge is reduced approximately to the predevelopment condition. The shortcuts in chapter 6 are useful in sizing a basin even though the final design may require a more detailed analysis.

## Selecting the appropriate procedures

Figure 1-1 is a flow chart that shows how to select the appropriate procedures to use in TR-55. In the figure, the diamond-shaped box labeled "Subareas required?" directs the user to the appropriate method based on whether the watershed needs to be divided into subareas. Watershed subdivision is required when significantly different conditions affecting runoff or timing are present in the watershed—for example, if the watershed has widely differing curve numbers or nonhomogeneous slope patterns.

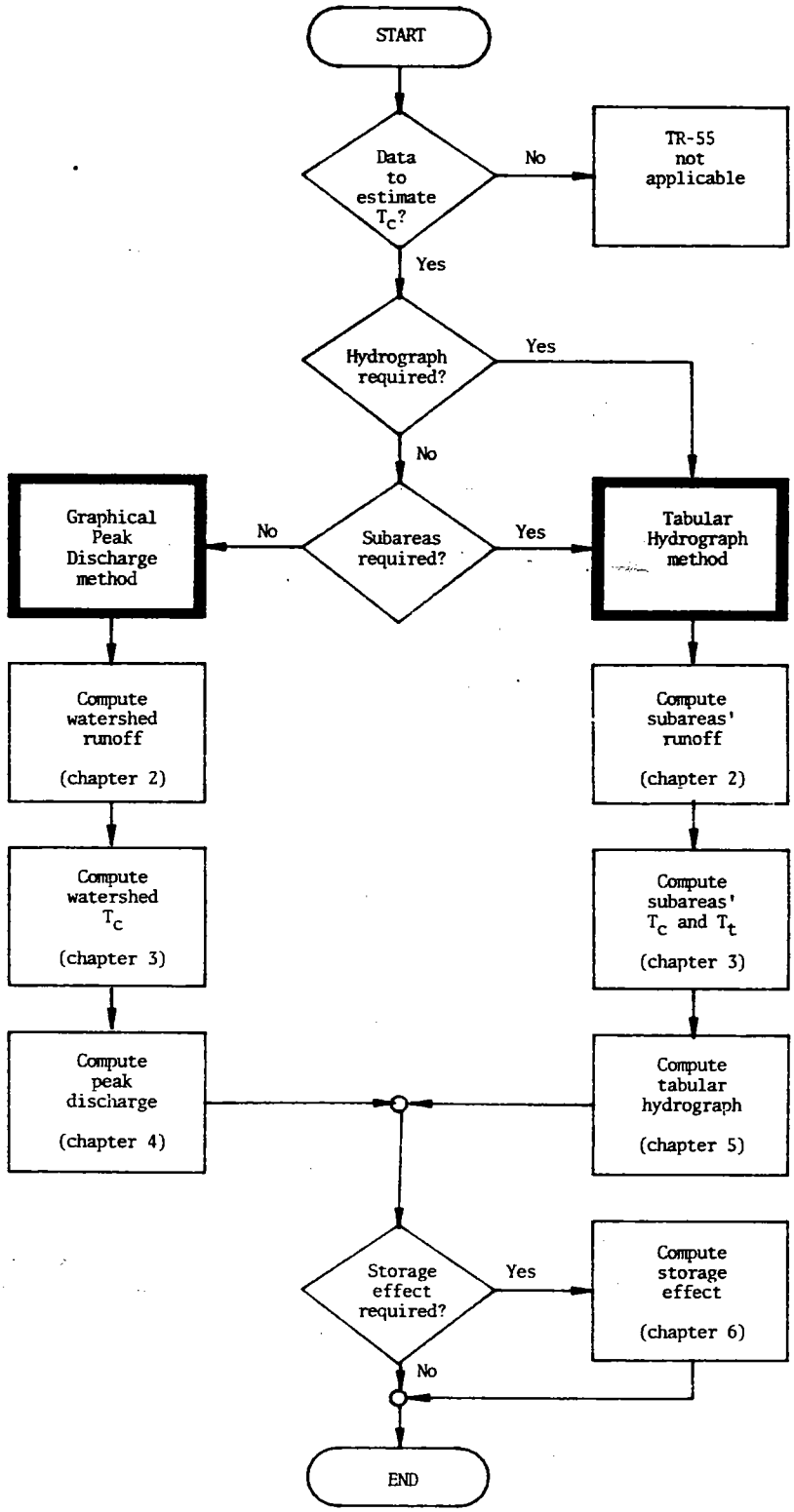



**Figure 1-1.—Flow chart for selecting the appropriate procedures in TR-55.**

## Limitations

To save time, the procedures in TR-55 are simplified by assumptions about some parameters. These simplifications, however, limit the use of the procedures and can provide results that are less accurate than more detailed methods. The user should examine the sensitivity of the analysis being conducted to a variation of the peak discharge or hydrograph. To ensure that the degree of error is tolerable, specific limitations are given in chapters 2 through 6. Additional general constraints to the use of TR-55 are as follows:

- The methods in this TR are based on open and unconfined flow over land or in channels. For large events during which flow is divided between sewer and overland flow, more information about hydraulics than is presented here is needed to determine $T_c$. After flow enters a closed system, the discharge can be assumed constant until another flow is encountered at a junction or another inlet.

- Both the Graphical Peak Discharge and Tabular Hydrograph methods are derived from TR-20 (SCS 1983) output. Their accuracy is comparable; they differ only in their products. The use of $T_c$ permits them to be used for any size watershed within the scope of the curves or tables. The Graphical method (chapter 4) is used only for hydrologically homogeneous watersheds because the procedure is limited to a single watershed subarea. The Tabular method (chapter 5) can be used for a heterogeneous watershed that is divided into a number of homogeneous subwatersheds. Hydrographs for the subwatersheds can be routed and added.

- The approximate storage-routing curves (chapter 6) should not be used if the adjustment for ponding (chapter 4) is used. These storage-routing curves, like the peak discharge and hydrograph procedures, are generalizations derived from TR-20 routings.

# Chapter 2: Estimating runoff

## SCS Runoff Curve Number method

The SCS Runoff Curve Number (CN) method is described in detail in NEH-4 (SCS 1985). The SCS runoff equation is

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{[Eq. 2-1]}$$

where

$Q$ = runoff (in),
$P$ = rainfall (in),
$S$ = potential maximum retention after runoff begins (in), and
$I_a$ = initial abstraction (in).

Initial abstraction ($I_a$) is all losses before runoff begins. It includes water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration. $I_a$ is highly variable but generally is correlated with soil and cover parameters. Through studies of many small agricultural watersheds, $I_a$ was found to be approximated by the following empirical equation:

$$I_a = 0.2S. \qquad \text{[Eq. 2-2]}$$

By removing $I_a$ as an independent parameter, this approximation allows use of a combination of S and P to produce a unique runoff amount. Substituting equation 2-2 into equation 2-1 gives

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}. \qquad \text{[Eq. 2-3]}$$

S is related to the soil and cover conditions of the watershed through the CN. CN has a range of 0 to 100, and S is related to CN by

$$S = \frac{1000}{CN} - 10. \qquad \text{[Eq. 2-4]}$$

Figure 2-1 and table 2-1 solve equations 2-3 and 2-4 for a range of CN's and rainfall.

## Factors considered in determining runoff curve numbers

The major factors that determine CN are the hydrologic soil group (HSG), cover type, treatment, hydrologic condition, and antecedent runoff condition (ARC). Another factor considered is whether impervious areas outlet directly to the drainage system (connected) or whether the flow spreads over pervious areas before entering the drainage system (unconnected). Figure 2-2 is provided to aid in selecting the appropriate figure or table for determining curve numbers.

CN's in table 2-2 (*a* to *d*) represent average antecedent runoff condition for urban, cultivated agricultural, other agricultural, and arid and semiarid rangeland uses. Table 2-2 assumes impervious areas are directly connected. The following sections explain how to determine CN's and how to modify them for urban conditions.

### Hydrologic soil groups

Infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates. Soils are classified into four HSG's (A, B, C, and D) according to their minimum infiltration rate, which is obtained for bare soil after prolonged wetting. Appendix A defines the four groups and provides a list of most of the soils in the United States and their group classification. The soils in the area of interest may be identified from a soil survey report, which can be obtained from local SCS offices or soil and water conservation district offices.

Most urban areas are only partially covered by impervious surfaces: the soil remains an important factor in runoff estimates. Urbanization has a greater effect on runoff in watersheds with soils having high infiltration rates (sands and gravels) than in watersheds predominantly of silts and clays, which generally have low infiltration rates.

Any disturbance of a soil profile can significantly change its infiltration characteristics. With urbanization, native soil profiles may be mixed or removed or fill material from other areas may be introduced. Therefore, a method based on soil

## Solution for runoff equation

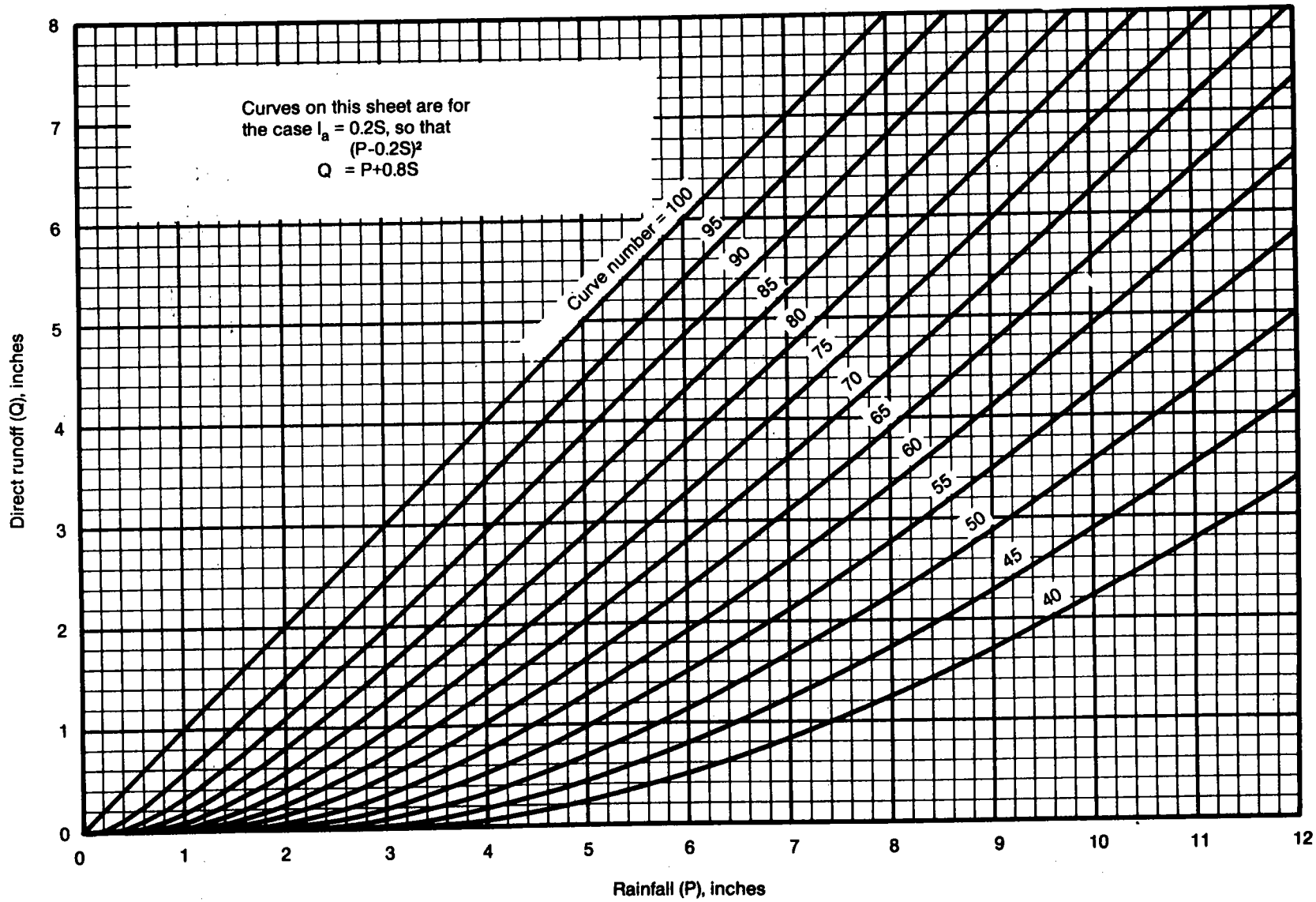



Figure 2-1.—Solution of runoff equation.

texture is given in appendix A for determining the HSG classification for disturbed soils.

### Cover type

Table 2-2 addresses most cover types, such as vegetation, bare soil, and impervious surfaces. There are a number of methods for determining cover type. The most common are field reconnaissance, aerial photographs, and land use maps.

### Treatment

*Treatment* is a cover type modifier (used only in table 2-2b) to describe the management of cultivated agricultural lands. It includes mechanical practices, such as contouring and terracing, and management practices, such as crop rotations and reduced or no tillage.

### Hydrologic condition

*Hydrologic condition* indicates the effects of cover type and treatment on infiltration and runoff and is generally estimated from density of plant and residue cover on sample areas. *Good* hydrologic condition indicates that the soil usually has a low runoff potential for that specific hydrologic soil group, cover type, and treatment. Some factors to consider in estimating the effect of cover on infiltration and runoff are (a) canopy or density of lawns, crops, or other vegetative areas; (b) amount of year-round cover; (c) amount of grass or close-seeded legumes in rotations; (d) percent of residue cover; and (e) degree of surface roughness.

**Table 2-1.—Runoff depth for selected CN's and rainfall amounts[1]**

| Rainfall | Runoff depth for curve number of— | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 40 | 45 | 50 | 55 | 60 | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 98 |
| | *inches* | | | | | | | | | | | | |
| 1.0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.03 | 0.08 | 0.17 | 0.32 | 0.56 | 0.79 |
| 1.2 | .00 | .00 | .00 | .00 | .00 | .00 | .03 | .07 | .15 | .27 | .46 | .74 | .99 |
| 1.4 | .00 | .00 | .00 | .00 | .00 | .02 | .06 | .13 | .24 | .39 | .61 | .92 | 1.18 |
| 1.6 | .00 | .00 | .00 | .00 | .01 | .05 | .11 | .20 | .34 | .52 | .76 | 1.11 | 1.38 |
| 1.8 | .00 | .00 | .00 | .00 | .03 | .09 | .17 | .29 | .44 | .65 | .93 | 1.29 | 1.58 |
| 2.0 | .00 | .00 | .00 | .02 | .06 | .14 | .24 | .38 | .56 | .80 | 1.09 | 1.48 | 1.77 |
| 2.5 | .00 | .00 | .02 | .08 | .17 | .30 | .46 | .65 | .89 | 1.18 | 1.53 | 1.96 | 2.27 |
| 3.0 | .00 | .02 | .09 | .19 | .33 | .51 | .71 | .96 | 1.25 | 1.59 | 1.98 | 2.45 | 2.77 |
| 3.5 | .02 | .08 | .20 | .35 | .53 | .75 | 1.01 | 1.30 | 1.64 | 2.02 | 2.45 | 2.94 | 3.27 |
| 4.0 | .06 | .18 | .33 | .53 | .76 | 1.03 | 1.33 | 1.67 | 2.04 | 2.46 | 2.92 | 3.43 | 3.77 |
| 4.5 | .14 | .30 | .50 | .74 | 1.02 | 1.33 | 1.67 | 2.05 | 2.46 | 2.91 | 3.40 | 3.92 | 4.26 |
| 5.0 | .24 | .44 | .69 | .98 | 1.30 | 1.65 | 2.04 | 2.45 | 2.89 | 3.37 | 3.88 | 4.42 | 4.76 |
| 6.0 | .50 | .80 | 1.14 | 1.52 | 1.92 | 2.35 | 2.81 | 3.28 | 3.78 | 4.30 | 4.85 | 5.41 | 5.76 |
| 7.0 | .84 | 1.24 | 1.68 | 2.12 | 2.60 | 3.10 | 3.62 | 4.15 | 4.69 | 5.25 | 5.82 | 6.41 | 6.76 |
| 8.0 | 1.25 | 1.74 | 2.25 | 2.78 | 3.33 | 3.89 | 4.46 | 5.04 | 5.63 | 6.21 | 6.81 | 7.40 | 7.76 |
| 9.0 | 1.71 | 2.29 | 2.88 | 3.49 | 4.10 | 4.72 | 5.33 | 5.95 | 6.57 | 7.18 | 7.79 | 8.40 | 8.76 |
| 10.0 | 2.23 | 2.89 | 3.56 | 4.23 | 4.90 | 5.56 | 6.22 | 6.88 | 7.52 | 8.16 | 8.78 | 9.40 | 9.76 |
| 11.0 | 2.78 | 3.52 | 4.26 | 5.00 | 5.72 | 6.43 | 7.13 | 7.81 | 8.48 | 9.13 | 9.77 | 10.39 | 10.76 |
| 12.0 | 3.38 | 4.19 | 5.00 | 5.79 | 6.56 | 7.32 | 8.05 | 8.76 | 9.45 | 10.11 | 10.76 | 11.39 | 11.76 |
| 13.0 | 4.00 | 4.89 | 5.76 | 6.61 | 7.42 | 8.21 | 8.98 | 9.71 | 10.42 | 11.10 | 11.76 | 12.39 | 12.76 |
| 14.0 | 4.65 | 5.62 | 6.55 | 7.44 | 8.30 | 9.12 | 9.91 | 10.67 | 11.39 | 12.08 | 12.75 | 13.39 | 13.76 |
| 15.0 | 5.33 | 6.36 | 7.35 | 8.29 | 9.19 | 10.04 | 10.85 | 11.63 | 12.37 | 13.07 | 13.74 | 14.39 | 14.76 |

[1]Interpolate the values shown to obtain runoff depths for CN's or rainfall amounts not shown.

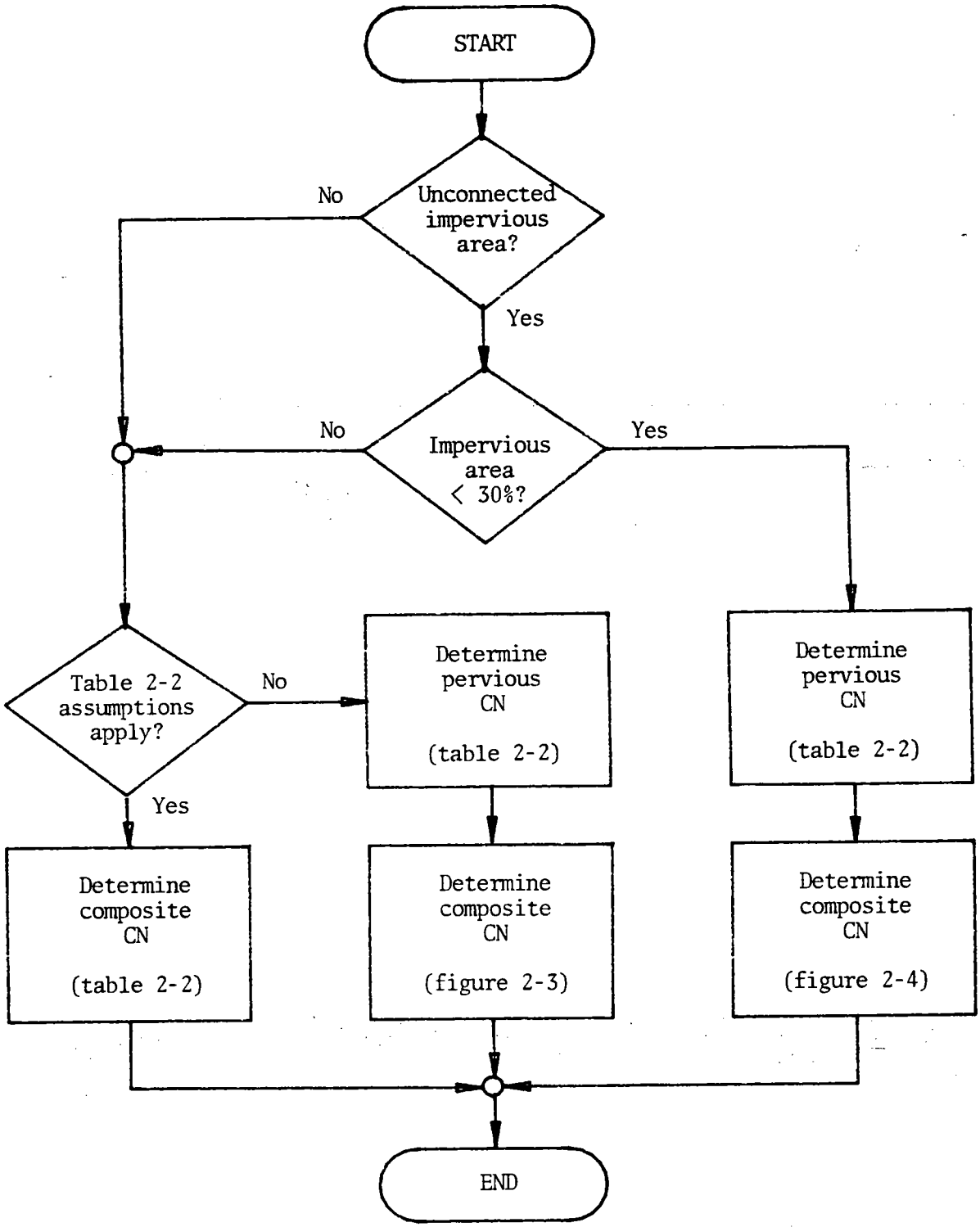



**Figure 2-2.—Flow chart for selecting the appropriate figure or table for determining runoff curve numbers.**

**Table 2-2a.—Runoff curve numbers for urban areas[1]**

| Cover description | | Curve numbers for hydrologic soil group— | | | |
|---|---|---|---|---|---|
| Cover type and hydrologic condition | Average percent impervious area[2] | A | B | C | D |
| *Fully developed urban areas (vegetation established)* | | | | | |
| Open space (lawns, parks, golf courses, cemeteries, etc.)[3]: | | | | | |
| Poor condition (grass cover < 50%) | | 68 | 79 | 86 | 89 |
| Fair condition (grass cover 50% to 75%) | | 49 | 69 | 79 | 84 |
| Good condition (grass cover > 75%) | | 39 | 61 | 74 | 80 |
| Impervious areas: | | | | | |
| Paved parking lots, roofs, driveways, etc. (excluding right-of-way) | | 98 | 98 | 98 | 98 |
| Streets and roads: | | | | | |
| Paved; curbs and storm sewers (excluding right-of-way) | | 98 | 98 | 98 | 98 |
| Paved; open ditches (including right-of-way) | | 83 | 89 | 92 | 93 |
| Gravel (including right-of-way) | | 76 | 85 | 89 | 91 |
| Dirt (including right-of-way) | | 72 | 82 | 87 | 89 |
| Western desert urban areas: | | | | | |
| Natural desert landscaping (pervious areas only)[4] | | 63 | 77 | 85 | 88 |
| Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders) | | 96 | 96 | 96 | 96 |
| Urban districts: | | | | | |
| Commercial and business | 85 | 89 | 92 | 94 | 95 |
| Industrial | 72 | 81 | 88 | 91 | 93 |
| Residential districts by average lot size: | | | | | |
| 1/8 acre or less (town houses) | 65 | 77 | 85 | 90 | 92 |
| 1/4 acre | 38 | 61 | 75 | 83 | 87 |
| 1/3 acre | 30 | 57 | 72 | 81 | 86 |
| 1/2 acre | 25 | 54 | 70 | 80 | 85 |
| 1 acre | 20 | 51 | 68 | 79 | 84 |
| 2 acres | 12 | 46 | 65 | 77 | 82 |
| *Developing urban areas* | | | | | |
| Newly graded areas (pervious areas only, no vegetation)[5] | | 77 | 86 | 91 | 94 |
| Idle lands (CN's are determined using cover types similar to those in table 2-2c). | | | | | |

[1]Average runoff condition, and $I_a = 0.2S$.
[2]The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
[3]CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
[4]Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
[5]Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

**Table 2-2b.—Runoff curve numbers for cultivated agricultural lands[1]**

| Cover description | | | Curve numbers for hydrologic soil group— | | | |
|---|---|---|---|---|---|---|
| Cover type | Treatment[2] | Hydrologic condition[3] | A | B | C | D |
| Fallow | Bare soil | — | 77 | 86 | 91 | 94 |
| | Crop residue cover (CR) | Poor | 76 | 85 | 90 | 93 |
| | | Good | 74 | 83 | 88 | 90 |
| Row crops | Straight row (SR) | Poor | 72 | 81 | 88 | 91 |
| | | Good | 67 | 78 | 85 | 89 |
| | SR + CR | Poor | 71 | 80 | 87 | 90 |
| | | Good | 64 | 75 | 82 | 85 |
| | Contoured (C) | Poor | 70 | 79 | 84 | 88 |
| | | Good | 65 | 75 | 82 | 86 |
| | C + CR | Poor | 69 | 78 | 83 | 87 |
| | | Good | 64 | 74 | 81 | 85 |
| | Contoured & terraced (C&T) | Poor | 66 | 74 | 80 | 82 |
| | | Good | 62 | 71 | 78 | 81 |
| | C&T + CR | Poor | 65 | 73 | 79 | 81 |
| | | Good | 61 | 70 | 77 | 80 |
| Small grain | SR | Poor | 65 | 76 | 84 | 88 |
| | | Good | 63 | 75 | 83 | 87 |
| | SR + CR | Poor | 64 | 75 | 83 | 86 |
| | | Good | 60 | 72 | 80 | 84 |
| | C | Poor | 63 | 74 | 82 | 85 |
| | | Good | 61 | 73 | 81 | 84 |
| | C + CR | Poor | 62 | 73 | 81 | 84 |
| | | Good | 60 | 72 | 80 | 83 |
| | C&T | Poor | 61 | 72 | 79 | 82 |
| | | Good | 59 | 70 | 78 | 81 |
| | C&T + CR | Poor | 60 | 71 | 78 | 81 |
| | | Good | 58 | 69 | 77 | 80 |
| Close-seeded or broadcast legumes or rotation meadow | SR | Poor | 66 | 77 | 85 | 89 |
| | | Good | 58 | 72 | 81 | 85 |
| | C | Poor | 64 | 75 | 83 | 85 |
| | | Good | 55 | 69 | 78 | 83 |
| | C&T | Poor | 63 | 73 | 80 | 83 |
| | | Good | 51 | 67 | 76 | 80 |

[1]Average runoff condition, and $I_a = 0.2S$.

[2]*Crop residue cover* applies only if residue is on at least 5% of the surface throughout the year.

[3]Hydrologic condition is based on combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes in rotations, (d) percent of residue cover on the land surface (good $\geq$ 20%), and (e) degree of surface roughness.

*Poor*: Factors impair infiltration and tend to increase runoff.

*Good*: Factors encourage average and better than average infiltration and tend to decrease runoff.

**Table 2-2c.—Runoff curve numbers for other agricultural lands[1]**

| Cover description | | Curve numbers for hydrologic soil group— | | | |
|---|---|---|---|---|---|
| Cover type | Hydrologic condition | A | B | C | D |
| Pasture, grassland, or range—continuous forage for grazing.[2] | Poor | 68 | 79 | 86 | 89 |
| | Fair | 49 | 69 | 79 | 84 |
| | Good | 39 | 61 | 74 | 80 |
| Meadow—continuous grass, protected from grazing and generally mowed for hay. | — | 30 | 58 | 71 | 78 |
| Brush—brush-weed-grass mixture with brush the major element.[3] | Poor | 48 | 67 | 77 | 83 |
| | Fair | 35 | 56 | 70 | 77 |
| | Good | [4]30 | 48 | 65 | 73 |
| Woods—grass combination (orchard or tree farm).[5] | Poor | 57 | 73 | 82 | 86 |
| | Fair | 43 | 65 | 76 | 82 |
| | Good | 32 | 58 | 72 | 79 |
| Woods.[6] | Poor | 45 | 66 | 77 | 83 |
| | Fair | 36 | 60 | 73 | 79 |
| | Good | [4]30 | 55 | 70 | 77 |
| Farmsteads—buildings, lanes, driveways, and surrounding lots. | — | 59 | 74 | 82 | 86 |

[1]Average runoff condition, and $I_a = 0.2S$.

[2]*Poor:* <50% ground cover or heavily grazed with no mulch.
*Fair:* 50 to 75% ground cover and not heavily grazed.
*Good:* >75% ground cover and lightly or only occasionally grazed.

[3]*Poor:* <50% ground cover.
*Fair:* 50 to 75% ground cover.
*Good:* >75% ground cover.

[4]Actual curve number is less than 30; use CN = 30 for runoff computations.

[5]CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

[6]*Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
*Fair:* Woods are grazed but not burned, and some forest litter covers the soil.
*Good:* Woods are protected from grazing, and litter and brush adequately cover the soil.

**Table 2-2d.—Runoff curve numbers for arid and semiarid rangelands[1]**

| Cover description | | Curve numbers for hydrologic soil group— | | | |
|---|---|---|---|---|---|
| Cover type | Hydrologic condition[2] | A[3] | B | C | D |
| Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element. | Poor | | 80 | 87 | 93 |
| | Fair | | 71 | 81 | 89 |
| | Good | | 62 | 74 | 85 |
| Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush. | Poor | | 66 | 74 | 79 |
| | Fair | | 48 | 57 | 63 |
| | Good | | 30 | 41 | 48 |
| Pinyon-juniper—pinyon, juniper, or both; grass understory. | Poor | | 75 | 85 | 89 |
| | Fair | | 58 | 73 | 80 |
| | Good | | 41 | 61 | 71 |
| Sagebrush with grass understory. | Poor | | 67 | 80 | 85 |
| | Fair | | 51 | 63 | 70 |
| | Good | | 35 | 47 | 55 |
| Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus. | Poor | 63 | 77 | 85 | 88 |
| | Fair | 55 | 72 | 81 | 86 |
| | Good | 49 | 68 | 79 | 84 |

[1]Average runoff condition, and $I_a = 0.2S$. For range in humid regions, use table 2-2c.

[2]*Poor:* <30% ground cover (litter, grass, and brush overstory).
*Fair:* 30 to 70% ground cover.
*Good:* >70% ground cover.

[3]Curve numbers for group A have been developed only for desert shrub.

### Antecedent runoff condition

The index of runoff potential before a storm event is the antecedent runoff condition (ARC). ARC is an attempt to account for the variation in CN at a site from storm to storm. CN for the average ARC at a site is the median value as taken from sample rainfall and runoff data. The CN's in table 2-2 are for the average ARC, which is used primarily for design applications. See NEH-4 (SCS 1985) and Rallison and Miller (1981) for more detailed discussion of storm-to-storm variation and a demonstration of upper and lower enveloping curves.

### Urban impervious area modifications

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for urban areas (Rawls et al., 1981). For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

#### Connected impervious areas

An impervious area is considered connected if runoff from it flows directly into the drainage system. It is also considered connected if runoff from it occurs as concentrated shallow flow that runs over a pervious area and then into a drainage system.

Urban CN's (table 2-2a) were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that (a) pervious urban areas are equivalent to pasture in good hydrologic condition and (b) impervious areas have a CN of 98 and are directly connected to the drainage system. Some assumed percentages of impervious area are shown in table 2-2a.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in table 2-2a are not applicable, use figure 2-3 to compute a composite CN. For example, table 2-2a gives a CN of 70 for a ½-acre lot in HSG B, with an assumed impervious area of 25 percent. However, if the lot has 20 percent impervious area and a pervious area CN of 61, the composite CN obtained from figure 2-3 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

#### Unconnected impervious areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use figure 2-4 if total impervious area is less than 30 percent or (2) use figure 2-3 if the total impervious area is equal to or greater than 30 percent, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30 percent, obtain the composite CN by entering the right half of figure 2-4 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a ½-acre lot with 20 percent total impervious area (75 percent of which is unconnected) and pervious CN of 61, the composite CN from figure 2-4 is 66. If all of the impervious area is connected, the resulting CN (from figure 2-3) would be 68.

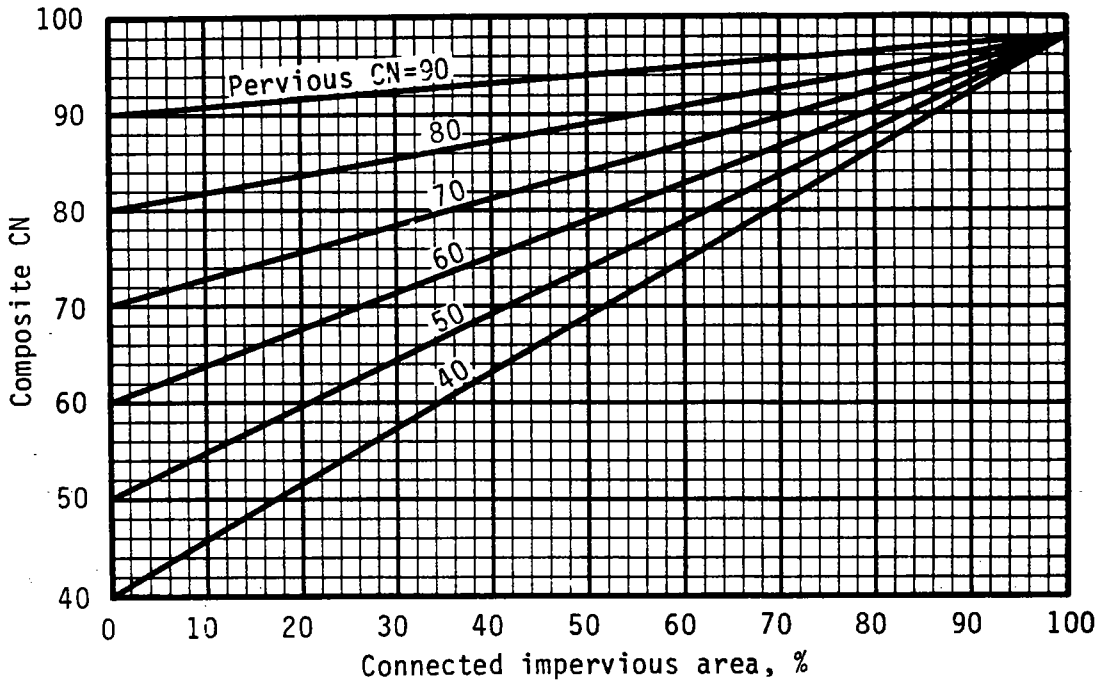



**Figure 2-3.—Composite CN with connected impervious area.**

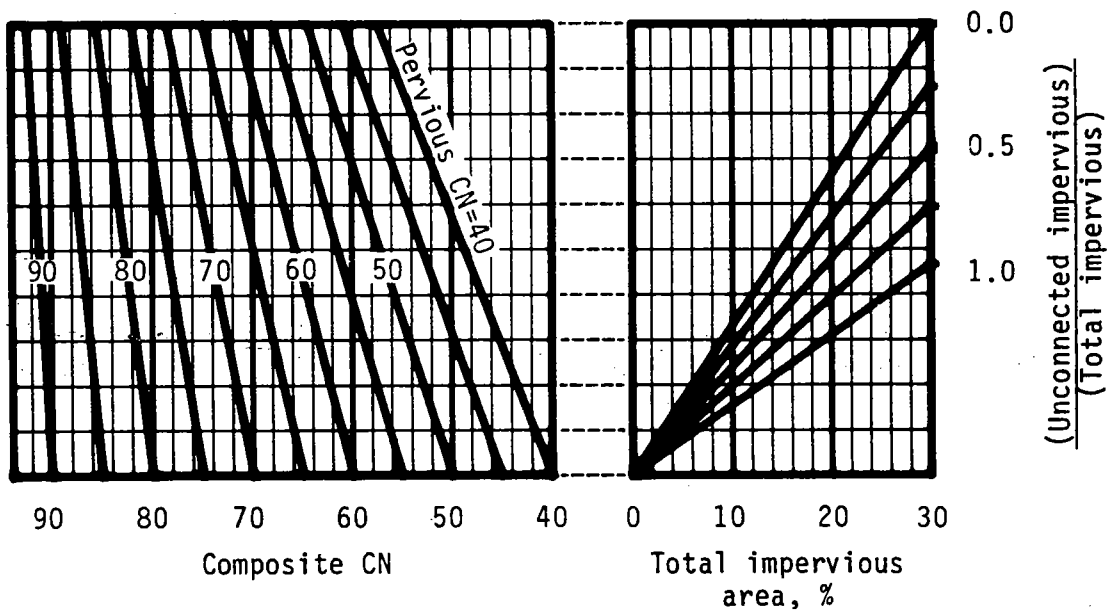



**Figure 2-4.—Composite CN with unconnected impervious areas and total impervious area less than 30%.**

## Runoff

When CN and the amount of rainfall have been determined for the watershed, determine runoff by using figure 2-1, table 2-1, or equations 2-3 and 2-4. The runoff is usually rounded to the nearest hundredth of an inch.

## Limitations

- Curve numbers describe average conditions that are useful for design purposes. If the rainfall event used is a historical storm, the modeling accuracy decreases.

- Use the runoff curve number equation with caution when recreating specific features of an actual storm. The equation does not contain an expression for time and, therefore, does not account for rainfall duration or intensity.

- The user should understand the assumption reflected in the initial abstraction term ($I_a$) and should ascertain that the assumption applies to the situation. $I_a$, which consists of interception, initial infiltration, surface depression storage, evapotranspiration, and other factors, was generalized as 0.2S based on data from agricultural watersheds (S is the potential maximum retention after runoff begins). This approximation can be especially important in an urban application because the combination of impervious areas with pervious areas can imply a significant initial loss that may not take place. The opposite effect, a greater initial loss, can occur if the impervious areas have surface depressions that store some runoff. To use a relationship other than $I_a = 0.2S$, one must redevelop equation 2-3, figure 2-1, table 2-1, and table 2-2 by using the original rainfall-runoff data to establish new S or CN relationships for each cover and hydrologic soil group.

- Runoff from snowmelt or rain on frozen ground cannot be estimated using these procedures.

- The CN procedure is less accurate when runoff is less than 0.5 inch. As a check, use another procedure to determine runoff.

- The SCS runoff procedures apply only to direct surface runoff: do not overlook large sources of subsurface flow or high ground water levels that contribute to runoff. These conditions are often related to HSG A soils and forest areas that have been assigned relatively low CN's in table 2-2. Good judgment and experience based on stream gage records are needed to adjust CN's as conditions warrant.

- When the weighted CN is less than 40, use another procedure to determine runoff.

## Examples

Four examples illustrate the procedure for computing runoff curve number (CN) and runoff (Q) in inches. Worksheet 2 in appendix D is provided to assist TR-55 users. Figures 2-5 to 2-8 represent the use of worksheet 2 for each example. All four examples are based on the same watershed and the same storm event.

The watershed covers 250 acres in Dyer County, northwestern Tennessee. Seventy percent (175 acres) is a Loring soil, which is in hydrologic soil group C. Thirty percent (75 acres) is a Memphis soil, which is in group B. The event is a 25-year frequency, 24-hour storm with total rainfall of 6 inches.

Cover type and conditions in the watershed are different for each example. The examples, therefore, illustrate how to compute CN and Q for various situations of proposed, planned, or present development.

### Example 2-1

The present cover type is pasture in good hydrologic condition. (See figure 2-5 for worksheet 2 information.)

**Example 2-2**

Seventy percent (175 acres) of the watershed, consisting of all the Memphis soil and 100 acres of the Loring soil, is ½-acre residential lots with lawns in good hydrologic condition. The rest of the watershed is scattered open space in good hydrologic condition. (See figure 2-6.)

**Example 2-3**

This example is the same as example 2-2, except that the ½-acre lots have a total impervious area of 35 percent. For these lots, the pervious area is lawns in good hydrologic condition. Since the impervious area percentage differs from the percentage assumed in table 2-2, use figure 2-3 to compute CN. (See figure 2-7.)

**Example 2-4**

This example is also based on example 2-2, except that 50 percent of the impervious area associated with the ½-acre lots on the Loring soil is "unconnected," that is, it is not directly connected to the drainage system. For these lots, the pervious area CN (lawn, good condition) is 74 and the impervious area is 25 percent. Use figure 2-4 to compute the CN for these lots. CN's for the ½-acre lots on Memphis soil and the open space on Loring soil are the same as those in example 2-2. (See figure 2-8.)

## Worksheet 2: Runoff curve number and runoff

Project Heavenly Acres By WJR Date 10/1/85

Location Dyer County, Tennessee Checked WM Date 10/3/85

Circle one: (Present) Developed

1. Runoff curve number (CN)

| Soil name and hydrologic group (appendix A) | Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio) | CN 1/ Table 2-2 | CN 1/ Fig. 2-3 | CN 1/ Fig. 2-4 | Area ☐acres ☐mi² ☒% | Product of CN x area |
|---|---|---|---|---|---|---|
| Memphis, B | Pasture, good condition | 61 | | | 30 | 1830 |
| Loring, C | Pasture, good condition | 74 | | | 70 | 5180 |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | Totals = | 100 | 7010 |

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{7010}{100} = 70.1$; Use CN = 70

2. Runoff

| | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|
| Frequency ..... yr | 25 | | |
| Rainfall, P (24-hour) ..... in | 6.0 | | |
| Runoff, Q ..... in (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.) | 2.81 | | |

Figure 2-5.—Worksheet 2 for example 2-1.

## Worksheet 2: Runoff curve number and runoff

Project Heavenly Acres By WJR Date 10/1/85

Location Dyer County, Tennessee Checked WJR Date 10/3/85

Circle one: Present (Developed) 175 acres residential

1. Runoff curve number (CN)

| Soil name and hydrologic group (appendix A) | Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio) | CN 1/ Table 2-2 | CN 1/ Fig. 2-3 | CN 1/ Fig. 2-4 | Area ☒ acres ☐ $mi^2$ ☐ % | Product of CN x area |
|---|---|---|---|---|---|---|
| Memphis, B | 25% impervious ½ acre lots, good condition | 70 | | | 75 | 5250 |
| Loring, C | 25% impervious ½ acre lots, good condition | 80 | | | 100 | 8000 |
| Loring, C | Open space, good condition | 74 | | | 75 | 5550 |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | Totals = | 250 | 18,800 |

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{18,800}{250} = 75.2$; Use CN = 75

2. Runoff

| | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|
| Frequency ..... yr | 25 | | |
| Rainfall, P (24-hour) ..... in | 6.0 | | |
| Runoff, Q ..... in (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.) | 3.28 | | |

Figure 2-6.—Worksheet 2 for example 2-2.

## Worksheet 2: Runoff curve number and runoff

Project Heavenly Acres By WJR Date 10/1/85

Location Dyer County, Tennessee Checked JM Date 10/3/85

Circle one: Present (Developed)

1. Runoff curve number (CN)

| Soil name and hydrologic group (appendix A) | Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio) | CN 1/ Table 2-2 | CN 1/ Fig. 2-3 | CN 1/ Fig. 2-4 | Area ☒ acres ☐ $mi^2$ ☐ % | Product of CN x area |
|---|---|---|---|---|---|---|
| Memphis, B | 35% impervious ½ acre lots, good condition | | 74 | | 75 | 5550 |
| Loring, C | 35% impervious ½ acre lots, good condition | | 82 | | 1000 | 8200 |
| Loring, C | Open space, good condition | 74 | | | 75 | 5550 |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| 1/ Use only one CN source per line. | | | | Totals = | 250 | 19,300 |

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{19,300}{250} = 77.2$; Use CN = 77

2. Runoff

| | | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|---|
| Frequency | yr | 25 | | |
| Rainfall, P (24-hour) | in | 6.0 | | |
| Runoff, Q (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.) | in | 3.48 | | |

Figure 2-7.—Worksheet 2 for example 2-3.

## Worksheet 2: Runoff curve number and runoff

Project Heavenly Acres By WJR Date 10/1/85

Location Dyer County, Tennessee Checked WJ Date 10/3/85

Circle one: Present (Developed)

1. Runoff curve number (CN)

| Soil name and hydrologic group (appendix A) | Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio) | CN 1/ Table 2-2 | CN 1/ Fig. 2-3 | CN 1/ Fig. 2-4 | Area ☒ acres ☐ $mi^2$ ☐ % | Product of CN x area |
|---|---|---|---|---|---|---|
| Memphis, B | 25% connected impervious 1/2 acre lots, good condition | 70 | | | 75 | 5250 |
| Loring, C | 25% impervious with 50% of unconnected 1/2 acre lots, good condition | | | 78 | 100 | 7800 |
| Loring, C | Open space, good condition | 74 | | | 75 | 5550 |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| 1/ Use only one CN source per line. | | | | Totals = | 250 | 18,600 |

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{18,600}{250} = 74.4$; Use CN = 74

2. Runoff

| | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|
| Frequency ..... yr | 25 | | |
| Rainfall, P (24-hour) ..... in | 6.0 | | |
| Runoff, Q ..... in (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.) | 3.19 | | |

Figure 2-8.—Worksheet 2 for example 2-4.

# Chapter 3: Time of concentration and travel time

Travel time ($T_t$) is the time it takes water to travel from one location to another in a watershed. $T_t$ is a component of time of concentration ($T_c$), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. $T_c$ is computed by summing all the travel times for consecutive components of the drainage conveyance system.

$T_c$ influences the shape and peak of the runoff hydrograph. Urbanization usually decreases $T_c$, thereby increasing the peak discharge. But $T_c$ can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading.

## Factors affecting time of concentration and travel time

### Surface roughness

One of the most significant effects of urban development on flow velocity is less retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by urban development: the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

### Channel shape and flow patterns

In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

### Slope

Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

## Computation of travel time and time of concentration

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time ($T_t$) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600\ V} \qquad \text{[Eq. 3-1]}$$

where

$T_t$ = travel time (hr),
L = flow length (ft),
V = average velocity (ft/s), and
3600 = conversion factor from seconds to hours.

Time of concentration ($T_c$) is the sum of $T_t$ values for the various consecutive flow segments:

$$T_c = T_{t_1} + T_{t_2} + \ldots T_{t_m} \qquad \text{[Eq. 3-2]}$$

where

$T_c$ = time of concentration (hr) and
m = number of flow segments.

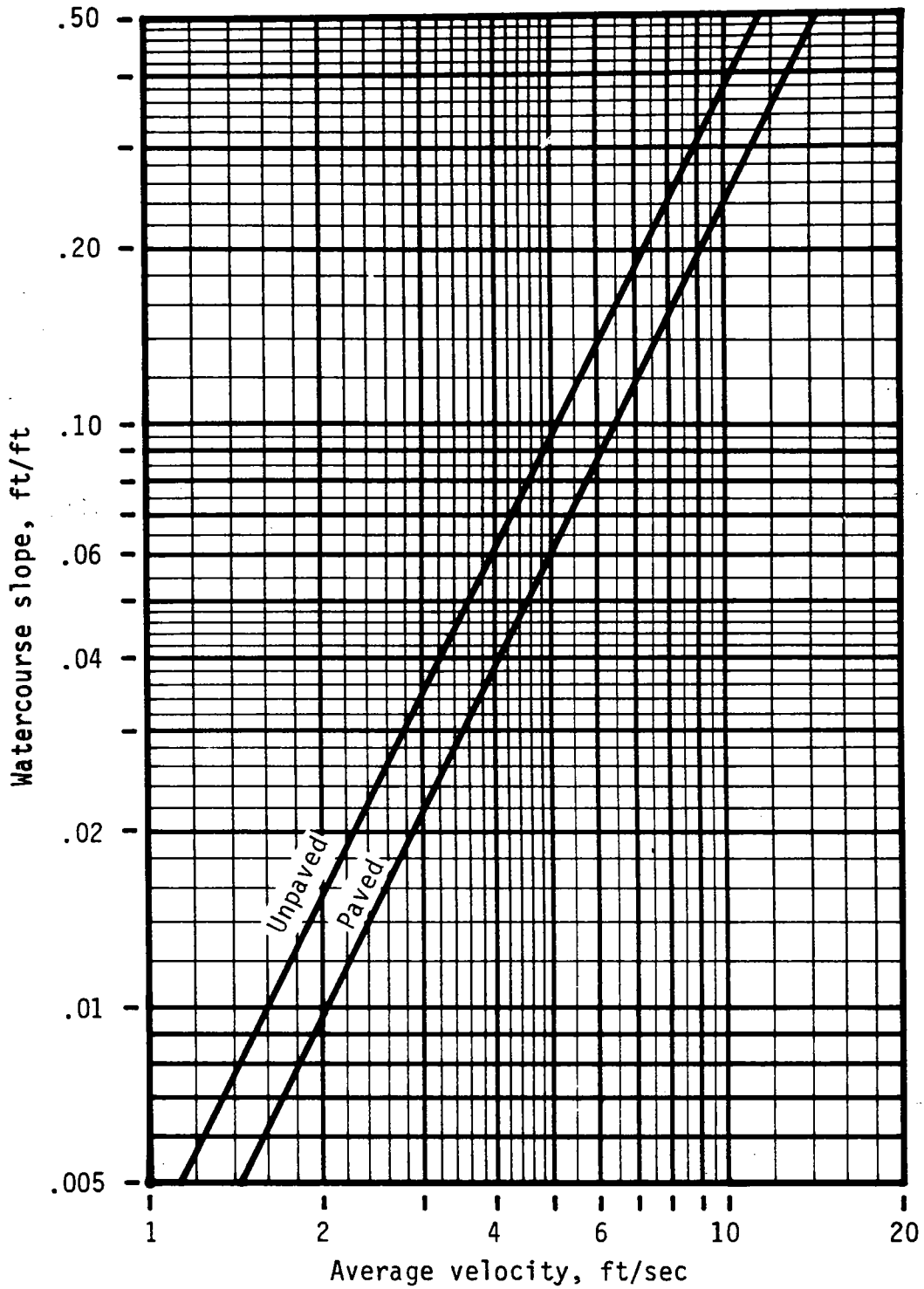



**Figure 3-1.—Average velocities for estimating travel time for shallow concentrated flow.**

### Sheet flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 3-1 gives Manning's n values for sheet flow for various surface conditions.

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overton and Meadows 1976) to compute $T_t$:

$$T_t = \frac{0.007\,(nL)^{0.8}}{(P_2)^{0.5}\,s^{0.4}} \qquad \text{[Eq. 3-3]}$$

**Table 3-1.—Roughness coefficients (Manning's n) for sheet flow**

| Surface description | n[1] |
|---|---|
| Smooth surfaces (concrete, asphalt, gravel, or bare soil) | 0.011 |
| Fallow (no residue) | 0.05 |
| Cultivated soils: | |
| Residue cover ≤20% | 0.06 |
| Residue cover >20% | 0.17 |
| Grass: | |
| Short grass prairie | 0.15 |
| Dense grasses[2] | 0.24 |
| Bermudagrass | 0.41 |
| Range (natural) | 0.13 |
| Woods:[3] | |
| Light underbrush | 0.40 |
| Dense underbrush | 0.80 |

[1]The n values are a composite of information compiled by Engman (1986).
[2]Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[3]When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

where

$T_t$ = travel time (hr),
n = Manning's roughness coefficient (table 3-1),
L = flow length (ft),
$P_2$ = 2-year, 24-hour rainfall (in), and
s = slope of hydraulic grade line (land slope, ft/ft).

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

### Shallow concentrated flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

### Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

Manning's equation is

$$V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n} \qquad \text{[Eq. 3-4]}$$

where

V = average velocity (ft/s),
r = hydraulic radius (ft) and is equal to $a/p_w$,
a = cross sectional flow area ($ft^2$),
$p_w$ = wetted perimeter (ft),
s = slope of the hydraulic grade line (channel slope, ft/ft), and
n = Manning's roughness coefficient for open channel flow.

Manning's n values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using equation 3-4, $T_t$ for the channel segment can be estimated using equation 3-1.

**Reservoirs or lakes**

Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero.

## Limitations

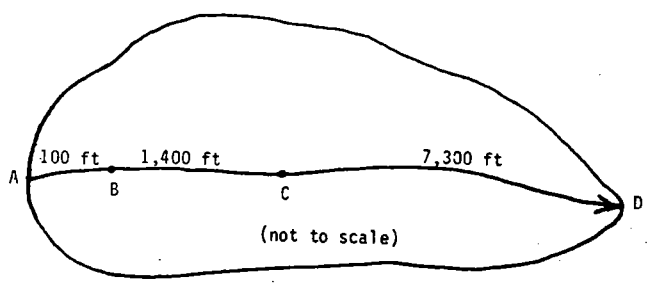
- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation 3-3 was developed for use with the four standard rainfall intensity-duration relationships.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate $T_c$. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.
- The minimum $T_c$ used in TR-55 is 0.1 hour.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. The procedures in TR-55 can be used to determine the peak flow upstream of the culvert. Detailed storage routing procedures should be used to determine the outflow through the culvert.

## Example 3-1

The sketch below shows a watershed in Dyer County, northwestern Tennessee. The problem is to compute $T_c$ at the outlet of the watershed (point D). The 2-year 24-hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute $T_c$, first determine $T_t$ for each segment from the following information:

Segment AB: Sheet flow; dense grass; slope (s) = 0.01 ft/ft; and length (L) = 100 ft.
Segment BC: Shallow concentrated flow; unpaved; s = 0.01 ft/ft; and L = 1400 ft.
Segment CD: Channel flow; Manning's n = .05; flow area (a) = 27 $ft^2$; wetted perimeter ($p_w$) = 28.2 ft; s = 0.005 ft/ft; and L = 7300 ft.

See figure 3-2 for the computations made on worksheet 3.



## Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project Heavenly Acres By DW Date 10/6/85

Location Dyer County, Tennessee Checked RM Date 10/8/85

Circle one: Present (Developed)

Circle one: ($T_c$) $T_t$ through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| | | | | | |
|---|---|---|---|---|---|
| **Sheet flow** (Applicable to $T_c$ only) | | Segment ID | AB | | |
| 1. Surface description (table 3-1) | | | DENSE GRASS | | |
| 2. Manning's roughness coeff., n (table 3-1) | | | 0.24 | | |
| 3. Flow length, L (total L ≤ 300 ft) | ft | | 100 | | |
| 4. Two-yr 24-hr rainfall, $P_2$ | in | | 3.6 | | |
| 5. Land slope, s | ft/ft | | 0.01 | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ | hr | Compute $T_t$ | 0.30 | + | = 0.30 |
| **Shallow concentrated flow** | | Segment ID | BC | | |
| 7. Surface description (paved or unpaved) | | | Unpaved | | |
| 8. Flow length, L | ft | | 1400 | | |
| 9. Watercourse slope, s | ft/ft | | 0.01 | | |
| 10. Average velocity, V (figure 3-1) | ft/s | | 1.6 | | |
| 11. $T_t = \frac{L}{3600\ V}$ | hr | Compute $T_t$ | 0.24 | + | = 0.24 |
| **Channel flow** | | Segment ID | CD | | |
| 12. Cross sectional flow area, a | $ft^2$ | | 27 | | |
| 13. Wetted perimeter, $p_w$ | ft | | 28.2 | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ | ft | Compute r | 0.957 | | |
| 15. Channel slope, s | ft/ft | | 0.005 | | |
| 16. Manning's roughness coeff., n | | | 0.05 | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ | ft/s | Compute V | 2.05 | | |
| 18. Flow length, L | ft | | 7300 | | |
| 19. $T_t = \frac{L}{3600\ V}$ | hr | Compute $T_t$ | 0.99 | + | = 0.99 |
| 20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) | hr | | | | 1.53 |

Figure 3-2.—Worksheet 3 for example 3-1.

# Chapter 4: Graphical Peak Discharge method

This chapter presents the Graphical Peak Discharge method for computing peak discharge from rural and urban areas. The Graphical method was developed from hydrograph analyses using TR-20, "Computer Program for Project Formulation–Hydrology" (SCS 1983). The peak discharge equation used is

$$q_p = q_u A_m Q F_p \qquad \text{[Eq. 4-1]}$$

where

$q_p$ = peak discharge (cfs);
$q_u$ = unit peak discharge (csm/in);
$A_m$ = drainage area ($mi^2$);
$Q$ = runoff (in); and
$F_p$ = pond and swamp adjustment factor.

The input requirements for the Graphical method are as follows: (1) $T_c$ (hr), (2) drainage area ($mi^2$), (3) appropriate rainfall distribution (I, IA, II, or III), (4) 24-hour rainfall (in), and (5) CN. If pond and swamp areas are spread throughout the watershed and are not considered in the $T_c$ computation, an adjustment for pond and swamp areas is also needed.

## Peak discharge computation

For a selected rainfall frequency, the 24-hour rainfall (P) is obtained from appendix B or more detailed local precipitation maps. CN and total runoff (Q) for the watershed are computed according to the methods outlined in chapter 2. The CN is used to determine the initial abstraction ($I_a$) from table 4-1. $I_a/P$ is then computed.

If the computed $I_a/P$ ratio is outside the range shown in exhibit 4 (4-I, 4-IA, 4-II, and 4-III) for the rainfall distribution of interest, then the limiting value should be used. If the ratio falls between the limiting values, use linear interpolation. Figure 4-1 illustrates the sensitivity of $I_a/P$ to CN and P.

Peak discharge per square mile per inch of runoff ($q_u$) is obtained from exhibit 4-I, 4-IA, 4-II, or 4-III by using $T_c$ (chapter 3), rainfall distribution type, and $I_a/P$ ratio. The pond and swamp adjustment factor is obtained from table 4-2 (rounded to the nearest table value). Use worksheet 4 in appendix D to aid in computing the peak discharge using the Graphical method.

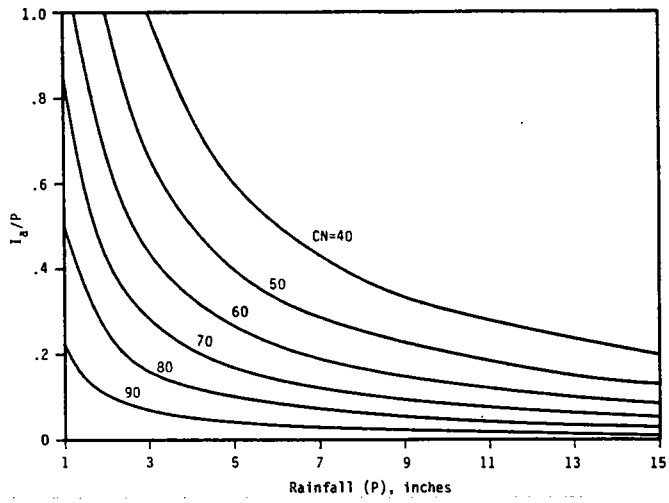



**Figure 4-1.—Variation of $I_a/P$ for P and CN.**

**Table 4-1.—$I_a$ values for runoff curve numbers**

| Curve number | $I_a$ (in) | Curve number | $I_a$ (in) |
|---|---|---|---|
| 40 | 3.000 | 70 | 0.857 |
| 41 | 2.878 | 71 | 0.817 |
| 42 | 2.762 | 72 | 0.778 |
| 43 | 2.651 | 73 | 0.740 |
| 44 | 2.545 | 74 | 0.703 |
| 45 | 2.444 | 75 | 0.667 |
| 46 | 2.348 | 76 | 0.632 |
| 47 | 2.255 | 77 | 0.597 |
| 48 | 2.167 | 78 | 0.564 |
| 49 | 2.082 | 79 | 0.532 |
| 50 | 2.000 | 80 | 0.500 |
| 51 | 1.922 | 81 | 0.469 |
| 52 | 1.846 | 82 | 0.439 |
| 53 | 1.774 | 83 | 0.410 |
| 54 | 1.704 | 84 | 0.381 |
| 55 | 1.636 | 85 | 0.353 |
| 56 | 1.571 | 86 | 0.326 |
| 57 | 1.509 | 87 | 0.299 |
| 58 | 1.448 | 88 | 0.273 |
| 59 | 1.390 | 89 | 0.247 |
| 60 | 1.333 | 90 | 0.222 |
| 61 | 1.279 | 91 | 0.198 |
| 62 | 1.226 | 92 | 0.174 |
| 63 | 1.175 | 93 | 0.151 |
| 64 | 1.125 | 94 | 0.128 |
| 65 | 1.077 | 95 | 0.105 |
| 66 | 1.030 | 96 | 0.083 |
| 67 | 0.985 | 97 | 0.062 |
| 68 | 0.941 | 98 | 0.041 |
| 69 | 0.899 | | |

**Table 4-2.—Adjustment factor ($F_p$) for pond and swamp areas that are spread throughout the watershed**

| Percentage of pond and swamp areas | $F_p$ |
|---|---|
| 0 | 1.00 |
| 0.2 | 0.97 |
| 1.0 | 0.87 |
| 3.0 | 0.75 |
| 5.0 | 0.72 |

## Limitations

The Graphical method provides a determination of peak discharge only. If a hydrograph is needed or watershed subdivision is required, use the Tabular Hydrograph method (chapter 5). Use TR-20 if the watershed is very complex or a higher degree of accuracy is required.

- The watershed must be hydrologically homogeneous, that is, describable by one CN. Land use, soils, and cover are distributed uniformly throughout the watershed.
- The watershed may have only one main stream or, if more than one, the branches must have nearly equal $T_c$'s.
- The method cannot perform valley or reservoir routing.
- The $F_p$ factor can be applied only for ponds or swamps that are not in the $T_c$ flow path.
- Accuracy of peak discharge estimated by this method will be reduced if $I_a/P$ values are used that are outside the range given in exhibit 4. The limiting $I_a/P$ values are recommended for use.
- This method should be used only if the weighted CN is greater than 40.
- When this method is used to develop estimates of peak discharge for both present and developed conditions of a watershed, use the same procedure for estimating $T_c$.
- $T_c$ values with this method may range from 0.1 to 10 hours.

## Example 4-1

Compute the 25-year peak discharge for the 250-acre watershed described in examples 2-2 and 3-1. Figure 4-2 shows how worksheet 4 is used to compute $q_p$ as 345 cfs.

## Worksheet 4: Graphical Peak Discharge method

Project Heavenly Acres By RHM Date 10/15/85

Location Dyer County, Tennessee Checked RHM Date 10/17/85

Circle one: Present (Developed)

1. Data:

   Drainage area ..... $A_m$ = 0.39 $mi^2$ (acres/640)

   Runoff curve number .... CN = 75 (From worksheet 2), Figure 2-6

   Time of concentration .. $T_c$ = 1.53 hr (From worksheet 3), Figure 3-2

   Rainfall distribution type = II (I, IA, II, III)

   Pond and swamp areas spread throughout watershed ..... = -- percent of $A_m$ (-- acres or $mi^2$ covered)

| | | | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|---|---|
| 2. | Frequency ..... | yr | 25 | | |
| 3. | Rainfall, P (24-hour) ..... | in | 6.0 | | |
| 4. | Initial abstraction, $I_a$ ..... (Use CN with table 4-1.) | in | 0.667 | | |
| 5. | Compute $I_a/P$ ..... | | 0.11 | | |
| 6. | Unit peak discharge, $q_u$ ..... (Use $T_c$ and $I_a/P$ with exhibit 4-II) | csm/in | 270 | | |
| 7. | Runoff, Q ..... (From worksheet 2). Figure 2-6 | in | 3.28 | | |
| 8. | Pond and swamp adjustment factor, $F_p$ .... (Use percent pond and swamp area with table 4-2. Factor is 1.0 for zero percent pond and swamp area.) | | 1.0 | | |
| 9. | Peak discharge, $q_p$ ..... (Where $q_p = q_u A_m Q F_p$) | cfs | 345 | | |

Figure 4-2.—Worksheet 4 for example 4-1.

**Exhibit 4-I: Unit peak discharge ($q_u$) for SCS type I rainfall distribution**

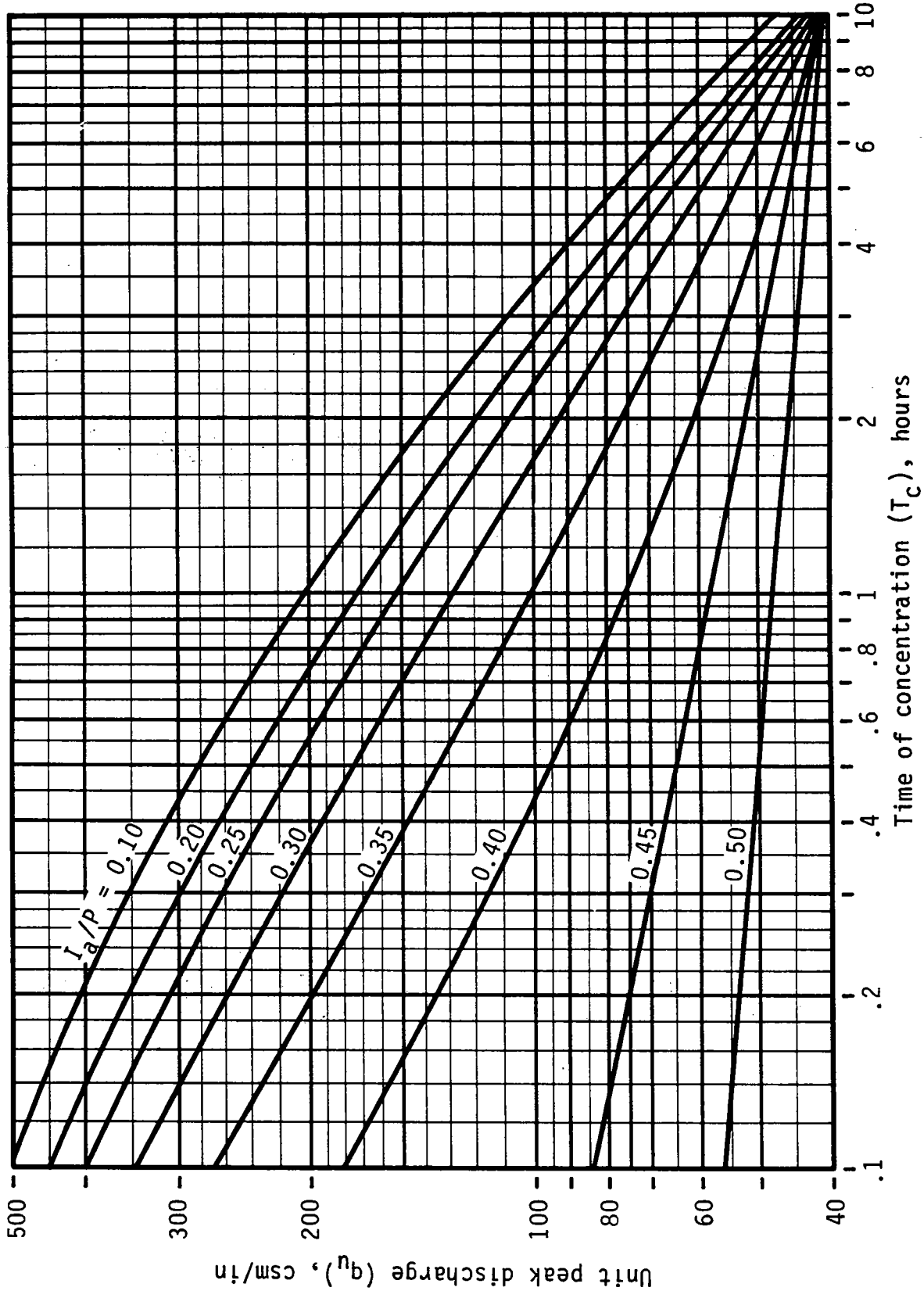


**Exhibit 4-IA: Unit peak discharge ($q_u$) for SCS type IA rainfall distribution**

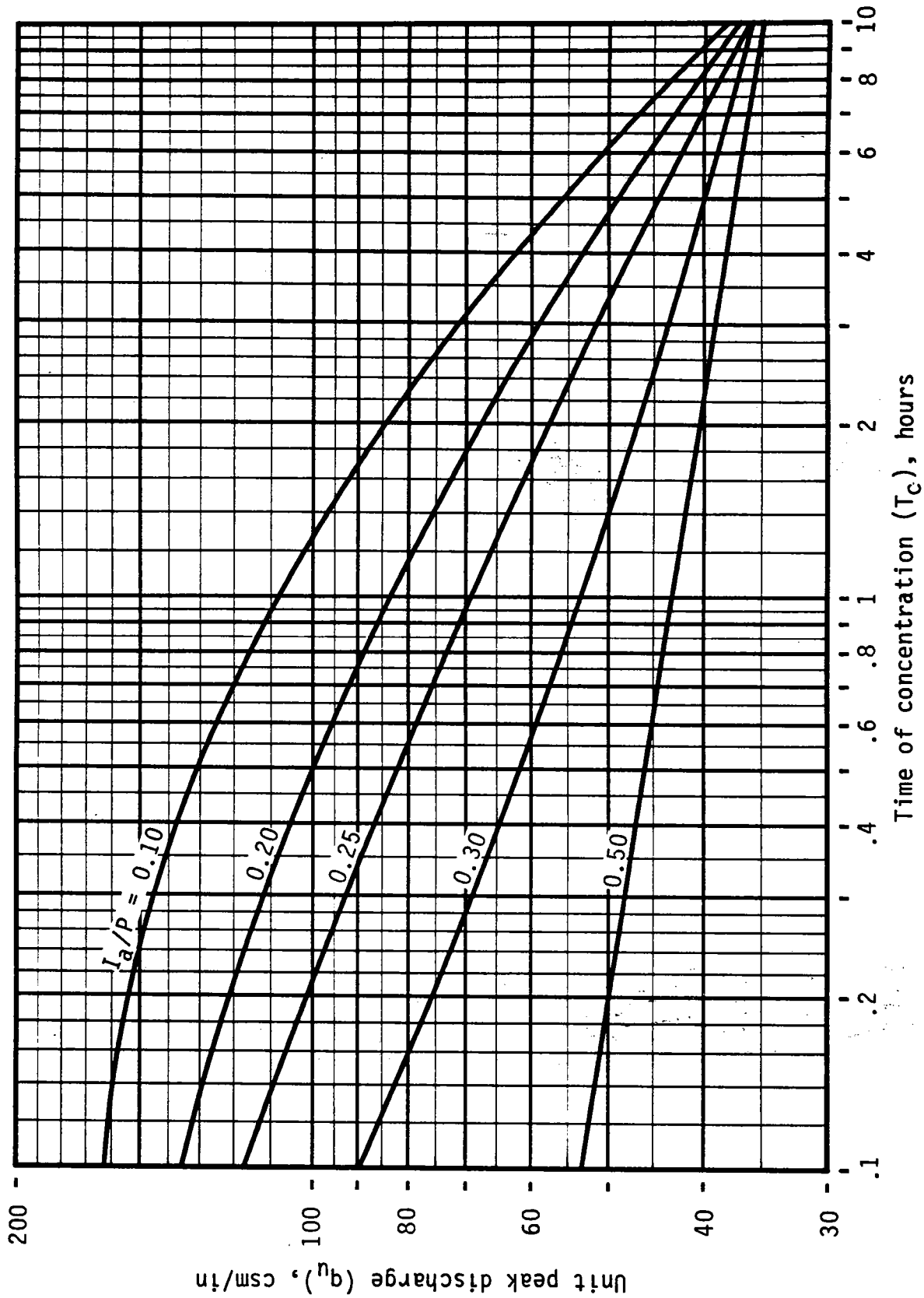


**Exhibit 4-II: Unit peak discharge ($q_u$) for SCS type II rainfall distribution**

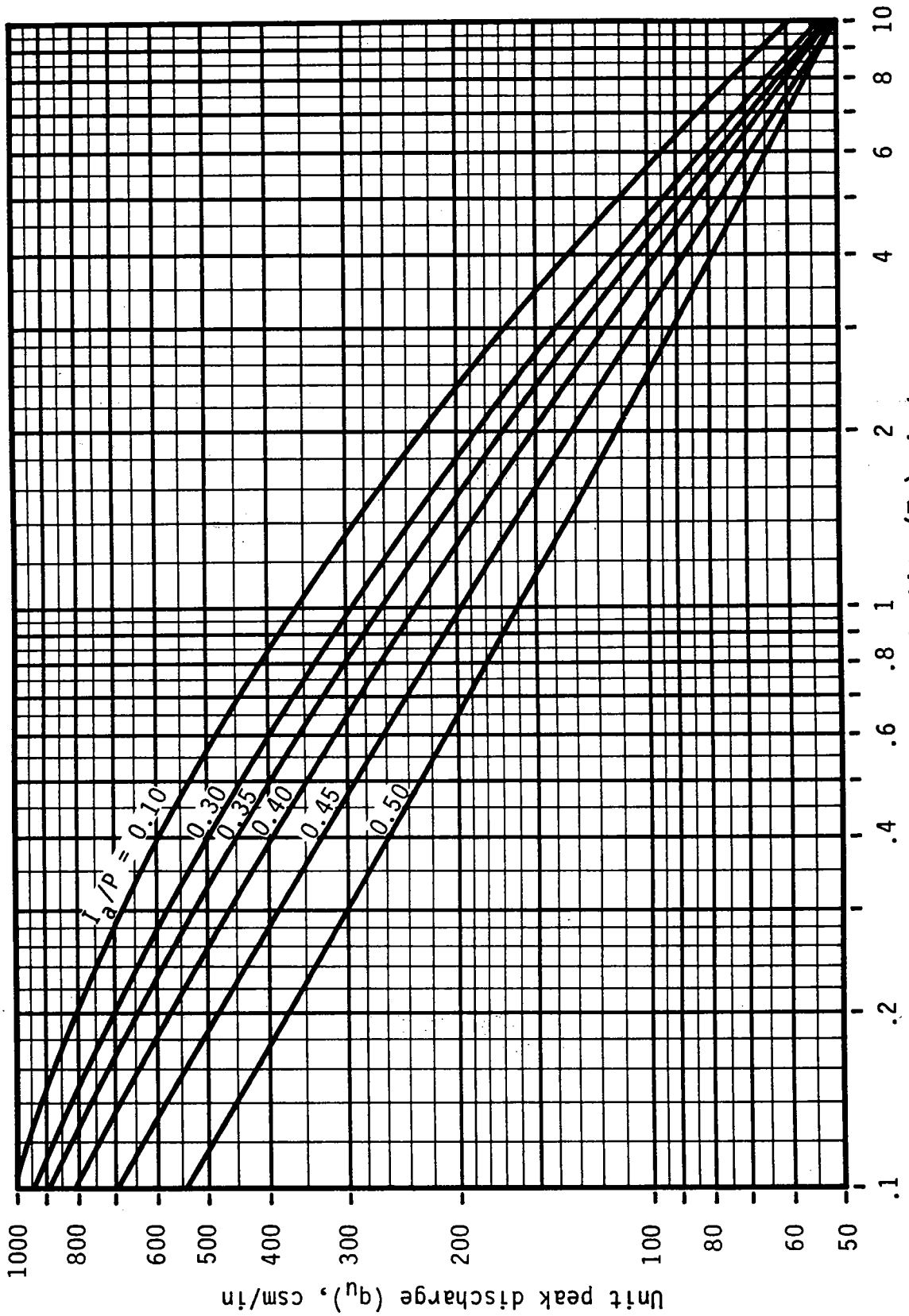
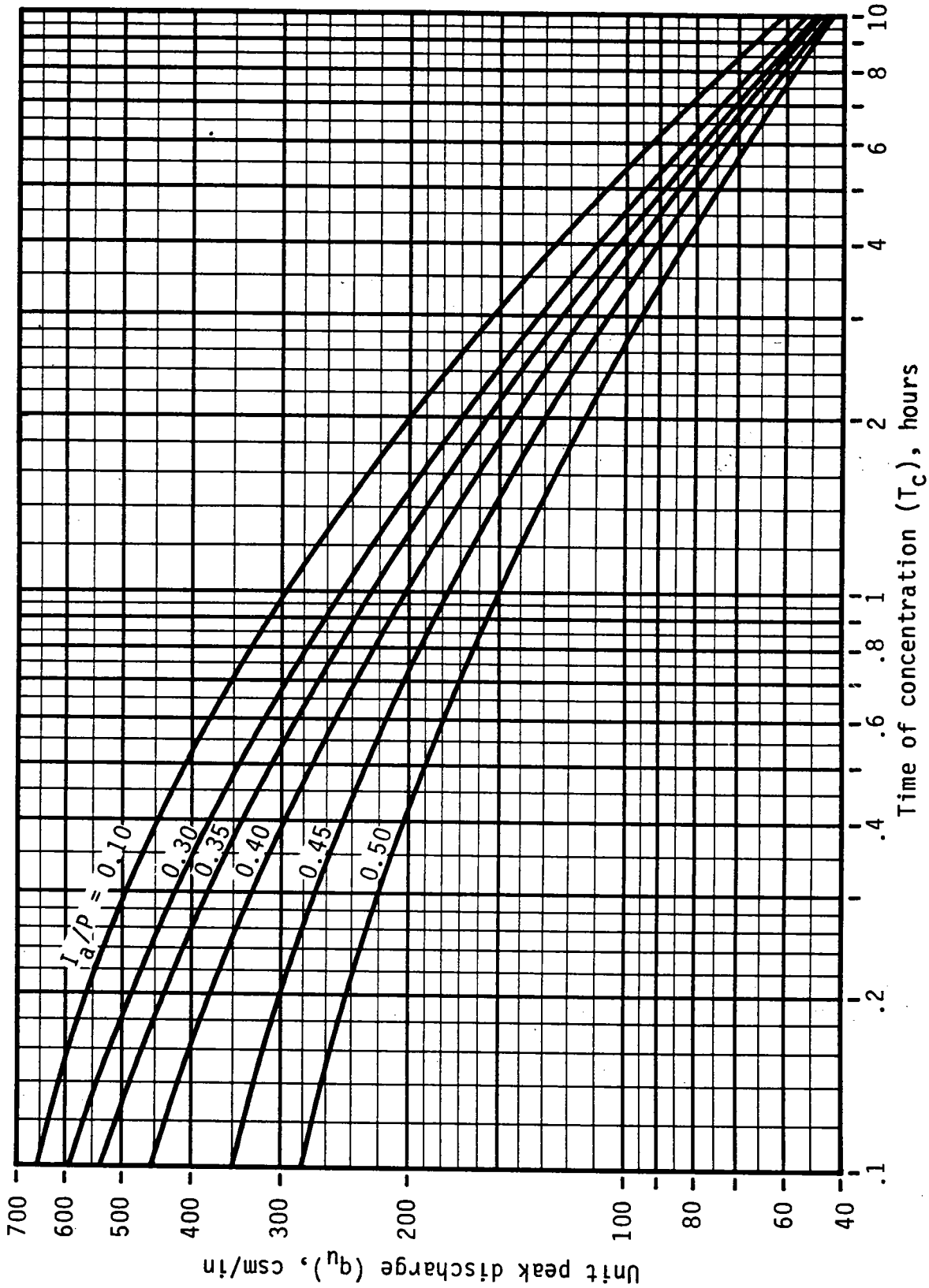


**Exhibit 4-III: Unit peak discharge ($q_u$) for SCS type III rainfall distribution**



# Chapter 5: Tabular Hydrograph method

This chapter presents the Tabular Hydrograph method of computing peak discharges from rural and urban areas, using time of concentration ($T_c$) and travel time ($T_t$) from a subarea as inputs. This method approximates TR-20, a more detailed hydrograph procedure (SCS 1983).

The Tabular method can develop partial composite flood hydrographs at any point in a watershed by dividing the watershed into homogeneous subareas. In this manner, the method can estimate runoff from nonhomogeneous watersheds. The method is especially applicable for estimating the effects of land use change in a portion of a watershed. It can also be used to estimate the effects of proposed structures.

Input data needed to develop a partial composite flood hydrograph include (1) 24-hour rainfall (in), (2) appropriate rainfall distribution (I, IA, II, or III), (3) CN, (4) $T_c$ (hr), (5) $T_t$ (hr), and (6) drainage area ($mi^2$).

## Tabular Hydrograph method exhibits

Exhibit 5 (5-I, 5-IA, 5-II, and 5-III) shows tabular discharge values for the various rainfall distributions. Tabular discharges expressed in csm/in (cubic feet of discharge per second per square mile of watershed per inch of runoff) are given for a range of subarea $T_c$'s from 0.1 to 2 hours and reach $T_t$'s from 0 to 3 hours.

The exhibit was developed by computing hydrographs for 1 square mile of drainage area for selected $T_c$'s and routing them through stream reaches with the range of $T_t$'s indicated. The Modified Att-Kin method for reach routing, formulated by SCS in the late 1970's, was used to compute the tabular hydrographs (Comer et al., 1981). A CN of 75 and rainfall amounts generating appropriate $I_a/P$ ratios were used. The resulting runoff estimate was used to convert the hydrographs in exhibits 5-I through 5-III to cubic feet per second per square mile per inch of runoff.

An assumption in development of the tabular hydrographs is that all discharges for a stream reach flow at the same velocity. By this assumption, the subarea flood hydrographs may be routed separately and added at the reference point. The tabular hydrographs in exhibit 5 are prerouted hydrographs. For $T_t$'s other than zero, the tabular discharge values represent the contribution from a single subarea to the composite hydrograph at $T_t$ downstream.

## Information required for Tabular Hydrograph method

The following information is required for the Tabular method:

1. Subdivision of the watershed into areas that are relatively homogeneous and have convenient routing reaches.
2. Drainage area of each subarea in square miles.
3. $T_c$ for each subarea in hours. The procedure for estimating $T_c$ is outlined in chapter 3. Worksheet 3 (appendix D) can be used to calculate $T_c$.
4. $T_t$ for each routing reach in hours. The procedure for estimating $T_t$ is outlined in chapter 3. Worksheet 3 can be used to calculate $T_t$ through a subarea for shallow concentrated and open channel flow.
5. Weighted CN for each subarea. Table 2-2 shows CN's for individual hydrologic soil cover combinations. Worksheet 2 can be used to calculate the weighted runoff curve number.
6. Appropriate rainfall distribution according to figure B-2 (appendix B).
7. The 24-hour rainfall for the selected frequency. Appendix B contains rainfall maps for various frequencies (figures B-3 to B-8).
8. Total runoff (Q) in inches computed from CN and rainfall.
9. $I_a$ for each subarea from table 5-1, which is the same as table 4-1.
10. Ratio of $I_a/P$ for each subarea. If the ratio for the rainfall distribution of interest is outside the range shown in exhibit 5, use the limiting value.

## Development of composite flood hydrograph

This section describes the procedure for developing the peak discharge and selected discharge values of a composite flood hydrograph.

### Selecting $T_c$ and $T_t$

First, use worksheet 5a to develop a summary of basic watershed data by subarea. Then use worksheet 5b to develop a tabular hydrograph discharge summary; this summary displays the effect of individual subarea hydrographs as routed to the watershed point of interest. Use $\Sigma T_t$ for each subarea as the total reach travel time from that subarea through the watershed to the point of interest. Compute the hydrograph coordinates for selected $\Sigma T_t$'s using the appropriate sheets in exhibit 5. The flow at any time is

$$q = q_t A_m Q \qquad \text{[Eq. 5-1]}$$

where

- $q$ = hydrograph coordinate (cfs) at hydrograph time t;
- $q_t$ = tabular hydrograph unit discharge from exhibit 5 (csm/in);
- $A_m$ = drainage area of individual subarea ($mi^2$); and
- $Q$ = runoff (in).

Since the timing of peak discharge changes with $T_c$ and $T_t$, interpolation of peak discharge for $T_c$ and $T_t$ values for use in exhibit 5 is not recommended. Interpolation may result in an estimate of peak discharge that would be invalid because it would be lower than either of the hydrographs. Therefore, round the actual values of $T_c$ and $T_t$ to values presented in exhibit 5. Perform this rounding so that the sum of the selected table values is close to the sum of actual $T_c$ and $T_t$. An acceptable procedure is to select the results of one of three rounding operations:

1. Round $T_c$ and $T_t$ separately to the nearest table value and sum;
2. Round $T_c$ down and $T_t$ up to nearest table value and sum; and
3. Round $T_c$ up and $T_t$ down to nearest table value and sum.

From these three alternatives, choose the pair of rounded $T_c$ and $T_t$ values whose sum is closest to the sum of the actual $T_c$ and $T_t$. If two rounding methods produce sums equally close to the actual sum, use the combination in which rounded $T_c$ is closest to actual $T_c$. An illustration of the rounding procedure is as follows:

**Table 5-1.—$I_a$ values for runoff curve numbers**

| Curve number | $I_a$ (in) | Curve number | $I_a$ (in) |
|---|---|---|---|
| 40 | 3.000 | 70 | 0.857 |
| 41 | 2.878 | 71 | 0.817 |
| 42 | 2.762 | 72 | 0.778 |
| 43 | 2.651 | 73 | 0.740 |
| 44 | 2.545 | 74 | 0.703 |
| 45 | 2.444 | 75 | 0.667 |
| 46 | 2.348 | 76 | 0.632 |
| 47 | 2.255 | 77 | 0.597 |
| 48 | 2.167 | 78 | 0.564 |
| 49 | 2.082 | 79 | 0.532 |
| 50 | 2.000 | 80 | 0.500 |
| 51 | 1.922 | 81 | 0.469 |
| 52 | 1.846 | 82 | 0.439 |
| 53 | 1.774 | 83 | 0.410 |
| 54 | 1.704 | 84 | 0.381 |
| 55 | 1.636 | 85 | 0.353 |
| 56 | 1.571 | 86 | 0.326 |
| 57 | 1.509 | 87 | 0.299 |
| 58 | 1.448 | 88 | 0.273 |
| 59 | 1.390 | 89 | 0.247 |
| 60 | 1.333 | 90 | 0.222 |
| 61 | 1.279 | 91 | 0.198 |
| 62 | 1.226 | 92 | 0.174 |
| 63 | 1.175 | 93 | 0.151 |
| 64 | 1.125 | 94 | 0.128 |
| 65 | 1.077 | 95 | 0.105 |
| 66 | 1.030 | 96 | 0.083 |
| 67 | 0.985 | 97 | 0.062 |
| 68 | 0.941 | 98 | 0.041 |
| 69 | 0.899 | | |

| | Actual values | Table values by rounding method— | | |
|---|---|---|---|---|
| | | 1 | 2 | 3 |
| $T_c$ | 1.1 | 1.0 | 1.0 | 1.25 |
| $T_t$ | 1.7 | 1.5 | 2.0 | 1.5 |
| Sum | 2.8 | 2.5 | 3.0 | 2.75 |

In this instance, the results from method 3 would be selected because the sum 2.75 is closest to the actual sum of 2.8.

### Selecting $I_a/P$

The computed $I_a/P$ value can be rounded to the nearest $I_a/P$ value in exhibits 5-I through 5-III, or the hydrograph values (csm/in) can be linearly interpolated because $I_a/P$ interpolation generally involves peaks that occur at the same time.

### Summing for the composite hydrograph

The composite hydrograph is the summation of prerouted individual subarea hydrographs at each time shown on worksheet 5b. Only the times encompassing the expected maximum composite discharge are summed to define a portion of the composite hydrograph.

If desired, the entire composite hydrograph can be approximated by linear extrapolation as follows:

1. Set up a table similar to worksheet 5b. Include on this table the full range of hydrograph times displayed in exhibit 5.
2. Compute the subarea discharge values for those times and insert them in the table.
3. Sum the values to obtain the composite hydrograph.
4. Apply linear extrapolation to the first two points and the last two points of the composite hydrograph. The volume under this approximation of the entire composite hydrograph may differ from the computed runoff volume.

## Limitations

The Tabular method is used to determine peak flows and hydrographs within a watershed. However, its accuracy decreases as the complexity of the watershed increases. If you want to compare present and developed conditions of a watershed, use the same procedure for estimating $T_c$ for both conditions.

Use the TR-20 computer program (SCS 1983) instead of the Tabular method if any of the following conditions applies:

- $T_t$ is greater than 3 hours (largest $T_t$ in exhibit 5).
- $T_c$ is greater than 2 hours (largest $T_c$ in exhibit 5).
- Drainage areas of individual subareas differ by a factor of 5 or more.
- The entire composite flood hydrograph or entire runoff volume is required for detailed flood routings. The hydrograph based on extrapolation is only an approximation of the entire hydrograph.
- The time of peak discharge must be more accurate than that obtained through the Tabular method.

The composite flood hydrograph should be compared with actual stream gage data where possible. The instantaneous peak flow value from the composite flood hydrograph can be compared with data from USGS curves of peak flow versus drainage area.

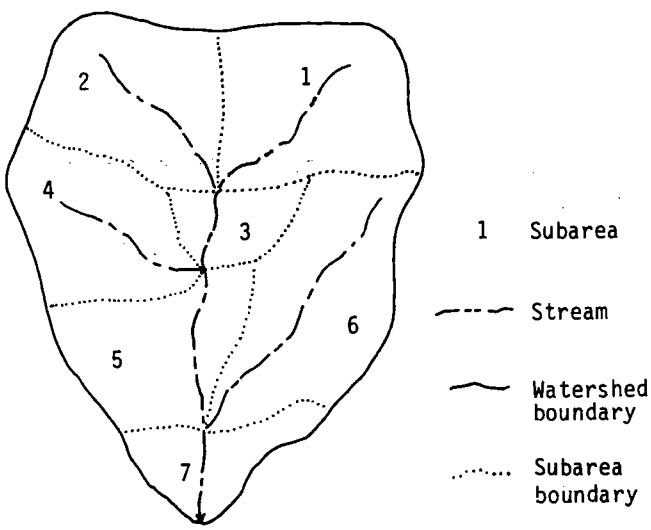
## Examples

A developer proposes to put a subdivision, Fallswood, in subareas 5, 6, and 7 of a watershed in Dyer County, northwestern Tennessee (see sketch below). Before approving the developer's proposal, the planning board wants to know how the development would affect the 25-year peak discharge at the downstream end of subarea 7. The rainfall distribution is type II (figure B-2), and the 24-hour rainfall (P) is 6.0 inches (figure B-6).

### Example 5-1

Compute the 25-year frequency peak discharge at the downstream end of subarea 7 for present conditions, using worksheets 5a and 5b. To do this, first calculate the present condition CN, $T_c$, and $T_t$ for each subarea, using the procedures in chapters 2 and 3. Enter the values on worksheet 5a (figure 5-1).

Next, compute the prerouted hydrograph points for each subarea hydrograph over a range of time near the peak discharge using worksheet 5b (figure 5-2) and the appropriate exhibit 5. For example, for subarea 4, in which $T_c = 0.75$ hr, refer to sheet 6 of exhibit 5-II. With $\Sigma T_t$ of 2.00 hr (the sum of downstream travel time through subareas 5 and 7 to the outlet) and $I_a/P$ of 0.1, the routed peak discharge of subarea 4 at the outlet of subarea 7 occurs at 14.6 hr and is 274 csm/in. Solving equation 5-1 with




appropriate values provides the peak discharge (q) for subarea 4 at 14.6 hr:

$$q = q_t(A_mQ) = (274)(0.70) = 192 \text{ cfs.}$$

Once all the prerouted subarea hydrographs have been tabulated on worksheet 5b, sum each of the time columns to obtain the composite hydrograph. The resulting 25-year frequency peak discharge is 720 cfs at 14.3 hr (figure 5-2).

### Example 5-2

Compute the 25-year frequency peak discharge at the downstream end of subarea 7 for the developed conditions, using worksheets 5a and 5b.

First, calculate the developed condition CN, $T_c$, and $T_t$ for each subarea, using the procedures in chapters 2 and 3. Enter the values on worksheet 5a (figure 5-3).

Next, compute the prerouted hydrograph points for each subarea hydrograph over a range of time near the peak discharge using worksheet 5b (figure 5-4) and the appropriate exhibit 5. For example, for subarea 6, in which $T_c = 1.0$ hr, refer to sheet 7 of exhibit 5-II. With $\Sigma T_t$ of 0.5 hr (downstream travel time through subarea 7 to the outlet) and $I_a/P$ of 0.1, the peak discharge of subarea 6 at the outlet of the watershed occurs at 13.2 hr and is 311 csm/in. Solving equation 5-1 provides the peak discharge (q):

$$q = q_t(A_mQ) = (311)(1.31) = 407 \text{ cfs.}$$

Once all the prerouted subarea hydrographs have been tabulated on worksheet 5b, sum each of the time columns to obtain the composite hydrograph. The resulting 25-year frequency peak discharge is 872 cfs at 13.6 hr (figure 5-4).

### Comparison

According to the results of the two examples, the proposed subdivision at the downstream end of subarea 7 is expected to increase peak discharge from 720 to 872 cfs and to decrease the time to peak from 14.3 to 13.6 hr.

## Worksheet 5a: Basic watershed data

Project Fallswood Location Dyer County, Tennessee By DW Date 10/1/85

Circle one: (Present) Developed Frequency (yr) 25 Checked JH Date 10/3/85

| Subarea name | Drainage area $A_m$ ($mi^2$) | Time of concentration $T_c$ (hr) | Travel time through subarea $T_t$ (hr) | Downstream subarea names | Travel time summation to outlet $\Sigma T_t$ (hr) | 24-hr Rainfall P (in) | Runoff curve number CN | Runoff Q (in) | $A_m Q$ ($mi^2$-in) | Initial abstraction $I_a$ (in) | $I_a/P$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.30 | 1.50 | -- | 3,5,7 | 2.50 | 6.0 | 65 | 2.35 | 0.71 | 1.077 | 0.18 |
| 2 | 0.20 | 1.25 | -- | 3,5,7 | 2.50 | 6.0 | 70 | 2.80 | 0.56 | 0.857 | 0.14 |
| 3 | 0.10 | 0.50 | 0.50 | 5,7 | 2.00 | 6.0 | 75 | 3.28 | 0.33 | 0.667 | 0.11 |
| 4 | 0.25 | 0.75 | -- | 5,7 | 2.00 | 6.0 | 70 | 2.80 | 0.70 | 0.857 | 0.14 |
| 5 | 0.20 | 1.50 | 1.25 | 7 | 0.75 | 6.0 | 75 | 3.28 | 0.66 | 0.667 | 0.11 |
| 6 | 0.40 | 1.50 | -- | 7 | 0.75 | 6.0 | 70 | 2.80 | 1.12 | 0.857 | 0.14 |
| 7 | 0.20 | 1.25 | 0.75 | -- | 0 | 6.0 | 75 | 3.28 | 0.66 | 0.667 | 0.11 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

From worksheet 3 | From worksheet 2 | From table 5-1

Figure 5-1.—Worksheet 5a for example 5-1.

## Worksheet 5b: Tabular hydrograph discharge summary

Project Fallswood Location Dyer County, Tennessee By DW Date 10/1/85

Circle one: (Present) Developed Frequency (yr) 25 Checked WH Date 10/3/85

| Subarea name | Basic watershed data used 1/ | | | | Select and enter hydrograph times in hours from exhibit 5-II 2/ | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sub-area $T_c$ (hr) | $\Sigma T_t$ to outlet (hr) | $I_a/P$ | $A_mQ$ ($mi^2$-in) | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 |
| | | | | | Discharges at selected hydrograph times 3/ (cfs) | | | | | | | | | | | |
| 1 | 1.50 | 2.50 | 0.10 | 0.71 | 4 | 4 | 5 | 6 | 6 | 8 | 10 | 13 | 24 | 49 | 100 | 149 |
| 2 | 1.25 | 2.50 | 0.10 | 0.56 | 3 | 4 | 4 | 6 | 7 | 8 | 11 | 16 | 32 | 64 | 110 | 127 |
| 3 | 0.50 | 2.00 | 0.10 | 0.33 | 5 | 5 | 6 | 8 | 12 | 21 | 41 | 67 | 98 | 92 | 60 | 29 |
| 4 | 0.75 | 2.00 | 0.10 | 0.70 | 8 | 9 | 11 | 14 | 20 | 34 | 62 | 106 | 172 | 192 | 149 | 81 |
| 5 | 1.50 | 0.75 | 0.10 | 0.66 | 21 | 28 | 50 | 83 | 118 | 147 | 158 | 154 | 127 | 98 | 67 | 44 |
| 6 | 1.50 | 0.75 | 0.10 | 1.12 | 36 | 47 | 85 | 140 | 200 | 249 | 269 | 261 | 216 | 166 | 114 | 75 |
| 7 | 1.25 | 0 | 0.10 | 0.66 | 169 | 187 | 205 | 176 | 140 | 108 | 85 | 69 | 51 | 40 | 31 | 24 |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| Composite hydrograph at outlet | | | | | 246 | 284 | 366 | 433 | 503 | 575 | 636 | 686 | 720 | 701 | 631 | 529 |

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is $A_mQ$ multiplied by tabular discharge from appropriate exhibit 5.

Figure 5-2.—Worksheet 5b for example 5-1.

## Worksheet 5a: Basic watershed data

Project Fallswood Location Dyer County, Tennessee By DW Date 10/1/85

Circle one: Present (Developed) _____ Frequency (yr) 25 Checked JM Date 10/3/85

| Subarea name | Drainage area $A_m$ ($mi^2$) | Time of concentration $T_c$ (hr) | Travel time through subarea $T_t$ (hr) | Downstream subarea names | Travel time summation to outlet $\Sigma T_t$ (hr) | 24-hr Rainfall P (in) | Runoff curve number CN | Runoff Q (in) | $A_m Q$ ($mi^2$-in) | Initial abstraction $I_a$ (in) | $I_a/P$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.30 | 1.50 | -- | 3,5,7 | 2.00 | 6.0 | 65 | 2.35 | 0.71 | 1.077 | 0.18 |
| 2 | 0.20 | 1.25 | -- | 3,5,7 | 2.00 | 6.0 | 70 | 2.80 | 0.56 | 0.857 | 0.14 |
| 3 | 0.10 | 0.50 | 0.50 | 5,7 | 1.50 | 6.0 | 75 | 3.28 | 0.33 | 0.667 | 0.11 |
| 4 | 0.25 | 0.75 | -- | 5,7 | 1.50 | 6.0 | 70 | 2.80 | 0.70 | 0.857 | 0.14 |
| 5 | 0.20 | 1.50 | 1.00 | 7 | 0.50 | 6.0 | 85 | 4.31 | 0.86 | 0.353 | 0.06 |
| 6 | 0.40 | 1.00 | -- | 7 | 0.50 | 6.0 | 75 | 3.28 | 1.31 | 0.857 | 0.14 |
| 7 | 0.20 | 0.75 | 0.50 | -- | 0 | 6.0 | 90 | 4.85 | 0.97 | 0.222 | 0.04 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

From worksheet 3 | From worksheet 2 | From table 5-1

Figure 5-3.—Worksheet 5a for example 5-2.

## Worksheet 5b: Tabular hydrograph discharge summary

Project Fallswood Location Dyer County, Tennessee By DW Date 10/1/85

Circle one: Present (Developed) Frequency (yr) 25 Checked TH Date 10/3/85

| Subarea name | Basic watershed data used 1/ | | | | Select and enter hydrograph times in hours from exhibit 5-II 2/ | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sub-area $T_c$ (hr) | $\Sigma T_t$ to outlet (hr) | $I_a/P$ | $A_mQ$ ($mi^2$-in) | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 |
| | | | | | Discharges at selected hydrograph times 3/ (cfs) | | | | | | | | | | | |
| 1 | 1.50 | 2.00 | 0.10 | 0.71 | 6 | 6 | 7 | 9 | 11 | 16 | 24 | 40 | 78 | 122 | 155 | 133 |
| 2 | 1.25 | 2.00 | 0.10 | 0.56 | 6 | 6 | 7 | 9 | 12 | 20 | 33 | 55 | 96 | 132 | 132 | 87 |
| 3 | 0.50 | 1.50 | 0.10 | 0.33 | 8 | 9 | 14 | 29 | 58 | 89 | 106 | 102 | 74 | 46 | 25 | 16 |
| 4 | 0.75 | 1.50 | 0.10 | 0.70 | 13 | 14 | 19 | 32 | 63 | 114 | 169 | 207 | 193 | 143 | 83 | 46 |
| 5 | 1.50 | 0.50 | 0.10 | 0.86 | 51 | 69 | 117 | 167 | 205 | 214 | 202 | 175 | 132 | 99 | 70 | 48 |
| 6 | 1.00 | 0.50 | 0.10 | 1.31 | 149 | 208 | 331 | 407 | 393 | 329 | 255 | 195 | 134 | 97 | 69 | 52 |
| 7 | 0.75 | 0 | 0.10 | 0.97 | 398 | 358 | 244 | 167 | 119 | 90 | 72 | 59 | 48 | 40 | 34 | 30 |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| Composite hydrograph at outlet | | | | | 631 | 670 | 739 | 820 | 861 | 872 | 861 | 833 | 755 | 679 | 568 | 412 |

1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is $A_mQ$ multiplied by tabular discharge from appropriate exhibit 5.



Figure 5-4.—Worksheet 5b for example 5-2.

# Exhibit 5-I: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 9.0 | 9.3 | 9.6 | 9.9 | 10.0 | 10.1 | 10.2 | 10.3 | 10.4 | 10.5 | 10.6 | 10.7 | 10.8 | 11.0 | 11.2 | 11.4 | 11.6 | 11.8 | 12.0 | 12.3 | 12.6 | 13.0 | 13.5 | 14.0 | 14.5 | 15.0 | 15.5 | 16.0 | 17.0 | 18.0 | 20.0 | 24.0 |
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 30 | 40 | 56 | 183 | 337 | 504 | 326 | 155 | 122 | 107 | 93 | 81 | 73 | 66 | 60 | 56 | 54 | 52 | 49 | 46 | 44 | 40 | 36 | 32 | 30 | 29 | 28 | 27 | 26 | 24 | 20 | 13 |
| .10 | 26 | 35 | 48 | 93 | 153 | 276 | 428 | 360 | 223 | 156 | 123 | 103 | 88 | 72 | 65 | 59 | 56 | 54 | 51 | 47 | 45 | 42 | 37 | 33 | 30 | 29 | 28 | 28 | 26 | 24 | 21 | 13 |
| .20 | 23 | 30 | 41 | 60 | 82 | 129 | 227 | 361 | 360 | 269 | 194 | 147 | 118 | 85 | 71 | 63 | 58 | 55 | 53 | 49 | 46 | 43 | 39 | 35 | 31 | 29 | 29 | 28 | 26 | 24 | 21 | 14 |
| .30 | 22 | 29 | 39 | 56 | 73 | 111 | 188 | 303 | 341 | 293 | 227 | 173 | 136 | 94 | 75 | 65 | 60 | 56 | 54 | 50 | 47 | 43 | 39 | 35 | 31 | 29 | 29 | 28 | 26 | 25 | 21 | 14 |
| .40 | 18 | 25 | 34 | 46 | 53 | 66 | 96 | 157 | 255 | 312 | 300 | 251 | 199 | 126 | 90 | 73 | 64 | 59 | 56 | 52 | 48 | 44 | 40 | 36 | 32 | 30 | 29 | 28 | 26 | 25 | 21 | 14 |
| .50 | 18 | 24 | 32 | 44 | 50 | 61 | 84 | 133 | 214 | 280 | 293 | 265 | 221 | 144 | 99 | 77 | 66 | 60 | 56 | 52 | 49 | 45 | 41 | 37 | 32 | 30 | 29 | 28 | 27 | 25 | 21 | 14 |
| .75 | 14 | 19 | 25 | 34 | 38 | 43 | 49 | 62 | 88 | 134 | 190 | 234 | 252 | 221 | 162 | 115 | 87 | 71 | 63 | 56 | 52 | 47 | 43 | 39 | 35 | 31 | 29 | 29 | 27 | 25 | 22 | 15 |
| 1.0 | 11 | 14 | 19 | 26 | 28 | 31 | 34 | 38 | 44 | 52 | 68 | 98 | 141 | 222 | 238 | 191 | 139 | 101 | 79 | 63 | 56 | 51 | 45 | 41 | 37 | 33 | 30 | 29 | 27 | 26 | 22 | 15 |
| 1.5 | 9 | 10 | 13 | 17 | 18 | 20 | 22 | 25 | 27 | 30 | 34 | 38 | 44 | 74 | 132 | 191 | 211 | 190 | 151 | 101 | 73 | 58 | 50 | 45 | 41 | 37 | 33 | 30 | 28 | 27 | 23 | 16 |
| 2.0 | 6 | 7 | 9 | 11 | 12 | 13 | 14 | 15 | 16 | 18 | 20 | 22 | 24 | 29 | 38 | 58 | 97 | 148 | 193 | 193 | 141 | 89 | 61 | 51 | 46 | 41 | 37 | 33 | 29 | 27 | 24 | 17 |
| 2.5 | 4 | 5 | 7 | 8 | 9 | 9 | 10 | 11 | 11 | 12 | 13 | 14 | 16 | 19 | 23 | 29 | 39 | 58 | 93 | 154 | 181 | 147 | 87 | 61 | 51 | 45 | 41 | 37 | 30 | 28 | 25 | 18 |
| 3.0 | 2 | 3 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 10 | 11 | 13 | 15 | 19 | 23 | 28 | 39 | 72 | 124 | 170 | 138 | 86 | 61 | 50 | 45 | 40 | 33 | 29 | 26 | 19 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 61 | 195 | 343 | 232 | 129 | 113 | 103 | 91 | 81 | 76 | 71 | 66 | 64 | 62 | 61 | 59 | 56 | 54 | 51 | 47 | 43 | 40 | 40 | 39 | 39 | 37 | 36 | 31 | 21 |
| .10 | 0 | 0 | 0 | 12 | 45 | 145 | 277 | 247 | 169 | 131 | 112 | 98 | 87 | 76 | 70 | 65 | 64 | 62 | 60 | 57 | 55 | 53 | 49 | 44 | 40 | 40 | 39 | 39 | 38 | 36 | 32 | 21 |
| .20 | 0 | 0 | 0 | 9 | 33 | 107 | 220 | 238 | 192 | 151 | 125 | 107 | 94 | 79 | 71 | 66 | 64 | 62 | 61 | 58 | 56 | 53 | 49 | 45 | 41 | 40 | 40 | 39 | 38 | 36 | 32 | 21 |
| .30 | 0 | 0 | 0 | 1 | 6 | 24 | 79 | 173 | 216 | 200 | 168 | 139 | 118 | 90 | 77 | 70 | 66 | 64 | 62 | 59 | 57 | 54 | 50 | 46 | 41 | 40 | 40 | 39 | 38 | 36 | 32 | 22 |
| .40 | 0 | 0 | 0 | 1 | 4 | 17 | 59 | 135 | 189 | 196 | 177 | 152 | 129 | 97 | 81 | 72 | 67 | 64 | 63 | 60 | 57 | 54 | 51 | 46 | 42 | 40 | 40 | 39 | 38 | 36 | 32 | 22 |
| .50 | 0 | 0 | 0 | 0 | 0 | 3 | 12 | 43 | 104 | 161 | 185 | 180 | 161 | 121 | 93 | 79 | 71 | 66 | 64 | 61 | 58 | 55 | 52 | 48 | 43 | 40 | 40 | 39 | 38 | 37 | 33 | 22 |
| .75 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 18 | 49 | 92 | 130 | 153 | 159 | 142 | 114 | 92 | 79 | 71 | 66 | 63 | 60 | 56 | 53 | 49 | 44 | 41 | 40 | 40 | 38 | 37 | 33 | 23 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 27 | 56 | 92 | 144 | 152 | 128 | 103 | 86 | 75 | 67 | 63 | 59 | 55 | 51 | 47 | 43 | 40 | 40 | 39 | 37 | 34 | 24 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 32 | 79 | 121 | 136 | 127 | 109 | 85 | 72 | 64 | 58 | 55 | 51 | 47 | 43 | 40 | 39 | 38 | 34 | 25 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 33 | 70 | 105 | 126 | 118 | 97 | 75 | 63 | 58 | 54 | 50 | 46 | 42 | 40 | 39 | 35 | 26 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 11 | 32 | 63 | 105 | 118 | 100 | 75 | 63 | 58 | 54 | 50 | 46 | 40 | 39 | 36 | 27 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 12 | 43 | 84 | 112 | 96 | 74 | 63 | 58 | 54 | 50 | 42 | 40 | 37 | 28 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 3 | 13 | 28 | 39 | 44 | 45 | 46 | 46 | 48 | 49 | 51 | 53 | 54 | 54 | 55 | 56 | 55 | 53 | 50 | 50 | 49 | 49 | 49 | 49 | 48 | 45 | 32 |
| .10 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 22 | 33 | 41 | 44 | 45 | 46 | 48 | 49 | 50 | 52 | 54 | 54 | 54 | 56 | 55 | 53 | 50 | 50 | 49 | 49 | 49 | 49 | 48 | 45 | 32 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | 17 | 28 | 36 | 41 | 44 | 46 | 48 | 49 | 51 | 53 | 54 | 54 | 56 | 56 | 54 | 51 | 50 | 50 | 49 | 49 | 49 | 48 | 45 | 32 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 13 | 23 | 32 | 38 | 44 | 47 | 48 | 50 | 51 | 53 | 54 | 54 | 55 | 54 | 52 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 10 | 18 | 27 | 34 | 43 | 46 | 48 | 49 | 51 | 53 | 54 | 54 | 55 | 54 | 52 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 15 | 23 | 36 | 43 | 46 | 48 | 50 | 51 | 53 | 54 | 55 | 55 | 53 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 34 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 13 | 27 | 37 | 43 | 46 | 48 | 50 | 52 | 54 | 55 | 55 | 54 | 51 | 50 | 50 | 49 | 49 | 48 | 46 | 34 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 20 | 32 | 40 | 45 | 47 | 50 | 52 | 54 | 55 | 54 | 52 | 50 | 50 | 49 | 49 | 49 | 47 | 36 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 13 | 23 | 33 | 42 | 47 | 51 | 54 | 55 | 54 | 53 | 50 | 50 | 49 | 49 | 48 | 37 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 12 | 25 | 37 | 46 | 51 | 54 | 55 | 54 | 52 | 50 | 49 | 49 | 48 | 39 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 15 | 31 | 45 | 51 | 53 | 55 | 54 | 52 | 50 | 49 | 48 | 41 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 14 | 32 | 45 | 51 | 53 | 54 | 54 | 51 | 49 | 48 | 42 |
| | RAINFALL TYPE = I | | | | | | | | | | | | | * * * TC = 0.1 HR * * * | | | | | | | | | | | | | | | | | SHEET 1 OF 10 |

## Exhibit 5-I, continued: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 9.0 | 9.3 | 9.6 | 9.9 | 10.0 | 10.1 | 10.2 | 10.3 | 10.4 | 10.5 | 10.6 | 10.7 | 10.8 | 11.0 | 11.2 | 11.4 | 11.6 | 11.8 | 12.0 | 12.3 | 12.6 | 13.0 | 13.5 | 14.0 | 14.5 | 15.0 | 15.5 | 16.0 | 17.0 | 18.0 | 20.0 | 24.0 |
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.2 HR * * * | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 28 | 37 | 52 | 126 | 220 | 379 | 405 | 267 | 168 | 129 | 108 | 92 | 80 | 69 | 62 | 58 | 55 | 53 | 50 | 47 | 44 | 41 | 37 | 33 | 30 | 29 | 28 | 27 | 26 | 24 | 21 | 13 |
| .10 | 24 | 32 | 44 | 71 | 108 | 182 | 313 | 375 | 303 | 213 | 157 | 124 | 103 | 78 | 68 | 61 | 57 | 55 | 52 | 49 | 46 | 42 | 38 | 34 | 30 | 29 | 29 | 28 | 26 | 24 | 21 | 14 |
| .20 | 21 | 28 | 38 | 53 | 65 | 94 | 153 | 260 | 336 | 314 | 247 | 187 | 145 | 97 | 77 | 67 | 60 | 57 | 54 | 50 | 47 | 44 | 39 | 35 | 31 | 30 | 29 | 28 | 26 | 25 | 21 | 14 |
| .30 | 20 | 27 | 36 | 50 | 60 | 82 | 129 | 216 | 296 | 308 | 267 | 214 | 168 | 110 | 82 | 69 | 62 | 58 | 55 | 51 | 47 | 44 | 40 | 36 | 32 | 30 | 29 | 28 | 26 | 25 | 21 | 14 |
| .40 | 17 | 23 | 31 | 42 | 47 | 56 | 74 | 111 | 181 | 258 | 291 | 275 | 234 | 152 | 103 | 79 | 68 | 61 | 57 | 53 | 49 | 45 | 41 | 37 | 33 | 30 | 29 | 28 | 27 | 25 | 22 | 14 |
| .50 | 16 | 22 | 30 | 40 | 45 | 52 | 66 | 96 | 153 | 223 | 269 | 273 | 247 | 171 | 115 | 86 | 71 | 63 | 58 | 54 | 50 | 46 | 42 | 37 | 33 | 30 | 29 | 28 | 27 | 25 | 22 | 14 |
| .75 | 13 | 17 | 24 | 32 | 35 | 39 | 44 | 52 | 68 | 99 | 145 | 194 | 229 | 240 | 183 | 132 | 97 | 77 | 67 | 58 | 53 | 48 | 44 | 39 | 35 | 31 | 30 | 29 | 27 | 26 | 22 | 15 |
| 1.0 | 11 | 13 | 17 | 24 | 26 | 29 | 32 | 35 | 39 | 45 | 56 | 75 | 107 | 189 | 229 | 206 | 158 | 115 | 87 | 67 | 58 | 52 | 46 | 42 | 38 | 34 | 31 | 29 | 28 | 26 | 23 | 15 |
| 1.5 | 8 | 10 | 12 | 15 | 17 | 19 | 21 | 23 | 25 | 28 | 31 | 35 | 39 | 60 | 106 | 166 | 204 | 197 | 161 | 112 | 79 | 60 | 51 | 46 | 42 | 37 | 33 | 30 | 28 | 27 | 23 | 16 |
| 2.0 | 5 | 7 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18 | 20 | 22 | 27 | 34 | 49 | 79 | 126 | 161 | 188 | 152 | 97 | 64 | 52 | 46 | 42 | 38 | 34 | 29 | 28 | 24 | 17 |
| 2.5 | 3 | 5 | 6 | 8 | 8 | 9 | 10 | 10 | 11 | 12 | 13 | 14 | 15 | 18 | 22 | 26 | 34 | 49 | 77 | 136 | 176 | 155 | 95 | 64 | 52 | 46 | 42 | 37 | 31 | 28 | 25 | 18 |
| 3.0 | 2 | 3 | 4 | 5 | 6 | 6 | 7 | 7 | 8 | 8 | 9 | 9 | 10 | 11 | 13 | 16 | 19 | 23 | 29 | 49 | 91 | 154 | 167 | 102 | 67 | 53 | 47 | 42 | 34 | 29 | 26 | 19 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.2 HR * * * | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 22 | 76 | 206 | 258 | 207 | 144 | 119 | 104 | 92 | 82 | 74 | 68 | 65 | 63 | 62 | 60 | 57 | 55 | 52 | 48 | 44 | 40 | 40 | 39 | 39 | 38 | 36 | 32 | 21 |
| .10 | 0 | 0 | 0 | 3 | 16 | 56 | 156 | 224 | 213 | 167 | 135 | 114 | 99 | 81 | 73 | 67 | 65 | 63 | 61 | 58 | 56 | 53 | 50 | 45 | 41 | 40 | 40 | 39 | 38 | 36 | 32 | 22 |
| .20 | 0 | 0 | 0 | 2 | 11 | 41 | 118 | 189 | 205 | 179 | 150 | 126 | 108 | 85 | 75 | 69 | 65 | 63 | 62 | 59 | 56 | 54 | 50 | 46 | 41 | 40 | 40 | 39 | 38 | 36 | 32 | 22 |
| .30 | 0 | 0 | 0 | 0 | 2 | 8 | 30 | 88 | 155 | 188 | 182 | 161 | 138 | 103 | 83 | 74 | 68 | 65 | 63 | 60 | 57 | 55 | 51 | 47 | 42 | 40 | 40 | 39 | 38 | 36 | 32 | 22 |
| .40 | 0 | 0 | 0 | 0 | 1 | 6 | 22 | 66 | 126 | 167 | 177 | 166 | 147 | 111 | 88 | 76 | 69 | 66 | 63 | 61 | 58 | 55 | 51 | 47 | 43 | 40 | 40 | 39 | 38 | 37 | 33 | 22 |
| .50 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 16 | 50 | 100 | 145 | 167 | 166 | 136 | 105 | 85 | 75 | 69 | 65 | 62 | 59 | 56 | 53 | 49 | 44 | 41 | 40 | 40 | 38 | 37 | 33 | 23 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 22 | 50 | 87 | 119 | 140 | 148 | 124 | 101 | 85 | 74 | 69 | 64 | 61 | 57 | 54 | 50 | 45 | 42 | 40 | 40 | 38 | 37 | 33 | 23 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 11 | 28 | 55 | 114 | 142 | 135 | 113 | 93 | 80 | 69 | 64 | 60 | 56 | 52 | 48 | 44 | 41 | 40 | 39 | 37 | 34 | 24 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 17 | 52 | 97 | 124 | 129 | 115 | 91 | 75 | 65 | 59 | 55 | 52 | 48 | 43 | 41 | 39 | 38 | 35 | 25 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 19 | 49 | 84 | 111 | 120 | 102 | 79 | 65 | 59 | 55 | 51 | 47 | 43 | 40 | 39 | 36 | 26 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 20 | 45 | 89 | 113 | 104 | 78 | 65 | 59 | 55 | 51 | 47 | 41 | 39 | 36 | 28 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 30 | 68 | 108 | 99 | 77 | 65 | 58 | 54 | 50 | 43 | 40 | 37 | 29 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.2 HR * * * | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 15 | 28 | 38 | 43 | 45 | 45 | 48 | 49 | 50 | 52 | 53 | 53 | 53 | 53 | 53 | 53 | 50 | 50 | 50 | 49 | 49 | 49 | 48 | 45 | 32 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 11 | 22 | 32 | 40 | 43 | 45 | 47 | 48 | 50 | 52 | 53 | 53 | 53 | 53 | 53 | 53 | 50 | 50 | 50 | 49 | 49 | 49 | 48 | 45 | 32 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 17 | 27 | 36 | 41 | 43 | 46 | 48 | 49 | 51 | 53 | 53 | 53 | 53 | 53 | 53 | 51 | 50 | 50 | 49 | 49 | 49 | 48 | 45 | 33 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 13 | 23 | 31 | 38 | 41 | 45 | 48 | 49 | 51 | 52 | 53 | 53 | 53 | 53 | 53 | 51 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 19 | 27 | 34 | 39 | 44 | 47 | 49 | 50 | 52 | 53 | 53 | 53 | 53 | 53 | 51 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 15 | 23 | 30 | 36 | 43 | 46 | 48 | 50 | 51 | 53 | 53 | 53 | 53 | 53 | 52 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 8 | 14 | 20 | 27 | 37 | 43 | 46 | 48 | 50 | 52 | 53 | 53 | 53 | 53 | 52 | 50 | 50 | 50 | 49 | 49 | 48 | 46 | 34 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 14 | 26 | 36 | 42 | 46 | 48 | 51 | 52 | 53 | 53 | 53 | 52 | 50 | 50 | 49 | 49 | 49 | 47 | 35 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 13 | 23 | 33 | 40 | 46 | 49 | 52 | 53 | 53 | 53 | 51 | 50 | 50 | 49 | 49 | 48 | 37 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 16 | 26 | 37 | 44 | 50 | 52 | 53 | 53 | 53 | 51 | 50 | 49 | 49 | 48 | 38 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 19 | 31 | 43 | 50 | 52 | 53 | 53 | 52 | 51 | 50 | 49 | 48 | 39 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 11 | 25 | 41 | 49 | 52 | 53 | 53 | 52 | 50 | 49 | 48 | 41 |
| | RAINFALL TYPE = I | | | | | | | | | | | | | * * * TC = 0.2 HR * * * | | | | | | | | | | | | | | | | SHEET 2 OF 10 | |

## Exhibit 5-I, continued: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

| TRVL TIME (HR) | 9.0 | 9.3 | 9.6 | 9.9 | 10.0 | 10.1 | 10.2 | 10.3 | 10.4 | 10.5 | 10.6 | 10.7 | 10.8 | 11.0 | 11.2 | 11.4 | 11.6 | 11.8 | 12.0 | 12.3 | 12.6 | 13.0 | 13.5 | 14.0 | 14.5 | 15.0 | 15.5 | 16.0 | 17.0 | 18.0 | 20.0 | 24.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.3 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 25 | 34 | 46 | 87 | 138 | 242 | 349 | 346 | 256 | 182 | 141 | 115 | 97 | 76 | 66 | 60 | 57 | 54 | 52 | 48 | 45 | 42 | 38 | 34 | 30 | 29 | 28 | 28 | 26 | 24 | 21 | 13 |
| .10 | 24 | 32 | 44 | 77 | 118 | 200 | 299 | 330 | 281 | 215 | 166 | 132 | 109 | 82 | 69 | 62 | 58 | 55 | 52 | 49 | 46 | 42 | 38 | 34 | 30 | 29 | 29 | 28 | 26 | 24 | 21 | 14 |
| .20 | 21 | 28 | 38 | 54 | 69 | 102 | 167 | 255 | 305 | 289 | 240 | 190 | 152 | 103 | 79 | 67 | 61 | 57 | 54 | 50 | 47 | 44 | 40 | 35 | 31 | 30 | 29 | 28 | 26 | 25 | 21 | 14 |
| .30 | 20 | 27 | 36 | 51 | 63 | 89 | 141 | 217 | 276 | 295 | 255 | 212 | 172 | 115 | 85 | 71 | 63 | 58 | 55 | 51 | 48 | 44 | 40 | 36 | 32 | 30 | 29 | 28 | 26 | 25 | 21 | 14 |
| .40 | 17 | 23 | 31 | 42 | 48 | 58 | 79 | 120 | 185 | 246 | 272 | 260 | 228 | 156 | 108 | 82 | 69 | 62 | 57 | 53 | 49 | 45 | 41 | 37 | 33 | 30 | 29 | 28 | 27 | 25 | 22 | 14 |
| .50 | 16 | 22 | 30 | 40 | 45 | 54 | 70 | 104 | 158 | 216 | 253 | 258 | 238 | 173 | 120 | 89 | 72 | 63 | 58 | 54 | 50 | 46 | 42 | 37 | 33 | 30 | 29 | 28 | 27 | 25 | 22 | 14 |
| .75 | 13 | 17 | 24 | 32 | 35 | 39 | 45 | 54 | 72 | 104 | 147 | 189 | 219 | 231 | 182 | 135 | 100 | 79 | 67 | 58 | 53 | 48 | 44 | 40 | 35 | 32 | 30 | 29 | 27 | 26 | 22 | 15 |
| 1.0 | 11 | 13 | 17 | 24 | 26 | 29 | 32 | 35 | 40 | 46 | 58 | 79 | 110 | 184 | 221 | 202 | 158 | 118 | 90 | 68 | 58 | 52 | 46 | 42 | 38 | 34 | 31 | 29 | 28 | 26 | 23 | 15 |
| 1.5 | 8 | 10 | 12 | 15 | 17 | 19 | 21 | 23 | 25 | 28 | 31 | 35 | 40 | 62 | 107 | 163 | 199 | 193 | 165 | 114 | 81 | 61 | 52 | 46 | 42 | 37 | 33 | 31 | 28 | 27 | 23 | 16 |
| 2.0 | 5 | 7 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18 | 20 | 22 | 27 | 35 | 50 | 80 | 125 | 165 | 184 | 152 | 99 | 64 | 53 | 47 | 42 | 38 | 34 | 29 | 28 | 24 | 17 |
| 2.5 | 3 | 5 | 6 | 8 | 8 | 9 | 10 | 10 | 11 | 12 | 13 | 14 | 15 | 18 | 22 | 27 | 35 | 50 | 78 | 134 | 173 | 154 | 96 | 64 | 52 | 46 | 42 | 37 | 31 | 28 | 25 | 18 |
| 3.0 | 2 | 3 | 4 | 6 | 6 | 7 | 7 | 8 | 8 | 9 | 9 | 10 | 11 | 12 | 14 | 17 | 21 | 26 | 34 | 61 | 107 | 164 | 145 | 94 | 64 | 52 | 46 | 41 | 33 | 29 | 26 | 19 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.3 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 8 | 32 | 98 | 198 | 217 | 192 | 148 | 125 | 108 | 95 | 79 | 72 | 67 | 64 | 63 | 61 | 58 | 56 | 53 | 49 | 45 | 41 | 40 | 40 | 39 | 38 | 36 | 32 | 22 |
| .10 | 0 | 0 | 0 | 1 | 6 | 23 | 73 | 156 | 197 | 194 | 164 | 138 | 118 | 91 | 77 | 70 | 66 | 64 | 62 | 59 | 57 | 54 | 50 | 46 | 41 | 40 | 40 | 39 | 38 | 36 | 32 | 22 |
| .20 | 0 | 0 | 0 | 1 | 4 | 17 | 54 | 122 | 172 | 187 | 171 | 149 | 128 | 98 | 81 | 72 | 67 | 64 | 63 | 60 | 57 | 54 | 51 | 46 | 42 | 40 | 40 | 39 | 38 | 36 | 32 | 22 |
| .30 | 0 | 0 | 0 | 0 | 0 | 3 | 12 | 40 | 95 | 146 | 173 | 172 | 157 | 120 | 94 | 79 | 71 | 67 | 64 | 61 | 58 | 55 | 52 | 48 | 43 | 40 | 40 | 39 | 38 | 37 | 33 | 23 |
| .40 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 30 | 73 | 122 | 156 | 167 | 160 | 128 | 100 | 83 | 73 | 68 | 65 | 62 | 59 | 56 | 52 | 48 | 44 | 41 | 40 | 39 | 38 | 37 | 33 | 23 |
| .50 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 22 | 56 | 100 | 137 | 157 | 159 | 135 | 107 | 87 | 76 | 69 | 65 | 62 | 59 | 56 | 53 | 49 | 44 | 41 | 40 | 40 | 38 | 37 | 33 | 23 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 9 | 26 | 53 | 86 | 115 | 143 | 134 | 113 | 93 | 80 | 72 | 65 | 62 | 58 | 54 | 51 | 46 | 42 | 40 | 40 | 39 | 37 | 33 | 23 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 14 | 32 | 57 | 111 | 138 | 132 | 113 | 94 | 81 | 69 | 64 | 60 | 56 | 52 | 48 | 44 | 41 | 40 | 39 | 37 | 34 | 24 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 34 | 74 | 111 | 126 | 121 | 99 | 80 | 67 | 61 | 56 | 53 | 49 | 44 | 41 | 39 | 38 | 35 | 26 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 11 | 33 | 66 | 97 | 118 | 108 | 84 | 67 | 60 | 56 | 52 | 48 | 44 | 40 | 39 | 36 | 27 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 12 | 32 | 74 | 106 | 112 | 83 | 67 | 60 | 56 | 52 | 48 | 41 | 39 | 37 | 28 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 21 | 54 | 98 | 107 | 81 | 67 | 60 | 55 | 51 | 43 | 40 | 37 | 29 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.3 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 16 | 27 | 36 | 41 | 44 | 46 | 48 | 49 | 51 | 52 | 52 | 52 | 52 | 52 | 52 | 51 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 12 | 22 | 31 | 38 | 42 | 45 | 48 | 49 | 51 | 52 | 52 | 52 | 52 | 52 | 52 | 51 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 9 | 18 | 27 | 34 | 39 | 44 | 47 | 49 | 50 | 51 | 52 | 52 | 52 | 52 | 52 | 51 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 14 | 23 | 30 | 36 | 43 | 46 | 48 | 50 | 51 | 52 | 52 | 52 | 52 | 52 | 51 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 11 | 19 | 26 | 33 | 41 | 46 | 48 | 49 | 51 | 52 | 52 | 52 | 52 | 52 | 52 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 9 | 15 | 23 | 30 | 39 | 44 | 47 | 49 | 50 | 51 | 52 | 52 | 52 | 52 | 52 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 8 | 14 | 20 | 31 | 40 | 44 | 47 | 49 | 51 | 52 | 52 | 52 | 52 | 52 | 50 | 50 | 50 | 49 | 49 | 48 | 46 | 34 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 9 | 20 | 31 | 39 | 44 | 47 | 50 | 51 | 52 | 52 | 52 | 52 | 50 | 50 | 50 | 49 | 49 | 47 | 36 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 9 | 18 | 28 | 36 | 44 | 48 | 51 | 52 | 52 | 52 | 51 | 50 | 50 | 49 | 49 | 48 | 37 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 12 | 21 | 33 | 42 | 48 | 51 | 52 | 52 | 52 | 51 | 50 | 49 | 49 | 48 | 38 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 15 | 27 | 40 | 48 | 51 | 52 | 52 | 52 | 51 | 50 | 49 | 48 | 40 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 11 | 25 | 41 | 48 | 51 | 52 | 52 | 51 | 50 | 49 | 48 | 41 |
| | RAINFALL TYPE = I | | | | | | | | | | | | | * * * TC = 0.3 HR * * * | | | | | | | | | | | | | | | | | SHEET 3 OF 10 |

## Exhibit 5-I, continued: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

| TRVL TIME (HR) | 9.0 | 9.3 | 9.6 | 9.9 | 10.0 | 10.1 | 10.2 | 10.3 | 10.4 | 10.5 | 10.6 | 10.7 | 10.8 | 11.0 | 11.2 | 11.4 | 11.6 | 11.8 | 12.0 | 12.3 | 12.6 | 13.0 | 13.5 | 14.0 | 14.5 | 15.0 | 15.5 | 16.0 | 17.0 | 18.0 | 20.0 | 24.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | IA/P = 0.10 | | | | | | | | | | | | * * * TC =0.4 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 23 | 31 | 42 | 66 | 96 | 157 | 250 | 310 | 304 | 244 | 186 | 149 | 122 | 89 | 73 | 64 | 59 | 56 | 53 | 49 | 46 | 43 | 39 | 35 | 31 | 29 | 29 | 28 | 26 | 25 | 21 | 14 |
| .10 | 22 | 29 | 40 | 61 | 84 | 133 | 211 | 277 | 295 | 261 | 211 | 170 | 138 | 98 | 78 | 66 | 60 | 56 | 54 | 50 | 47 | 43 | 39 | 35 | 31 | 29 | 29 | 28 | 26 | 25 | 21 | 14 |
| .20 | 19 | 26 | 34 | 47 | 56 | 75 | 114 | 178 | 244 | 278 | 267 | 230 | 190 | 128 | 93 | 75 | 65 | 59 | 56 | 52 | 48 | 44 | 40 | 36 | 32 | 30 | 29 | 28 | 27 | 25 | 21 | 14 |
| .30 | 18 | 24 | 33 | 45 | 53 | 67 | 98 | 152 | 213 | 257 | 263 | 241 | 207 | 143 | 102 | 80 | 68 | 61 | 57 | 52 | 49 | 45 | 41 | 37 | 32 | 30 | 29 | 28 | 27 | 25 | 21 | 14 |
| .40 | 15 | 21 | 28 | 38 | 43 | 49 | 61 | 86 | 130 | 185 | 233 | 253 | 245 | 188 | 132 | 97 | 77 | 66 | 60 | 55 | 51 | 46 | 42 | 38 | 34 | 30 | 29 | 28 | 27 | 25 | 22 | 14 |
| .50 | 15 | 20 | 27 | 36 | 41 | 46 | 56 | 76 | 112 | 161 | 209 | 238 | 243 | 201 | 146 | 106 | 82 | 69 | 61 | 55 | 51 | 47 | 42 | 38 | 34 | 31 | 29 | 29 | 27 | 25 | 22 | 14 |
| .75 | 12 | 16 | 21 | 29 | 32 | 35 | 39 | 46 | 57 | 77 | 108 | 147 | 184 | 220 | 200 | 157 | 118 | 91 | 74 | 61 | 55 | 50 | 45 | 40 | 36 | 32 | 30 | 29 | 27 | 26 | 22 | 15 |
| 1.0 | 10 | 12 | 16 | 21 | 24 | 26 | 29 | 32 | 36 | 40 | 48 | 61 | 83 | 147 | 202 | 212 | 178 | 138 | 105 | 75 | 62 | 54 | 47 | 43 | 39 | 35 | 31 | 29 | 28 | 26 | 23 | 16 |
| 1.5 | 8 | 9 | 11 | 14 | 16 | 17 | 19 | 21 | 23 | 25 | 28 | 31 | 35 | 50 | 83 | 134 | 179 | 193 | 177 | 131 | 92 | 65 | 53 | 47 | 42 | 38 | 34 | 31 | 29 | 27 | 24 | 16 |
| 2.0 | 5 | 6 | 8 | 10 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18 | 20 | 25 | 31 | 42 | 64 | 102 | 144 | 179 | 163 | 111 | 70 | 55 | 48 | 43 | 39 | 35 | 30 | 28 | 25 | 17 |
| 2.5 | 3 | 4 | 6 | 7 | 8 | 8 | 9 | 9 | 10 | 11 | 12 | 13 | 14 | 16 | 20 | 24 | 30 | 42 | 63 | 114 | 160 | 170 | 107 | 70 | 54 | 47 | 43 | 38 | 31 | 29 | 25 | 18 |
| 3.0 | 2 | 3 | 4 | 5 | 6 | 6 | 7 | 7 | 8 | 8 | 9 | 9 | 10 | 11 | 13 | 16 | 19 | 23 | 30 | 51 | 90 | 148 | 161 | 104 | 69 | 54 | 47 | 42 | 34 | 29 | 26 | 19 |
| | | IA/P = 0.30 | | | | | | | | | | | | * * * TC =0.4 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 3 | 14 | 48 | 115 | 184 | 192 | 178 | 148 | 127 | 111 | 88 | 77 | 70 | 66 | 64 | 62 | 59 | 56 | 54 | 50 | 46 | 41 | 40 | 40 | 39 | 38 | 36 | 32 | 22 |
| .10 | 0 | 0 | 0 | 0 | 2 | 10 | 35 | 89 | 152 | 179 | 178 | 158 | 137 | 105 | 85 | 75 | 69 | 65 | 63 | 60 | 58 | 55 | 51 | 47 | 42 | 40 | 40 | 39 | 38 | 36 | 32 | 22 |
| .20 | 0 | 0 | 0 | 0 | 2 | 7 | 26 | 68 | 124 | 161 | 172 | 163 | 146 | 113 | 90 | 78 | 70 | 66 | 64 | 61 | 58 | 55 | 52 | 47 | 43 | 40 | 40 | 39 | 38 | 37 | 33 | 22 |
| .30 | 0 | 0 | 0 | 0 | 1 | 5 | 19 | 52 | 100 | 140 | 162 | 162 | 151 | 120 | 96 | 81 | 72 | 67 | 64 | 61 | 59 | 55 | 52 | 48 | 43 | 41 | 40 | 39 | 38 | 37 | 33 | 23 |
| .40 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 14 | 39 | 80 | 120 | 148 | 158 | 142 | 114 | 92 | 79 | 71 | 67 | 63 | 60 | 57 | 53 | 49 | 45 | 41 | 40 | 40 | 38 | 37 | 33 | 23 |
| .50 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 10 | 29 | 63 | 101 | 132 | 152 | 145 | 120 | 97 | 82 | 73 | 68 | 63 | 60 | 57 | 53 | 50 | 45 | 41 | 40 | 40 | 38 | 37 | 33 | 23 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 13 | 31 | 58 | 87 | 130 | 138 | 123 | 103 | 87 | 77 | 68 | 63 | 59 | 55 | 51 | 47 | 43 | 41 | 40 | 39 | 37 | 34 | 24 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 18 | 36 | 86 | 125 | 134 | 122 | 104 | 88 | 73 | 66 | 61 | 57 | 53 | 49 | 45 | 41 | 40 | 39 | 38 | 34 | 24 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 10 | 36 | 75 | 109 | 124 | 120 | 100 | 81 | 68 | 61 | 56 | 53 | 49 | 44 | 41 | 40 | 38 | 35 | 26 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 22 | 50 | 83 | 116 | 116 | 91 | 70 | 61 | 57 | 53 | 49 | 45 | 40 | 39 | 36 | 27 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 22 | 60 | 97 | 111 | 88 | 70 | 61 | 56 | 53 | 48 | 42 | 39 | 37 | 28 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 14 | 43 | 89 | 106 | 86 | 69 | 61 | 56 | 52 | 44 | 40 | 37 | 29 |
| | | IA/P = 0.50 | | | | | | | | | | | | * * * TC =0.4 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 17 | 27 | 34 | 40 | 44 | 47 | 49 | 50 | 51 | 51 | 51 | 51 | 51 | 51 | 51 | 50 | 50 | 49 | 49 | 49 | 48 | 46 | 33 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 13 | 22 | 30 | 40 | 45 | 47 | 49 | 50 | 51 | 51 | 51 | 51 | 51 | 51 | 50 | 50 | 50 | 49 | 49 | 48 | 46 | 34 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 18 | 33 | 41 | 45 | 48 | 49 | 51 | 51 | 51 | 51 | 51 | 51 | 50 | 50 | 50 | 49 | 49 | 49 | 46 | 34 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 22 | 35 | 42 | 46 | 48 | 50 | 51 | 51 | 51 | 51 | 51 | 51 | 50 | 50 | 49 | 49 | 49 | 47 | 35 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 19 | 32 | 40 | 45 | 47 | 49 | 51 | 51 | 51 | 51 | 51 | 51 | 50 | 50 | 49 | 49 | 49 | 47 | 35 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 23 | 34 | 41 | 45 | 48 | 50 | 51 | 51 | 51 | 51 | 51 | 50 | 50 | 49 | 49 | 49 | 47 | 35 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 14 | 26 | 35 | 42 | 46 | 49 | 50 | 51 | 51 | 51 | 51 | 50 | 50 | 50 | 49 | 49 | 47 | 36 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 14 | 25 | 35 | 44 | 48 | 50 | 51 | 51 | 51 | 51 | 50 | 50 | 49 | 49 | 48 | 37 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 13 | 27 | 39 | 47 | 50 | 51 | 51 | 51 | 51 | 50 | 49 | 49 | 48 | 39 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 21 | 36 | 47 | 50 | 51 | 51 | 51 | 51 | 50 | 49 | 48 | 40 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 19 | 37 | 47 | 50 | 51 | 51 | 51 | 50 | 49 | 48 | 42 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 25 | 40 | 48 | 50 | 51 | 51 | 50 | 50 | 49 | 43 |
| | | RAINFALL TYPE = I | | | | | | | | | | | | * * * TC = 0.4 HR * * * | | | | | | | | | | | | | | | | | | SHEET 4 OF 10 |

## Exhibit 5-I, continued: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 9.0 | 9.3 | 9.6 | 9.9 | 10.0 | 10.1 | 10.2 | 10.3 | 10.4 | 10.5 | 10.6 | 10.7 | 10.8 | 11.0 | 11.2 | 11.4 | 11.6 | 11.8 | 12.0 | 12.3 | 12.6 | 13.0 | 13.5 | 14.0 | 14.5 | 15.0 | 15.5 | 16.0 | 17.0 | 18.0 | 20.0 | 24.0 | | | | |
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | | | | | IA/P = 0.10 | | | |
| 0.0 | 21 | 28 | 38 | 56 | 73 | 111 | 175 | 247 | 282 | 271 | 229 | 186 | 152 | 108 | 84 | 71 | 63 | 58 | 55 | 51 | 47 | 44 | 40 | 35 | 31 | 30 | 29 | 28 | 26 | 25 | 21 | 14 | | | | |
| .10 | 20 | 27 | 36 | 52 | 66 | 96 | 149 | 214 | 259 | 267 | 242 | 204 | 169 | 119 | 90 | 74 | 65 | 59 | 55 | 51 | 48 | 44 | 40 | 36 | 32 | 30 | 29 | 28 | 26 | 25 | 21 | 14 | | | | |
| .20 | 17 | 23 | 31 | 42 | 49 | 60 | 84 | 127 | 185 | 234 | 256 | 247 | 219 | 156 | 112 | 86 | 72 | 63 | 58 | 53 | 49 | 45 | 41 | 37 | 33 | 30 | 29 | 28 | 27 | 25 | 22 | 14 | | | | |
| .30 | 16 | 22 | 30 | 40 | 46 | 55 | 74 | 110 | 160 | 210 | 241 | 245 | 227 | 171 | 123 | 93 | 76 | 66 | 59 | 54 | 50 | 46 | 42 | 38 | 33 | 30 | 29 | 28 | 27 | 25 | 22 | 14 | | | | |
| .40 | 14 | 19 | 26 | 35 | 38 | 43 | 51 | 67 | 95 | 138 | 186 | 222 | 237 | 210 | 157 | 115 | 89 | 73 | 64 | 57 | 52 | 47 | 43 | 39 | 35 | 31 | 29 | 29 | 27 | 25 | 22 | 15 | | | | |
| .50 | 14 | 18 | 25 | 33 | 37 | 41 | 48 | 61 | 84 | 120 | 164 | 203 | 229 | 216 | 170 | 126 | 96 | 77 | 66 | 58 | 53 | 48 | 43 | 39 | 35 | 31 | 30 | 29 | 27 | 25 | 22 | 15 | | | | |
| .75 | 11 | 14 | 19 | 26 | 29 | 32 | 35 | 40 | 48 | 61 | 83 | 114 | 148 | 202 | 210 | 176 | 137 | 106 | 84 | 66 | 57 | 51 | 46 | 41 | 37 | 33 | 30 | 29 | 28 | 26 | 22 | 15 | | | | |
| 1.0 | 10 | 11 | 15 | 19 | 21 | 24 | 26 | 29 | 32 | 36 | 41 | 50 | 65 | 116 | 176 | 203 | 190 | 156 | 122 | 85 | 67 | 56 | 49 | 44 | 40 | 35 | 32 | 30 | 28 | 26 | 23 | 16 | | | | |
| 1.5 | 7 | 9 | 11 | 13 | 14 | 16 | 17 | 19 | 21 | 23 | 25 | 28 | 31 | 42 | 67 | 110 | 157 | 187 | 183 | 146 | 104 | 72 | 55 | 48 | 43 | 39 | 35 | 32 | 29 | 27 | 24 | 17 | | | | |
| 2.0 | 4 | 6 | 7 | 9 | 10 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18 | 22 | 27 | 36 | 53 | 83 | 123 | 175 | 175 | 124 | 77 | 57 | 49 | 44 | 40 | 35 | 30 | 28 | 25 | 18 | | | | |
| 2.5 | 3 | 4 | 5 | 7 | 7 | 8 | 8 | 9 | 9 | 10 | 11 | 12 | 13 | 15 | 18 | 22 | 27 | 36 | 53 | 97 | 146 | 166 | 119 | 76 | 57 | 49 | 44 | 39 | 32 | 29 | 25 | 18 | | | | |
| 3.0 | 1 | 2 | 3 | 5 | 5 | 6 | 6 | 6 | 7 | 8 | 8 | 9 | 9 | 11 | 12 | 15 | 17 | 21 | 27 | 43 | 76 | 135 | 158 | 114 | 75 | 57 | 48 | 43 | 35 | 30 | 26 | 19 | | | | |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | | | | | IA/P = 0.30 | | | |
| 0.0 | 0 | 0 | 0 | 1 | 6 | 22 | 64 | 123 | 168 | 174 | 165 | 143 | 126 | 100 | 84 | 74 | 68 | 65 | 63 | 60 | 57 | 54 | 51 | 47 | 42 | 40 | 40 | 39 | 38 | 36 | 32 | 22 | | | | |
| .10 | 0 | 0 | 0 | 1 | 5 | 16 | 48 | 98 | 144 | 164 | 165 | 150 | 134 | 106 | 87 | 77 | 70 | 66 | 63 | 60 | 58 | 55 | 51 | 47 | 42 | 40 | 40 | 39 | 38 | 36 | 32 | 22 | | | | |
| .20 | 0 | 0 | 0 | 0 | 1 | 3 | 12 | 36 | 77 | 122 | 150 | 160 | 154 | 126 | 101 | 85 | 75 | 69 | 65 | 62 | 59 | 56 | 52 | 48 | 44 | 41 | 40 | 39 | 38 | 37 | 33 | 23 | | | | |
| .30 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 27 | 60 | 101 | 134 | 151 | 153 | 132 | 107 | 89 | 77 | 70 | 66 | 63 | 60 | 56 | 53 | 49 | 44 | 41 | 40 | 40 | 38 | 37 | 33 | 23 | | | | |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 20 | 47 | 83 | 117 | 140 | 148 | 125 | 102 | 86 | 76 | 69 | 64 | 61 | 57 | 54 | 50 | 46 | 42 | 40 | 40 | 39 | 37 | 33 | 23 | | | | |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 15 | 36 | 68 | 101 | 127 | 144 | 130 | 108 | 90 | 78 | 71 | 65 | 62 | 58 | 54 | 50 | 46 | 42 | 40 | 40 | 39 | 37 | 33 | 23 | | | | |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 17 | 36 | 60 | 87 | 125 | 133 | 121 | 103 | 88 | 78 | 68 | 63 | 59 | 55 | 52 | 47 | 43 | 41 | 40 | 39 | 37 | 34 | 24 | | | | |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 9 | 21 | 62 | 106 | 130 | 126 | 112 | 96 | 78 | 69 | 62 | 57 | 54 | 50 | 46 | 42 | 40 | 39 | 38 | 34 | 25 | | | | |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 23 | 56 | 92 | 115 | 120 | 106 | 87 | 71 | 62 | 57 | 53 | 49 | 45 | 42 | 40 | 38 | 35 | 26 | | | | |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 14 | 36 | 67 | 104 | 113 | 96 | 74 | 63 | 58 | 54 | 50 | 46 | 40 | 39 | 36 | 27 | | | | |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 15 | 47 | 85 | 109 | 93 | 73 | 63 | 57 | 53 | 49 | 42 | 40 | 37 | 28 | | | | |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 32 | 77 | 105 | 90 | 72 | 62 | 57 | 53 | 45 | 40 | 38 | 30 | | | | |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | | | | | IA/P = 0.50 | | | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 18 | 26 | 33 | 41 | 45 | 47 | 49 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 49 | 49 | 48 | 46 | 33 | | | | |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 15 | 22 | 30 | 39 | 44 | 47 | 49 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 49 | 49 | 48 | 46 | 34 | | | | |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 12 | 19 | 26 | 37 | 43 | 46 | 48 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 49 | 49 | 48 | 46 | 34 | | | | |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 9 | 16 | 23 | 34 | 41 | 45 | 48 | 49 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 49 | 49 | 49 | 46 | 34 | | | | |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 13 | 19 | 31 | 39 | 44 | 47 | 49 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 49 | 49 | 49 | 46 | 34 | | | | |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 10 | 16 | 28 | 37 | 43 | 46 | 48 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 49 | 49 | 49 | 46 | 34 | | | | |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 10 | 20 | 30 | 38 | 43 | 46 | 48 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 49 | 49 | 49 | 47 | 35 | | | | |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 10 | 20 | 30 | 38 | 43 | 47 | 49 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 49 | 49 | 47 | 36 | | | | |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 14 | 23 | 31 | 41 | 46 | 49 | 50 | 50 | 50 | 50 | 50 | 50 | 49 | 49 | 48 | 38 | | | | |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 13 | 25 | 36 | 45 | 49 | 50 | 50 | 50 | 50 | 50 | 49 | 49 | 48 | 39 | | | | |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 12 | 23 | 37 | 46 | 49 | 50 | 50 | 50 | 50 | 50 | 49 | 48 | 40 | | | | |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 21 | 37 | 46 | 49 | 50 | 50 | 50 | 50 | 50 | 49 | 48 | 42 | | | | |
| | RAINFALL TYPE = I | | | | | | | | | | | | | * * * TC = 0.5 HR * * * | | | | | | | | | | | | | | | | | | | SHEET 5 OF 10 | | | |

## Exhibit 5-I, continued: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

| TRVL TIME (HR) | 9.0 | 9.3 | 9.6 | 9.9 | 10.0 | 10.1 | 10.2 | 10.3 | 10.4 | 10.5 | 10.6 | 10.7 | 10.8 | 11.0 | 11.2 | 11.4 | 11.6 | 11.8 | 12.0 | 12.3 | 12.6 | 13.0 | 13.5 | 14.0 | 14.5 | 15.0 | 15.5 | 16.0 | 17.0 | 18.0 | 20.0 | 24.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.75 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| C.0 | 16 | 22 | 30 | 41 | 47 | 59 | 82 | 119 | 167 | 209 | 229 | 233 | 212 | 161 | 122 | 97 | 80 | 69 | 62 | 56 | 51 | 46 | 42 | 38 | 33 | 31 | 29 | 28 | 27 | 25 | 22 | 14 |
| .10 | 14 | 19 | 26 | 35 | 39 | 44 | 54 | 72 | 104 | 146 | 188 | 215 | 227 | 197 | 150 | 116 | 93 | 77 | 68 | 59 | 53 | 48 | 43 | 39 | 35 | 31 | 30 | 29 | 27 | 25 | 22 | 15 |
| .20 | 13 | 16 | 22 | 30 | 33 | 37 | 42 | 50 | 65 | 91 | 128 | 168 | 200 | 218 | 183 | 142 | 110 | 89 | 75 | 63 | 56 | 49 | 44 | 40 | 36 | 32 | 30 | 29 | 27 | 26 | 22 | 15 |
| .30 | 12 | 16 | 21 | 28 | 31 | 35 | 39 | 46 | 59 | 80 | 112 | 149 | 183 | 214 | 191 | 151 | 118 | 94 | 79 | 65 | 57 | 50 | 45 | 41 | 36 | 32 | 30 | 29 | 27 | 26 | 22 | 15 |
| .40 | 12 | 15 | 20 | 27 | 30 | 33 | 37 | 43 | 54 | 71 | 98 | 132 | 166 | 207 | 197 | 161 | 126 | 100 | 83 | 67 | 58 | 51 | 45 | 41 | 37 | 33 | 30 | 29 | 27 | 26 | 22 | 15 |
| .50 | 11 | 13 | 18 | 24 | 26 | 29 | 32 | 36 | 41 | 49 | 64 | 87 | 117 | 178 | 204 | 186 | 152 | 120 | 96 | 74 | 62 | 53 | 47 | 42 | 38 | 34 | 31 | 29 | 28 | 26 | 23 | 15 |
| .75 | 10 | 12 | 15 | 21 | 23 | 25 | 28 | 31 | 35 | 40 | 49 | 63 | 83 | 136 | 180 | 190 | 171 | 142 | 115 | 85 | 69 | 57 | 49 | 44 | 39 | 35 | 32 | 30 | 28 | 26 | 23 | 16 |
| 1.0 | 8 | 10 | 12 | 16 | 17 | 19 | 21 | 23 | 25 | 28 | 31 | 35 | 42 | 66 | 110 | 158 | 186 | 180 | 157 | 116 | 86 | 65 | 53 | 46 | 42 | 38 | 34 | 31 | 28 | 27 | 23 | 16 |
| 1.5 | 5 | 7 | 8 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18 | 20 | 22 | 27 | 35 | 52 | 83 | 124 | 159 | 174 | 147 | 102 | 69 | 54 | 47 | 42 | 38 | 34 | 30 | 28 | 24 | 17 |
| 2.0 | 3 | 5 | 6 | 8 | 8 | 9 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 18 | 22 | 27 | 36 | 52 | 80 | 131 | 165 | 148 | 99 | 68 | 54 | 47 | 42 | 38 | 31 | 28 | 25 | 18 |
| 2.5 | 2 | 3 | 4 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 11 | 12 | 14 | 17 | 21 | 26 | 35 | 63 | 107 | 157 | 141 | 96 | 67 | 54 | 46 | 42 | 34 | 29 | 26 | 19 |
| 3.0 | 1 | 1 | 2 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 10 | 11 | 13 | 15 | 19 | 26 | 42 | 86 | 151 | 141 | 101 | 71 | 55 | 47 | 38 | 31 | 27 | 20 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.75 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| C.0 | 0 | 0 | 0 | 0 | 1 | 4 | 14 | 35 | 69 | 105 | 131 | 143 | 146 | 124 | 104 | 89 | 79 | 72 | 68 | 64 | 60 | 56 | 53 | 49 | 44 | 41 | 40 | 40 | 38 | 37 | 33 | 23 |
| .10 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 10 | 27 | 55 | 88 | 117 | 134 | 142 | 118 | 100 | 87 | 77 | 71 | 66 | 62 | 58 | 54 | 50 | 46 | 42 | 40 | 40 | 39 | 37 | 33 | 23 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 20 | 43 | 73 | 102 | 123 | 137 | 123 | 105 | 90 | 80 | 73 | 66 | 62 | 58 | 54 | 50 | 46 | 42 | 40 | 40 | 39 | 37 | 33 | 23 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 15 | 34 | 60 | 88 | 128 | 134 | 118 | 101 | 88 | 78 | 69 | 64 | 60 | 55 | 52 | 48 | 43 | 41 | 40 | 39 | 37 | 34 | 24 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 12 | 27 | 49 | 75 | 118 | 131 | 121 | 105 | 91 | 80 | 71 | 65 | 60 | 56 | 52 | 48 | 44 | 41 | 40 | 39 | 37 | 34 | 24 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 9 | 21 | 39 | 87 | 122 | 128 | 116 | 101 | 88 | 75 | 68 | 62 | 57 | 53 | 49 | 45 | 42 | 40 | 39 | 38 | 34 | 24 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 21 | 56 | 95 | 118 | 121 | 110 | 98 | 82 | 72 | 64 | 58 | 54 | 50 | 46 | 42 | 41 | 39 | 38 | 34 | 25 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 12 | 39 | 76 | 106 | 119 | 115 | 98 | 82 | 69 | 61 | 57 | 53 | 49 | 44 | 42 | 40 | 38 | 35 | 26 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 14 | 37 | 67 | 95 | 112 | 105 | 85 | 69 | 61 | 56 | 52 | 48 | 44 | 40 | 39 | 36 | 27 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 15 | 35 | 74 | 102 | 107 | 84 | 69 | 61 | 56 | 52 | 48 | 41 | 39 | 37 | 28 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 23 | 55 | 94 | 103 | 82 | 68 | 60 | 56 | 52 | 44 | 40 | 37 | 29 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 15 | 50 | 93 | 100 | 81 | 68 | 60 | 55 | 47 | 41 | 38 | 30 | |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.75 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| C.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 17 | 29 | 37 | 42 | 45 | 48 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 47 | 34 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 8 | 14 | 26 | 35 | 41 | 45 | 47 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 47 | 35 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 11 | 23 | 33 | 39 | 44 | 47 | 48 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 47 | 35 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 20 | 30 | 38 | 43 | 46 | 48 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 47 | 35 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 8 | 17 | 28 | 36 | 41 | 45 | 47 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 47 | 35 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 15 | 25 | 34 | 40 | 44 | 47 | 48 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 47 | 35 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 3 | 9 | 18 | 27 | 35 | 40 | 44 | 47 | 48 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 47 | 36 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 18 | 27 | 35 | 40 | 45 | 48 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 48 | 37 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 13 | 21 | 29 | 38 | 44 | 48 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 48 | 38 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 9 | 15 | 26 | 36 | 44 | 48 | 49 | 49 | 49 | 49 | 49 | 49 | 49 | 48 | 39 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 11 | 21 | 34 | 44 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 40 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 19 | 35 | 44 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 42 |
| | RAINFALL TYPE = I | | | | | | | | | | | | | * * * TC = 0.75 HR * * * | | | | | | | | | | | | | | | | | SHEET 6 OF 10 |

## Exhibit 5-I, continued: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

| TRVL TIME (HR) | 9.0 | 9.3 | 9.6 | 9.9 | 10.0 | 10.1 | 10.2 | 10.3 | 10.4 | 10.5 | 10.6 | 10.7 | 10.8 | 11.0 | 11.2 | 11.4 | 11.6 | 11.8 | 12.0 | 12.3 | 12.6 | 13.0 | 13.5 | 14.0 | 14.5 | 15.0 | 15.5 | 16.0 | 17.0 | 18.0 | 20.0 | 24.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =1.0 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 14 | 19 | 25 | 34 | 38 | 46 | 58 | 79 | 106 | 136 | 165 | 186 | 202 | 185 | 153 | 122 | 101 | 85 | 74 | 63 | 56 | 50 | 44 | 40 | 35 | 32 | 30 | 29 | 27 | 26 | 22 | 15 |
| .10 | 13 | 18 | 24 | 32 | 36 | 42 | 53 | 70 | 94 | 122 | 151 | 174 | 194 | 188 | 160 | 130 | 106 | 89 | 77 | 65 | 57 | 50 | 44 | 40 | 36 | 32 | 30 | 29 | 27 | 26 | 22 | 15 |
| .20 | 12 | 15 | 21 | 28 | 31 | 34 | 40 | 49 | 63 | 83 | 109 | 137 | 162 | 190 | 180 | 152 | 124 | 102 | 86 | 70 | 61 | 53 | 46 | 41 | 37 | 33 | 31 | 29 | 27 | 26 | 22 | 15 |
| .30 | 12 | 15 | 20 | 26 | 29 | 33 | 37 | 45 | 57 | 75 | 98 | 124 | 149 | 187 | 183 | 159 | 131 | 107 | 90 | 73 | 62 | 53 | 47 | 42 | 37 | 33 | 31 | 29 | 28 | 26 | 23 | 15 |
| .40 | 10 | 13 | 17 | 23 | 25 | 28 | 31 | 35 | 42 | 52 | 67 | 88 | 112 | 160 | 183 | 176 | 151 | 125 | 103 | 81 | 67 | 56 | 48 | 43 | 39 | 35 | 31 | 30 | 28 | 26 | 23 | 15 |
| .50 | 10 | 13 | 16 | 22 | 24 | 27 | 30 | 33 | 39 | 48 | 61 | 79 | 101 | 148 | 181 | 181 | 157 | 131 | 108 | 84 | 69 | 58 | 49 | 43 | 39 | 35 | 32 | 30 | 28 | 26 | 23 | 16 |
| .75 | 9 | 11 | 14 | 19 | 21 | 23 | 26 | 29 | 33 | 38 | 47 | 59 | 75 | 115 | 153 | 172 | 167 | 148 | 125 | 96 | 77 | 62 | 51 | 45 | 40 | 36 | 33 | 30 | 28 | 27 | 23 | 16 |
| 1.0 | 8 | 9 | 11 | 15 | 16 | 17 | 19 | 21 | 23 | 26 | 29 | 33 | 40 | 61 | 95 | 134 | 162 | 169 | 157 | 126 | 97 | 72 | 57 | 49 | 43 | 39 | 35 | 32 | 29 | 27 | 24 | 16 |
| 1.5 | 5 | 6 | 8 | 10 | 10 | 11 | 12 | 13 | 14 | 16 | 17 | 19 | 21 | 25 | 33 | 48 | 74 | 107 | 138 | 160 | 148 | 111 | 76 | 59 | 50 | 44 | 39 | 35 | 30 | 28 | 25 | 17 |
| 2.0 | 3 | 4 | 6 | 7 | 8 | 8 | 9 | 9 | 10 | 11 | 12 | 13 | 14 | 17 | 20 | 25 | 33 | 48 | 71 | 115 | 153 | 153 | 108 | 75 | 58 | 49 | 43 | 39 | 32 | 29 | 25 | 18 |
| 2.5 | 2 | 3 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 12 | 14 | 16 | 20 | 25 | 33 | 56 | 94 | 139 | 147 | 104 | 74 | 58 | 49 | 43 | 34 | 30 | 26 | 19 |
| 3.0 | 1 | 1 | 2 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 9 | 10 | 12 | 14 | 17 | 24 | 38 | 76 | 131 | 143 | 108 | 78 | 60 | 50 | 39 | 32 | 27 | 20 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =1.0 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 0 | 1 | 2 | 7 | 17 | 34 | 55 | 79 | 99 | 114 | 128 | 114 | 100 | 89 | 80 | 74 | 68 | 63 | 59 | 54 | 51 | 46 | 43 | 41 | 40 | 39 | 37 | 33 | 23 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 13 | 27 | 46 | 68 | 89 | 116 | 124 | 110 | 98 | 87 | 79 | 71 | 65 | 60 | 56 | 52 | 48 | 43 | 41 | 40 | 39 | 37 | 34 | 24 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 21 | 37 | 58 | 78 | 109 | 121 | 113 | 101 | 90 | 81 | 72 | 66 | 61 | 56 | 52 | 48 | 44 | 41 | 40 | 39 | 37 | 34 | 24 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 17 | 31 | 49 | 87 | 113 | 118 | 109 | 98 | 87 | 76 | 69 | 63 | 57 | 53 | 49 | 45 | 42 | 40 | 39 | 38 | 34 | 24 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 13 | 25 | 41 | 78 | 107 | 117 | 111 | 101 | 90 | 78 | 70 | 63 | 58 | 54 | 50 | 45 | 42 | 41 | 39 | 38 | 34 | 25 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 10 | 20 | 34 | 69 | 100 | 115 | 113 | 103 | 93 | 80 | 71 | 64 | 58 | 54 | 50 | 46 | 42 | 41 | 39 | 38 | 34 | 25 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 31 | 61 | 90 | 107 | 110 | 104 | 90 | 79 | 69 | 61 | 56 | 52 | 48 | 44 | 41 | 40 | 38 | 35 | 25 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 12 | 33 | 61 | 89 | 105 | 109 | 99 | 86 | 73 | 64 | 58 | 54 | 50 | 45 | 42 | 40 | 38 | 35 | 26 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 13 | 31 | 55 | 80 | 104 | 104 | 88 | 73 | 63 | 58 | 53 | 49 | 45 | 41 | 39 | 36 | 27 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 20 | 51 | 82 | 100 | 90 | 74 | 64 | 58 | 54 | 50 | 42 | 40 | 37 | 28 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 13 | 37 | 76 | 98 | 88 | 73 | 64 | 58 | 53 | 45 | 41 | 38 | 30 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 34 | 77 | 95 | 86 | 73 | 63 | 58 | 49 | 42 | 38 | 31 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =1.0 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 5 | 9 | 18 | 28 | 35 | 40 | 44 | 47 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 47 | 35 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 7 | 16 | 25 | 33 | 39 | 43 | 46 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 47 | 35 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 6 | 14 | 23 | 31 | 38 | 42 | 45 | 47 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 47 | 36 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 5 | 12 | 21 | 29 | 36 | 41 | 44 | 47 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 47 | 36 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 10 | 19 | 27 | 34 | 40 | 44 | 47 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 47 | 36 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 8 | 16 | 25 | 33 | 38 | 43 | 46 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 47 | 36 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 8 | 15 | 23 | 30 | 36 | 41 | 45 | 47 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 47 | 36 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 11 | 19 | 27 | 33 | 41 | 45 | 47 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 37 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 11 | 17 | 28 | 37 | 44 | 47 | 48 | 48 | 48 | 48 | 48 | 48 | 48 | 39 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 16 | 26 | 38 | 45 | 47 | 48 | 48 | 48 | 48 | 48 | 48 | 40 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 12 | 24 | 38 | 45 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 41 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 14 | 29 | 40 | 46 | 47 | 47 | 47 | 47 | 47 | 47 | 42 |
| | RAINFALL TYPE = I | | | | | | | | | | | | | * * * TC = 1.0 HR * * * | | | | | | | | | | | | | | | | | SHEET 7 OF 10 |

# Exhibit 5-I, continued: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

| TRVL TIME (HR) | 9.0 | 9.3 | 9.6 | 9.9 | 10.0 | 10.1 | 10.2 | 10.3 | 10.4 | 10.5 | 10.6 | 10.7 | 10.8 | 11.0 | 11.2 | 11.4 | 11.6 | 11.8 | 12.0 | 12.3 | 12.6 | 13.0 | 13.5 | 14.0 | 14.5 | 15.0 | 15.5 | 16.0 | 17.0 | 18.0 | 20.0 | 24.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | IA/P = 0.10 | | | | | | | | | | | | * * * TC =1.25 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 12 | 16 | 22 | 29 | 33 | 38 | 47 | 61 | 80 | 103 | 127 | 149 | 164 | 180 | 163 | 138 | 115 | 98 | 85 | 71 | 62 | 53 | 47 | 41 | 37 | 33 | 31 | 29 | 27 | 26 | 22 | 15 |
| .10 | 11 | 14 | 19 | 25 | 28 | 31 | 36 | 44 | 55 | 72 | 92 | 116 | 138 | 167 | 175 | 156 | 132 | 111 | 95 | 77 | 66 | 56 | 48 | 43 | 38 | 34 | 31 | 30 | 28 | 26 | 23 | 15 |
| .20 | 11 | 14 | 18 | 24 | 27 | 30 | 34 | 40 | 50 | 65 | 83 | 105 | 127 | 160 | 172 | 160 | 138 | 116 | 99 | 80 | 68 | 57 | 49 | 43 | 39 | 34 | 31 | 30 | 28 | 26 | 23 | 15 |
| .30 | 10 | 12 | 16 | 21 | 23 | 25 | 28 | 32 | 38 | 46 | 58 | 75 | 95 | 136 | 164 | 169 | 154 | 132 | 112 | 89 | 74 | 61 | 51 | 45 | 40 | 36 | 32 | 30 | 28 | 26 | 23 | 16 |
| .40 | 9 | 12 | 15 | 20 | 22 | 24 | 27 | 30 | 35 | 42 | 53 | 68 | 86 | 126 | 157 | 167 | 157 | 137 | 117 | 92 | 76 | 62 | 52 | 46 | 40 | 36 | 33 | 30 | 28 | 26 | 23 | 16 |
| .50 | 9 | 11 | 13 | 17 | 19 | 21 | 23 | 26 | 29 | 33 | 39 | 49 | 61 | 97 | 134 | 160 | 165 | 152 | 132 | 104 | 84 | 67 | 55 | 47 | 42 | 37 | 33 | 31 | 28 | 27 | 23 | 16 |
| .75 | 7 | 9 | 11 | 14 | 15 | 17 | 18 | 20 | 22 | 25 | 28 | 32 | 38 | 59 | 90 | 124 | 150 | 159 | 152 | 126 | 101 | 77 | 60 | 51 | 45 | 40 | 35 | 32 | 29 | 27 | 24 | 17 |
| 1.0 | 6 | 8 | 10 | 12 | 13 | 14 | 15 | 17 | 19 | 20 | 23 | 25 | 29 | 40 | 60 | 91 | 123 | 148 | 157 | 144 | 118 | 88 | 66 | 54 | 47 | 41 | 37 | 33 | 29 | 27 | 24 | 17 |
| 1.5 | 4 | 6 | 7 | 9 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 18 | 22 | 28 | 40 | 59 | 85 | 114 | 145 | 150 | 121 | 86 | 65 | 54 | 46 | 41 | 37 | 31 | 28 | 25 | 18 |
| 2.0 | 2 | 3 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 11 | 11 | 13 | 16 | 19 | 24 | 33 | 47 | 80 | 119 | 144 | 124 | 90 | 68 | 55 | 47 | 42 | 33 | 29 | 26 | 19 |
| 2.5 | 1 | 2 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 8 | 10 | 11 | 13 | 16 | 19 | 24 | 38 | 65 | 111 | 140 | 120 | 88 | 67 | 54 | 47 | 37 | 31 | 27 | 20 |
| 3.0 | 1 | 1 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 5 | 6 | 7 | 8 | 9 | 11 | 13 | 15 | 21 | 32 | 62 | 114 | 136 | 116 | 86 | 66 | 54 | 41 | 33 | 27 | 20 |
| | | IA/P = 0.30 | | | | | | | | | | | | * * * TC =1.25 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 21 | 35 | 53 | 71 | 86 | 106 | 115 | 104 | 94 | 86 | 79 | 72 | 66 | 61 | 56 | 52 | 48 | 44 | 41 | 40 | 39 | 37 | 34 | 24 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 16 | 29 | 45 | 62 | 92 | 107 | 113 | 101 | 92 | 84 | 75 | 69 | 63 | 58 | 53 | 49 | 45 | 42 | 41 | 39 | 38 | 34 | 24 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 13 | 23 | 37 | 54 | 84 | 104 | 110 | 104 | 94 | 86 | 77 | 70 | 64 | 58 | 54 | 49 | 45 | 42 | 41 | 39 | 38 | 34 | 24 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 10 | 19 | 31 | 62 | 90 | 106 | 109 | 101 | 92 | 81 | 73 | 66 | 60 | 55 | 51 | 46 | 43 | 41 | 39 | 38 | 34 | 25 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 15 | 26 | 55 | 83 | 102 | 107 | 103 | 94 | 83 | 74 | 67 | 60 | 56 | 51 | 47 | 43 | 41 | 39 | 38 | 35 | 25 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 12 | 33 | 62 | 88 | 103 | 106 | 100 | 88 | 78 | 69 | 62 | 57 | 53 | 48 | 44 | 42 | 40 | 38 | 35 | 25 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 12 | 31 | 56 | 80 | 97 | 103 | 97 | 87 | 75 | 65 | 59 | 55 | 50 | 46 | 43 | 40 | 39 | 35 | 26 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 13 | 32 | 56 | 79 | 95 | 101 | 94 | 80 | 69 | 61 | 56 | 52 | 48 | 44 | 40 | 39 | 36 | 27 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 13 | 30 | 51 | 81 | 98 | 94 | 79 | 68 | 61 | 56 | 52 | 47 | 42 | 39 | 36 | 28 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 13 | 37 | 66 | 95 | 95 | 78 | 68 | 61 | 56 | 51 | 44 | 40 | 37 | 29 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 26 | 61 | 93 | 89 | 77 | 67 | 60 | 55 | 47 | 41 | 38 | 30 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 24 | 64 | 91 | 87 | 76 | 66 | 60 | 50 | 43 | 39 | 31 |
| | | IA/P = 0.50 | | | | | | | | | | | | * * * TC =1.25 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 6 | 13 | 21 | 29 | 35 | 40 | 43 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 36 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 5 | 11 | 19 | 27 | 34 | 39 | 42 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 36 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 9 | 17 | 25 | 32 | 37 | 41 | 46 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 36 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 8 | 15 | 23 | 30 | 36 | 40 | 45 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 36 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 7 | 13 | 21 | 28 | 35 | 39 | 45 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 37 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 12 | 19 | 27 | 33 | 38 | 44 | 46 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 37 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 14 | 21 | 28 | 34 | 41 | 45 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 37 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 11 | 17 | 24 | 34 | 41 | 46 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 38 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 19 | 29 | 39 | 45 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 40 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 9 | 18 | 30 | 41 | 46 | 47 | 47 | 47 | 47 | 47 | 47 | 47 | 41 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 17 | 31 | 41 | 46 | 47 | 47 | 47 | 47 | 47 | 47 | 42 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 18 | 32 | 42 | 46 | 47 | 47 | 47 | 47 | 47 | 43 |
| | | RAINFALL TYPE = I | | | | | | | | | | | | * * * TC = 1.25 HR * * * | | | | | | | | | | | | | | | | | SHEET 8 OF 10 | |

## Exhibit 5-I, continued: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 9.0 | 9.3 | 9.6 | 9.9 | 10.0 | 10.1 | 10.2 | 10.3 | 10.4 | 10.5 | 10.6 | 10.7 | 10.8 | 11.0 | 11.2 | 11.4 | 11.6 | 11.8 | 12.0 | 12.3 | 12.6 | 13.0 | 13.5 | 14.0 | 14.5 | 15.0 | 15.5 | 16.0 | 17.0 | 18.0 | 20.0 | 24.0 |
| | IA/P = 0.10 | | | | | | | | | | | * * * TC =1.5 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 11 | 14 | 19 | 25 | 28 | 32 | 39 | 48 | 60 | 76 | 94 | 112 | 129 | 151 | 163 | 147 | 130 | 112 | 97 | 81 | 69 | 59 | 50 | 44 | 39 | 35 | 32 | 30 | 28 | 26 | 23 | 15 |
| .10 | 10 | 12 | 16 | 22 | 24 | 27 | 30 | 36 | 44 | 55 | 69 | 86 | 103 | 135 | 152 | 159 | 143 | 125 | 108 | 88 | 75 | 62 | 52 | 46 | 41 | 36 | 32 | 30 | 28 | 26 | 23 | 16 |
| .20 | 9 | 11 | 14 | 19 | 21 | 23 | 25 | 29 | 34 | 41 | 50 | 63 | 78 | 112 | 140 | 155 | 152 | 138 | 121 | 98 | 81 | 67 | 55 | 48 | 42 | 37 | 33 | 31 | 28 | 27 | 23 | 16 |
| .30 | 9 | 11 | 14 | 18 | 20 | 22 | 24 | 27 | 31 | 38 | 46 | 57 | 71 | 104 | 133 | 151 | 154 | 141 | 125 | 101 | 84 | 68 | 56 | 48 | 43 | 38 | 34 | 31 | 29 | 27 | 23 | 16 |
| .40 | 8 | 10 | 13 | 17 | 19 | 21 | 23 | 26 | 30 | 35 | 42 | 52 | 65 | 96 | 126 | 147 | 152 | 144 | 129 | 105 | 87 | 70 | 57 | 49 | 43 | 38 | 34 | 31 | 29 | 27 | 23 | 16 |
| .50 | 8 | 9 | 12 | 15 | 17 | 18 | 20 | 22 | 25 | 28 | 33 | 39 | 48 | 73 | 104 | 131 | 148 | 151 | 140 | 117 | 95 | 75 | 60 | 51 | 45 | 40 | 35 | 32 | 29 | 27 | 24 | 17 |
| .75 | 7 | 8 | 11 | 13 | 15 | 16 | 18 | 19 | 21 | 24 | 27 | 32 | 38 | 56 | 82 | 110 | 133 | 146 | 144 | 127 | 106 | 83 | 65 | 54 | 47 | 41 | 36 | 33 | 29 | 27 | 24 | 17 |
| 1.0 | 5 | 7 | 9 | 11 | 11 | 12 | 13 | 15 | 16 | 18 | 19 | 22 | 24 | 32 | 47 | 69 | 96 | 122 | 139 | 145 | 127 | 99 | 74 | 60 | 51 | 44 | 39 | 35 | 30 | 28 | 24 | 17 |
| 1.5 | 4 | 5 | 6 | 8 | 8 | 9 | 10 | 10 | 11 | 12 | 13 | 14 | 16 | 19 | 24 | 32 | 46 | 66 | 90 | 124 | 139 | 128 | 97 | 73 | 59 | 50 | 44 | 39 | 32 | 29 | 25 | 18 |
| 2.0 | 2 | 3 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 12 | 14 | 17 | 21 | 27 | 38 | 63 | 96 | 135 | 129 | 100 | 76 | 61 | 51 | 44 | 35 | 30 | 26 | 19 |
| 2.5 | 1 | 2 | 2 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 10 | 11 | 14 | 16 | 20 | 31 | 51 | 91 | 132 | 124 | 98 | 75 | 60 | 51 | 39 | 32 | 27 | 20 |
| 3.0 | 0 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 12 | 16 | 23 | 42 | 86 | 123 | 128 | 101 | 77 | 62 | 45 | 35 | 28 | 21 |
| | IA/P = 0.30 | | | | | | | | | | | * * * TC =1.5 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 13 | 21 | 33 | 46 | 60 | 83 | 97 | 106 | 97 | 90 | 84 | 76 | 70 | 64 | 59 | 54 | 50 | 45 | 43 | 41 | 39 | 38 | 34 | 25 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 18 | 28 | 40 | 66 | 86 | 98 | 103 | 95 | 88 | 80 | 73 | 66 | 60 | 55 | 51 | 47 | 43 | 41 | 39 | 38 | 35 | 25 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 8 | 14 | 23 | 34 | 60 | 81 | 95 | 101 | 97 | 90 | 81 | 74 | 67 | 61 | 56 | 52 | 47 | 44 | 41 | 40 | 38 | 35 | 25 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 12 | 19 | 41 | 65 | 85 | 98 | 100 | 95 | 85 | 77 | 69 | 62 | 57 | 53 | 48 | 44 | 42 | 40 | 38 | 35 | 25 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 16 | 36 | 59 | 80 | 94 | 99 | 96 | 87 | 79 | 70 | 63 | 58 | 53 | 49 | 45 | 42 | 40 | 38 | 35 | 26 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 8 | 13 | 31 | 54 | 75 | 91 | 98 | 97 | 88 | 80 | 71 | 64 | 58 | 54 | 49 | 45 | 42 | 40 | 38 | 35 | 26 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 13 | 28 | 49 | 69 | 85 | 95 | 95 | 87 | 76 | 67 | 61 | 56 | 51 | 47 | 44 | 40 | 39 | 36 | 26 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 13 | 29 | 49 | 69 | 84 | 95 | 92 | 81 | 71 | 63 | 58 | 53 | 49 | 45 | 41 | 39 | 36 | 27 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 9 | 20 | 35 | 63 | 84 | 92 | 83 | 72 | 64 | 58 | 54 | 49 | 43 | 40 | 37 | 28 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 9 | 25 | 50 | 79 | 90 | 81 | 71 | 63 | 58 | 53 | 45 | 41 | 38 | 29 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 18 | 46 | 79 | 88 | 80 | 70 | 63 | 57 | 49 | 42 | 38 | 31 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 17 | 50 | 79 | 87 | 79 | 70 | 62 | 52 | 45 | 39 | 32 |
| | IA/P = 0.50 | | | | | | | | | | | * * * TC =1.5 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 8 | 15 | 22 | 29 | 34 | 39 | 44 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 37 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 17 | 24 | 30 | 35 | 41 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 37 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 4 | 9 | 15 | 22 | 28 | 34 | 40 | 45 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 37 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 13 | 20 | 27 | 33 | 39 | 44 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 38 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 12 | 18 | 25 | 31 | 38 | 44 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 38 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 17 | 23 | 30 | 37 | 43 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 38 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 12 | 19 | 25 | 33 | 40 | 45 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 38 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 15 | 25 | 33 | 42 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 39 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 14 | 23 | 34 | 43 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 40 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 11 | 22 | 35 | 43 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 42 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 10 | 23 | 36 | 43 | 46 | 46 | 46 | 46 | 46 | 46 | 43 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 14 | 27 | 38 | 44 | 46 | 46 | 46 | 46 | 46 | 44 |
| | RAINFALL TYPE = I | | | | | | | | | | | * * * TC = 1.5 HR * * * | | | | | | | | | | | | | | | | | | SHEET 9 OF 10 | |

## Exhibit 5-I, continued: Tabular hydrograph unit discharges (csm/in) for type I rainfall distribution

| TRVL TIME (HR) | 9.0 | 9.3 | 9.6 | 9.9 | 10.0 | 10.1 | 10.2 | 10.3 | 10.4 | 10.5 | 10.6 | 10.7 | 10.8 | 11.0 | 11.2 | 11.4 | 11.6 | 11.8 | 12.0 | 12.3 | 12.6 | 13.0 | 13.5 | 14.0 | 14.5 | 15.0 | 15.5 | 16.0 | 17.0 | 18.0 | 20.0 | 24.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =2.0 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 9 | 11 | 14 | 19 | 21 | 24 | 28 | 33 | 40 | 49 | 59 | 70 | 82 | 106 | 123 | 138 | 138 | 125 | 115 | 97 | 83 | 70 | 59 | 50 | 44 | 39 | 35 | 32 | 29 | 27 | 23 | 16 |
| .10 | 8 | 10 | 13 | 17 | 18 | 20 | 23 | 26 | 31 | 37 | 45 | 54 | 65 | 89 | 110 | 125 | 135 | 130 | 123 | 106 | 90 | 75 | 62 | 53 | 46 | 40 | 36 | 33 | 29 | 27 | 24 | 16 |
| .20 | 8 | 10 | 12 | 16 | 17 | 19 | 22 | 25 | 29 | 34 | 41 | 50 | 60 | 83 | 105 | 121 | 132 | 131 | 125 | 109 | 92 | 77 | 63 | 53 | 46 | 41 | 36 | 33 | 29 | 27 | 24 | 17 |
| .30 | 7 | 9 | 11 | 14 | 15 | 17 | 18 | 21 | 23 | 27 | 32 | 38 | 46 | 66 | 89 | 109 | 123 | 131 | 129 | 117 | 100 | 82 | 66 | 56 | 48 | 42 | 38 | 34 | 30 | 28 | 24 | 17 |
| .40 | 7 | 8 | 10 | 13 | 15 | 16 | 18 | 20 | 22 | 25 | 30 | 35 | 43 | 61 | 83 | 104 | 120 | 131 | 131 | 119 | 103 | 84 | 68 | 57 | 49 | 43 | 38 | 34 | 30 | 28 | 24 | 17 |
| .50 | 6 | 8 | 10 | 13 | 14 | 15 | 17 | 19 | 21 | 24 | 28 | 33 | 39 | 56 | 78 | 99 | 116 | 127 | 130 | 121 | 106 | 86 | 69 | 58 | 50 | 44 | 39 | 35 | 30 | 28 | 24 | 17 |
| .75 | 5 | 7 | 8 | 11 | 12 | 13 | 14 | 15 | 16 | 18 | 20 | 23 | 27 | 38 | 53 | 73 | 93 | 111 | 123 | 127 | 118 | 98 | 77 | 63 | 54 | 47 | 41 | 37 | 31 | 28 | 25 | 17 |
| 1.0 | 4 | 5 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18 | 24 | 32 | 45 | 63 | 83 | 102 | 122 | 126 | 113 | 89 | 71 | 59 | 51 | 44 | 39 | 32 | 29 | 25 | 18 |
| 1.5 | 3 | 4 | 5 | 6 | 7 | 7 | 8 | 8 | 9 | 10 | 10 | 11 | 12 | 15 | 18 | 23 | 32 | 44 | 60 | 88 | 111 | 123 | 110 | 87 | 70 | 58 | 50 | 44 | 35 | 30 | 26 | 19 |
| 2.0 | 1 | 2 | 3 | 4 | 5 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 10 | 12 | 14 | 18 | 23 | 31 | 49 | 74 | 106 | 121 | 107 | 86 | 69 | 58 | 49 | 38 | 32 | 27 | 20 |
| 2.5 | 1 | 1 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 11 | 13 | 16 | 22 | 35 | 62 | 101 | 118 | 108 | 88 | 71 | 59 | 44 | 35 | 28 | 21 |
| 3.0 | 0 | 0 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 3 | 4 | 5 | 5 | 6 | 8 | 9 | 10 | 14 | 19 | 34 | 67 | 103 | 116 | 105 | 86 | 70 | 50 | 39 | 29 | 21 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =2.0 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 10 | 16 | 23 | 31 | 49 | 65 | 77 | 84 | 92 | 86 | 80 | 75 | 69 | 63 | 58 | 53 | 49 | 45 | 43 | 40 | 38 | 35 | 26 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 13 | 19 | 35 | 53 | 68 | 79 | 85 | 90 | 83 | 77 | 71 | 64 | 59 | 55 | 50 | 46 | 43 | 40 | 39 | 35 | 26 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 7 | 11 | 24 | 40 | 57 | 71 | 81 | 89 | 89 | 80 | 73 | 66 | 61 | 56 | 51 | 47 | 44 | 41 | 39 | 35 | 26 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 3 | 6 | 9 | 20 | 36 | 53 | 68 | 78 | 84 | 88 | 81 | 74 | 67 | 61 | 56 | 52 | 48 | 44 | 41 | 39 | 36 | 26 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 12 | 24 | 40 | 57 | 70 | 80 | 87 | 84 | 76 | 69 | 63 | 58 | 53 | 49 | 45 | 41 | 39 | 36 | 27 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 10 | 21 | 36 | 53 | 67 | 77 | 87 | 85 | 77 | 69 | 63 | 58 | 54 | 49 | 46 | 41 | 39 | 36 | 27 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 14 | 26 | 41 | 56 | 69 | 82 | 85 | 80 | 72 | 65 | 60 | 55 | 51 | 47 | 42 | 40 | 36 | 27 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 20 | 34 | 49 | 68 | 81 | 85 | 77 | 69 | 63 | 58 | 53 | 49 | 43 | 40 | 37 | 28 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 19 | 38 | 58 | 78 | 83 | 76 | 68 | 62 | 57 | 53 | 45 | 41 | 37 | 29 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 13 | 29 | 55 | 77 | 82 | 75 | 68 | 62 | 57 | 48 | 43 | 38 | 30 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 22 | 51 | 75 | 81 | 75 | 68 | 62 | 53 | 45 | 39 | 32 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 25 | 54 | 75 | 80 | 75 | 68 | 57 | 48 | 40 | 33 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =2.0 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 8 | 13 | 18 | 24 | 29 | 36 | 41 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 38 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 3 | 7 | 12 | 17 | 22 | 28 | 35 | 40 | 45 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 38 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 8 | 13 | 18 | 24 | 31 | 38 | 44 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 39 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 7 | 12 | 17 | 22 | 30 | 37 | 43 | 45 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 39 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 11 | 16 | 21 | 29 | 36 | 42 | 45 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 46 | 39 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 5 | 9 | 14 | 20 | 28 | 35 | 42 | 45 | 45 | 45 | 45 | 45 | 45 | 45 | 45 | 45 | 39 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 13 | 18 | 26 | 33 | 41 | 45 | 45 | 45 | 45 | 45 | 45 | 45 | 45 | 45 | 39 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 5 | 9 | 16 | 24 | 33 | 42 | 45 | 45 | 45 | 45 | 45 | 45 | 45 | 45 | 40 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 8 | 15 | 25 | 36 | 43 | 45 | 45 | 45 | 45 | 45 | 45 | 45 | 42 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 14 | 26 | 37 | 43 | 45 | 45 | 45 | 45 | 45 | 45 | 45 | 43 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 15 | 27 | 37 | 43 | 45 | 45 | 45 | 45 | 45 | 45 | 44 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 19 | 30 | 39 | 44 | 45 | 45 | 45 | 45 | 45 | 45 |
| | RAINFALL TYPE = I | | | | | | | | | | | | | * * * TC = 2.0 HR * * * | | | | | | | | | | | | | | | | | | SHEET 10 OF 10 |

## Exhibit 5-IA: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 7.0 | 7.3 | 7.6 | 7.9 | 8.0 | 8.1 | 8.2 | 8.3 | 8.4 | 8.5 | 8.6 | 8.7 | 8.8 | 9.0 | 9.2 | 9.4 | 9.6 | 9.8 | 10.0 | 10.3 | 10.6 | 11.0 | 11.5 | 12.0 | 12.5 | 13.0 | 13.5 | 14.0 | 15.0 | 16.0 | 18.0 | 22.0 |
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 28 | 36 | 50 | 143 | 154 | 163 | 140 | 103 | 87 | 76 | 68 | 67 | 65 | 61 | 54 | 49 | 45 | 44 | 44 | 41 | 40 | 39 | 36 | 33 | 32 | 32 | 31 | 30 | 30 | 29 | 26 | 21 |
| .10 | 27 | 32 | 43 | 104 | 130 | 146 | 157 | 145 | 117 | 97 | 83 | 73 | 69 | 65 | 59 | 53 | 48 | 45 | 45 | 42 | 40 | 39 | 37 | 34 | 33 | 32 | 32 | 31 | 30 | 29 | 27 | 22 |
| .20 | 26 | 29 | 37 | 59 | 89 | 116 | 136 | 150 | 147 | 127 | 107 | 91 | 79 | 68 | 63 | 57 | 52 | 47 | 45 | 44 | 41 | 39 | 37 | 35 | 33 | 32 | 32 | 31 | 30 | 29 | 27 | 22 |
| .30 | 25 | 28 | 35 | 53 | 77 | 103 | 125 | 142 | 145 | 133 | 116 | 99 | 86 | 71 | 65 | 59 | 53 | 48 | 46 | 44 | 41 | 39 | 38 | 35 | 33 | 32 | 32 | 31 | 30 | 29 | 27 | 22 |
| .40 | 24 | 26 | 31 | 41 | 49 | 68 | 91 | 114 | 132 | 141 | 136 | 122 | 107 | 82 | 70 | 63 | 57 | 52 | 48 | 45 | 43 | 40 | 38 | 36 | 33 | 32 | 32 | 31 | 30 | 29 | 27 | 22 |
| .50 | 24 | 26 | 30 | 39 | 46 | 60 | 81 | 103 | 123 | 135 | 135 | 127 | 114 | 88 | 73 | 65 | 59 | 53 | 49 | 45 | 43 | 40 | 39 | 36 | 33 | 33 | 32 | 31 | 30 | 29 | 27 | 22 |
| .75 | 20 | 24 | 27 | 32 | 35 | 39 | 47 | 60 | 76 | 94 | 111 | 122 | 125 | 114 | 94 | 79 | 68 | 61 | 55 | 48 | 45 | 42 | 39 | 37 | 35 | 33 | 32 | 32 | 30 | 30 | 27 | 22 |
| 1.0 | 16 | 20 | 24 | 27 | 28 | 30 | 32 | 36 | 41 | 49 | 62 | 77 | 94 | 118 | 122 | 104 | 86 | 74 | 65 | 55 | 49 | 44 | 41 | 39 | 36 | 34 | 33 | 32 | 31 | 30 | 28 | 23 |
| 1.5 | 12 | 15 | 18 | 22 | 23 | 24 | 25 | 27 | 28 | 30 | 32 | 36 | 42 | 61 | 86 | 107 | 112 | 104 | 91 | 73 | 60 | 50 | 44 | 41 | 38 | 36 | 34 | 33 | 31 | 30 | 28 | 23 |
| 2.0 | 7 | 10 | 12 | 15 | 16 | 18 | 19 | 20 | 21 | 22 | 24 | 25 | 26 | 30 | 36 | 50 | 69 | 90 | 106 | 103 | 87 | 67 | 52 | 45 | 41 | 39 | 36 | 34 | 32 | 31 | 29 | 24 |
| 2.5 | 4 | 6 | 8 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 21 | 23 | 26 | 30 | 36 | 49 | 66 | 91 | 101 | 89 | 66 | 51 | 44 | 41 | 38 | 36 | 33 | 31 | 29 | 25 |
| 3.0 | 2 | 3 | 5 | 7 | 8 | 9 | 10 | 11 | 11 | 12 | 13 | 14 | 16 | 18 | 20 | 23 | 25 | 29 | 36 | 55 | 79 | 98 | 86 | 65 | 51 | 44 | 41 | 38 | 34 | 32 | 30 | 25 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 1 | 45 | 65 | 89 | 78 | 64 | 59 | 54 | 54 | 53 | 51 | 50 | 48 | 45 | 44 | 43 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 33 |
| .10 | 0 | 0 | 0 | 18 | 36 | 55 | 78 | 78 | 69 | 62 | 57 | 55 | 53 | 51 | 50 | 47 | 44 | 44 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 34 |
| .20 | 0 | 0 | 0 | 13 | 29 | 46 | 67 | 74 | 71 | 65 | 60 | 56 | 54 | 52 | 50 | 48 | 45 | 44 | 43 | 43 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 34 |
| .30 | 0 | 0 | 0 | 10 | 22 | 38 | 58 | 69 | 70 | 67 | 62 | 58 | 56 | 52 | 51 | 48 | 46 | 44 | 44 | 43 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 34 |
| .40 | 0 | 0 | 0 | 2 | 7 | 17 | 31 | 49 | 62 | 67 | 67 | 64 | 60 | 55 | 52 | 50 | 48 | 45 | 44 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 34 |
| .50 | 0 | 0 | 0 | 1 | 5 | 13 | 25 | 41 | 55 | 63 | 66 | 64 | 61 | 56 | 53 | 51 | 48 | 46 | 44 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 34 |
| .75 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 13 | 24 | 36 | 47 | 55 | 61 | 61 | 57 | 54 | 51 | 49 | 47 | 44 | 43 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 15 | 25 | 36 | 46 | 53 | 60 | 59 | 55 | 53 | 50 | 48 | 45 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 38 | 35 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 15 | 31 | 47 | 55 | 57 | 56 | 53 | 50 | 46 | 44 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 39 | 39 | 35 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 15 | 28 | 42 | 51 | 55 | 54 | 51 | 47 | 44 | 43 | 42 | 41 | 41 | 40 | 40 | 39 | 39 | 36 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 14 | 26 | 38 | 50 | 54 | 52 | 47 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 39 | 36 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 13 | 29 | 43 | 53 | 51 | 47 | 44 | 43 | 42 | 41 | 40 | 40 | 39 | 37 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 12 | 14 | 18 | 21 | 23 | 26 | 30 | 32 | 33 | 37 | 38 | 41 | 42 | 46 | 48 | 53 | 49 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 9 | 12 | 15 | 19 | 22 | 25 | 29 | 32 | 32 | 35 | 39 | 41 | 42 | 45 | 48 | 53 | 49 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 7 | 11 | 14 | 18 | 22 | 24 | 28 | 32 | 32 | 35 | 39 | 41 | 42 | 45 | 48 | 53 | 49 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 8 | 12 | 15 | 20 | 23 | 27 | 31 | 31 | 34 | 39 | 40 | 41 | 45 | 48 | 53 | 49 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 9 | 13 | 18 | 22 | 25 | 30 | 31 | 33 | 37 | 39 | 41 | 44 | 47 | 53 | 49 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 10 | 15 | 20 | 24 | 29 | 31 | 32 | 36 | 39 | 41 | 44 | 47 | 53 | 53 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 11 | 16 | 21 | 26 | 30 | 31 | 34 | 38 | 40 | 43 | 46 | 53 | 53 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 11 | 17 | 23 | 28 | 31 | 32 | 35 | 39 | 42 | 45 | 53 | 53 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 17 | 23 | 28 | 31 | 32 | 35 | 40 | 43 | 49 | 52 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 17 | 23 | 28 | 31 | 32 | 38 | 42 | 48 | 52 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 10 | 18 | 24 | 28 | 31 | 36 | 40 | 47 | 52 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 11 | 18 | 24 | 28 | 33 | 39 | 45 | 52 |
| | RAINFALL TYPE = IA | | | | | | | | | | | | | * * * TC = 0.1 HR * * * | | | | | | | | | | | | | | | | | | SHEET 1 OF 10 |

# Exhibit 5-IA, continued: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

| TRVL TIME (HR) | 7.0 | 7.3 | 7.6 | 7.9 | 8.0 | 8.1 | 8.2 | 8.3 | 8.4 | 8.5 | 8.6 | 8.7 | 8.8 | HYDROGRAPH TIME (HOURS) 9.0 | 9.2 | 9.4 | 9.6 | 9.8 | 10.0 | 10.3 | 10.6 | 11.0 | 11.5 | 12.0 | 12.5 | 13.0 | 13.5 | 14.0 | 15.0 | 16.0 | 18.0 | 22.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | IA/P = 0.10 | | | | | | | | | | | | * * * TC =0.2 HR * * * | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 27 | 34 | 46 | 124 | 143 | 153 | 153 | 127 | 103 | 87 | 76 | 68 | 67 | 63 | 57 | 51 | 47 | 45 | 44 | 42 | 40 | 39 | 36 | 33 | 33 | 32 | 32 | 31 | 30 | 29 | 27 | 21 |
| .10 | 27 | 33 | 44 | 108 | 131 | 146 | 151 | 135 | 113 | 96 | 83 | 73 | 69 | 65 | 59 | 53 | 48 | 45 | 45 | 42 | 40 | 39 | 37 | 33 | 33 | 32 | 32 | 31 | 30 | 29 | 27 | 22 |
| .20 | 26 | 29 | 38 | 67 | 95 | 119 | 137 | 146 | 139 | 122 | 105 | 90 | 79 | 69 | 63 | 57 | 52 | 47 | 45 | 44 | 41 | 39 | 37 | 34 | 33 | 32 | 32 | 31 | 30 | 29 | 27 | 22 |
| .30 | 25 | 27 | 33 | 45 | 60 | 83 | 107 | 127 | 140 | 139 | 128 | 112 | 97 | 76 | 68 | 62 | 56 | 50 | 47 | 45 | 42 | 40 | 38 | 35 | 33 | 33 | 32 | 31 | 30 | 29 | 27 | 22 |
| .40 | 24 | 27 | 32 | 43 | 54 | 73 | 96 | 117 | 132 | 137 | 131 | 118 | 104 | 81 | 70 | 63 | 57 | 52 | 48 | 45 | 43 | 40 | 38 | 36 | 33 | 33 | 32 | 32 | 30 | 29 | 27 | 22 |
| .50 | 23 | 25 | 29 | 37 | 41 | 50 | 65 | 86 | 106 | 124 | 132 | 131 | 123 | 98 | 79 | 69 | 62 | 56 | 51 | 46 | 44 | 41 | 39 | 37 | 34 | 33 | 32 | 32 | 30 | 29 | 27 | 22 |
| .75 | 20 | 24 | 27 | 33 | 36 | 41 | 50 | 63 | 80 | 97 | 112 | 121 | 123 | 111 | 93 | 78 | 68 | 61 | 55 | 48 | 45 | 42 | 39 | 37 | 34 | 33 | 32 | 32 | 30 | 30 | 27 | 22 |
| 1.0 | 16 | 20 | 24 | 27 | 29 | 31 | 33 | 37 | 43 | 52 | 65 | 81 | 96 | 120 | 120 | 102 | 85 | 73 | 64 | 55 | 49 | 44 | 41 | 39 | 36 | 34 | 33 | 32 | 31 | 30 | 28 | 23 |
| 1.5 | 12 | 15 | 19 | 22 | 23 | 24 | 26 | 27 | 28 | 30 | 33 | 37 | 44 | 63 | 88 | 107 | 111 | 102 | 89 | 72 | 60 | 50 | 44 | 41 | 38 | 36 | 34 | 33 | 32 | 30 | 28 | 23 |
| 2.0 | 7 | 10 | 12 | 16 | 17 | 18 | 19 | 20 | 22 | 23 | 24 | 25 | 27 | 30 | 38 | 52 | 71 | 91 | 105 | 101 | 86 | 66 | 52 | 45 | 41 | 39 | 36 | 34 | 32 | 31 | 29 | 24 |
| 2.5 | 4 | 6 | 9 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 19 | 20 | 21 | 23 | 26 | 30 | 38 | 50 | 68 | 92 | 101 | 88 | 65 | 51 | 44 | 41 | 38 | 36 | 33 | 31 | 29 | 25 |
| 3.0 | 2 | 3 | 5 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 19 | 22 | 24 | 28 | 33 | 49 | 72 | 97 | 89 | 68 | 53 | 45 | 41 | 39 | 34 | 32 | 30 | 25 |
| | | IA/P = 0.30 | | | | | | | | | | | | * * * TC =0.2 HR * * * | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 26 | 45 | 64 | 76 | 74 | 65 | 59 | 55 | 53 | 52 | 51 | 49 | 46 | 44 | 44 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 33 |
| .10 | 0 | 0 | 0 | 20 | 37 | 55 | 69 | 72 | 67 | 62 | 57 | 54 | 53 | 52 | 50 | 47 | 44 | 44 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 34 |
| .20 | 0 | 0 | 0 | 5 | 15 | 30 | 47 | 61 | 68 | 68 | 64 | 59 | 56 | 53 | 52 | 49 | 46 | 44 | 44 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 34 |
| .30 | 0 | 0 | 0 | 4 | 11 | 23 | 39 | 54 | 64 | 66 | 65 | 61 | 58 | 54 | 52 | 50 | 47 | 45 | 44 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 34 |
| .40 | 0 | 0 | 0 | 3 | 8 | 18 | 32 | 47 | 58 | 64 | 64 | 62 | 59 | 55 | 52 | 50 | 48 | 45 | 44 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .50 | 0 | 0 | 0 | 0 | 2 | 6 | 14 | 26 | 40 | 52 | 60 | 63 | 62 | 58 | 54 | 52 | 50 | 47 | 45 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .75 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 14 | 24 | 35 | 45 | 53 | 57 | 59 | 56 | 54 | 51 | 49 | 47 | 44 | 43 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 8 | 15 | 25 | 35 | 44 | 56 | 58 | 56 | 54 | 51 | 49 | 46 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 35 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 23 | 38 | 50 | 55 | 56 | 54 | 51 | 47 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 39 | 39 | 35 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 6 | 15 | 27 | 40 | 49 | 54 | 54 | 51 | 47 | 44 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 39 | 36 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 14 | 25 | 36 | 49 | 53 | 51 | 47 | 44 | 43 | 42 | 41 | 41 | 40 | 40 | 39 | 36 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 13 | 28 | 42 | 52 | 51 | 47 | 44 | 43 | 42 | 41 | 40 | 40 | 39 | 37 |
| | | IA/P = 0.50 | | | | | | | | | | | | * * * TC =0.2 HR * * * | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 10 | 13 | 16 | 19 | 23 | 25 | 29 | 32 | 33 | 36 | 39 | 41 | 42 | 46 | 48 | 50 | 49 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 10 | 14 | 17 | 21 | 24 | 28 | 31 | 32 | 35 | 40 | 41 | 42 | 45 | 48 | 50 | 49 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 7 | 11 | 15 | 19 | 23 | 26 | 31 | 31 | 34 | 39 | 40 | 41 | 45 | 48 | 50 | 49 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 8 | 12 | 17 | 21 | 25 | 30 | 31 | 33 | 37 | 39 | 41 | 44 | 47 | 50 | 50 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 9 | 15 | 19 | 23 | 28 | 31 | 32 | 35 | 39 | 41 | 44 | 47 | 50 | 50 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 6 | 12 | 17 | 22 | 27 | 31 | 32 | 34 | 39 | 40 | 43 | 46 | 50 | 50 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 9 | 14 | 20 | 25 | 30 | 31 | 33 | 37 | 39 | 42 | 46 | 50 | 50 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 9 | 16 | 22 | 27 | 31 | 32 | 35 | 38 | 42 | 45 | 49 | 50 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 8 | 17 | 23 | 28 | 31 | 32 | 35 | 40 | 43 | 49 | 50 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 17 | 23 | 28 | 31 | 32 | 38 | 42 | 48 | 50 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 16 | 22 | 27 | 30 | 35 | 40 | 46 | 50 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 9 | 17 | 23 | 27 | 33 | 38 | 45 | 50 |
| | | RAINFALL TYPE = IA | | | | | | | | | | | | * * * TC = 0.2 HR * * * | | | | | | | | | | | | | | | | SHEET 2 OF 10 |

## Exhibit 5-IA, continued: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

| TRVL TIME (HR) | 7.0 | 7.3 | 7.6 | 7.9 | 8.0 | 8.1 | 8.2 | 8.3 | 8.4 | 8.5 | 8.6 | 8.7 | 8.8 | 9.0 | 9.2 | 9.4 | 9.6 | 9.8 | 10.0 | 10.3 | 10.6 | 11.0 | 11.5 | 12.0 | 12.5 | 13.0 | 13.5 | 14.0 | 15.0 | 16.0 | 18.0 | 22.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | IA/P = 0.10 | | | | | | | | | | | | * * * TC =0.3 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 26 | 31 | 41 | 92 | 120 | 138 | 145 | 144 | 125 | 105 | 90 | 78 | 71 | 66 | 60 | 54 | 49 | 46 | 45 | 43 | 40 | 39 | 37 | 34 | 33 | 32 | 32 | 31 | 30 | 29 | 27 | 22 |
| .10 | 26 | 30 | 39 | 80 | 106 | 128 | 139 | 142 | 131 | 113 | 98 | 85 | 76 | 68 | 62 | 56 | 51 | 47 | 45 | 43 | 41 | 39 | 37 | 34 | 33 | 32 | 32 | 31 | 30 | 29 | 27 | 22 |
| .20 | 25 | 28 | 34 | 51 | 70 | 94 | 117 | 131 | 139 | 133 | 120 | 105 | 92 | 74 | 67 | 60 | 54 | 49 | 46 | 44 | 42 | 40 | 38 | 35 | 33 | 33 | 32 | 31 | 30 | 29 | 27 | 22 |
| .30 | 24 | 26 | 31 | 40 | 47 | 62 | 84 | 106 | 123 | 133 | 133 | 124 | 112 | 87 | 73 | 65 | 59 | 53 | 49 | 45 | 43 | 40 | 39 | 36 | 33 | 33 | 32 | 32 | 30 | 29 | 27 | 22 |
| .40 | 23 | 26 | 30 | 38 | 44 | 56 | 75 | 95 | 114 | 127 | 131 | 127 | 117 | 93 | 76 | 67 | 60 | 55 | 50 | 46 | 43 | 40 | 39 | 36 | 34 | 33 | 32 | 32 | 30 | 29 | 27 | 22 |
| .50 | 21 | 25 | 28 | 34 | 37 | 42 | 51 | 67 | 86 | 104 | 119 | 127 | 127 | 110 | 88 | 74 | 66 | 59 | 53 | 47 | 45 | 41 | 39 | 37 | 34 | 33 | 32 | 32 | 30 | 30 | 27 | 22 |
| .75 | 19 | 23 | 26 | 31 | 33 | 36 | 42 | 51 | 64 | 80 | 96 | 109 | 119 | 116 | 102 | 85 | 73 | 65 | 58 | 50 | 46 | 43 | 40 | 38 | 35 | 33 | 33 | 32 | 31 | 30 | 28 | 23 |
| 1.0 | 15 | 19 | 23 | 26 | 27 | 29 | 31 | 33 | 37 | 44 | 53 | 66 | 80 | 106 | 116 | 109 | 93 | 79 | 69 | 58 | 51 | 45 | 41 | 39 | 37 | 34 | 33 | 32 | 31 | 30 | 28 | 23 |
| 1.5 | 11 | 14 | 17 | 21 | 22 | 23 | 24 | 26 | 27 | 29 | 31 | 34 | 38 | 53 | 75 | 97 | 109 | 106 | 96 | 78 | 64 | 52 | 45 | 41 | 39 | 36 | 34 | 33 | 32 | 30 | 28 | 24 |
| 2.0 | 6 | 9 | 11 | 14 | 16 | 17 | 18 | 19 | 20 | 21 | 23 | 24 | 25 | 28 | 34 | 44 | 61 | 81 | 97 | 103 | 91 | 71 | 54 | 46 | 42 | 39 | 37 | 34 | 32 | 31 | 29 | 24 |
| 2.5 | 4 | 5 | 8 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 19 | 20 | 22 | 25 | 28 | 34 | 44 | 58 | 84 | 99 | 92 | 70 | 54 | 46 | 41 | 39 | 36 | 33 | 32 | 29 | 25 |
| 3.0 | 2 | 3 | 5 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 19 | 22 | 24 | 28 | 33 | 49 | 71 | 96 | 89 | 68 | 53 | 45 | 41 | 39 | 34 | 32 | 30 | 25 |
| | | IA/P = 0.30 | | | | | | | | | | | | * * * TC =0.3 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 13 | 28 | 45 | 63 | 69 | 69 | 65 | 60 | 55 | 53 | 53 | 51 | 48 | 45 | 44 | 43 | 43 | 42 | 42 | 42 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 38 | 34 |
| .10 | 0 | 0 | 0 | 9 | 21 | 37 | 54 | 64 | 67 | 65 | 62 | 57 | 54 | 53 | 51 | 49 | 46 | 44 | 43 | 43 | 42 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .20 | 0 | 0 | 0 | 7 | 17 | 30 | 46 | 58 | 64 | 65 | 63 | 59 | 56 | 53 | 52 | 49 | 46 | 44 | 44 | 43 | 42 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .30 | 0 | 0 | 0 | 1 | 5 | 13 | 25 | 39 | 52 | 60 | 63 | 63 | 60 | 55 | 53 | 51 | 49 | 46 | 44 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .40 | 0 | 0 | 0 | 1 | 4 | 10 | 20 | 33 | 45 | 55 | 61 | 62 | 61 | 56 | 54 | 52 | 49 | 46 | 45 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .50 | 0 | 0 | 0 | 1 | 3 | 7 | 15 | 27 | 39 | 50 | 57 | 60 | 61 | 57 | 54 | 52 | 50 | 47 | 45 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .75 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 15 | 24 | 34 | 43 | 51 | 55 | 58 | 56 | 54 | 52 | 49 | 47 | 44 | 43 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 9 | 16 | 25 | 34 | 49 | 56 | 57 | 55 | 53 | 50 | 47 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 35 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 5 | 10 | 23 | 37 | 49 | 54 | 55 | 54 | 51 | 47 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 39 | 39 | 35 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 10 | 21 | 33 | 44 | 51 | 54 | 52 | 48 | 44 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 39 | 39 | 36 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 19 | 30 | 45 | 53 | 53 | 48 | 44 | 43 | 42 | 41 | 41 | 40 | 40 | 39 | 37 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 9 | 23 | 37 | 50 | 52 | 47 | 44 | 43 | 42 | 41 | 40 | 40 | 39 | 37 |
| | | IA/P = 0.50 | | | | | | | | | | | | * * * TC =0.3 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 7 | 11 | 14 | 18 | 22 | 24 | 28 | 32 | 32 | 35 | 40 | 41 | 42 | 45 | 48 | 48 | 48 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 8 | 12 | 15 | 20 | 23 | 27 | 31 | 31 | 34 | 39 | 40 | 41 | 45 | 48 | 48 | 48 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 9 | 13 | 18 | 21 | 25 | 30 | 31 | 33 | 37 | 39 | 41 | 44 | 47 | 48 | 48 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 8 | 12 | 17 | 21 | 25 | 30 | 31 | 33 | 37 | 39 | 41 | 44 | 47 | 48 | 48 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 9 | 14 | 19 | 23 | 28 | 31 | 32 | 35 | 39 | 41 | 44 | 47 | 48 | 48 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 15 | 21 | 26 | 30 | 31 | 33 | 38 | 40 | 43 | 46 | 48 | 48 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 13 | 19 | 24 | 29 | 31 | 33 | 36 | 39 | 42 | 46 | 48 | 48 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 9 | 16 | 22 | 27 | 31 | 32 | 35 | 38 | 42 | 45 | 48 | 48 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 8 | 16 | 23 | 27 | 31 | 32 | 35 | 40 | 43 | 48 | 48 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 16 | 22 | 27 | 30 | 32 | 38 | 42 | 47 | 48 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 16 | 22 | 27 | 30 | 35 | 40 | 46 | 48 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 9 | 17 | 23 | 27 | 33 | 38 | 45 | 48 |
| | | RAINFALL TYPE = IA | | | | | | | | | | | | * * * TC = 0.3 HR * * * | | | | | | | | | | | | | | | | | SHEET 3 OF 10 | |

## Exhibit 5-IA, continued: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

| TRVL TIME (HR) | 7.0 | 7.3 | 7.6 | 7.9 | 8.0 | 8.1 | 8.2 | 8.3 | 8.4 | 8.5 | 8.6 | 8.7 | 8.8 | 9.0 | 9.2 | 9.4 | 9.6 | 9.8 | 10.0 | 10.3 | 10.6 | 11.0 | 11.5 | 12.0 | 12.5 | 13.0 | 13.5 | 14.0 | 15.0 | 16.0 | 18.0 | 22.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.4 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 26 | 29 | 37 | 67 | 93 | 117 | 134 | 137 | 136 | 121 | 105 | 91 | 81 | 70 | 64 | 58 | 52 | 48 | 46 | 44 | 41 | 40 | 37 | 34 | 33 | 33 | 32 | 31 | 30 | 29 | 27 | 22 |
| .10 | 25 | 29 | 36 | 60 | 82 | 105 | 124 | 133 | 135 | 126 | 112 | 98 | 87 | 73 | 66 | 59 | 53 | 49 | 46 | 44 | 41 | 40 | 38 | 34 | 33 | 33 | 32 | 31 | 30 | 29 | 27 | 22 |
| .20 | 24 | 27 | 32 | 43 | 55 | 73 | 94 | 114 | 127 | 132 | 128 | 117 | 104 | 83 | 71 | 64 | 58 | 52 | 48 | 45 | 43 | 40 | 38 | 35 | 33 | 33 | 32 | 32 | 30 | 29 | 27 | 22 |
| .30 | 24 | 26 | 31 | 41 | 50 | 65 | 85 | 104 | 119 | 128 | 128 | 121 | 110 | 88 | 74 | 66 | 59 | 54 | 49 | 46 | 43 | 40 | 39 | 35 | 34 | 33 | 32 | 32 | 30 | 29 | 27 | 22 |
| .40 | 22 | 25 | 28 | 35 | 39 | 46 | 59 | 76 | 95 | 111 | 122 | 126 | 122 | 104 | 85 | 72 | 64 | 58 | 52 | 47 | 44 | 41 | 39 | 37 | 34 | 33 | 32 | 32 | 30 | 29 | 27 | 22 |
| .50 | 20 | 23 | 26 | 31 | 34 | 37 | 43 | 54 | 69 | 86 | 103 | 116 | 123 | 117 | 98 | 82 | 71 | 63 | 56 | 49 | 46 | 42 | 40 | 38 | 34 | 33 | 33 | 32 | 30 | 30 | 27 | 22 |
| .75 | 16 | 20 | 24 | 27 | 29 | 31 | 33 | 37 | 43 | 53 | 65 | 80 | 94 | 116 | 116 | 101 | 86 | 74 | 65 | 56 | 49 | 44 | 41 | 39 | 36 | 34 | 33 | 32 | 31 | 30 | 28 | 23 |
| 1.0 | 14 | 18 | 22 | 25 | 26 | 27 | 29 | 31 | 34 | 38 | 45 | 54 | 67 | 93 | 111 | 113 | 101 | 87 | 75 | 62 | 53 | 47 | 42 | 40 | 37 | 34 | 33 | 33 | 31 | 30 | 28 | 23 |
| 1.5 | 10 | 13 | 16 | 20 | 21 | 22 | 23 | 24 | 26 | 27 | 29 | 31 | 34 | 45 | 64 | 86 | 102 | 107 | 101 | 84 | 69 | 55 | 46 | 42 | 39 | 37 | 34 | 33 | 32 | 30 | 28 | 24 |
| 2.0 | 6 | 8 | 11 | 13 | 14 | 16 | 17 | 18 | 19 | 20 | 21 | 23 | 24 | 27 | 31 | 39 | 52 | 71 | 88 | 102 | 96 | 76 | 57 | 47 | 42 | 40 | 37 | 35 | 33 | 31 | 29 | 24 |
| 2.5 | 3 | 5 | 7 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 21 | 23 | 26 | 31 | 38 | 51 | 75 | 94 | 98 | 74 | 57 | 47 | 42 | 39 | 37 | 33 | 32 | 30 | 25 |
| 3.0 | 1 | 2 | 4 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 16 | 18 | 20 | 23 | 26 | 30 | 43 | 64 | 89 | 95 | 72 | 56 | 47 | 42 | 39 | 34 | 33 | 30 | 25 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.4 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 6 | 15 | 29 | 46 | 60 | 65 | 65 | 63 | 59 | 56 | 53 | 52 | 49 | 47 | 44 | 44 | 43 | 42 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .10 | 0 | 0 | 0 | 4 | 12 | 23 | 38 | 53 | 61 | 63 | 63 | 60 | 57 | 54 | 53 | 50 | 47 | 45 | 44 | 43 | 42 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .20 | 0 | 0 | 0 | 3 | 9 | 19 | 32 | 46 | 56 | 61 | 62 | 61 | 59 | 55 | 53 | 51 | 48 | 45 | 44 | 43 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .30 | 0 | 0 | 0 | 2 | 7 | 15 | 26 | 39 | 50 | 57 | 61 | 61 | 59 | 56 | 53 | 51 | 49 | 46 | 44 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .40 | 0 | 0 | 0 | 0 | 2 | 5 | 11 | 21 | 33 | 44 | 53 | 58 | 60 | 58 | 55 | 53 | 51 | 48 | 45 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .50 | 0 | 0 | 0 | 0 | 1 | 4 | 9 | 17 | 28 | 39 | 48 | 55 | 59 | 59 | 56 | 53 | 51 | 49 | 46 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .75 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 9 | 16 | 25 | 34 | 43 | 49 | 57 | 57 | 55 | 53 | 50 | 48 | 45 | 44 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 35 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 17 | 25 | 34 | 48 | 55 | 56 | 55 | 53 | 50 | 47 | 45 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 35 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 16 | 30 | 43 | 51 | 55 | 55 | 52 | 49 | 45 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 36 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 15 | 27 | 39 | 47 | 53 | 53 | 49 | 45 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 39 | 36 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 7 | 14 | 25 | 40 | 50 | 52 | 49 | 45 | 43 | 42 | 41 | 41 | 40 | 40 | 39 | 37 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 18 | 32 | 47 | 52 | 48 | 45 | 43 | 42 | 41 | 40 | 40 | 39 | 37 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.4 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 9 | 13 | 16 | 20 | 23 | 27 | 31 | 32 | 34 | 39 | 40 | 42 | 45 | 47 | 47 | 47 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 10 | 13 | 18 | 22 | 26 | 30 | 31 | 33 | 38 | 39 | 41 | 45 | 47 | 47 | 47 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 11 | 16 | 20 | 24 | 29 | 31 | 33 | 36 | 39 | 41 | 44 | 47 | 47 | 47 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 8 | 13 | 18 | 23 | 28 | 31 | 32 | 35 | 39 | 41 | 43 | 47 | 47 | 47 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 11 | 16 | 21 | 26 | 30 | 32 | 34 | 38 | 40 | 43 | 46 | 47 | 47 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 13 | 19 | 25 | 29 | 31 | 33 | 37 | 39 | 42 | 46 | 47 | 47 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 11 | 17 | 23 | 28 | 31 | 32 | 35 | 39 | 42 | 45 | 47 | 47 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 12 | 20 | 25 | 29 | 31 | 33 | 37 | 41 | 44 | 47 | 47 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 13 | 20 | 26 | 30 | 31 | 34 | 40 | 43 | 47 | 47 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 14 | 21 | 26 | 30 | 32 | 37 | 41 | 47 | 47 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 15 | 21 | 26 | 30 | 34 | 40 | 46 | 47 | |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 15 | 22 | 27 | 32 | 38 | 45 | 47 | |
| | RAINFALL TYPE = IA | | | | | | | | | | | | | * * * TC = C.4 HR * * * | | | | | | | | | | | | | | | | | SHEET 4 OF 10 |

## Exhibit 5-IA, continued: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

| TRVL TIME (HR) | 7.0 | 7.3 | 7.6 | 7.9 | 8.0 | 8.1 | 8.2 | 8.3 | 8.4 | 8.5 | 8.6 | 8.7 | 8.8 | 9.0 | 9.2 | 9.4 | 9.6 | 9.8 | 10.0 | 10.3 | 10.6 | 11.0 | 11.5 | 12.0 | 12.5 | 13.0 | 13.5 | 14.0 | 15.0 | 16.0 | 18.0 | 22.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 25 | 28 | 34 | 53 | 72 | 94 | 115 | 129 | 130 | 129 | 117 | 104 | 92 | 76 | 68 | 61 | 55 | 50 | 47 | 44 | 42 | 40 | 38 | 34 | 33 | 33 | 32 | 31 | 30 | 29 | 27 | 22 |
| .10 | 24 | 27 | 33 | 49 | 64 | 84 | 105 | 121 | 127 | 128 | 121 | 109 | 98 | 80 | 70 | 63 | 57 | 51 | 48 | 45 | 42 | 40 | 38 | 34 | 33 | 33 | 32 | 32 | 30 | 29 | 27 | 22 |
| .20 | 23 | 26 | 30 | 39 | 46 | 58 | 75 | 95 | 112 | 122 | 126 | 123 | 114 | 93 | 78 | 69 | 61 | 55 | 50 | 46 | 44 | 41 | 39 | 36 | 34 | 33 | 32 | 32 | 30 | 29 | 27 | 22 |
| .30 | 21 | 24 | 27 | 34 | 37 | 43 | 53 | 68 | 86 | 103 | 116 | 123 | 123 | 108 | 89 | 76 | 67 | 60 | 54 | 48 | 45 | 42 | 39 | 37 | 34 | 33 | 33 | 32 | 30 | 30 | 27 | 22 |
| .40 | 20 | 24 | 27 | 32 | 35 | 40 | 49 | 61 | 78 | 95 | 109 | 118 | 121 | 111 | 93 | 79 | 69 | 62 | 55 | 49 | 45 | 42 | 40 | 37 | 34 | 33 | 33 | 32 | 30 | 30 | 27 | 22 |
| .50 | 18 | 22 | 25 | 29 | 31 | 34 | 38 | 45 | 56 | 70 | 87 | 101 | 113 | 118 | 106 | 89 | 77 | 67 | 60 | 52 | 47 | 43 | 40 | 38 | 35 | 33 | 33 | 32 | 31 | 30 | 28 | 23 |
| .75 | 15 | 19 | 23 | 26 | 27 | 29 | 31 | 34 | 38 | 45 | 54 | 67 | 80 | 104 | 112 | 106 | 93 | 80 | 70 | 59 | 51 | 46 | 41 | 39 | 36 | 34 | 33 | 33 | 31 | 30 | 28 | 23 |
| 1.0 | 13 | 16 | 20 | 24 | 25 | 26 | 27 | 29 | 31 | 34 | 39 | 46 | 56 | 80 | 102 | 110 | 105 | 93 | 81 | 66 | 56 | 48 | 43 | 40 | 38 | 35 | 33 | 33 | 31 | 30 | 28 | 23 |
| 1.5 | 9 | 12 | 15 | 18 | 20 | 21 | 22 | 23 | 24 | 26 | 27 | 29 | 31 | 39 | 55 | 75 | 94 | 104 | 103 | 89 | 74 | 58 | 48 | 43 | 40 | 37 | 35 | 33 | 32 | 31 | 29 | 24 |
| 2.0 | 5 | 7 | 10 | 12 | 13 | 14 | 15 | 17 | 18 | 19 | 20 | 21 | 23 | 25 | 29 | 35 | 45 | 62 | 79 | 100 | 100 | 81 | 61 | 49 | 43 | 40 | 38 | 35 | 33 | 31 | 29 | 25 |
| 2.5 | 3 | 4 | 6 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 20 | 22 | 25 | 28 | 34 | 45 | 67 | 88 | 96 | 79 | 60 | 49 | 43 | 40 | 37 | 33 | 32 | 30 | 25 |
| 3.0 | 1 | 2 | 3 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 15 | 17 | 19 | 21 | 24 | 28 | 38 | 56 | 83 | 93 | 77 | 59 | 48 | 43 | 40 | 35 | 33 | 30 | 26 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 2 | 8 | 18 | 31 | 46 | 57 | 61 | 61 | 61 | 58 | 54 | 53 | 51 | 48 | 45 | 44 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .10 | 0 | 0 | 0 | 2 | 6 | 14 | 25 | 39 | 51 | 58 | 60 | 61 | 59 | 55 | 53 | 51 | 49 | 46 | 44 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .20 | 0 | 0 | 0 | 0 | 1 | 4 | 11 | 21 | 33 | 45 | 54 | 58 | 60 | 58 | 55 | 53 | 51 | 48 | 46 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 38 | 34 |
| .30 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 16 | 27 | 39 | 49 | 55 | 59 | 59 | 55 | 53 | 51 | 49 | 46 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 34 |
| .40 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 13 | 23 | 34 | 44 | 51 | 56 | 58 | 56 | 54 | 52 | 49 | 47 | 44 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 34 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 10 | 18 | 29 | 39 | 47 | 56 | 57 | 55 | 53 | 51 | 49 | 45 | 44 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 35 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 17 | 25 | 34 | 48 | 54 | 56 | 55 | 53 | 50 | 47 | 45 | 43 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 39 | 35 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 11 | 26 | 41 | 51 | 55 | 55 | 54 | 50 | 47 | 44 | 43 | 42 | 41 | 41 | 40 | 40 | 40 | 40 | 39 | 35 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 11 | 23 | 36 | 47 | 52 | 54 | 52 | 48 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 36 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 16 | 27 | 38 | 49 | 53 | 51 | 47 | 44 | 42 | 42 | 41 | 40 | 40 | 40 | 39 | 36 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 15 | 30 | 43 | 52 | 50 | 46 | 44 | 42 | 42 | 41 | 40 | 40 | 39 | 37 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 10 | 23 | 40 | 51 | 50 | 46 | 44 | 42 | 42 | 40 | 40 | 39 | 37 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 11 | 14 | 19 | 22 | 26 | 30 | 31 | 34 | 38 | 40 | 41 | 45 | 46 | 46 | 46 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 8 | 12 | 17 | 21 | 25 | 29 | 31 | 33 | 37 | 39 | 41 | 44 | 46 | 46 | 46 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 9 | 14 | 19 | 23 | 28 | 31 | 32 | 35 | 39 | 41 | 43 | 46 | 46 | 46 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 12 | 17 | 22 | 27 | 31 | 32 | 34 | 38 | 40 | 43 | 46 | 46 | 46 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 11 | 16 | 21 | 26 | 30 | 32 | 34 | 38 | 40 | 43 | 46 | 46 | 46 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 13 | 19 | 25 | 29 | 31 | 33 | 37 | 39 | 42 | 46 | 46 | 46 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 11 | 17 | 23 | 28 | 31 | 32 | 35 | 39 | 42 | 45 | 46 | 46 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 11 | 18 | 24 | 29 | 31 | 33 | 36 | 41 | 44 | 46 | 46 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 11 | 19 | 24 | 29 | 31 | 33 | 39 | 42 | 46 | 46 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 12 | 19 | 25 | 29 | 31 | 37 | 41 | 46 | 46 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 13 | 20 | 25 | 29 | 34 | 39 | 45 | 46 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 14 | 20 | 26 | 32 | 37 | 44 | 46 |
| | RAINFALL TYPE = IA | | | | | | | | | | | | | * * * TC = 0.5 HR * * * | | | | | | | | | | | | | | | | | SHEET 5 OF 10 |

## Exhibit 5-IA, continued: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

| TRVL TIME (HR) | 7.0 | 7.3 | 7.6 | 7.9 | 8.0 | 8.1 | 8.2 | 8.3 | 8.4 | 8.5 | 8.6 | 8.7 | 8.8 | 9.0 | 9.2 | 9.4 | 9.6 | 9.8 | 10.0 | 10.3 | 10.6 | 11.0 | 11.5 | 12.0 | 12.5 | 13.0 | 13.5 | 14.0 | 15.0 | 16.0 | 18.0 | 22.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.75 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 22 | 25 | 29 | 38 | 45 | 56 | 71 | 87 | 103 | 114 | 117 | 117 | 111 | 95 | 81 | 72 | 65 | 58 | 53 | 48 | 45 | 41 | 39 | 36 | 33 | 33 | 33 | 32 | 30 | 29 | 27 | 22 |
| .10 | 21 | 25 | 28 | 36 | 42 | 51 | 64 | 79 | 95 | 108 | 114 | 116 | 113 | 99 | 85 | 74 | 66 | 60 | 54 | 48 | 45 | 42 | 39 | 37 | 34 | 33 | 33 | 32 | 30 | 30 | 27 | 22 |
| .20 | 19 | 23 | 26 | 32 | 35 | 39 | 47 | 58 | 72 | 87 | 101 | 110 | 114 | 109 | 95 | 82 | 72 | 65 | 58 | 51 | 47 | 43 | 40 | 38 | 34 | 33 | 33 | 32 | 31 | 30 | 27 | 23 |
| .30 | 19 | 22 | 26 | 31 | 33 | 37 | 44 | 54 | 66 | 80 | 94 | 104 | 112 | 110 | 98 | 85 | 75 | 67 | 60 | 52 | 47 | 43 | 40 | 38 | 34 | 33 | 33 | 32 | 31 | 30 | 28 | 23 |
| .40 | 17 | 21 | 24 | 28 | 30 | 32 | 36 | 41 | 49 | 61 | 74 | 87 | 99 | 111 | 106 | 94 | 82 | 73 | 65 | 56 | 49 | 45 | 41 | 39 | 35 | 33 | 33 | 33 | 31 | 30 | 28 | 23 |
| .50 | 16 | 20 | 23 | 27 | 29 | 31 | 34 | 39 | 46 | 56 | 68 | 81 | 93 | 109 | 107 | 97 | 85 | 75 | 67 | 57 | 50 | 45 | 41 | 39 | 36 | 33 | 33 | 33 | 31 | 30 | 28 | 23 |
| .75 | 15 | 18 | 22 | 25 | 27 | 28 | 31 | 34 | 38 | 45 | 54 | 64 | 75 | 95 | 104 | 102 | 93 | 83 | 74 | 62 | 54 | 47 | 42 | 40 | 37 | 34 | 33 | 33 | 31 | 30 | 28 | 23 |
| 1.0 | 12 | 15 | 18 | 22 | 23 | 24 | 25 | 27 | 29 | 31 | 34 | 39 | 46 | 65 | 85 | 100 | 103 | 98 | 88 | 74 | 62 | 52 | 45 | 41 | 39 | 35 | 33 | 33 | 32 | 30 | 28 | 23 |
| 1.5 | 7 | 10 | 12 | 15 | 17 | 18 | 19 | 20 | 21 | 22 | 24 | 25 | 26 | 31 | 39 | 53 | 71 | 87 | 99 | 99 | 85 | 68 | 53 | 46 | 41 | 39 | 36 | 34 | 33 | 31 | 29 | 24 |
| 2.0 | 4 | 6 | 8 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 21 | 23 | 26 | 31 | 39 | 51 | 67 | 88 | 95 | 86 | 67 | 53 | 45 | 41 | 38 | 36 | 33 | 32 | 29 | 25 |
| 2.5 | 2 | 3 | 5 | 7 | 8 | 9 | 10 | 11 | 11 | 12 | 13 | 14 | 16 | 18 | 20 | 23 | 26 | 30 | 38 | 56 | 77 | 93 | 84 | 66 | 52 | 45 | 41 | 38 | 34 | 33 | 30 | 25 |
| 3.0 | 0 | 1 | 2 | 4 | 4 | 5 | 6 | 6 | 7 | 8 | 8 | 9 | 10 | 12 | 14 | 16 | 18 | 21 | 24 | 30 | 42 | 67 | 90 | 84 | 68 | 54 | 46 | 42 | 36 | 33 | 30 | 26 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.75 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 0 | 2 | 4 | 10 | 19 | 29 | 39 | 48 | 53 | 56 | 56 | 54 | 53 | 51 | 49 | 46 | 44 | 44 | 42 | 42 | 40 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 34 |
| .10 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 15 | 24 | 34 | 43 | 50 | 54 | 56 | 54 | 53 | 52 | 49 | 47 | 45 | 44 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 34 |
| .20 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 12 | 20 | 30 | 39 | 46 | 54 | 55 | 54 | 53 | 51 | 49 | 46 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 35 |
| .30 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 17 | 25 | 34 | 42 | 52 | 55 | 54 | 53 | 51 | 49 | 46 | 44 | 43 | 42 | 42 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 35 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 8 | 14 | 21 | 30 | 38 | 50 | 55 | 55 | 53 | 52 | 50 | 47 | 45 | 43 | 42 | 42 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 35 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 11 | 18 | 26 | 34 | 47 | 53 | 54 | 54 | 52 | 50 | 47 | 45 | 43 | 42 | 42 | 40 | 40 | 40 | 40 | 40 | 39 | 39 | 35 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 10 | 16 | 23 | 37 | 47 | 52 | 53 | 53 | 52 | 49 | 46 | 44 | 42 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 7 | 17 | 30 | 42 | 50 | 53 | 53 | 51 | 49 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 36 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 11 | 21 | 33 | 43 | 49 | 52 | 51 | 48 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 36 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 11 | 20 | 30 | 43 | 50 | 51 | 48 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 39 | 37 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 10 | 23 | 36 | 48 | 51 | 47 | 44 | 43 | 42 | 41 | 40 | 40 | 39 | 37 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 17 | 34 | 48 | 50 | 47 | 44 | 43 | 42 | 40 | 40 | 39 | 38 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.75 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 10 | 15 | 19 | 23 | 28 | 31 | 32 | 36 | 39 | 41 | 44 | 44 | 44 | 44 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 7 | 12 | 17 | 22 | 27 | 30 | 32 | 34 | 38 | 40 | 43 | 44 | 44 | 44 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 15 | 20 | 25 | 30 | 31 | 33 | 37 | 39 | 43 | 44 | 44 | 44 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 12 | 18 | 24 | 29 | 31 | 33 | 36 | 39 | 42 | 44 | 44 | 44 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 10 | 16 | 22 | 27 | 30 | 32 | 35 | 38 | 42 | 44 | 44 | 44 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 14 | 21 | 26 | 30 | 31 | 34 | 38 | 41 | 44 | 44 | 44 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 11 | 19 | 24 | 29 | 31 | 33 | 36 | 41 | 44 | 44 | 44 | 44 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 16 | 22 | 27 | 30 | 32 | 35 | 40 | 43 | 44 | 44 | 44 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 9 | 16 | 23 | 27 | 30 | 32 | 38 | 42 | 44 | 44 | 44 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 10 | 17 | 23 | 28 | 30 | 36 | 40 | 44 | 44 | 44 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 11 | 18 | 23 | 28 | 33 | 39 | 44 | 44 | 44 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 17 | 23 | 30 | 36 | 43 | 44 | 44 |
| | RAINFALL TYPE = IA | | | | | | | | | | | | | * * * TC = 0.75 HR * * * | | | | | | | | | | | | | | | | | SHEET 6 OF 10 | |

## Exhibit 5-IA, continued: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

| TRVL TIME (HR) | 7.0 | 7.3 | 7.6 | 7.9 | 8.0 | 8.1 | 8.2 | 8.3 | 8.4 | 8.5 | 8.6 | 8.7 | 8.8 | 9.0 | 9.2 | 9.4 | 9.6 | 9.8 | 10.0 | 10.3 | 10.6 | 11.0 | 11.5 | 12.0 | 12.5 | 13.0 | 13.5 | 14.0 | 15.0 | 16.0 | 18.0 | 22.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | IA/P = 0.10 | | | | | | | | | | | | * * * TC =1.0 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 19 | 23 | 26 | 33 | 37 | 43 | 52 | 63 | 76 | 88 | 97 | 104 | 108 | 102 | 91 | 81 | 72 | 65 | 59 | 52 | 48 | 43 | 40 | 38 | 34 | 33 | 33 | 32 | 31 | 30 | 28 | 23 |
| .10 | 19 | 22 | 26 | 32 | 35 | 41 | 48 | 58 | 70 | 82 | 92 | 100 | 105 | 103 | 94 | 83 | 75 | 67 | 61 | 53 | 48 | 44 | 40 | 38 | 35 | 33 | 33 | 32 | 31 | 30 | 28 | 23 |
| .20 | 17 | 21 | 24 | 28 | 30 | 34 | 38 | 45 | 54 | 65 | 76 | 87 | 96 | 104 | 100 | 91 | 81 | 73 | 66 | 57 | 51 | 45 | 41 | 39 | 36 | 33 | 33 | 33 | 31 | 30 | 28 | 23 |
| .30 | 16 | 20 | 23 | 27 | 29 | 32 | 36 | 42 | 50 | 60 | 71 | 81 | 91 | 103 | 101 | 93 | 84 | 75 | 67 | 58 | 52 | 46 | 42 | 39 | 36 | 34 | 33 | 33 | 31 | 30 | 28 | 23 |
| .40 | 15 | 18 | 22 | 25 | 27 | 29 | 31 | 35 | 40 | 46 | 55 | 65 | 76 | 94 | 102 | 99 | 91 | 81 | 73 | 63 | 55 | 48 | 43 | 40 | 37 | 34 | 33 | 33 | 31 | 30 | 28 | 23 |
| .50 | 14 | 18 | 21 | 25 | 26 | 28 | 30 | 33 | 37 | 43 | 51 | 61 | 71 | 90 | 101 | 101 | 93 | 84 | 75 | 64 | 56 | 49 | 43 | 40 | 37 | 34 | 33 | 33 | 31 | 30 | 28 | 23 |
| .75 | 13 | 16 | 19 | 23 | 24 | 25 | 27 | 29 | 32 | 37 | 42 | 49 | 58 | 76 | 91 | 98 | 96 | 89 | 81 | 70 | 60 | 51 | 45 | 41 | 38 | 35 | 34 | 33 | 32 | 30 | 28 | 23 |
| 1.0 | 10 | 13 | 16 | 19 | 20 | 22 | 23 | 24 | 26 | 27 | 30 | 33 | 37 | 50 | 67 | 83 | 94 | 97 | 93 | 81 | 70 | 58 | 48 | 43 | 40 | 37 | 34 | 33 | 32 | 31 | 29 | 24 |
| 1.5 | 6 | 8 | 10 | 13 | 14 | 15 | 16 | 18 | 19 | 20 | 21 | 22 | 24 | 27 | 33 | 42 | 56 | 71 | 84 | 93 | 89 | 75 | 59 | 49 | 44 | 40 | 37 | 35 | 33 | 31 | 29 | 24 |
| 2.0 | 3 | 5 | 7 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 21 | 23 | 27 | 32 | 41 | 53 | 74 | 88 | 91 | 74 | 59 | 49 | 43 | 40 | 37 | 34 | 32 | 30 | 25 |
| 2.5 | 1 | 2 | 4 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 11 | 12 | 13 | 16 | 18 | 20 | 23 | 26 | 32 | 45 | 64 | 84 | 89 | 72 | 58 | 48 | 43 | 40 | 34 | 33 | 30 | 26 |
| 3.0 | 0 | 1 | 2 | 3 | 3 | 4 | 4 | 5 | 6 | 6 | 7 | 8 | 9 | 10 | 12 | 14 | 16 | 18 | 21 | 26 | 35 | 55 | 81 | 87 | 73 | 59 | 50 | 44 | 37 | 34 | 31 | 26 |
| | | IA/P = 0.30 | | | | | | | | | | | | * * * TC =1.0 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 16 | 23 | 31 | 38 | 44 | 51 | 53 | 52 | 52 | 50 | 48 | 46 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .10 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 8 | 13 | 20 | 27 | 35 | 46 | 51 | 53 | 52 | 51 | 50 | 47 | 45 | 44 | 42 | 42 | 40 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 11 | 17 | 24 | 31 | 43 | 50 | 52 | 52 | 51 | 50 | 47 | 45 | 44 | 42 | 42 | 40 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 14 | 21 | 28 | 40 | 48 | 52 | 52 | 52 | 50 | 48 | 46 | 44 | 43 | 42 | 40 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 7 | 12 | 18 | 31 | 42 | 49 | 52 | 52 | 51 | 49 | 47 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 3 | 6 | 10 | 15 | 28 | 39 | 47 | 51 | 52 | 51 | 49 | 47 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 3 | 6 | 9 | 19 | 31 | 41 | 47 | 50 | 51 | 50 | 48 | 46 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 36 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 14 | 25 | 36 | 44 | 49 | 51 | 50 | 47 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 36 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 13 | 23 | 33 | 44 | 49 | 50 | 47 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 39 | 37 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 13 | 25 | 38 | 48 | 50 | 47 | 44 | 43 | 42 | 41 | 40 | 40 | 39 | 37 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 9 | 19 | 36 | 48 | 49 | 47 | 44 | 43 | 42 | 40 | 40 | 39 | 38 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 18 | 37 | 47 | 49 | 46 | 44 | 43 | 41 | 40 | 40 | 38 |
| | | IA/P = 0.50 | | | | | | | | | | | | * * * TC =1.0 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 7 | 11 | 16 | 21 | 26 | 30 | 32 | 34 | 38 | 40 | 43 | 43 | 43 | 43 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 14 | 19 | 25 | 29 | 31 | 33 | 37 | 39 | 42 | 43 | 43 | 43 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 11 | 17 | 23 | 28 | 30 | 32 | 36 | 39 | 42 | 43 | 43 | 43 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 9 | 15 | 21 | 27 | 30 | 32 | 35 | 38 | 41 | 43 | 43 | 43 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 13 | 20 | 25 | 29 | 31 | 34 | 37 | 41 | 43 | 43 | 43 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 11 | 18 | 24 | 28 | 31 | 33 | 36 | 41 | 43 | 43 | 43 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 16 | 22 | 27 | 30 | 32 | 35 | 40 | 43 | 43 | 43 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 13 | 20 | 25 | 29 | 31 | 34 | 39 | 42 | 43 | 43 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 14 | 20 | 25 | 29 | 31 | 37 | 41 | 43 | 43 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 14 | 20 | 26 | 29 | 34 | 39 | 43 | 43 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 8 | 15 | 21 | 26 | 32 | 37 | 43 | 43 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 8 | 14 | 20 | 29 | 34 | 42 | 43 |
| | | RAINFALL TYPE = IA | | | | | | | | | | | | * * * TC = 1.0 HR * * * | | | | | | | | | | | | | | | | | SHEET 7 OF 10 | |

## Exhibit 5-IA, continued: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 7.0 | 7.3 | 7.6 | 7.9 | 8.0 | 8.1 | 8.2 | 8.3 | 8.4 | 8.5 | 8.6 | 8.7 | 8.8 | 9.0 | 9.2 | 9.4 | 9.6 | 9.8 | 10.0 | 10.3 | 10.6 | 11.0 | 11.5 | 12.0 | 12.5 | 13.0 | 13.5 | 14.0 | 15.0 | 16.0 | 18.0 | 22.0 |
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =1.25 HR * * * | | | | | | | | | | | | | | | IA/P = 0.10 | | |
| 0.0 | 17 | 21 | 24 | 29 | 33 | 37 | 43 | 52 | 61 | 71 | 81 | 89 | 95 | 100 | 94 | 86 | 78 | 71 | 64 | 57 | 51 | 46 | 42 | 39 | 35 | 34 | 33 | 33 | 31 | 30 | 28 | 23 |
| .10 | 16 | 19 | 23 | 27 | 29 | 31 | 35 | 41 | 48 | 57 | 66 | 76 | 85 | 96 | 99 | 92 | 84 | 76 | 69 | 60 | 54 | 47 | 43 | 40 | 36 | 34 | 34 | 33 | 31 | 30 | 28 | 23 |
| .20 | 15 | 19 | 22 | 26 | 28 | 30 | 33 | 38 | 45 | 53 | 62 | 71 | 80 | 93 | 97 | 93 | 86 | 78 | 71 | 62 | 55 | 48 | 43 | 40 | 37 | 34 | 34 | 33 | 31 | 30 | 28 | 23 |
| .30 | 14 | 17 | 20 | 24 | 25 | 27 | 29 | 32 | 36 | 42 | 49 | 58 | 67 | 84 | 95 | 96 | 91 | 84 | 76 | 66 | 58 | 50 | 44 | 41 | 38 | 35 | 34 | 33 | 31 | 30 | 28 | 23 |
| .40 | 13 | 16 | 20 | 23 | 25 | 26 | 28 | 31 | 34 | 39 | 46 | 54 | 62 | 80 | 92 | 96 | 93 | 86 | 78 | 68 | 59 | 51 | 45 | 41 | 38 | 35 | 34 | 33 | 31 | 30 | 28 | 23 |
| .50 | 12 | 15 | 18 | 21 | 23 | 24 | 25 | 27 | 29 | 33 | 37 | 43 | 50 | 67 | 83 | 93 | 95 | 91 | 84 | 73 | 63 | 54 | 46 | 42 | 39 | 36 | 34 | 33 | 32 | 30 | 28 | 23 |
| .75 | 10 | 13 | 16 | 20 | 21 | 22 | 23 | 25 | 27 | 29 | 32 | 36 | 42 | 55 | 71 | 84 | 93 | 93 | 88 | 78 | 68 | 57 | 49 | 43 | 40 | 37 | 34 | 34 | 32 | 31 | 28 | 24 |
| 1.0 | 8 | 10 | 13 | 16 | 17 | 19 | 20 | 21 | 22 | 23 | 25 | 27 | 29 | 37 | 49 | 63 | 78 | 88 | 92 | 88 | 78 | 65 | 53 | 46 | 42 | 39 | 36 | 34 | 33 | 31 | 29 | 24 |
| 1.5 | 5 | 7 | 9 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 22 | 25 | 29 | 36 | 47 | 60 | 73 | 87 | 89 | 79 | 64 | 53 | 46 | 42 | 38 | 36 | 33 | 32 | 29 | 25 |
| 2.0 | 2 | 4 | 6 | 8 | 9 | 10 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 19 | 21 | 24 | 29 | 36 | 45 | 64 | 80 | 87 | 77 | 63 | 52 | 46 | 41 | 38 | 34 | 32 | 30 | 25 |
| 2.5 | 1 | 2 | 3 | 4 | 5 | 6 | 6 | 7 | 8 | 8 | 9 | 10 | 11 | 13 | 15 | 17 | 19 | 22 | 26 | 35 | 49 | 72 | 85 | 78 | 65 | 53 | 46 | 42 | 36 | 33 | 30 | 26 |
| 3.0 | 0 | 1 | 1 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 9 | 11 | 12 | 14 | 17 | 19 | 23 | 31 | 47 | 73 | 84 | 76 | 63 | 53 | 46 | 38 | 34 | 31 | 26 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =1.25 HR * * * | | | | | | | | | | | | | | | IA/P = 0.30 | | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 10 | 15 | 22 | 28 | 34 | 43 | 48 | 51 | 51 | 51 | 49 | 47 | 45 | 44 | 43 | 42 | 40 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 8 | 13 | 19 | 25 | 36 | 45 | 49 | 51 | 50 | 50 | 48 | 46 | 44 | 43 | 42 | 40 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 7 | 11 | 16 | 22 | 34 | 43 | 48 | 50 | 50 | 50 | 48 | 46 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 3 | 5 | 9 | 14 | 19 | 31 | 40 | 46 | 49 | 50 | 50 | 48 | 46 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 8 | 12 | 17 | 28 | 38 | 45 | 49 | 50 | 50 | 49 | 47 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 6 | 10 | 14 | 25 | 36 | 43 | 48 | 50 | 50 | 49 | 47 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 6 | 13 | 23 | 33 | 41 | 46 | 49 | 50 | 48 | 46 | 44 | 43 | 42 | 40 | 40 | 40 | 40 | 40 | 39 | 36 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 14 | 23 | 32 | 40 | 46 | 49 | 49 | 47 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 36 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 13 | 21 | 30 | 41 | 47 | 49 | 47 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 39 | 37 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 12 | 23 | 35 | 45 | 48 | 47 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 37 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 18 | 33 | 45 | 48 | 46 | 44 | 43 | 42 | 40 | 40 | 40 | 38 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 17 | 34 | 45 | 48 | 46 | 44 | 43 | 41 | 40 | 40 | 38 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =1.25 HR * * * | | | | | | | | | | | | | | | IA/P = 0.50 | | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 5 | 9 | 14 | 19 | 24 | 28 | 31 | 33 | 37 | 39 | 42 | 42 | 42 | 42 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 11 | 17 | 23 | 27 | 30 | 32 | 36 | 38 | 42 | 42 | 42 | 42 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 15 | 21 | 26 | 29 | 32 | 35 | 38 | 41 | 42 | 42 | 42 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 13 | 19 | 25 | 29 | 31 | 34 | 37 | 41 | 42 | 42 | 42 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 11 | 17 | 23 | 28 | 30 | 33 | 36 | 40 | 42 | 42 | 42 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 15 | 22 | 26 | 30 | 32 | 35 | 40 | 42 | 42 | 42 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 7 | 13 | 20 | 25 | 29 | 31 | 34 | 39 | 42 | 42 | 42 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 11 | 17 | 23 | 27 | 30 | 33 | 38 | 42 | 42 | 42 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 11 | 18 | 23 | 28 | 30 | 36 | 40 | 42 | 42 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 12 | 18 | 24 | 28 | 33 | 39 | 42 | 42 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 13 | 19 | 24 | 31 | 37 | 42 | 42 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 12 | 18 | 28 | 33 | 42 | 42 |
| | RAINFALL TYPE = IA | | | | | | | | | | | | | * * * TC = 1.25 HR * * * | | | | | | | | | | | | | | | SHEET 8 OF 10 | | |

## Exhibit 5-IA, continued: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

| TRVL TIME (HR) | 7.0 | 7.3 | 7.6 | 7.9 | 8.0 | 8.1 | 8.2 | 8.3 | 8.4 | 8.5 | 8.6 | 8.7 | 8.8 | 9.0 | 9.2 | 9.4 | 9.6 | 9.8 | 10.0 | 10.3 | 10.6 | 11.0 | 11.5 | 12.0 | 12.5 | 13.0 | 13.5 | 14.0 | 15.0 | 16.0 | 18.0 | 22.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | IA/P = 0.10 | | | | | | | | | | | | * * * TC =1.5 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 15 | 19 | 22 | 27 | 29 | 32 | 37 | 42 | 49 | 57 | 65 | 73 | 80 | 89 | 94 | 89 | 83 | 76 | 70 | 62 | 55 | 49 | 44 | 40 | 37 | 35 | 34 | 33 | 31 | 30 | 28 | 23 |
| .10 | 14 | 17 | 20 | 24 | 26 | 28 | 31 | 35 | 40 | 46 | 53 | 61 | 69 | 82 | 90 | 92 | 87 | 81 | 74 | 65 | 58 | 51 | 45 | 41 | 38 | 35 | 34 | 33 | 31 | 30 | 28 | 23 |
| .20 | 13 | 16 | 20 | 23 | 25 | 27 | 29 | 33 | 37 | 43 | 50 | 57 | 65 | 79 | 88 | 91 | 88 | 82 | 76 | 67 | 59 | 52 | 46 | 42 | 38 | 35 | 34 | 33 | 32 | 30 | 28 | 23 |
| .30 | 12 | 15 | 18 | 22 | 23 | 24 | 26 | 28 | 31 | 35 | 41 | 47 | 54 | 69 | 81 | 89 | 90 | 86 | 81 | 71 | 63 | 54 | 47 | 43 | 39 | 36 | 34 | 34 | 32 | 30 | 28 | 24 |
| .40 | 11 | 14 | 18 | 21 | 22 | 24 | 25 | 27 | 30 | 34 | 38 | 44 | 51 | 65 | 78 | 87 | 90 | 87 | 82 | 73 | 64 | 55 | 48 | 43 | 40 | 36 | 34 | 34 | 32 | 30 | 28 | 24 |
| .50 | 10 | 13 | 16 | 19 | 20 | 22 | 23 | 24 | 26 | 29 | 32 | 36 | 41 | 54 | 69 | 81 | 88 | 89 | 86 | 77 | 68 | 58 | 50 | 44 | 41 | 37 | 35 | 34 | 32 | 31 | 29 | 24 |
| .75 | 9 | 11 | 14 | 18 | 19 | 20 | 21 | 22 | 24 | 26 | 28 | 31 | 35 | 46 | 58 | 71 | 82 | 88 | 88 | 81 | 73 | 62 | 52 | 46 | 42 | 38 | 35 | 34 | 32 | 31 | 29 | 24 |
| 1.0 | 7 | 9 | 11 | 14 | 15 | 16 | 18 | 19 | 20 | 21 | 22 | 24 | 26 | 32 | 40 | 52 | 65 | 76 | 84 | 87 | 81 | 70 | 58 | 49 | 44 | 40 | 37 | 35 | 33 | 31 | 29 | 24 |
| 1.5 | 4 | 6 | 8 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 22 | 26 | 31 | 39 | 50 | 62 | 77 | 85 | 81 | 69 | 57 | 49 | 44 | 40 | 37 | 34 | 32 | 30 | 25 |
| 2.0 | 2 | 3 | 5 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 12 | 13 | 14 | 17 | 19 | 22 | 25 | 31 | 38 | 53 | 69 | 83 | 80 | 68 | 56 | 48 | 43 | 40 | 35 | 33 | 30 | 25 |
| 2.5 | 1 | 1 | 2 | 3 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 9 | 9 | 11 | 13 | 15 | 17 | 20 | 23 | 30 | 41 | 61 | 82 | 82 | 69 | 58 | 49 | 44 | 37 | 34 | 31 | 26 |
| 3.0 | 0 | 0 | 1 | 2 | 2 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 8 | 9 | 11 | 13 | 15 | 17 | 21 | 27 | 40 | 63 | 80 | 78 | 68 | 57 | 49 | 40 | 35 | 31 | 27 |
| | | IA/P = 0.30 | | | | | | | | | | | | * * * TC =1.5 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 6 | 10 | 14 | 19 | 24 | 34 | 41 | 46 | 48 | 49 | 48 | 47 | 46 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 35 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 3 | 5 | 8 | 12 | 17 | 27 | 36 | 42 | 46 | 48 | 49 | 48 | 46 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 36 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 7 | 10 | 15 | 24 | 33 | 41 | 45 | 48 | 49 | 48 | 47 | 45 | 43 | 43 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 36 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 6 | 9 | 13 | 22 | 31 | 39 | 44 | 47 | 48 | 48 | 47 | 45 | 44 | 43 | 41 | 40 | 40 | 40 | 40 | 40 | 39 | 36 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 3 | 5 | 7 | 15 | 24 | 33 | 40 | 45 | 47 | 48 | 47 | 46 | 44 | 43 | 42 | 40 | 40 | 40 | 40 | 40 | 39 | 36 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 6 | 13 | 22 | 31 | 38 | 44 | 47 | 48 | 48 | 46 | 44 | 43 | 42 | 40 | 40 | 40 | 40 | 40 | 39 | 36 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 2 | 4 | 9 | 16 | 24 | 33 | 39 | 44 | 47 | 48 | 47 | 45 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 39 | 36 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 12 | 20 | 29 | 36 | 44 | 47 | 48 | 46 | 44 | 43 | 42 | 40 | 40 | 40 | 40 | 40 | 36 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 12 | 19 | 30 | 39 | 46 | 47 | 46 | 44 | 43 | 42 | 40 | 40 | 40 | 40 | 37 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 8 | 17 | 28 | 40 | 46 | 47 | 45 | 44 | 42 | 41 | 40 | 40 | 40 | 37 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 13 | 26 | 40 | 47 | 47 | 45 | 43 | 42 | 40 | 40 | 40 | 38 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 12 | 28 | 41 | 46 | 46 | 45 | 43 | 41 | 40 | 40 | 38 |
| | | IA/P = 0.50 | | | | | | | | | | | | * * * TC =1.5 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 7 | 11 | 16 | 22 | 27 | 30 | 32 | 35 | 38 | 42 | 42 | 42 | 42 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 4 | 7 | 13 | 19 | 24 | 28 | 31 | 34 | 37 | 41 | 42 | 42 | 42 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 12 | 19 | 24 | 28 | 31 | 33 | 36 | 41 | 42 | 42 | 42 | 42 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 12 | 18 | 23 | 28 | 30 | 33 | 36 | 40 | 42 | 42 | 42 | 42 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 11 | 17 | 23 | 27 | 30 | 33 | 36 | 40 | 42 | 42 | 42 | 42 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 17 | 22 | 27 | 30 | 32 | 36 | 40 | 41 | 41 | 41 | 41 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 10 | 16 | 22 | 26 | 29 | 32 | 35 | 40 | 41 | 41 | 41 | 41 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 11 | 17 | 23 | 27 | 30 | 33 | 38 | 41 | 41 | 41 | 41 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 12 | 18 | 23 | 27 | 30 | 36 | 40 | 41 | 41 | 41 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 12 | 18 | 24 | 28 | 33 | 39 | 41 | 41 | 41 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 13 | 19 | 24 | 31 | 36 | 41 | 41 | 41 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 13 | 19 | 28 | 34 | 41 | 41 | 41 |
| | | RAINFALL TYPE = IA | | | | | | | | | | | | * * * TC = 1.5 HR * * * | | | | | | | | | | | | | | | | | SHEET 9 OF 10 | |

## Exhibit 5-IA, continued: Tabular hydrograph unit discharges (csm/in) for type IA rainfall distribution

| TRVL TIME (HR) | 7.0 | 7.3/ 7.6 | 7.9 8.0 | 8.1 8.2 | 8.3 8.4 | 8.5 8.6 | 8.7 8.8 | 9.0 9.2 | 9.4 9.6 | 9.8 10.0 | 10.3 10.6 | 11.0 11.5 | 12.0 12.5 | 13.0 13.5 | 14.0 15.0 | 16.0 18.0 | 22.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | IA/P = 0.10 | | | | | | | * * * TC =2.0 HR * * * | | | | | | | | IA/P = 0.10 | |
| 0.0 | 12 | 15 | 18 | 22 | 24 | 26 | 28 | 32 | 36 | 41 | 46 | 52 | 58 | 68 | 76 | 80 | 84 | 79 | 76 | 68 | 62 | 55 | 48 | 44 | 40 | 37 | 35 | 34 | 32 | 31 | 28 | 24 |
| .10 | 11 | 13 | 16 | 20 | 21 | 23 | 25 | 27 | 30 | 34 | 38 | 43 | 49 | 60 | 70 | 77 | 83 | 83 | 78 | 72 | 65 | 57 | 50 | 45 | 41 | 38 | 36 | 34 | 32 | 31 | 29 | 24 |
| .20 | 9 | 12 | 15 | 18 | 19 | 21 | 22 | 24 | 26 | 29 | 32 | 36 | 41 | 52 | 63 | 72 | 78 | 82 | 81 | 75 | 68 | 60 | 52 | 46 | 42 | 39 | 36 | 35 | 32 | 31 | 29 | 24 |
| .30 | 9 | 12 | 14 | 18 | 19 | 20 | 21 | 23 | 25 | 28 | 31 | 34 | 39 | 49 | 60 | 70 | 77 | 81 | 81 | 76 | 70 | 61 | 53 | 47 | 43 | 39 | 36 | 35 | 33 | 31 | 29 | 24 |
| .40 | 9 | 11 | 14 | 17 | 18 | 19 | 21 | 22 | 24 | 26 | 29 | 33 | 37 | 47 | 57 | 67 | 75 | 80 | 81 | 77 | 71 | 62 | 53 | 47 | 43 | 39 | 37 | 35 | 33 | 31 | 29 | 24 |
| .50 | 7 | 10 | 12 | 15 | 17 | 18 | 19 | 20 | 21 | 23 | 25 | 28 | 31 | 39 | 49 | 60 | 69 | 76 | 81 | 79 | 74 | 65 | 56 | 49 | 44 | 40 | 37 | 35 | 33 | 31 | 29 | 24 |
| .75 | 6 | 9 | 11 | 14 | 15 | 16 | 17 | 18 | 20 | 21 | 23 | 25 | 27 | 34 | 42 | 52 | 62 | 70 | 76 | 79 | 76 | 68 | 59 | 51 | 46 | 41 | 38 | 36 | 33 | 31 | 29 | 24 |
| 1.0 | 5 | 6 | 9 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 20 | 21 | 25 | 30 | 38 | 47 | 57 | 66 | 76 | 79 | 75 | 64 | 55 | 49 | 44 | 40 | 37 | 34 | 32 | 29 | 25 |
| 1.5 | 3 | 4 | 6 | 8 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 18 | 21 | 24 | 30 | 37 | 45 | 59 | 71 | 78 | 73 | 63 | 55 | 48 | 43 | 40 | 35 | 33 | 30 | 25 |
| 2.0 | 1 | 2 | 3 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 10 | 12 | 14 | 16 | 19 | 22 | 26 | 36 | 48 | 65 | 77 | 73 | 64 | 56 | 49 | 44 | 37 | 34 | 31 | 26 |
| 2.5 | 0 | 1 | 1 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 10 | 12 | 14 | 16 | 18 | 23 | 31 | 46 | 66 | 76 | 72 | 63 | 55 | 48 | 40 | 35 | 31 | 27 |
| 3.0 | 0 | 0 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 7 | 8 | 10 | 11 | 13 | 17 | 21 | 30 | 49 | 67 | 75 | 71 | 63 | 54 | 43 | 37 | 32 | 27 |
| | IA/P = 0.30 | | | | | | | * * * TC =2.0 HR * * * | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 5 | 7 | 10 | 13 | 20 | 27 | 34 | 39 | 42 | 44 | 47 | 45 | 45 | 44 | 43 | 42 | 40 | 40 | 40 | 40 | 40 | 39 | 36 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 2 | 4 | 6 | 8 | 15 | 22 | 29 | 35 | 40 | 43 | 45 | 46 | 45 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 36 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 2 | 3 | 5 | 10 | 17 | 24 | 31 | 36 | 41 | 44 | 46 | 45 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 36 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 3 | 4 | 9 | 15 | 22 | 29 | 35 | 39 | 44 | 46 | 45 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 36 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 2 | 4 | 8 | 14 | 20 | 27 | 33 | 38 | 43 | 46 | 46 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 40 | 36 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 2 | 5 | 9 | 15 | 22 | 29 | 35 | 41 | 44 | 45 | 45 | 44 | 43 | 42 | 40 | 40 | 40 | 40 | 40 | 36 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 3 | 6 | 11 | 17 | 24 | 30 | 38 | 42 | 45 | 45 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 40 | 37 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 14 | 21 | 30 | 38 | 43 | 45 | 44 | 44 | 43 | 41 | 40 | 40 | 40 | 40 | 37 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 8 | 16 | 25 | 36 | 43 | 45 | 44 | 43 | 42 | 41 | 40 | 40 | 40 | 38 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 12 | 24 | 36 | 43 | 45 | 44 | 43 | 42 | 40 | 40 | 40 | 38 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 12 | 25 | 37 | 43 | 45 | 44 | 43 | 41 | 40 | 40 | 38 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 13 | 26 | 37 | 43 | 44 | 44 | 42 | 40 | 40 | 39 |
| | IA/P = 0.50 | | | | | | | * * * TC =2.0 HR * * * | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 7 | 12 | 18 | 23 | 27 | 30 | 33 | 36 | 40 | 40 | 40 | 40 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 5 | 10 | 16 | 21 | 26 | 29 | 32 | 35 | 40 | 40 | 40 | 40 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 8 | 14 | 20 | 25 | 28 | 31 | 34 | 39 | 40 | 40 | 40 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 12 | 18 | 23 | 27 | 30 | 33 | 38 | 40 | 40 | 40 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 11 | 16 | 22 | 26 | 29 | 32 | 38 | 40 | 40 | 40 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 9 | 15 | 20 | 25 | 28 | 31 | 37 | 40 | 40 | 40 | 40 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 13 | 19 | 24 | 27 | 30 | 36 | 40 | 40 | 40 | 40 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 14 | 20 | 24 | 28 | 34 | 39 | 40 | 40 | 40 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 10 | 16 | 21 | 25 | 32 | 37 | 40 | 40 | 40 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 11 | 16 | 21 | 29 | 35 | 40 | 40 | 40 | 40 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 11 | 17 | 26 | 32 | 40 | 40 | 40 | 40 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 11 | 21 | 29 | 39 | 40 | 40 | 40 | 40 |
| | RAINFALL TYPE = IA | | | | | | | * * * TC = 2.0 HR * * * | | | | | | | | SHEET 10 OF 10 | |

## Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
| IA/P = 0.10 | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 24 | 34 | 53 | 334 | 647 | 1010 | 623 | 217 | 147 | 123 | 104 | 86 | 76 | 66 | 57 | 51 | 46 | 42 | 38 | 34 | 32 | 29 | 26 | 23 | 21 | 20 | 19 | 18 | 15 | 13 | 12 | 0 |
| .10 | 21 | 29 | 43 | 134 | 267 | 520 | 847 | 701 | 378 | 224 | 157 | 122 | 98 | 75 | 64 | 56 | 50 | 45 | 41 | 36 | 33 | 30 | 27 | 24 | 21 | 20 | 19 | 18 | 16 | 13 | 12 | 0 |
| .20 | 18 | 25 | 35 | 61 | 110 | 215 | 418 | 704 | 702 | 486 | 312 | 209 | 151 | 94 | 73 | 62 | 54 | 49 | 44 | 38 | 34 | 31 | 28 | 25 | 22 | 21 | 19 | 18 | 16 | 14 | 12 | 0 |
| .30 | 17 | 23 | 33 | 56 | 92 | 174 | 337 | 582 | 662 | 545 | 389 | 269 | 190 | 109 | 79 | 65 | 56 | 50 | 45 | 39 | 35 | 32 | 29 | 25 | 22 | 21 | 20 | 18 | 16 | 14 | 12 | 0 |
| .40 | 15 | 20 | 28 | 41 | 51 | 78 | 142 | 272 | 478 | 601 | 563 | 447 | 328 | 172 | 104 | 76 | 63 | 55 | 49 | 42 | 37 | 33 | 29 | 26 | 23 | 21 | 20 | 19 | 17 | 14 | 12 | 0 |
| .50 | 14 | 19 | 26 | 39 | 47 | 68 | 117 | 220 | 392 | 531 | 553 | 482 | 380 | 209 | 121 | 84 | 67 | 57 | 51 | 43 | 38 | 33 | 30 | 27 | 23 | 21 | 20 | 19 | 17 | 14 | 12 | 0 |
| .75 | 12 | 15 | 21 | 29 | 33 | 38 | 49 | 73 | 126 | 224 | 343 | 432 | 464 | 385 | 252 | 156 | 103 | 76 | 62 | 50 | 43 | 36 | 31 | 28 | 25 | 22 | 21 | 19 | 17 | 15 | 12 | 0 |
| 1.0 | 9 | 12 | 15 | 21 | 23 | 26 | 29 | 33 | 40 | 55 | 86 | 148 | 238 | 406 | 434 | 317 | 205 | 130 | 89 | 62 | 50 | 41 | 34 | 30 | 27 | 24 | 22 | 20 | 18 | 16 | 12 | 0 |
| 1.5 | 7 | 8 | 10 | 14 | 15 | 16 | 18 | 20 | 22 | 25 | 29 | 34 | 45 | 101 | 220 | 339 | 373 | 320 | 234 | 131 | 80 | 53 | 40 | 34 | 30 | 27 | 24 | 21 | 19 | 17 | 12 | 2 |
| 2.0 | 4 | 6 | 7 | 9 | 9 | 10 | 11 | 12 | 13 | 15 | 16 | 18 | 20 | 25 | 37 | 72 | 150 | 252 | 336 | 312 | 216 | 109 | 58 | 42 | 34 | 30 | 27 | 24 | 20 | 18 | 13 | 8 |
| 2.5 | 3 | 4 | 5 | 6 | 7 | 7 | 8 | 8 | 9 | 10 | 11 | 12 | 13 | 16 | 19 | 25 | 39 | 75 | 142 | 262 | 308 | 229 | 108 | 58 | 41 | 34 | 30 | 27 | 22 | 19 | 14 | 11 |
| 3.0 | 1 | 2 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 8 | 10 | 12 | 14 | 17 | 22 | 31 | 76 | 169 | 288 | 236 | 122 | 64 | 43 | 35 | 30 | 24 | 20 | 16 | 11 |
| IA/P = 0.30 | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 154 | 568 | 936 | 524 | 217 | 172 | 149 | 126 | 107 | 97 | 86 | 76 | 69 | 63 | 58 | 53 | 48 | 46 | 42 | 38 | 34 | 31 | 30 | 28 | 27 | 24 | 20 | 19 | 0 |
| .10 | 0 | 0 | 0 | 19 | 109 | 415 | 762 | 603 | 346 | 230 | 176 | 143 | 119 | 96 | 84 | 74 | 68 | 62 | 57 | 50 | 47 | 44 | 40 | 35 | 32 | 30 | 29 | 27 | 24 | 21 | 19 | 0 |
| .20 | 0 | 0 | 0 | 0 | 13 | 77 | 302 | 609 | 605 | 432 | 297 | 217 | 167 | 115 | 94 | 81 | 73 | 66 | 60 | 53 | 48 | 45 | 41 | 37 | 33 | 31 | 29 | 28 | 25 | 21 | 19 | 0 |
| .30 | 0 | 0 | 0 | 0 | 9 | 54 | 219 | 479 | 563 | 476 | 357 | 263 | 199 | 129 | 99 | 85 | 75 | 68 | 62 | 54 | 49 | 45 | 41 | 37 | 33 | 31 | 29 | 28 | 25 | 21 | 19 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 6 | 38 | 159 | 372 | 500 | 484 | 399 | 309 | 183 | 123 | 96 | 82 | 73 | 66 | 58 | 51 | 46 | 42 | 38 | 34 | 31 | 30 | 28 | 25 | 22 | 19 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 4 | 27 | 115 | 287 | 429 | 465 | 421 | 346 | 213 | 138 | 103 | 86 | 76 | 68 | 59 | 52 | 47 | 43 | 39 | 34 | 32 | 30 | 29 | 25 | 22 | 19 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 10 | 46 | 132 | 246 | 338 | 381 | 341 | 243 | 165 | 119 | 94 | 80 | 67 | 58 | 50 | 45 | 41 | 37 | 33 | 31 | 29 | 26 | 23 | 19 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 22 | 69 | 149 | 241 | 357 | 331 | 246 | 170 | 122 | 96 | 76 | 64 | 54 | 47 | 42 | 38 | 34 | 32 | 30 | 27 | 24 | 19 | 0 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 41 | 142 | 258 | 310 | 285 | 224 | 142 | 97 | 71 | 55 | 47 | 43 | 39 | 35 | 32 | 29 | 25 | 20 | 4 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 10 | 49 | 130 | 221 | 279 | 255 | 182 | 108 | 70 | 55 | 47 | 42 | 38 | 34 | 30 | 27 | 20 | 11 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 14 | 52 | 119 | 224 | 256 | 193 | 107 | 70 | 55 | 47 | 42 | 38 | 32 | 28 | 22 | 17 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 52 | 141 | 240 | 199 | 117 | 74 | 56 | 48 | 43 | 35 | 30 | 24 | 18 |
| IA/P = 0.50 | | | | | | | | | | * * * TC =0.1 HR * * * | | | | | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 70 | 539 | 377 | 196 | 171 | 154 | 134 | 117 | 108 | 99 | 89 | 83 | 77 | 72 | 67 | 61 | 59 | 56 | 51 | 46 | 43 | 42 | 40 | 38 | 34 | 30 | 28 | 0 |
| .10 | 0 | 0 | 0 | 0 | 47 | 375 | 376 | 256 | 199 | 169 | 146 | 126 | 114 | 102 | 92 | 85 | 79 | 73 | 68 | 62 | 59 | 56 | 52 | 47 | 43 | 42 | 40 | 38 | 34 | 30 | 28 | 0 |
| .20 | 0 | 0 | 0 | 0 | 0 | 31 | 260 | 338 | 283 | 227 | 189 | 160 | 138 | 112 | 99 | 90 | 83 | 77 | 72 | 64 | 60 | 57 | 53 | 48 | 44 | 42 | 41 | 39 | 35 | 30 | 28 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 21 | 180 | 285 | 284 | 246 | 208 | 176 | 131 | 110 | 97 | 88 | 82 | 76 | 68 | 62 | 59 | 54 | 50 | 45 | 43 | 41 | 39 | 36 | 31 | 28 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 14 | 125 | 232 | 266 | 253 | 223 | 192 | 142 | 115 | 100 | 91 | 83 | 77 | 69 | 63 | 59 | 55 | 50 | 45 | 43 | 41 | 40 | 36 | 31 | 28 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 9 | 86 | 183 | 239 | 248 | 231 | 205 | 154 | 122 | 104 | 93 | 85 | 79 | 71 | 64 | 59 | 55 | 51 | 46 | 43 | 41 | 40 | 36 | 32 | 28 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 31 | 87 | 147 | 190 | 211 | 213 | 184 | 147 | 121 | 103 | 92 | 84 | 75 | 67 | 61 | 57 | 52 | 47 | 44 | 42 | 40 | 37 | 32 | 28 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 13 | 45 | 92 | 141 | 205 | 197 | 165 | 134 | 112 | 98 | 84 | 75 | 65 | 59 | 55 | 50 | 46 | 43 | 41 | 38 | 34 | 28 | 0 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 51 | 118 | 170 | 183 | 167 | 143 | 111 | 92 | 77 | 65 | 59 | 54 | 50 | 45 | 43 | 39 | 35 | 28 | 2 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 15 | 51 | 103 | 148 | 168 | 156 | 127 | 96 | 76 | 65 | 58 | 54 | 49 | 45 | 41 | 37 | 29 | 12 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 31 | 69 | 131 | 159 | 140 | 101 | 78 | 66 | 59 | 54 | 50 | 43 | 39 | 31 | 24 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 11 | 46 | 101 | 151 | 134 | 99 | 77 | 65 | 59 | 54 | 45 | 41 | 33 | 26 |
| RAINFALL TYPE = II | | | | | | | | | | * * * TC = 0.1 HR * * * | | | | | | | | | | | | | | | | | | | | SHEET 1 OF 10 | |

# Exhibit 5-II, continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | 11.3 | 11.9 | 12.1 | 12.3 | 12.5 | 12.7 | 13.0 | 13.4 | 13.8 | 14.3 | 15.0 | 16.0 | 17.0 | 18.0 | 20.0 | 26.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.6 | 12.0 | 12.2 | 12.4 | 12.6 | 12.8 | 13.2 | 13.6 | 14.0 | 14.6 | 15.5 | 16.5 | 17.5 | 19.0 | 22.0 |

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 — * * * TC =0.2 HR * * * — IA/P = 0.10

| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | 23 | 31 | 47 | 209 | 403 | 739 | 800 | 481 | 250 | 166 | 128 | 102 | 86 | 70 | 61 | 54 | 49 | 44 | 40 | 35 | 33 | 30 | 27 | 24 | 21 | 20 | 19 | 18 | 16 | 13 | 12 | 0 |
| .10 | 19 | 26 | 39 | 86 | 168 | 325 | 601 | 733 | 565 | 355 | 229 | 161 | 122 | 83 | 69 | 59 | 53 | 47 | 43 | 37 | 34 | 31 | 28 | 25 | 22 | 21 | 19 | 18 | 16 | 14 | 12 | 0 |
| .20 | 17 | 23 | 32 | 49 | 74 | 136 | 262 | 488 | 652 | 594 | 435 | 298 | 207 | 115 | 81 | 67 | 58 | 51 | 46 | 40 | 35 | 32 | 29 | 26 | 23 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .30 | 16 | 22 | 30 | 46 | 64 | 112 | 212 | 396 | 566 | 585 | 485 | 360 | 258 | 139 | 90 | 71 | 60 | 53 | 48 | 41 | 36 | 32 | 29 | 26 | 23 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .40 | 14 | 19 | 25 | 37 | 43 | 57 | 94 | 173 | 322 | 485 | 551 | 507 | 409 | 227 | 129 | 87 | 68 | 58 | 52 | 44 | 38 | 33 | 30 | 27 | 24 | 21 | 20 | 19 | 17 | 14 | 12 | 0 |
| .50 | 13 | 18 | 24 | 35 | 40 | 52 | 80 | 142 | 262 | 410 | 504 | 506 | 441 | 269 | 153 | 98 | 73 | 61 | 53 | 45 | 39 | 34 | 30 | 27 | 24 | 22. | 20 | 19 | 17 | 15 | 12 | 0 |
| .75 | 10 | 13 | 17 | 23 | 26 | 30 | 34 | 40 | 55 | 86 | 150 | 247 | 349 | 438 | 360 | 240 | 151 | 101 | 75 | 57 | 47 | 39 | 33 | 29 | 26 | 23 | 21 | 20 | 18 | 15 | 12 | 0 |
| 1.0 | 9 | 11 | 14 | 19 | 21 | 24 | 26 | 30 | 35 | 44 | 62 | 101 | 167 | 337 | 413 | 353 | 245 | 157 | 104 | 68 | 53 | 42 | 35 | 31 | 28 | 24 | 22 | 20 | 18 | 16 | 12 | 0 |
| 1.5 | 6 | 8 | 10 | 13 | 14 | 15 | 17 | 19 | 21 | 23 | 26 | 30 | 37 | 73 | 166 | 288 | 356 | 337 | 264 | 154 | 91 | 57 | 42 | 35 | 30 | 27 | 24 | 22 | 19 | 17 | 13 | 3 |
| 2.0 | 4 | 5 | 7 | 8 | 9 | 10 | 10 | 11 | 12 | 14 | 15 | 16 | 18 | 23 | 31 | 55 | 114 | 206 | 291 | 324 | 239 | 125 | 63 | 44 | 35 | 31 | 28 | 24 | 20 | 18 | 14 | 9 |
| 2.5 | 3 | 4 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 11 | 12 | 15 | 18 | 22 | 32 | 58 | 111 | 227 | 298 | 246 | 122 | 63 | 43 | 35 | 31 | 27 | 22 | 19 | 15 | 11 |
| 3.0 | 1 | 2 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 11 | 13 | 16 | 19 | 27 | 59 | 138 | 280 | 248 | 137 | 70 | 46 | 36 | 31 | 25 | 21 | 16 | 11 |

IA/P = 0.30 — * * * TC =0.2 HR * * * — IA/P = 0.30

| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | 0 | 0 | 0 | 39 | 180 | 545 | 697 | 497 | 276 | 198 | 158 | 130 | 110 | 93 | 81 | 73 | 67 | 61 | 56 | 49 | 46 | 43 | 39 | 35 | 32 | 30 | 29 | 27 | 24 | 21 | 19 | 0 |
| .10 | 0 | 0 | 0 | 2 | 27 | 129 | 407 | 600 | 532 | 361 | 252 | 190 | 150 | 108 | 90 | 79 | 71 | 65 | 59 | 52 | 48 | 44 | 41 | 36 | 32 | 31 | 29 | 28 | 25 | 21 | 19 | 0 |
| .20 | 0 | 0 | 0 | 2 | 19 | 92 | 302 | 501 | 521 | 415 | 306 | 228 | 176 | 119 | 95 | 82 | 73 | 67 | 61 | 53 | 48 | 45 | 41 | 37 | 33 | 31 | 29 | 28 | 25 | 21 | 19 | 0 |
| .30 | 0 | 0 | 0 | 0 | 1 | 13 | 66 | 223 | 408 | 484 | 438 | 350 | 269 | 163 | 114 | 93 | 80 | 72 | 65 | 57 | 51 | 46 | 42 | 38 | 34 | 31 | 30 | 28 | 25 | 22 | 19 | 0 |
| .40 | 0 | 0 | 0 | 0 | 1 | 9 | 47 | 164 | 327 | 431 | 436 | 379 | 306 | 189 | 127 | 98 | 83 | 74 | 67 | 58 | 52 | 47 | 43 | 38 | 34 | 31 | 30 | 28 | 25 | 22 | 19 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 6 | 33 | 120 | 258 | 374 | 415 | 391 | 271 | 173 | 121 | 95 | 81 | 72 | 62 | 55 | 48 | 44 | 40 | 35 | 32 | 30 | 29 | 26 | 22 | 19 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 13 | 50 | 126 | 221 | 302 | 348 | 323 | 240 | 167 | 121 | 96 | 81 | 68 | 59 | 50 | 45 | 41 | 37 | 33 | 31 | 29 | 26 | 23 | 19 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 24 | 69 | 139 | 285 | 331 | 280 | 204 | 145 | 109 | 82 | 68 | 56 | 48 | 43 | 39 | 35 | 32 | 30 | 27 | 24 | 19 | 0 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 16 | 79 | 186 | 271 | 288 | 247 | 165 | 110 | 76 | 58 | 49 | 44 | 40 | 35 | 32 | 29 | 26 | 20 | 5 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 24 | 80 | 163 | 235 | 262 | 202 | 123 | 76 | 58 | 49 | 43 | 39 | 35 | 30 | 27 | 21 | 13 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 28 | 77 | 179 | 242 | 207 | 120 | 75 | 57 | 48 | 43 | 39 | 32 | 29 | 22 | 17 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 30 | 101 | 207 | 227 | 130 | 80 | 59 | 49 | 44 | 35 | 30 | 24 | 18 |

IA/P = 0.50 — * * * TC =0.2 HR * * * — IA/P = 0.50

| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | 0 | 0 | 0 | 0 | 7 | 98 | 371 | 322 | 221 | 182 | 158 | 137 | 120 | 104 | 94 | 86 | 80 | 74 | 69 | 62 | 60 | 57 | 52 | 47 | 44 | 42 | 40 | 39 | 35 | 30 | 28 | 0 |
| .10 | 0 | 0 | 0 | 0 | 4 | 67 | 270 | 305 | 249 | 204 | 174 | 149 | 130 | 108 | 97 | 88 | 82 | 76 | 71 | 64 | 60 | 57 | 53 | 48 | 44 | 42 | 41 | 39 | 35 | 30 | 28 | 0 |
| .20 | 0 | 0 | 0 | 0 | 0 | 3 | 45 | 195 | 268 | 255 | 221 | 189 | 163 | 125 | 106 | 95 | 87 | 80 | 75 | 67 | 62 | 58 | 54 | 49 | 45 | 43 | 41 | 39 | 35 | 31 | 28 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 2 | 31 | 140 | 226 | 245 | 229 | 203 | 176 | 134 | 111 | 98 | 89 | 82 | 76 | 68 | 62 | 59 | 55 | 50 | 45 | 43 | 41 | 39 | 36 | 31 | 28 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 1 | 21 | 101 | 184 | 225 | 228 | 211 | 188 | 144 | 117 | 101 | 91 | 84 | 78 | 69 | 63 | 59 | 55 | 50 | 45 | 43 | 41 | 40 | 36 | 31 | 28 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 14 | 72 | 146 | 199 | 218 | 213 | 175 | 137 | 113 | 99 | 89 | 82 | 73 | 66 | 60 | 56 | 52 | 47 | 43 | 42 | 40 | 36 | 32 | 28 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 28 | 71 | 121 | 162 | 186 | 193 | 161 | 133 | 112 | 98 | 88 | 78 | 70 | 62 | 57 | 53 | 48 | 44 | 42 | 41 | 37 | 33 | 28 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 13 | 38 | 77 | 154 | 186 | 174 | 147 | 122 | 105 | 89 | 78 | 68 | 60 | 56 | 51 | 46 | 43 | 42 | 38 | 34 | 28 | 0 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 22 | 71 | 129 | 163 | 168 | 150 | 120 | 98 | 80 | 67 | 60 | 55 | 51 | 46 | 43 | 40 | 36 | 28 | 4 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 25 | 65 | 112 | 146 | 157 | 134 | 103 | 79 | 67 | 60 | 55 | 50 | 46 | 41 | 38 | 29 | 14 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 26 | 60 | 117 | 148 | 136 | 101 | 79 | 66 | 59 | 54 | 50 | 43 | 39 | 31 | 24 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 40 | 90 | 142 | 130 | 99 | 78 | 66 | 59 | 54 | 45 | 41 | 33 | 26 |

RAINFALL TYPE = II — * * * TC = 0.2 HR * * * — SHEET 2 OF 10

## Exhibit 5-II, continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.3 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 20 | 28 | 41 | 118 | 235 | 447 | 676 | 676 | 459 | 283 | 196 | 146 | 114 | 80 | 66 | 57 | 51 | 46 | 42 | 37 | 33 | 31 | 28 | 24 | 22 | 20 | 19 | 18 | 16 | 13 | 12 | 0 |
| .10 | 19 | 26 | 39 | 99 | 189 | 361 | 571 | 641 | 520 | 362 | 251 | 181 | 136 | 89 | 70 | 60 | 53 | 48 | 43 | 37 | 34 | 31 | 28 | 25 | 22 | 21 | 19 | 18 | 16 | 14 | 12 | 0 |
| .20 | 17 | 23 | 32 | 53 | 83 | 154 | 292 | 478 | 587 | 542 | 422 | 308 | 223 | 127 | 86 | 68 | 58 | 52 | 46 | 40 | 35 | 32 | 29 | 26 | 23 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .30 | 16 | 22 | 30 | 49 | 72 | 127 | 237 | 398 | 524 | 536 | 460 | 359 | 268 | 151 | 97 | 73 | 61 | 53 | 48 | 41 | 36 | 32 | 29 | 26 | 23 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .40 | 14 | 19 | 25 | 37 | 45 | 63 | 105 | 193 | 330 | 459 | 510 | 477 | 398 | 237 | 139 | 92 | 70 | 59 | 52 | 44 | 38 | 34 | 30 | 27 | 24 | 21 | 20 | 19 | 17 | 14 | 12 | 0 |
| .50 | 13 | 18 | 24 | 35 | 42 | 56 | 89 | 158 | 272 | 397 | 472 | 475 | 424 | 274 | 163 | 104 | 76 | 62 | 54 | 46 | 39 | 34 | 30 | 27 | 24 | 22 | 20 | 19 | 17 | 15 | 12 | 0 |
| .75 | 11 | 14 | 19 | 26 | 30 | 34 | 42 | 59 | 95 | 160 | 250 | 339 | 417 | 398 | 299 | 196 | 128 | 89 | 69 | 54 | 45 | 37 | 32 | 29 | 26 | 23 | 21 | 20 | 17 | 15 | 12 | 0 |
| 1.0 | 9 | 11 | 14 | 19 | 21 | 24 | 27 | 30 | 36 | 46 | 68 | 109 | 174 | 328 | 396 | 346 | 248 | 163 | 109 | 70 | 54 | 43 | 35 | 31 | 28 | 24 | 22 | 20 | 18 | 16 | 12 | 0 |
| 1.5 | 6 | 8 | 10 | 13 | 14 | 15 | 17 | 19 | 21 | 23 | 26 | 31 | 38 | 77 | 169 | 282 | 347 | 330 | 264 | 158 | 94 | 58 | 42 | 35 | 31 | 27 | 24 | 22 | 19 | 17 | 13 | 3 |
| 2.0 | 4 | 5 | 7 | 8 | 9 | 10 | 10 | 11 | 12 | 14 | 15 | 16 | 18 | 23 | 32 | 57 | 116 | 205 | 285 | 317 | 239 | 128 | 64 | 44 | 36 | 31 | 28 | 25 | 20 | 18 | 14 | 9 |
| 2.5 | 2 | 4 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 11 | 12 | 15 | 18 | 23 | 33 | 60 | 113 | 223 | 293 | 245 | 125 | 65 | 44 | 35 | 31 | 27 | 22 | 19 | 15 | 11 |
| 3.0 | 1 | 2 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 11 | 13 | 16 | 20 | 27 | 61 | 138 | 275 | 246 | 139 | 72 | 46 | 36 | 31 | 25 | 21 | 16 | 11 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.3 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 11 | 64 | 251 | 525 | 574 | 454 | 303 | 221 | 173 | 140 | 104 | 88 | 77 | 70 | 64 | 58 | 51 | 47 | 44 | 40 | 36 | 32 | 31 | 29 | 28 | 24 | 21 | 19 | 0 |
| .10 | 0 | 0 | 0 | 0 | 7 | 45 | 183 | 411 | 520 | 476 | 360 | 268 | 205 | 133 | 101 | 85 | 76 | 69 | 62 | 55 | 49 | 45 | 41 | 37 | 33 | 31 | 30 | 28 | 25 | 21 | 19 | 0 |
| .20 | 0 | 0 | 0 | 0 | 5 | 32 | 132 | 318 | 452 | 468 | 396 | 310 | 240 | 151 | 109 | 90 | 78 | 70 | 64 | 56 | 50 | 46 | 42 | 38 | 33 | 31 | 30 | 28 | 25 | 22 | 19 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 3 | 22 | 96 | 244 | 383 | 440 | 411 | 344 | 217 | 142 | 105 | 87 | 76 | 69 | 60 | 53 | 47 | 43 | 39 | 35 | 32 | 30 | 29 | 26 | 22 | 19 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 2 | 16 | 69 | 186 | 317 | 399 | 407 | 365 | 246 | 160 | 115 | 92 | 79 | 71 | 61 | 54 | 48 | 43 | 39 | 35 | 32 | 30 | 29 | 26 | 22 | 19 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 11 | 50 | 140 | 258 | 352 | 389 | 327 | 223 | 149 | 110 | 89 | 77 | 66 | 57 | 50 | 45 | 41 | 36 | 33 | 31 | 29 | 26 | 23 | 19 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 20 | 63 | 135 | 219 | 290 | 335 | 281 | 205 | 146 | 110 | 89 | 72 | 62 | 52 | 46 | 42 | 38 | 34 | 31 | 30 | 27 | 23 | 19 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 32 | 78 | 216 | 320 | 306 | 243 | 176 | 128 | 90 | 72 | 59 | 49 | 44 | 40 | 36 | 33 | 31 | 28 | 24 | 19 | 1 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 20 | 84 | 185 | 264 | 281 | 246 | 168 | 112 | 77 | 58 | 49 | 44 | 40 | 36 | 32 | 29 | 26 | 20 | 5 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 12 | 50 | 121 | 200 | 257 | 224 | 141 | 83 | 61 | 50 | 44 | 40 | 36 | 31 | 28 | 21 | 14 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 16 | 51 | 145 | 239 | 223 | 137 | 82 | 60 | 50 | 44 | 40 | 33 | 29 | 22 | 17 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 19 | 74 | 184 | 224 | 146 | 89 | 63 | 51 | 45 | 36 | 31 | 24 | 18 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.3 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 1 | 25 | 151 | 299 | 277 | 219 | 187 | 162 | 141 | 113 | 100 | 90 | 84 | 78 | 72 | 65 | 61 | 58 | 53 | 48 | 44 | 42 | 41 | 39 | 35 | 31 | 28 | 0 |
| .10 | 0 | 0 | 0 | 0 | 1 | 17 | 106 | 235 | 263 | 234 | 202 | 175 | 152 | 120 | 104 | 93 | 85 | 79 | 73 | 66 | 61 | 58 | 54 | 49 | 44 | 42 | 41 | 39 | 35 | 31 | 28 | 0 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 12 | 75 | 182 | 236 | 234 | 213 | 188 | 144 | 116 | 101 | 91 | 84 | 78 | 70 | 63 | 59 | 55 | 50 | 45 | 43 | 41 | 40 | 36 | 31 | 28 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 8 | 52 | 138 | 203 | 224 | 217 | 197 | 154 | 123 | 105 | 94 | 86 | 79 | 71 | 64 | 59 | 55 | 51 | 46 | 43 | 42 | 40 | 36 | 32 | 28 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 37 | 105 | 170 | 206 | 213 | 203 | 164 | 131 | 110 | 97 | 88 | 81 | 72 | 65 | 60 | 56 | 51 | 46 | 43 | 42 | 40 | 36 | 32 | 28 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 26 | 78 | 140 | 184 | 203 | 191 | 155 | 126 | 107 | 95 | 86 | 76 | 69 | 62 | 57 | 53 | 48 | 44 | 42 | 41 | 37 | 33 | 28 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 10 | 34 | 73 | 117 | 153 | 184 | 173 | 146 | 122 | 105 | 94 | 82 | 73 | 64 | 58 | 54 | 49 | 45 | 43 | 41 | 37 | 33 | 28 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 17 | 42 | 114 | 168 | 178 | 159 | 134 | 114 | 94 | 82 | 70 | 61 | 57 | 52 | 47 | 44 | 42 | 39 | 35 | 28 | 0 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 10 | 44 | 98 | 144 | 163 | 157 | 130 | 105 | 84 | 69 | 61 | 56 | 52 | 47 | 44 | 40 | 36 | 29 | 6 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 14 | 44 | 87 | 127 | 153 | 141 | 110 | 83 | 69 | 61 | 56 | 51 | 47 | 42 | 38 | 30 | 17 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 16 | 42 | 97 | 138 | 145 | 107 | 82 | 68 | 60 | 55 | 51 | 43 | 40 | 32 | 25 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 27 | 71 | 127 | 139 | 105 | 81 | 68 | 60 | 55 | 46 | 41 | 33 | 27 |
| | RAINFALL TYPE = II | | | | | | | | | | | | | * * * TC = 0.3 HR * * * | | | | | | | | | | | | | | | | | SHEET 3 OF 10 |

# Exhibit 5-II, continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
| | IA/P = 0.10 | | | | | | | | | * * * TC =0.4 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.10 | | | | |
| 0.0 | 18 | 25 | 36 | 77 | 141 | 271 | 468 | 592 | 574 | 431 | 298 | 216 | 163 | 104 | 77 | 63 | 55 | 49 | 44 | 38 | 34 | 31 | 28 | 25 | 22 | 21 | 20 | 18 | 16 | 14 | 12 | 0 |
| .10 | 18 | 24 | 34 | 67 | 116 | 219 | 385 | 523 | 557 | 473 | 357 | 263 | 196 | 119 | 84 | 67 | 57 | 51 | 46 | 39 | 35 | 32 | 29 | 25 | 22 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .20 | 15 | 20 | 28 | 44 | 59 | 97 | 179 | 316 | 454 | 523 | 489 | 401 | 309 | 178 | 112 | 81 | 65 | 56 | 49 | 42 | 37 | 33 | 30 | 26 | 23 | 21 | 20 | 19 | 17 | 14 | 12 | 0 |
| .30 | 15 | 20 | 27 | 41 | 53 | 82 | 147 | 260 | 389 | 478 | 486 | 429 | 349 | 210 | 129 | 89 | 69 | 58 | 51 | 43 | 38 | 33 | 30 | 27 | 24 | 21 | 20 | 19 | 17 | 14 | 12 | 0 |
| .40 | 13 | 17 | 23 | 33 | 38 | 48 | 71 | 121 | 214 | 331 | 429 | 467 | 442 | 308 | 189 | 120 | 85 | 66 | 56 | 47 | 41 | 35 | 31 | 28 | 24 | 22 | 20 | 19 | 17 | 15 | 12 | 0 |
| .50 | 12 | 16 | 22 | 31 | 36 | 44 | 62 | 102 | 176 | 279 | 379 | 438 | 440 | 339 | 218 | 137 | 94 | 71 | 59 | 49 | 42 | 35 | 31 | 28 | 25 | 22 | 21 | 19 | 17 | 15 | 12 | 0 |
| .75 | 10 | 13 | 17 | 24 | 26 | 30 | 35 | 45 | 65 | 106 | 170 | 251 | 326 | 393 | 341 | 245 | 164 | 112 | 81 | 59 | 48 | 39 | 33 | 30 | 26 | 23 | 21 | 20 | 18 | 15 | 12 | 0 |
| 1.0 | 8 | 10 | 13 | 17 | 19 | 21 | 24 | 27 | 31 | 37 | 50 | 75 | 118 | 251 | 360 | 376 | 292 | 205 | 138 | 83 | 60 | 45 | 36 | 32 | 28 | 25 | 22 | 21 | 18 | 16 | 12 | 1 |
| 1.5 | 6 | 7 | 9 | 12 | 13 | 14 | 15 | 17 | 19 | 21 | 23 | 26 | 31 | 56 | 121 | 224 | 311 | 333 | 293 | 192 | 115 | 66 | 45 | 36 | 31 | 28 | 25 | 22 | 19 | 17 | 13 | 4 |
| 2.0 | 4 | 5 | 6 | 8 | 8 | 9 | 10 | 10 | 11 | 12 | 14 | 15 | 16 | 20 | 27 | 43 | 85 | 159 | 243 | 306 | 264 | 154 | 74 | 47 | 37 | 32 | 28 | 25 | 21 | 18 | 14 | 9 |
| 2.5 | 2 | 3 | 4 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 11 | 13 | 16 | 20 | 27 | 46 | 85 | 184 | 285 | 262 | 147 | 74 | 47 | 37 | 32 | 28 | 22 | 19 | 15 | 11 |
| 3.0 | 1 | 2 | 2 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 10 | 12 | 14 | 17 | 23 | 47 | 109 | 227 | 268 | 160 | 83 | 50 | 38 | 32 | 25 | 21 | 16 | 11 |
| | IA/P = 0.30 | | | | | | | | | * * * TC =0.4 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.30 | | | | |
| 0.0 | 0 | 0 | 0 | 4 | 26 | 113 | 296 | 480 | 495 | 413 | 306 | 234 | 186 | 127 | 100 | 84 | 74 | 67 | 61 | 54 | 49 | 45 | 41 | 37 | 33 | 31 | 29 | 28 | 25 | 21 | 19 | 0 |
| .10 | 0 | 0 | 0 | 0 | 2 | 18 | 81 | 224 | 395 | 462 | 430 | 347 | 272 | 172 | 121 | 96 | 82 | 73 | 66 | 57 | 51 | 46 | 42 | 38 | 34 | 31 | 30 | 28 | 25 | 22 | 19 | 0 |
| .20 | 0 | 0 | 0 | 0 | 2 | 13 | 59 | 169 | 320 | 414 | 424 | 373 | 305 | 196 | 134 | 103 | 85 | 75 | 67 | 59 | 52 | 47 | 43 | 39 | 34 | 32 | 30 | 29 | 25 | 22 | 19 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 42 | 127 | 255 | 361 | 403 | 383 | 274 | 181 | 127 | 99 | 83 | 73 | 63 | 55 | 48 | 44 | 40 | 36 | 32 | 30 | 29 | 26 | 23 | 19 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 30 | 94 | 202 | 308 | 372 | 379 | 298 | 203 | 141 | 106 | 87 | 76 | 65 | 56 | 49 | 44 | 40 | 36 | 32 | 31 | 29 | 26 | 23 | 19 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 21 | 70 | 158 | 258 | 334 | 364 | 270 | 187 | 133 | 102 | 85 | 70 | 60 | 51 | 46 | 41 | 37 | 33 | 31 | 30 | 26 | 23 | 19 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 30 | 76 | 145 | 219 | 321 | 305 | 241 | 177 | 130 | 102 | 78 | 65 | 55 | 47 | 43 | 38 | 34 | 32 | 30 | 27 | 24 | 19 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 15 | 42 | 150 | 267 | 308 | 272 | 209 | 154 | 103 | 79 | 62 | 51 | 45 | 41 | 37 | 33 | 31 | 28 | 25 | 19 | 1 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 10 | 51 | 136 | 226 | 274 | 263 | 195 | 131 | 85 | 62 | 51 | 45 | 41 | 36 | 33 | 29 | 26 | 20 | 6 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 31 | 86 | 162 | 252 | 239 | 162 | 93 | 64 | 52 | 45 | 41 | 37 | 31 | 28 | 21 | 15 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 33 | 112 | 202 | 235 | 155 | 92 | 64 | 52 | 45 | 41 | 33 | 29 | 23 | 18 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 21 | 76 | 182 | 221 | 148 | 90 | 63 | 51 | 45 | 36 | 31 | 24 | 18 |
| | IA/P = 0.50 | | | | | | | | | * * * TC =0.4 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.50 | | | | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 7 | 59 | 168 | 245 | 257 | 213 | 186 | 163 | 128 | 109 | 96 | 88 | 81 | 75 | 67 | 62 | 58 | 54 | 50 | 45 | 43 | 41 | 39 | 35 | 31 | 28 | 0 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 41 | 125 | 205 | 240 | 222 | 198 | 154 | 123 | 106 | 94 | 86 | 79 | 71 | 64 | 60 | 56 | 51 | 46 | 43 | 42 | 40 | 36 | 32 | 28 | 0 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 28 | 93 | 168 | 216 | 220 | 205 | 164 | 131 | 110 | 97 | 88 | 81 | 72 | 65 | 60 | 56 | 51 | 46 | 43 | 42 | 40 | 36 | 32 | 28 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 20 | 69 | 135 | 189 | 209 | 192 | 155 | 126 | 107 | 95 | 86 | 77 | 69 | 62 | 57 | 53 | 48 | 44 | 42 | 41 | 37 | 33 | 28 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 14 | 50 | 106 | 161 | 193 | 202 | 163 | 133 | 112 | 98 | 89 | 78 | 70 | 62 | 58 | 53 | 48 | 44 | 42 | 41 | 37 | 33 | 28 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 37 | 83 | 135 | 174 | 194 | 171 | 140 | 117 | 102 | 91 | 80 | 71 | 63 | 58 | 54 | 49 | 45 | 43 | 41 | 37 | 33 | 28 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 15 | 40 | 76 | 147 | 177 | 169 | 146 | 124 | 107 | 90 | 79 | 68 | 60 | 56 | 51 | 47 | 43 | 42 | 38 | 34 | 28 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | 21 | 78 | 141 | 173 | 167 | 146 | 125 | 101 | 86 | 73 | 63 | 58 | 53 | 48 | 45 | 42 | 39 | 35 | 28 | 1 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 26 | 71 | 121 | 153 | 159 | 139 | 113 | 89 | 72 | 63 | 57 | 53 | 48 | 44 | 40 | 37 | 29 | 7 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 16 | 45 | 86 | 138 | 150 | 125 | 93 | 74 | 64 | 58 | 53 | 48 | 42 | 39 | 31 | 20 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 17 | 59 | 112 | 143 | 121 | 91 | 73 | 63 | 57 | 53 | 45 | 40 | 32 | 26 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 11 | 40 | 101 | 138 | 117 | 90 | 73 | 63 | 57 | 48 | 42 | 34 | 27 |
| | RAINFALL TYPE = II | | | | | | | | | * * * TC = 0.4 HR * * * | | | | | | | | | | | | | | | | | | SHEET 4 OF 10 | | | | |

## Exhibit 5-II, continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
| IA/P = 0.10 | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 17 | 23 | 32 | 57 | 94 | 170 | 308 | 467 | 529 | 507 | 402 | 297 | 226 | 140 | 96 | 74 | 61 | 53 | 47 | 41 | 36 | 32 | 29 | 26 | 23 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .10 | 16 | 22 | 30 | 51 | 80 | 140 | 252 | 395 | 484 | 499 | 434 | 343 | 265 | 162 | 108 | 80 | 65 | 55 | 49 | 42 | 36 | 33 | 29 | 26 | 23 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .20 | 14 | 19 | 25 | 38 | 47 | 69 | 116 | 207 | 332 | 434 | 477 | 449 | 378 | 238 | 149 | 101 | 77 | 62 | 53 | 45 | 39 | 34 | 30 | 27 | 24 | 22 | 20 | 19 | 17 | 14 | 12 | 0 |
| .30 | 13 | 18 | 24 | 35 | 43 | 60 | 97 | 170 | 278 | 382 | 446 | 448 | 401 | 270 | 171 | 114 | 83 | 66 | 56 | 46 | 40 | 34 | 31 | 27 | 24 | 22 | 20 | 19 | 17 | 15 | 12 | 0 |
| .40 | 12 | 15 | 21 | 29 | 33 | 40 | 53 | 83 | 141 | 233 | 332 | 408 | 434 | 361 | 243 | 157 | 107 | 79 | 64 | 51 | 43 | 36 | 32 | 28 | 25 | 22 | 21 | 20 | 17 | 15 | 12 | 0 |
| .50 | 11 | 15 | 20 | 28 | 31 | 37 | 48 | 71 | 118 | 194 | 286 | 367 | 412 | 378 | 271 | 178 | 119 | 86 | 68 | 53 | 44 | 37 | 32 | 29 | 25 | 23 | 21 | 20 | 17 | 15 | 12 | 0 |
| .75 | 9 | 11 | 14 | 19 | 21 | 24 | 27 | 31 | 37 | 49 | 74 | 118 | 182 | 319 | 374 | 328 | 244 | 169 | 117 | 76 | 56 | 43 | 35 | 31 | 28 | 25 | 22 | 21 | 18 | 16 | 12 | 1 |
| 1.0 | 7 | 9 | 12 | 16 | 17 | 19 | 21 | 24 | 27 | 32 | 40 | 55 | 83 | 188 | 309 | 359 | 322 | 245 | 172 | 102 | 68 | 49 | 38 | 32 | 29 | 26 | 23 | 21 | 19 | 16 | 12 | 1 |
| 1.5 | 5 | 7 | 8 | 11 | 12 | 13 | 14 | 15 | 17 | 19 | 21 | 23 | 27 | 43 | 89 | 175 | 269 | 322 | 309 | 225 | 140 | 77 | 49 | 38 | 32 | 29 | 25 | 23 | 20 | 17 | 13 | 5 |
| 2.0 | 3 | 4 | 6 | 7 | 8 | 8 | 9 | 10 | 10 | 11 | 12 | 14 | 15 | 18 | 23 | 35 | 65 | 123 | 202 | 297 | 280 | 181 | 88 | 52 | 39 | 33 | 29 | 26 | 21 | 19 | 14 | 10 |
| 2.5 | 2 | 3 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 12 | 15 | 18 | 24 | 36 | 66 | 150 | 244 | 278 | 171 | 87 | 52 | 39 | 33 | 29 | 23 | 20 | 15 | 11 |
| 3.0 | 1 | 1 | 2 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 9 | 11 | 13 | 16 | 20 | 37 | 86 | 198 | 263 | 182 | 96 | 56 | 40 | 33 | 26 | 21 | 16 | 11 |
| IA/P = 0.30 | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 1 | 9 | 53 | 157 | 314 | 433 | 439 | 379 | 299 | 237 | 159 | 118 | 95 | 81 | 71 | 65 | 56 | 50 | 46 | 42 | 38 | 34 | 31 | 30 | 28 | 25 | 22 | 19 | 0 |
| .10 | 0 | 0 | 0 | 0 | 1 | 6 | 37 | 117 | 248 | 372 | 416 | 391 | 330 | 218 | 150 | 113 | 92 | 79 | 70 | 60 | 53 | 47 | 43 | 39 | 35 | 32 | 30 | 29 | 26 | 22 | 19 | 0 |
| .20 | 0 | 0 | 0 | 0 | 1 | 4 | 26 | 87 | 194 | 313 | 382 | 388 | 349 | 244 | 167 | 122 | 97 | 82 | 72 | 62 | 54 | 48 | 43 | 39 | 35 | 32 | 30 | 29 | 26 | 22 | 19 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 19 | 64 | 151 | 259 | 341 | 372 | 316 | 223 | 156 | 117 | 94 | 80 | 67 | 58 | 50 | 45 | 41 | 36 | 33 | 31 | 29 | 26 | 23 | 19 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 13 | 47 | 116 | 211 | 298 | 354 | 328 | 245 | 172 | 127 | 100 | 83 | 69 | 59 | 51 | 45 | 41 | 37 | 33 | 31 | 29 | 26 | 23 | 19 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 34 | 89 | 170 | 255 | 341 | 303 | 225 | 161 | 120 | 96 | 76 | 64 | 54 | 47 | 42 | 38 | 34 | 31 | 30 | 27 | 24 | 19 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 14 | 41 | 89 | 152 | 270 | 305 | 268 | 207 | 155 | 118 | 87 | 70 | 57 | 48 | 44 | 39 | 35 | 32 | 30 | 27 | 24 | 19 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 22 | 98 | 212 | 295 | 285 | 237 | 181 | 120 | 88 | 67 | 53 | 46 | 42 | 38 | 34 | 31 | 28 | 25 | 19 | 2 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 30 | 95 | 183 | 249 | 265 | 217 | 152 | 96 | 66 | 53 | 46 | 41 | 37 | 34 | 30 | 26 | 20 | 8 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 18 | 59 | 125 | 221 | 245 | 182 | 105 | 69 | 54 | 47 | 42 | 38 | 32 | 28 | 22 | 16 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 21 | 84 | 174 | 230 | 172 | 103 | 69 | 54 | 46 | 42 | 34 | 30 | 23 | 18 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 13 | 56 | 157 | 217 | 163 | 101 | 68 | 53 | 46 | 37 | 31 | 25 | 18 |
| IA/P = 0.50 | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 2 | 26 | 89 | 170 | 217 | 229 | 200 | 179 | 144 | 119 | 104 | 93 | 85 | 78 | 70 | 64 | 59 | 55 | 51 | 46 | 43 | 41 | 40 | 36 | 32 | 28 | 0 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 18 | 65 | 135 | 190 | 216 | 205 | 170 | 137 | 115 | 101 | 91 | 83 | 74 | 67 | 61 | 56 | 52 | 47 | 44 | 42 | 40 | 36 | 32 | 28 | 0 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 12 | 47 | 106 | 162 | 198 | 203 | 178 | 145 | 121 | 105 | 94 | 85 | 76 | 68 | 61 | 57 | 52 | 48 | 44 | 42 | 40 | 37 | 32 | 28 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 34 | 82 | 135 | 177 | 194 | 168 | 139 | 117 | 102 | 92 | 80 | 71 | 63 | 58 | 54 | 49 | 45 | 43 | 41 | 37 | 33 | 28 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 6 | 25 | 63 | 111 | 155 | 189 | 174 | 146 | 122 | 106 | 94 | 82 | 73 | 64 | 58 | 54 | 50 | 45 | 43 | 41 | 37 | 33 | 28 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 18 | 48 | 90 | 133 | 184 | 177 | 152 | 128 | 110 | 97 | 84 | 74 | 65 | 59 | 55 | 50 | 45 | 43 | 41 | 38 | 33 | 28 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | 22 | 47 | 80 | 142 | 169 | 164 | 144 | 124 | 108 | 91 | 79 | 68 | 61 | 56 | 51 | 47 | 44 | 42 | 38 | 34 | 28 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 11 | 51 | 112 | 155 | 166 | 154 | 134 | 109 | 91 | 76 | 65 | 59 | 54 | 49 | 45 | 43 | 39 | 35 | 28 | 2 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 16 | 50 | 97 | 136 | 154 | 145 | 121 | 95 | 75 | 64 | 58 | 54 | 49 | 45 | 41 | 37 | 29 | 10 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 18 | 47 | 86 | 134 | 146 | 125 | 94 | 75 | 64 | 58 | 53 | 49 | 42 | 39 | 31 | 21 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 11 | 44 | 95 | 140 | 127 | 97 | 77 | 65 | 58 | 54 | 45 | 41 | 33 | 26 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0. | 0 | 1 | 7 | 29 | 86 | 135 | 122 | 95 | 76 | 65 | 58 | 49 | 43 | 35 | 27 |
| RAINFALL TYPE = II | | | | | | | | | | * * * TC = 0.5 HR * * * | | | | | | | | | | | | | | | | | | | | SHEET 5 OF 10 |

## Exhibit 5-II, continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
| IA/P = 0.10 | * * * TC =0.75 HR * * * | | | | | | | | | | | | | | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 13 | 18 | 24 | 36 | 46 | 68 | 115 | 194 | 294 | 380 | 424 | 410 | 369 | 252 | 172 | 123 | 93 | 74 | 61 | 49 | 41 | 35 | 31 | 27 | 24 | 22 | 20 | 19 | 17 | 15 | 12 | 0 |
| .10 | 13 | 17 | 23 | 34 | 42 | 59 | 97 | 162 | 250 | 337 | 395 | 405 | 381 | 279 | 191 | 135 | 100 | 79 | 65 | 51 | 42 | 36 | 31 | 28 | 25 | 22 | 21 | 19 | 17 | 15 | 12 | 0 |
| .20 | 11 | 15 | 20 | 28 | 32 | 39 | 52 | 82 | 135 | 211 | 295 | 362 | 391 | 351 | 255 | 178 | 127 | 95 | 75 | 57 | 46 | 38 | 32 | 29 | 26 | 23 | 21 | 20 | 17 | 15 | 12 | 0 |
| .30 | 11 | 14 | 19 | 26 | 30 | 36 | 47 | 70 | 113 | 179 | 256 | 326 | 379 | 360 | 277 | 196 | 140 | 103 | 80 | 60 | 48 | 38 | 33 | 29 | 26 | 23 | 21 | 20 | 18 | 15 | 12 | 0 |
| .40 | 10 | 12 | 16 | 22 | 25 | 28 | 33 | 42 | 61 | 96 | 151 | 221 | 291 | 367 | 336 | 255 | 182 | 131 | 98 | 69 | 54 | 42 | 34 | 30 | 27 | 24 | 22 | 20 | 18 | 16 | 12 | 0 |
| .50 | 9 | 12 | 16 | 21 | 24 | 27 | 31 | 39 | 53 | 82 | 128 | 190 | 258 | 358 | 343 | 274 | 200 | 144 | 106 | 74 | 56 | 43 | 35 | 30 | 27 | 24 | 22 | 20 | 18 | 16 | 12 | 0 |
| .75 | 8 | 10 | 13 | 17 | 18 | 21 | 23 | 26 | 31 | 39 | 55 | 82 | 122 | 230 | 314 | 329 | 281 | 217 | 161 | 104 | 72 | 51 | 38 | 33 | 29 | 26 | 23 | 21 | 19 | 16 | 12 | 1 |
| 1.0 | 6 | 8 | 10 | 13 | 14 | 15 | 17 | 19 | 21 | 23 | 27 | 32 | 42 | 89 | 177 | 272 | 319 | 303 | 249 | 163 | 105 | 66 | 45 | 36 | 31 | 27 | 24 | 22 | 19 | 17 | 13 | 3 |
| 1.5 | 4 | 6 | 7 | 9 | 10 | 10 | 11 | 12 | 14 | 15 | 16 | 18 | 20 | 27 | 46 | 90 | 163 | 241 | 295 | 275 | 204 | 119 | 66 | 45 | 35 | 31 | 27 | 24 | 20 | 18 | 13 | 7 |
| 2.0 | 3 | 4 | 5 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 11 | 12 | 13 | 16 | 20 | 28 | 48 | 89 | 151 | 245 | 274 | 213 | 115 | 65 | 44 | 35 | 30 | 27 | 22 | 19 | 14 | 10 |
| 2.5 | 1 | 2 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 8 | 10 | 12 | 14 | 17 | 24 | 37 | 86 | 170 | 260 | 219 | 127 | 71 | 47 | 36 | 31 | 24 | 20 | 16 | 11 |
| 3.0 | 1 | 1 | 2 | 3 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 5 | 6 | 7 | 8 | 10 | 11 | 14 | 17 | 30 | 64 | 157 | 247 | 205 | 122 | 70 | 46 | 36 | 27 | 22 | 17 | 12 |
| IA/P = 0.30 | * * * TC =0.75 HR * * * | | | | | | | | | | | | | | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 0 | 1 | 6 | 30 | 86 | 174 | 266 | 326 | 348 | 328 | 246 | 181 | 138 | 110 | 92 | 79 | 66 | 57 | 49 | 44 | 40 | 36 | 32 | 31 | 29 | 26 | 23 | 19 | 0 |
| .10 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 22 | 65 | 137 | 223 | 292 | 329 | 303 | 228 | 170 | 131 | 106 | 89 | 73 | 61 | 52 | 46 | 41 | 37 | 33 | 31 | 29 | 26 | 23 | 19 | 0 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 15 | 48 | 108 | 185 | 256 | 305 | 321 | 245 | 184 | 141 | 112 | 93 | 75 | 63 | 53 | 46 | 42 | 37 | 34 | 31 | 30 | 27 | 23 | 19 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 11 | 36 | 84 | 151 | 221 | 277 | 308 | 260 | 199 | 152 | 120 | 98 | 78 | 65 | 54 | 47 | 42 | 38 | 34 | 31 | 30 | 27 | 23 | 19 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 27 | 65 | 122 | 188 | 286 | 301 | 243 | 187 | 144 | 114 | 87 | 71 | 57 | 48 | 43 | 39 | 35 | 32 | 30 | 27 | 24 | 19 | 1 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 20 | 50 | 98 | 158 | 263 | 292 | 254 | 200 | 155 | 122 | 91 | 74 | 59 | 49 | 44 | 40 | 35 | 32 | 30 | 27 | 24 | 19 | 1 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 23 | 51 | 140 | 231 | 269 | 253 | 211 | 167 | 119 | 90 | 68 | 53 | 46 | 42 | 37 | 34 | 31 | 28 | 25 | 19 | 2 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 29 | 96 | 186 | 249 | 261 | 231 | 169 | 120 | 84 | 61 | 50 | 44 | 40 | 36 | 33 | 29 | 26 | 20 | 5 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 34 | 91 | 163 | 220 | 241 | 197 | 131 | 83 | 61 | 50 | 44 | 40 | 35 | 31 | 27 | 21 | 12 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 11 | 36 | 85 | 174 | 226 | 200 | 127 | 82 | 60 | 49 | 44 | 39 | 32 | 29 | 22 | 17 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 37 | 105 | 196 | 214 | 135 | 87 | 62 | 51 | 44 | 36 | 31 | 24 | 18 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 24 | 96 | 205 | 189 | 130 | 85 | 62 | 50 | 39 | 32 | 26 | 18 |
| IA/P = 0.50 | * * * TC =0.75 HR * * * | | | | | | | | | | | | | | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 16 | 45 | 92 | 137 | 166 | 185 | 170 | 146 | 125 | 110 | 98 | 89 | 79 | 70 | 63 | 58 | 53 | 48 | 44 | 42 | 41 | 37 | 33 | 28 | 0 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 11 | 34 | 73 | 115 | 149 | 180 | 163 | 141 | 122 | 107 | 96 | 84 | 74 | 65 | 59 | 54 | 50 | 45 | 43 | 41 | 38 | 33 | 28 | 0 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 25 | 57 | 96 | 131 | 173 | 166 | 146 | 126 | 111 | 99 | 86 | 76 | 66 | 59 | 55 | 50 | 46 | 43 | 41 | 38 | 34 | 28 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 18 | 44 | 79 | 143 | 170 | 160 | 141 | 122 | 108 | 92 | 81 | 69 | 61 | 56 | 52 | 47 | 44 | 42 | 38 | 34 | 28 | 1 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 14 | 34 | 64 | 127 | 166 | 162 | 145 | 127 | 111 | 95 | 82 | 70 | 62 | 57 | 52 | 47 | 44 | 42 | 38 | 34 | 28 | 1 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 10 | 26 | 82 | 138 | 162 | 157 | 140 | 123 | 103 | 88 | 75 | 64 | 58 | 53 | 49 | 45 | 43 | 39 | 35 | 28 | 2 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 12 | 47 | 98 | 139 | 154 | 148 | 135 | 113 | 96 | 80 | 67 | 60 | 55 | 50 | 46 | 43 | 39 | 36 | 29 | 3 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 6 | 30 | 73 | 119 | 146 | 151 | 134 | 113 | 91 | 74 | 63 | 58 | 53 | 48 | 45 | 41 | 37 | 29 | 7 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 30 | 66 | 105 | 143 | 143 | 117 | 90 | 73 | 63 | 57 | 52 | 48 | 42 | 39 | 30 | 18 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 11 | 30 | 77 | 121 | 137 | 114 | 88 | 72 | 63 | 57 | 52 | 44 | 40 | 32 | 25 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 19 | 55 | 111 | 132 | 111 | 87 | 71 | 62 | 56 | 47 | 42 | 34 | 27 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 12 | 51 | 112 | 128 | 108 | 86 | 71 | 62 | 51 | 44 | 36 | 27 |
| RAINFALL TYPE = II | * * * TC = 0.75 HR * * * | | | | | | | | | | | | | | | | | | | | | | | | | | | | | SHEET 6 OF 10 | |

## Exhibit 5-II, continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
| IA/P = 0.10 | | | | | | | | | | | | * * * TC =1.0 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 11 | 15 | 20 | 29 | 35 | 47 | 72 | 112 | 168 | 231 | 289 | 329 | 357 | 313 | 239 | 175 | 133 | 103 | 83 | 63 | 50 | 40 | 33 | 29 | 26 | 23 | 21 | 20 | 17 | 15 | 12 | 0 |
| .10 | 10 | 13 | 17 | 24 | 27 | 33 | 42 | 62 | 95 | 144 | 202 | 260 | 306 | 340 | 293 | 222 | 165 | 126 | 98 | 72 | 56 | 43 | 35 | 30 | 27 | 24 | 22 | 20 | 18 | 15 | 12 | 0 |
| .20 | 10 | 13 | 17 | 23 | 26 | 30 | 38 | 54 | 82 | 123 | 176 | 232 | 281 | 332 | 303 | 238 | 179 | 136 | 105 | 76 | 59 | 45 | 35 | 30 | 27 | 24 | 22 | 20 | 18 | 16 | 12 | 1 |
| .30 | 9 | 12 | 16 | 22 | 24 | 28 | 35 | 48 | 70 | 105 | 152 | 205 | 256 | 323 | 310 | 254 | 193 | 146 | 113 | 81 | 61 | 46 | 36 | 31 | 27 | 24 | 22 | 20 | 18 | 16 | 12 | 1 |
| .40 | 8 | 11 | 14 | 19 | 21 | 23 | 27 | 32 | 42 | 61 | 91 | 132 | 181 | 276 | 318 | 294 | 237 | 181 | 138 | 95 | 70 | 51 | 39 | 32 | 28 | 25 | 23 | 21 | 18 | 16 | 12 | 1 |
| .50 | 8 | 10 | 13 | 18 | 20 | 22 | 25 | 30 | 38 | 53 | 78 | 114 | 159 | 253 | 311 | 300 | 251 | 195 | 149 | 102 | 74 | 53 | 40 | 33 | 29 | 25 | 23 | 21 | 18 | 16 | 12 | 1 |
| .75 | 7 | 8 | 11 | 14 | 16 | 17 | 19 | 21 | 25 | 30 | 38 | 53 | 76 | 146 | 228 | 284 | 293 | 256 | 208 | 143 | 99 | 66 | 46 | 36 | 31 | 27 | 24 | 22 | 19 | 17 | 13 | 2 |
| 1.0 | 5 | 7 | 8 | 11 | 12 | 13 | 14 | 16 | 17 | 19 | 22 | 25 | 31 | 57 | 111 | 188 | 256 | 286 | 272 | 208 | 144 | 90 | 56 | 41 | 33 | 29 | 26 | 23 | 20 | 17 | 13 | 4 |
| 1.5 | 4 | 5 | 6 | 8 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 22 | 33 | 59 | 107 | 171 | 231 | 268 | 235 | 157 | 88 | 56 | 41 | 33 | 29 | 25 | 21 | 18 | 14 | 8 |
| 2.0 | 2 | 3 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 12 | 15 | 19 | 27 | 44 | 78 | 157 | 231 | 252 | 167 | 96 | 59 | 42 | 34 | 29 | 23 | 20 | 15 | 11 |
| 2.5 | 1 | 2 | 2 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 10 | 12 | 15 | 19 | 27 | 58 | 120 | 214 | 241 | 159 | 94 | 59 | 42 | 34 | 26 | 21 | 16 | 11 |
| 3.0 | 0 | 1 | 1 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 10 | 12 | 14 | 22 | 44 | 113 | 214 | 231 | 152 | 91 | 58 | 42 | 29 | 23 | 17 | 12 |
| IA/P = 0.30 | | | | | | | | | | | | * * * TC =1.0 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 0 | 1 | 4 | 16 | 42 | 83 | 137 | 195 | 243 | 271 | 292 | 227 | 178 | 143 | 117 | 98 | 79 | 66 | 55 | 47 | 42 | 38 | 34 | 31 | 30 | 27 | 23 | 19 | 0 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 12 | 32 | 66 | 113 | 168 | 218 | 279 | 260 | 213 | 169 | 136 | 113 | 88 | 72 | 59 | 49 | 43 | 39 | 35 | 32 | 30 | 27 | 24 | 19 | 1 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 24 | 52 | 93 | 143 | 193 | 271 | 271 | 225 | 180 | 145 | 119 | 92 | 75 | 60 | 50 | 44 | 39 | 35 | 32 | 30 | 27 | 24 | 19 | 1 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 18 | 41 | 75 | 120 | 169 | 246 | 264 | 234 | 191 | 153 | 125 | 96 | 78 | 62 | 51 | 44 | 40 | 36 | 33 | 31 | 27 | 24 | 19 | 1 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 14 | 32 | 61 | 100 | 190 | 251 | 259 | 222 | 181 | 146 | 109 | 86 | 67 | 53 | 46 | 41 | 37 | 33 | 31 | 28 | 25 | 19 | 2 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 10 | 24 | 49 | 83 | 168 | 237 | 254 | 230 | 191 | 155 | 115 | 90 | 69 | 54 | 47 | 42 | 37 | 34 | 31 | 28 | 25 | 19 | 2 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 12 | 25 | 76 | 150 | 213 | 239 | 228 | 198 | 149 | 112 | 82 | 61 | 50 | 44 | 39 | 35 | 32 | 29 | 26 | 20 | 4 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 15 | 51 | 113 | 182 | 226 | 234 | 197 | 150 | 104 | 72 | 56 | 47 | 42 | 38 | 34 | 30 | 27 | 20 | 7 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 18 | 51 | 104 | 162 | 220 | 210 | 158 | 102 | 71 | 56 | 47 | 42 | 37 | 31 | 28 | 22 | 13 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 20 | 49 | 121 | 187 | 209 | 152 | 100 | 70 | 55 | 47 | 41 | 34 | 29 | 23 | 17 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 32 | 87 | 171 | 199 | 146 | 98 | 69 | 54 | 46 | 37 | 31 | 24 | 18 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 13 | 62 | 158 | 192 | 151 | 103 | 73 | 56 | 41 | 34 | 26 | 18 |
| IA/P = 0.50 | | | | | | | | | | | | * * * TC =1.0 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | 21 | 42 | 71 | 101 | 126 | 160 | 154 | 138 | 123 | 110 | 100 | 87 | 77 | 67 | 60 | 55 | 50 | 46 | 43 | 41 | 38 | 34 | 28 | 1 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 15 | 33 | 58 | 87 | 134 | 156 | 149 | 134 | 120 | 108 | 93 | 82 | 71 | 62 | 57 | 52 | 47 | 44 | 42 | 38 | 34 | 28 | 1 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 12 | 26 | 48 | 74 | 123 | 153 | 153 | 137 | 123 | 111 | 95 | 84 | 72 | 63 | 57 | 52 | 47 | 44 | 42 | 38 | 34 | 28 | 1 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 9 | 20 | 38 | 62 | 111 | 143 | 150 | 140 | 127 | 114 | 98 | 86 | 73 | 63 | 58 | 53 | 48 | 45 | 42 | 39 | 35 | 28 | 1 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 16 | 31 | 75 | 120 | 145 | 148 | 137 | 123 | 106 | 91 | 77 | 66 | 59 | 54 | 49 | 45 | 43 | 39 | 35 | 29 | 2 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 12 | 25 | 64 | 109 | 139 | 146 | 139 | 127 | 108 | 94 | 79 | 67 | 60 | 55 | 50 | 46 | 43 | 39 | 36 | 29 | 3 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 12 | 39 | 78 | 115 | 136 | 140 | 134 | 117 | 101 | 84 | 70 | 62 | 56 | 51 | 47 | 44 | 40 | 36 | 29 | 4 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | 26 | 59 | 96 | 125 | 139 | 133 | 117 | 97 | 78 | 66 | 59 | 54 | 49 | 46 | 41 | 37 | 29 | 8 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 26 | 54 | 86 | 123 | 133 | 119 | 95 | 77 | 66 | 59 | 54 | 49 | 43 | 39 | 31 | 17 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 10 | 25 | 64 | 104 | 129 | 116 | 93 | 76 | 65 | 58 | 53 | 45 | 41 | 33 | 24 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 10 | 34 | 84 | 125 | 117 | 96 | 78 | 66 | 59 | 49 | 43 | 35 | 27 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 32 | 89 | 122 | 114 | 94 | 77 | 66 | 53 | 45 | 37 | 27 |
| RAINFALL TYPE = II | | | | | | | | | | | | * * * TC = 1.0 HR * * * | | | | | | | | | | | | | | | | | | SHEET 7 OF 10 | |

## Exhibit 5-II, continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
| | IA/P = 0.10 | | | | | | | | | | | | | * * * TC =1.25 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 10 | 13 | 18 | 25 | 29 | 38 | 54 | 81 | 118 | 163 | 213 | 256 | 284 | 311 | 266 | 212 | 163 | 129 | 104 | 78 | 61 | 47 | 37 | 31 | 27 | 24 | 22 | 20 | 18 | 16 | 12 | 1 |
| .10 | 10 | 13 | 17 | 23 | 27 | 34 | 47 | 69 | 102 | 143 | 189 | 234 | 267 | 297 | 274 | 226 | 175 | 138 | 111 | 82 | 64 | 48 | 38 | 31 | 27 | 24 | 22 | 20 | 18 | 16 | 12 | 1 |
| .20 | 9 | 11 | 15 | 20 | 22 | 26 | 31 | 42 | 60 | 88 | 124 | 168 | 212 | 280 | 292 | 261 | 212 | 166 | 131 | 95 | 72 | 53 | 40 | 33 | 28 | 25 | 23 | 21 | 18 | 16 | 12 | 1 |
| .30 | 8 | 11 | 14 | 19 | 21 | 24 | 29 | 38 | 53 | 76 | 108 | 148 | 190 | 263 | 288 | 268 | 224 | 177 | 140 | 101 | 76 | 55 | 41 | 34 | 29 | 25 | 23 | 21 | 18 | 16 | 12 | 2 |
| .40 | 8 | 10 | 13 | 18 | 20 | 23 | 27 | 34 | 46 | 66 | 94 | 130 | 170 | 245 | 282 | 273 | 235 | 188 | 149 | 107 | 80 | 58 | 42 | 34 | 29 | 26 | 23 | 21 | 19 | 16 | 12 | 2 |
| .50 | 7 | 9 | 12 | 16 | 17 | 19 | 22 | 25 | 31 | 41 | 58 | 82 | 114 | 190 | 256 | 279 | 262 | 222 | 178 | 127 | 93 | 65 | 46 | 36 | 31 | 27 | 24 | 22 | 19 | 17 | 13 | 2 |
| .75 | 6 | 8 | 10 | 14 | 15 | 17 | 19 | 21 | 25 | 31 | 41 | 56 | 78 | 139 | 207 | 254 | 265 | 245 | 208 | 152 | 110 | 75 | 51 | 39 | 32 | 28 | 25 | 22 | 19 | 17 | 13 | 3 |
| 1.0 | 5 | 6 | 8 | 10 | 11 | 13 | 14 | 15 | 17 | 19 | 22 | 26 | 33 | 60 | 109 | 173 | 230 | 261 | 255 | 208 | 153 | 100 | 64 | 46 | 36 | 30 | 26 | 24 | 20 | 18 | 13 | 5 |
| 1.5 | 3 | 4 | 5 | 7 | 7 | 8 | 9 | 9 | 10 | 11 | 12 | 13 | 15 | 19 | 27 | 45 | 79 | 130 | 186 | 247 | 239 | 180 | 108 | 68 | 48 | 37 | 31 | 27 | 22 | 19 | 14 | 10 |
| 2.0 | 2 | 3 | 4 | 5 | 6 | 6 | 7 | 7 | 8 | 8 | 9 | 10 | 11 | 13 | 16 | 22 | 35 | 59 | 98 | 171 | 236 | 236 | 156 | 95 | 62 | 44 | 35 | 30 | 23 | 20 | 15 | 11 |
| 2.5 | 1 | 2 | 2 | 3 | 4 | 4 | 4 | 5 | 5 | 5 | 6 | 6 | 7 | 8 | 10 | 12 | 14 | 19 | 28 | 58 | 114 | 197 | 226 | 163 | 102 | 65 | 46 | 36 | 26 | 21 | 16 | 11 |
| 3.0 | 0 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 4 | 5 | 6 | 7 | 9 | 10 | 13 | 19 | 35 | 88 | 184 | 218 | 169 | 109 | 70 | 49 | 31 | 24 | 18 | 12 |
| | IA/P = 0.30 | | | | | | | | | | | | | * * * TC =1.25 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 2 | 9 | 25 | 50 | 86 | 130 | 174 | 208 | 253 | 235 | 201 | 164 | 136 | 115 | 92 | 76 | 61 | 51 | 44 | 39 | 35 | 32 | 30 | 27 | 24 | 19 | 1 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 19 | 40 | 71 | 110 | 153 | 217 | 247 | 227 | 191 | 157 | 131 | 103 | 84 | 66 | 53 | 46 | 41 | 36 | 33 | 31 | 28 | 24 | 19 | 2 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 14 | 31 | 58 | 93 | 133 | 202 | 239 | 231 | 199 | 165 | 138 | 108 | 87 | 68 | 55 | 47 | 41 | 37 | 33 | 31 | 28 | 25 | 19 | 2 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 10 | 24 | 46 | 77 | 152 | 210 | 236 | 222 | 190 | 158 | 122 | 97 | 74 | 58 | 49 | 43 | 38 | 34 | 32 | 28 | 25 | 20 | 3 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 19 | 37 | 64 | 134 | 196 | 232 | 225 | 198 | 166 | 127 | 101 | 77 | 59 | 50 | 43 | 38 | 35 | 32 | 28 | 25 | 20 | 3 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 14 | 30 | 82 | 151 | 206 | 228 | 217 | 189 | 146 | 113 | 85 | 64 | 52 | 45 | 40 | 36 | 33 | 29 | 26 | 20 | 5 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 7 | 15 | 49 | 105 | 164 | 205 | 218 | 205 | 166 | 129 | 95 | 69 | 55 | 47 | 41 | 37 | 33 | 29 | 26 | 20 | 6 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 32 | 77 | 134 | 185 | 214 | 203 | 166 | 120 | 83 | 63 | 52 | 45 | 39 | 35 | 30 | 27 | 21 | 10 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 11 | 33 | 72 | 121 | 184 | 203 | 171 | 117 | 82 | 62 | 51 | 44 | 39 | 32 | 29 | 22 | 15 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | 21 | 67 | 132 | 194 | 174 | 123 | 86 | 64 | 52 | 45 | 35 | 31 | 24 | 18 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 13 | 46 | 121 | 187 | 166 | 119 | 84 | 63 | 52 | 39 | 32 | 25 | 18 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 44 | 129 | 180 | 160 | 116 | 83 | 63 | 44 | 35 | 27 | 18 |
| | IA/P = 0.50 | | | | | | | | | | | | | * * * TC =1.25 HR * * * | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 13 | 26 | 44 | 68 | 91 | 125 | 142 | 142 | 128 | 117 | 107 | 94 | 83 | 72 | 63 | 57 | 52 | 47 | 44 | 42 | 38 | 34 | 28 | 2 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 10 | 20 | 36 | 57 | 100 | 129 | 140 | 136 | 125 | 114 | 100 | 88 | 76 | 65 | 59 | 54 | 49 | 45 | 43 | 39 | 35 | 29 | 3 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 16 | 30 | 48 | 90 | 122 | 139 | 139 | 127 | 117 | 102 | 90 | 77 | 66 | 60 | 54 | 49 | 45 | 43 | 39 | 35 | 29 | 3 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 12 | 24 | 59 | 98 | 126 | 137 | 134 | 125 | 109 | 96 | 82 | 69 | 61 | 56 | 51 | 46 | 44 | 40 | 36 | 29 | 4 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 19 | 51 | 89 | 119 | 134 | 136 | 127 | 112 | 98 | 83 | 70 | 62 | 56 | 51 | 47 | 44 | 40 | 36 | 29 | 5 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 15 | 43 | 79 | 112 | 131 | 135 | 129 | 114 | 100 | 85 | 71 | 63 | 57 | 52 | 47 | 44 | 40 | 36 | 29 | 6 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 15 | 39 | 71 | 102 | 123 | 130 | 125 | 112 | 94 | 78 | 67 | 60 | 54 | 49 | 46 | 41 | 37 | 29 | 9 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 17 | 40 | 71 | 101 | 121 | 129 | 121 | 103 | 84 | 71 | 62 | 56 | 51 | 47 | 42 | 38 | 30 | 13 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 10 | 26 | 51 | 92 | 119 | 125 | 105 | 86 | 72 | 63 | 57 | 52 | 44 | 40 | 32 | 23 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 11 | 35 | 72 | 112 | 122 | 103 | 85 | 71 | 63 | 56 | 47 | 42 | 34 | 26 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | 24 | 66 | 111 | 119 | 101 | 83 | 71 | 62 | 51 | 44 | 36 | 27 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 23 | 71 | 110 | 116 | 99 | 82 | 70 | 55 | 46 | 37 | 27 |
| | RAINFALL TYPE = II | | | | | | | | | | | | | * * * TC = 1.25 HR * * * | | | | | | | | | | | | | | | | | SHEET 8 OF 10 |

## Exhibit 5-II, continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | IA/P = 0.10 | | | | | | | | | | | * * * TC =1.5 HR * * * | | | | | | | | | | | | | | | | | | | IA/P = 0.10 |
| 0.0 | 9 | 11 | 15 | 21 | 25 | 31 | 41 | 58 | 82 | 112 | 147 | 184 | 216 | 255 | 275 | 236 | 198 | 159 | 129 | 98 | 76 | 57 | 43 | 35 | 30 | 25 | 23 | 21 | 18 | 16 | 12 | 1 |
| .10 | 8 | 10 | 13 | 18 | 20 | 23 | 28 | 37 | 51 | 72 | 98 | 131 | 166 | 226 | 265 | 254 | 226 | 187 | 151 | 113 | 86 | 63 | 46 | 37 | 31 | 26 | 23 | 21 | 19 | 16 | 13 | 2 |
| .20 | 8 | 10 | 13 | 17 | 19 | 22 | 26 | 33 | 45 | 63 | 87 | 116 | 149 | 212 | 259 | 259 | 233 | 197 | 160 | 119 | 90 | 66 | 48 | 38 | 32 | 27 | 24 | 22 | 19 | 16 | 13 | 2 |
| .30 | 7 | 9 | 12 | 16 | 18 | 21 | 24 | 30 | 40 | 55 | 76 | 103 | 134 | 197 | 244 | 255 | 238 | 206 | 169 | 125 | 95 | 68 | 49 | 38 | 32 | 27 | 24 | 22 | 19 | 17 | 13 | 2 |
| .40 | 7 | 8 | 11 | 14 | 15 | 17 | 19 | 23 | 28 | 36 | 49 | 67 | 91 | 151 | 208 | 247 | 252 | 230 | 196 | 146 | 109 | 77 | 54 | 41 | 34 | 29 | 25 | 22 | 19 | 17 | 13 | 3 |
| .50 | 6 | 8 | 10 | 13 | 15 | 16 | 18 | 21 | 26 | 33 | 43 | 59 | 80 | 136 | 194 | 238 | 249 | 235 | 204 | 154 | 115 | 81 | 56 | 42 | 34 | 29 | 25 | 23 | 20 | 17 | 13 | 3 |
| .75 | 5 | 7 | 8 | 11 | 12 | 13 | 14 | 16 | 18 | 21 | 25 | 32 | 42 | 76 | 125 | 179 | 222 | 240 | 233 | 193 | 148 | 102 | 67 | 48 | 38 | 32 | 27 | 24 | 20 | 18 | 13 | 5 |
| 1.0 | 4 | 5 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 16 | 18 | 22 | 34 | 59 | 101 | 152 | 201 | 236 | 230 | 193 | 135 | 86 | 59 | 44 | 35 | 30 | 26 | 21 | 18 | 14 | 7 |
| 1.5 | 3 | 4 | 5 | 6 | 6 | 7 | 8 | 8 | 9 | 10 | 11 | 12 | 13 | 16 | 22 | 34 | 58 | 95 | 141 | 203 | 226 | 197 | 131 | 84 | 58 | 43 | 35 | 29 | 23 | 20 | 15 | 10 |
| 2.0 | 1 | 2 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 10 | 12 | 16 | 22 | 34 | 56 | 110 | 172 | 218 | 187 | 126 | 82 | 57 | 43 | 34 | 25 | 21 | 16 | 11 |
| 2.5 | 1 | 1 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 11 | 14 | 18 | 34 | 69 | 141 | 210 | 190 | 133 | 87 | 60 | 44 | 30 | 23 | 17 | 12 |
| 3.0 | 0 | 0 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 3 | 3 | 4 | 5 | 5 | 6 | 8 | 9 | 11 | 16 | 27 | 66 | 149 | 204 | 181 | 128 | 85 | 58 | 35 | 25 | 18 | 12 |
| | IA/P = 0.30 | | | | | | | | | | | * * * TC =1.5 HR * * * | | | | | | | | | | | | | | | | | | | IA/P = 0.30 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 1 | 6 | 15 | 31 | 53 | 80 | 112 | 144 | 193 | 225 | 208 | 186 | 157 | 134 | 108 | 89 | 70 | 56 | 48 | 42 | 37 | 34 | 31 | 28 | 25 | 20 | 2 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 12 | 25 | 43 | 68 | 97 | 157 | 198 | 219 | 203 | 178 | 151 | 120 | 98 | 77 | 60 | 50 | 44 | 38 | 35 | 32 | 28 | 25 | 20 | 3 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 9 | 19 | 35 | 57 | 114 | 168 | 201 | 213 | 196 | 171 | 135 | 108 | 84 | 64 | 53 | 46 | 40 | 36 | 33 | 29 | 26 | 20 | 4 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 7 | 15 | 29 | 48 | 100 | 155 | 193 | 210 | 200 | 177 | 140 | 113 | 87 | 66 | 54 | 46 | 41 | 36 | 33 | 29 | 26 | 20 | 5 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 12 | 23 | 39 | 87 | 141 | 184 | 207 | 202 | 182 | 146 | 117 | 89 | 68 | 55 | 47 | 41 | 36 | 33 | 29 | 26 | 20 | 5 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 9 | 18 | 51 | 101 | 153 | 190 | 205 | 197 | 164 | 131 | 99 | 73 | 58 | 49 | 43 | 38 | 34 | 30 | 26 | 20 | 7 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 9 | 30 | 68 | 116 | 160 | 189 | 197 | 179 | 147 | 110 | 80 | 62 | 52 | 45 | 39 | 35 | 30 | 27 | 21 | 8 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 20 | 49 | 92 | 138 | 175 | 195 | 178 | 137 | 97 | 72 | 57 | 48 | 42 | 37 | 31 | 28 | 21 | 12 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | 21 | 47 | 85 | 145 | 187 | 178 | 133 | 95 | 71 | 57 | 48 | 42 | 34 | 29 | 23 | 16 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 13 | 45 | 97 | 162 | 180 | 138 | 99 | 74 | 58 | 49 | 38 | 32 | 25 | 18 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 31 | 89 | 161 | 174 | 133 | 97 | 72 | 58 | 42 | 34 | 26 | 18 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 29 | 98 | 160 | 169 | 129 | 95 | 71 | 48 | 37 | 28 | 19 |
| | IA/P = 0.50 | | | | | | | | | | | * * * TC =1.5 HR * * * | | | | | | | | | | | | | | | | | | | IA/P = 0.50 |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 8 | 16 | 27 | 42 | 59 | 92 | 116 | 128 | 130 | 121 | 112 | 100 | 90 | 78 | 67 | 60 | 55 | 50 | 46 | 43 | 39 | 35 | 29 | 4 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 12 | 22 | 35 | 51 | 84 | 110 | 125 | 128 | 123 | 114 | 102 | 91 | 79 | 68 | 61 | 55 | 50 | 46 | 43 | 39 | 35 | 29 | 4 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 18 | 29 | 60 | 91 | 114 | 126 | 128 | 120 | 108 | 97 | 83 | 71 | 63 | 57 | 52 | 47 | 44 | 40 | 36 | 29 | 5 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 14 | 24 | 52 | 83 | 108 | 123 | 126 | 122 | 110 | 98 | 85 | 72 | 63 | 57 | 52 | 48 | 44 | 40 | 36 | 29 | 6 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 12 | 31 | 60 | 90 | 112 | 124 | 126 | 116 | 104 | 90 | 75 | 66 | 59 | 54 | 49 | 45 | 41 | 37 | 29 | 8 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 9 | 26 | 53 | 83 | 106 | 121 | 125 | 118 | 106 | 91 | 77 | 67 | 60 | 54 | 49 | 46 | 41 | 37 | 29 | 8 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 16 | 36 | 62 | 88 | 108 | 119 | 122 | 112 | 97 | 81 | 69 | 62 | 56 | 51 | 47 | 42 | 38 | 30 | 11 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 10 | 26 | 49 | 75 | 98 | 118 | 121 | 108 | 90 | 76 | 66 | 59 | 54 | 49 | 43 | 39 | 31 | 16 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 11 | 25 | 45 | 80 | 107 | 118 | 106 | 89 | 75 | 65 | 59 | 53 | 45 | 41 | 32 | 23 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 11 | 32 | 63 | 100 | 115 | 104 | 87 | 74 | 65 | 58 | 48 | 42 | 34 | 26 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 16 | 48 | 94 | 113 | 105 | 89 | 76 | 66 | 53 | 45 | 36 | 27 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 15 | 54 | 96 | 111 | 103 | 88 | 75 | 58 | 48 | 38 | 28 |
| | RAINFALL TYPE = II | | | | | | | | | | | * * * TC = 1.5 HR * * * | | | | | | | | | | | | | | | | | | | SHEET 9 OF 10 |

## Exhibit 5-II, continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
| | IA/P = 0.10 | | | | | | | | | | | * * * TC =2.0 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.10 | |
| 0.0 | 7 | 9 | 12 | 16 | 18 | 21 | 27 | 36 | 49 | 64 | 82 | 104 | 127 | 171 | 201 | 226 | 208 | 193 | 171 | 132 | 105 | 79 | 58 | 45 | 36 | 30 | 26 | 23 | 20 | 17 | 13 | 3 |
| .10 | 6 | 8 | 10 | 14 | 15 | 17 | 20 | 25 | 33 | 43 | 57 | 74 | 94 | 139 | 179 | 204 | 218 | 205 | 188 | 150 | 118 | 88 | 63 | 48 | 38 | 32 | 27 | 24 | 20 | 17 | 13 | 4 |
| .20 | 6 | 8 | 10 | 13 | 14 | 16 | 19 | 23 | 29 | 39 | 51 | 66 | 84 | 128 | 169 | 198 | 213 | 207 | 192 | 157 | 123 | 91 | 65 | 49 | 39 | 33 | 28 | 24 | 20 | 17 | 13 | 4 |
| .30 | 6 | 7 | 9 | 12 | 14 | 15 | 18 | 21 | 27 | 35 | 45 | 59 | 76 | 117 | 159 | 191 | 211 | 208 | 196 | 163 | 128 | 95 | 68 | 51 | 40 | 33 | 28 | 25 | 20 | 18 | 13 | 4 |
| .40 | 5 | 6 | 8 | 11 | 12 | 13 | 15 | 17 | 20 | 24 | 31 | 41 | 53 | 87 | 128 | 167 | 197 | 209 | 205 | 180 | 145 | 106 | 75 | 55 | 43 | 35 | 30 | 26 | 21 | 18 | 14 | 5 |
| .50 | 5 | 6 | 8 | 10 | 11 | 13 | 14 | 16 | 18 | 22 | 28 | 37 | 48 | 78 | 118 | 158 | 190 | 208 | 208 | 185 | 151 | 111 | 77 | 57 | 44 | 36 | 30 | 26 | 21 | 18 | 14 | 5 |
| .75 | 4 | 6 | 7 | 9 | 10 | 11 | 12 | 13 | 15 | 18 | 22 | 27 | 35 | 58 | 91 | 129 | 164 | 191 | 202 | 194 | 167 | 125 | 87 | 63 | 48 | 38 | 32 | 27 | 22 | 18 | 14 | 6 |
| 1.0 | 3 | 4 | 6 | 7 | 8 | 8 | 9 | 10 | 11 | 12 | 14 | 16 | 18 | 28 | 46 | 74 | 110 | 147 | 178 | 201 | 193 | 156 | 108 | 76 | 56 | 43 | 35 | 30 | 23 | 19 | 14 | 8 |
| 1.5 | 2 | 3 | 3 | 5 | 5 | 5 | 6 | 6 | 7 | 8 | 8 | 9 | 10 | 12 | 16 | 23 | 36 | 57 | 86 | 137 | 178 | 195 | 160 | 113 | 79 | 58 | 45 | 36 | 26 | 21 | 16 | 11 |
| 2.0 | 1 | 2 | 2 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 10 | 12 | 16 | 23 | 35 | 67 | 112 | 169 | 190 | 154 | 110 | 78 | 57 | 44 | 30 | 23 | 17 | 11 |
| 2.5 | 0 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 4 | 5 | 6 | 7 | 8 | 9 | 12 | 16 | 28 | 52 | 105 | 170 | 185 | 149 | 107 | 76 | 56 | 35 | 26 | 18 | 12 |
| 3.0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 5 | 6 | 7 | 8 | 12 | 18 | 41 | 99 | 161 | 180 | 152 | 112 | 80 | 45 | 30 | 19 | 12 |
| | IA/P = 0.30 | | | | | | | | | | | * * * TC =2.0 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.30 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 8 | 15 | 25 | 38 | 54 | 74 | 115 | 148 | 168 | 185 | 170 | 159 | 131 | 110 | 89 | 70 | 57 | 49 | 42 | 38 | 34 | 29 | 26 | 20 | 5 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 6 | 12 | 21 | 32 | 47 | 85 | 124 | 153 | 169 | 180 | 168 | 145 | 120 | 96 | 75 | 60 | 51 | 44 | 39 | 35 | 30 | 26 | 20 | 6 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 10 | 17 | 27 | 41 | 75 | 114 | 146 | 165 | 175 | 170 | 149 | 124 | 99 | 76 | 62 | 52 | 45 | 39 | 35 | 30 | 27 | 21 | 6 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 7 | 14 | 23 | 49 | 86 | 122 | 151 | 170 | 174 | 160 | 136 | 107 | 82 | 66 | 54 | 47 | 41 | 37 | 31 | 27 | 21 | 8 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 6 | 11 | 19 | 43 | 77 | 113 | 144 | 165 | 173 | 163 | 140 | 111 | 85 | 67 | 55 | 47 | 41 | 37 | 31 | 27 | 21 | 8 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 9 | 16 | 37 | 68 | 104 | 136 | 160 | 171 | 165 | 144 | 114 | 87 | 69 | 56 | 48 | 42 | 37 | 31 | 27 | 21 | 9 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 15 | 34 | 62 | 96 | 127 | 152 | 167 | 160 | 132 | 100 | 77 | 62 | 52 | 45 | 40 | 32 | 28 | 22 | 11 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 10 | 24 | 48 | 79 | 111 | 150 | 166 | 153 | 118 | 90 | 71 | 58 | 49 | 43 | 34 | 29 | 23 | 14 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 10 | 24 | 45 | 88 | 130 | 161 | 148 | 115 | 88 | 70 | 57 | 48 | 37 | 31 | 24 | 17 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 10 | 32 | 68 | 122 | 157 | 143 | 113 | 87 | 68 | 56 | 42 | 34 | 26 | 18 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 16 | 51 | 114 | 153 | 144 | 116 | 89 | 70 | 49 | 38 | 27 | 19 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 15 | 59 | 118 | 150 | 140 | 113 | 88 | 57 | 42 | 29 | 19 |
| | IA/P = 0.50 | | | | | | | | | | | * * * TC =2.0 HR * * * | | | | | | | | | | | | | | | | | | IA/P = 0.50 | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 8 | 13 | 20 | 28 | 51 | 73 | 92 | 104 | 111 | 112 | 106 | 97 | 86 | 75 | 66 | 60 | 54 | 49 | 46 | 41 | 37 | 30 | 7 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 11 | 17 | 24 | 45 | 68 | 87 | 101 | 109 | 112 | 107 | 98 | 88 | 76 | 67 | 60 | 55 | 50 | 46 | 41 | 37 | 30 | 8 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 5 | 9 | 14 | 21 | 40 | 62 | 82 | 98 | 107 | 111 | 108 | 100 | 89 | 77 | 68 | 61 | 55 | 50 | 47 | 41 | 37 | 30 | 8 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 4 | 7 | 12 | 26 | 46 | 67 | 86 | 100 | 108 | 111 | 104 | 93 | 80 | 70 | 63 | 57 | 52 | 48 | 42 | 38 | 30 | 10 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 6 | 10 | 22 | 41 | 62 | 81 | 96 | 106 | 110 | 105 | 94 | 81 | 71 | 63 | 57 | 52 | 48 | 42 | 38 | 30 | 11 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 4 | 13 | 27 | 46 | 67 | 85 | 99 | 110 | 108 | 98 | 85 | 74 | 66 | 59 | 54 | 49 | 43 | 39 | 31 | 13 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 2 | 7 | 18 | 33 | 52 | 71 | 88 | 104 | 108 | 102 | 89 | 77 | 68 | 61 | 55 | 50 | 44 | 39 | 31 | 15 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 13 | 25 | 43 | 62 | 87 | 103 | 108 | 97 | 84 | 73 | 65 | 59 | 53 | 45 | 41 | 32 | 20 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 12 | 24 | 48 | 74 | 99 | 106 | 95 | 83 | 72 | 64 | 58 | 48 | 43 | 34 | 25 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 5 | 17 | 37 | 69 | 99 | 104 | 94 | 82 | 72 | 64 | 52 | 45 | 36 | 27 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 8 | 27 | 65 | 95 | 102 | 95 | 83 | 73 | 58 | 49 | 38 | 28 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 32 | 68 | 95 | 101 | 93 | 82 | 64 | 52 | 40 | 28 |
| | RAINFALL TYPE = II | | | | | | | | | | | * * * TC = 2.0 HR * * * | | | | | | | | | | | | | | | | | | SHEET 10 OF 10 | |

ODOT Hydraulic Manual Users: Exhibit 5-III, the Type III rainfall distribution, is omitted from this publication. This rainfall distribution does not occur in Oregon.

# Chapter 6: Storage volume for detention basins

As rural areas become urbanized, the resulting increases in peak discharges can adversely affect downstream flood plains. Increasingly, planners, developers, and the public want these downstream areas to be protected. Many local governments are adopting ordinances to control the type of development and its allowable impacts on the watershed. One of the most common controls requires that postdevelopment discharges do not exceed present-condition discharges for one or more storm frequencies at specified points along a channel.

This chapter discusses ways to manage peak discharges by delaying runoff. It also presents a procedure for estimating the storage capacity required to maintain the peaks within a specified level.

Efforts to reduce the effects of increased runoff from urban areas have been innovative and diverse. Many methods have been used effectively, such as infiltration trenches, porous pavement, rooftop storage, and cisterns. But these solutions can be expensive or require site conditions that cannot be provided.

The detention basin is the most widely used measure for controlling peak discharge. It is generally the least expensive and most reliable of the measures that have been considered. It can be designed to fit a wide variety of sites and can accommodate multiple-outlet spillways to meet requirements for multifrequency control of outflow. Measures other than a detention basin may be preferred in some locations; their omission here is not intended to discourage their use. Any device selected, however, should be assessed as to its function, maintenance needs, and impact.

## Estimating the effect of storage

When a detention basin is installed, hydraulic routing procedures can be used to estimate the effect on hydrographs. Both the TR-20 (SCS 1983) and DAMS2 (SCS 1982) computer programs provide accurate methods of analysis. Programmable calculator and computer programs are available for routing hydrographs through dams.

This chapter contains a manual method for quick estimates of the effects of temporary detention on peak discharges. The method is based on average storage and routing effects for many structures.

Figure 6-1 relates two ratios: peak outflow to peak inflow discharge ($q_o/q_i$) and storage volume to runoff volume ($V_s/V_r$) for all four rainfall distributions.

The relationships in figure 6-1 were determined on the basis of single stage outflow devices. Some were controlled by pipe flow, others by weir flow. Verification runs were made using multiple stage outflow devices, and the variance was similar to that in the base data. The method can therefore be used for both single- and multiple-stage outflow devices. The only constraints are that (1) each stage requires a design storm and a computation of the storage required for it and (2) the discharge of the upper stage(s) includes the discharge of the lower stage(s).

The brevity of the procedure allows the planner to examine many combinations of detention basins. When combined with the Tabular Hydrograph method, the procedure's usefulness is increased. Its principal use is to develop preliminary indications of storage adequacy and to allocate control to a group of detention basins. It is also adequate, however, for final design of small detention basins.

## Input requirements and procedures

Use figure 6-1 to estimate storage volume ($V_s$) required or peak outflow discharge ($q_o$). The most frequent application is to estimate $V_s$, for which the required inputs are runoff volume ($V_r$), $q_o$, and peak inflow discharge ($q_i$). To estimate $q_o$, the required inputs are $V_r$, $V_s$, and $q_i$.

### Estimating $V_s$

Use worksheet 6a to estimate $V_s$, storage volume required, by the following procedure.

1. Determine $q_o$. Many factors may dictate the selection of peak outflow discharge. The most common is to limit downstream discharges to a desired level, such as predevelopment discharge. Another factor may be that the outflow device has already been selected.
2. Estimate $q_i$ by procedures in chapters 4 or 5. Do not use peak discharges developed by any other procedure. When using the Tabular Hydrograph method to estimate $q_i$ for a subarea, only use

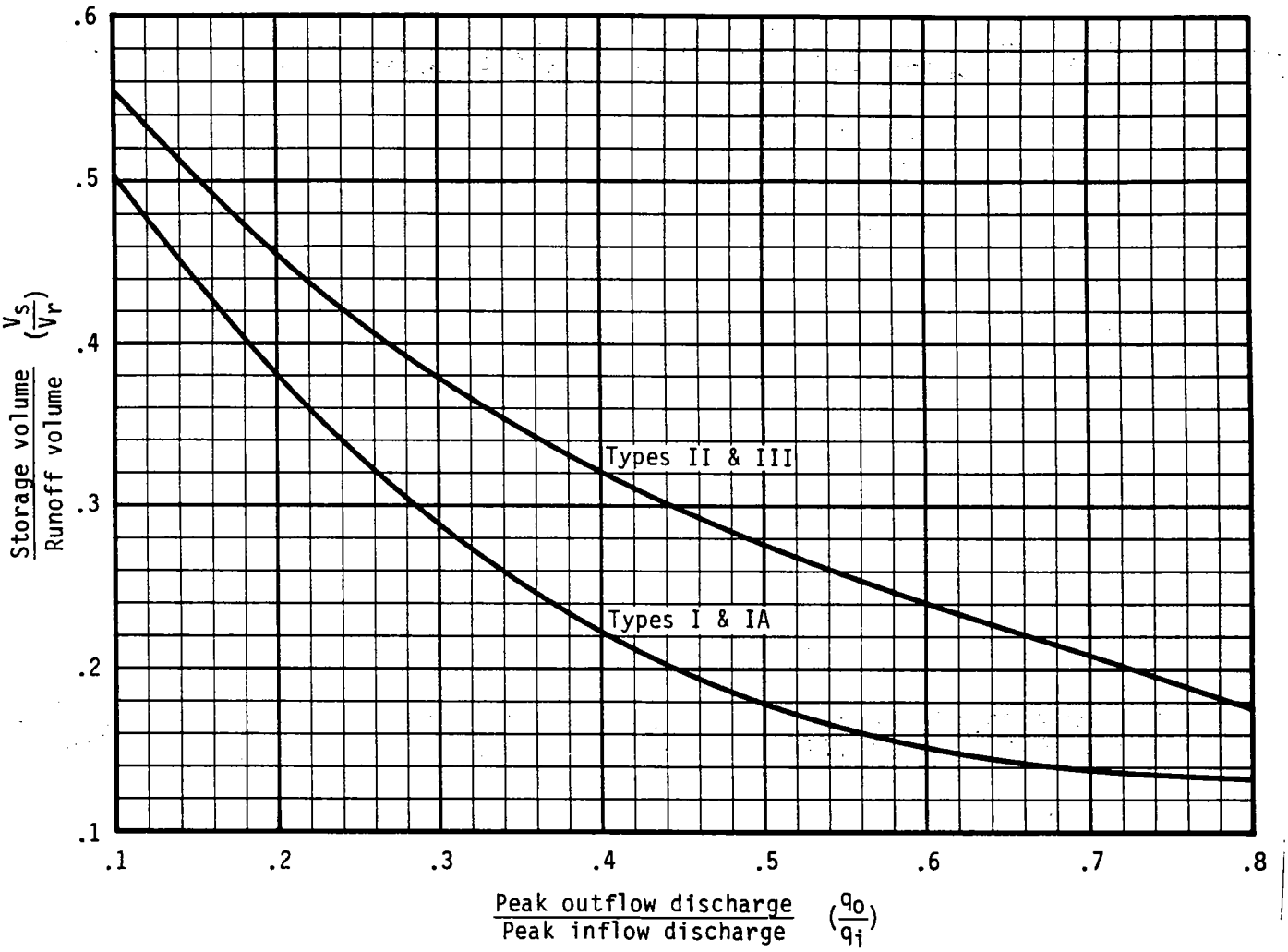



Figure 6-1.—Approximate detention basin routing for rainfall types I, IA, II, and III.

peak discharge associated with $T_t = 0$.

3. Compute $q_o/q_i$ and determine $V_s/V_r$ from figure 6-1.
4. Q (in inches) was determined when computing $q_i$ in step 2, but now it must be converted to the units in which $V_s$ is to be expressed—most likely, acre-feet or cubic feet. The most common conversion of Q to $V_r$ is expressed in acre-feet:

$$V_r = 53.33Q(A_m) \qquad \text{[Eq. 6-1]}$$

where

$V_r$ = runoff volume (acre-ft),
Q = runoff (in),
$A_m$ = drainage area ($mi_2$), and
53.33 = conversion factor from in-$mi^2$ to acre-ft.

5. Use the results of steps 3 and 4 to compute $V_s$:

$$V_s = V_r\left(\frac{V_s}{V_r}\right) \qquad \text{[Eq. 6-2]}$$

where $V_s$ = storage volume required (acre-ft).

6. The stage in the detention basin corresponding to $V_s$ must be equal to the stage used to generate $q_o$. In most situations a minor modification of the outflow device can be made. If the outflow device has been preselected, repeat the calculations with a modified $q_o$ value.

### Estimating $q_o$

Use worksheet 6b to estimate $q_o$, required peak outflow discharge, by the following procedure.

1. Determine $V_s$. If the maximum stage in the detention basin is constrained, set $V_s$ by the maximum permissible stage.
2. Compute Q (in inches) by the procedures in chapter 2, and convert it to the same units as $V_s$ (see step 4 in "Estimating $V_s$").
3. Compute $V_s/V_r$ and determine $q_o/q_i$ from figure 6-1.
4. Estimate $q_i$ by the procedures in chapters 4 or 5. Do not use peak discharges developed by any other method. When using the Tabular method to estimate $q_i$ for a subarea, use only the peak discharge associated with $T_t = 0$.
5. From steps 3 and 4, compute $q_o$:

$$q_o = q_i\left(\frac{q_o}{q_i}\right) \qquad \text{[Eq. 6-3]}$$

6. Proportion the outflow device so that the stage at $q_o$ is equal to the stage corresponding to $V_s$. If $q_o$ cannot be calibrated except in discrete steps (i.e., pipe sizes), repeat the procedure until the stages for $q_o$ and $V_s$ are approximately equal.

## Limitations

- This routing method is less accurate as the $q_o/q_i$ ratio approaches the limits shown in figure 6-1. The curves in figure 6-1 depend on the relationship between available storage, outflow device, inflow volume, and shape of the inflow hydrograph. When storage volume ($V_s$) required is small, the shape of the outflow hydrograph is sensitive to the rate of rise of the inflow hydrograph. Conversely, when $V_s$ is large, the inflow hydrograph shape has little effect on the outflow hydrograph. In such instances, the outflow hydrograph is controlled by the hydraulics of the outflow device and the procedure therefore yields consistent results. When the peak outflow discharge ($q_o$) approaches the peak inflow discharge ($q_i$), parameters that affect the rate of rise of a hydrograph, such as rainfall volume, curve number, and time of concentration, become especially significant.

- The procedure should not be used to perform final design if an error in storage of 25 percent cannot be tolerated. Figure 6-1 is biased to prevent undersizing of outflow devices, but it may significantly overestimate the required storage capacity. More detailed hydrograph development and routing will often pay for itself through reduced construction costs.

## Examples

Four examples illustrate the use of figure 6-1. Examples 6-1 through 6-4, respectively, show estimation of $V_s$, use of a two-stage structure, estimation of $q_o$, and use with the Tabular Hydrograph method.

### Example 6-1: Estimating $V_s$, single-stage structure

A development is being planned in a 75-acre (0.117-mi$^2$) watershed that outlets into an existing concrete-lined channel designed for present conditions. If the channel capacity is exceeded, damages will be substantial. The watershed is in the type II storm distribution region. The present channel capacity, 180 cfs, was established by computing discharge for the 25-year-frequency storm by the Graphical Peak Discharge method (chapter 4).

The developed-condition peak discharge ($q_i$) computed by the same method is 360 cfs, and runoff (Q) is 3.4 inches. Since outflow must be held to 180 cfs, a detention basin having that maximum outflow discharge ($q_o$) will be built at the watershed outlet.

How much storage ($V_s$) will be required to meet the maximum outflow discharge ($q_o$) of 180 cfs, and what will be the approximate dimensions of a rectangular weir outflow structure? Figure 6-2 shows how worksheet 6a is used to estimate required storage ($V_s$ = 5.9 acre-ft) and maximum stage ($E_{max}$ = 105.7 ft).

The rectangular weir was chosen for its simplicity; however, several types of outlets can meet the outflow device proportion requirement. Most hydraulic references, along with considerable research data that are available, provide more guidance on variations of outlet devices than can be summarized here.

An outlet device should be proportioned to meet specific objectives. A single-stage device was specified in this example because only one storm was considered. A weir is suitable here because of the low head. The weir crest elevation is 100.0 ft.

Using $V_s$ = 5.9 acre-ft (figure 6-2, step 9) and the elevation-storage curve, the maximum stage ($E_{max}$) is 105.7 ft.

The rectangular weir equation is

$$q_o = 3.2\ L_w H_w^{1.5} \qquad \text{[Eq. 6-4]}$$

where

$q_o$ = peak outflow discharge (cfs),
$L_w$ = weir crest length (ft), and
$H_w$ = head over weir crest (ft).

$H_w$ and $q_o$ are computed as follows:

$$H_w = E_{max} - \text{weir crest elevation}$$
$$= 105.7 - 100.0 = 5.7 \text{ ft.}$$

Since $q_o$ is known to be 180 cfs, solving equation 6-4 for $L_w$ yields

$$L_w = \frac{q_o}{3.2\ H_w^{1.5}} \qquad \text{[Eq. 6-5]}$$

$$= \frac{180}{3.2\ (5.7)^{1.5}} = 4.1 \text{ ft.}$$

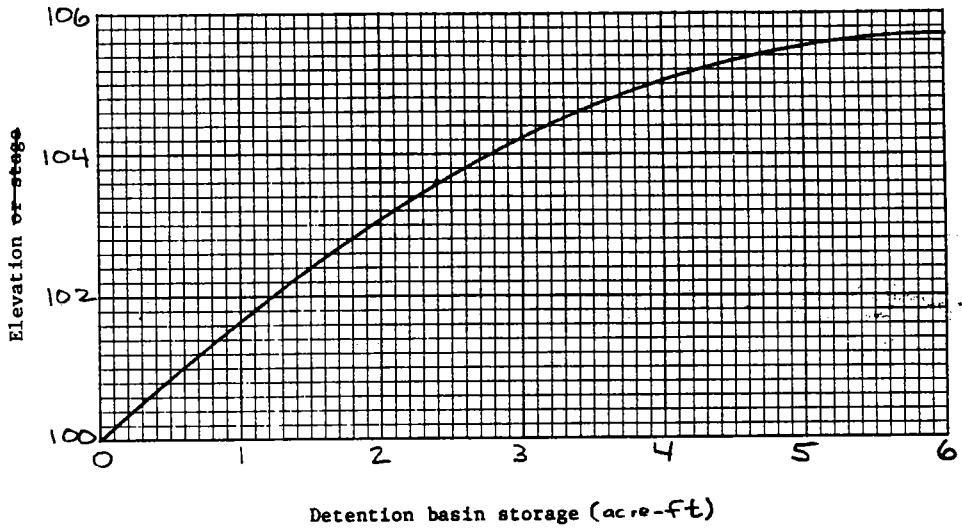
In summary, the outlet structure is a rectangular weir with crest length of 4.1 ft, $H_w$ = 5.7 ft, and $q_o$ = 180 cfs corresponding to a $V_s$ = 5.9 acre-ft.

## Worksheet 6a: Detention basin storage, peak outflow discharge ($q_o$) known

Project Robbinsville By SWR Date 11/5/85

Location Dyer County, Tennessee Checked RGC Date 11/8/85

Circle one: Present (Developed) Single-stage-structure




1. Data:
   Drainage area ..... $A_m$ = 0.117 $mi^2$
   Rainfall distribution type (I, IA, II, III) = II

| | 1st stage | 2nd stage |
|---|---|---|
| 2. Frequency ..... yr | 25 | |
| 3. Peak inflow discharge, $q_i$ .... cfs (From worksheet 4 or 5b) | 360 | |
| 4. Peak outflow discharge, $q_o$ .... cfs 1/ | 180 | |
| 5. Compute $\frac{q_o}{q_i}$ ..... | 0.50 | |

6. $\frac{V_s}{V_r}$ ..... 0.28 (Use $\frac{q_o}{q_i}$ with figure 6-1)
7. Runoff, Q ..... in 3.4 (From worksheet 2)
8. Runoff volume, $V_r$ ..... ac-ft 21.2 ($V_r = QA_m 53.33$)
9. Storage volume, $V_s$ ..... ac-ft 5.9 ($V_s = V_r(\frac{V_s}{V_r})$)
10. Maximum stage, $E_{max}$ 105.7 (From plot)

1/ 2nd stage $q_o$ includes 1st stage $q_o$.

Figure 6-2.—Worksheet 6a for example 6-1.

**Example 6-2: Estimating $V_s$, two-stage structure**

In addition to the requirements for a 25-year peak outflow discharge of 180 cfs stated in example 6-1, a decision was made to limit the 2-year outflow discharge to 50 cfs because of potential damages to agricultural property below the lined channel. By the method in chapter 4, the estimated 2-year peak discharge for developed conditions will be 91 cfs and runoff (Q) will be 1.5 inches.

Again, a rectangular concrete weir outflow device was selected; the device could have been another type, but it is important to remember that the flows through the first stage are part of the total discharge of the higher stage.

Figure 6-3 shows how worksheet 6a is used to compute the $V_s$ of 2.4 acre-ft and $E_{max}$ of 103.6 for the first stage. $E_{max}$ of 103.6 is the weir crest elevation for the second stage.

Equation 6-5 is again used to compute $L_w$ for the first stage. The weir crest elevation for the first stage is 100.00 ft and $q_o$ = 50 cfs. The first-stage computations for $H_w$ and $L_w$ are

$$H_w = E_{max} - \text{weir crest elevation}$$
$$= 103.6 - 100.0 = 3.6 \text{ ft;}$$

and, from equation 6-5,

$$L_w = \frac{50}{3.2(3.6)^{1.5}} = 2.3 \text{ ft.}$$

The second stage is then proportioned to discharge the correct amount at 105.7 ft (figure 6-2, step 10). Compute the discharge through the first stage for elevation 105.7 ft using

$$L_w = 2.3 \text{ ft (first stage)}$$

and

$$H_w = 105.7 - 100.0 = 5.7 \text{ ft.}$$

By substituting these values in equation 6-4, discharge ($q_o$) through the first stage at 105.7 ft is calculated:

$$q_o = 3.2(2.3)(5.7)^{1.5} = 100 \text{ cfs.}$$

Now compute the required weir crest length ($L_w$) for the second stage, using equation 6-5. Since the second stage crest elevation is 103.6 ft,

$$H_w = 105.7 - 103.6 = 2.1 \text{ ft;}$$

and, since $q_o$ for the second stage equals the total discharge from example 6-1 minus discharge through the first stage,

$$q_o = 180 - 100 = 80 \text{ cfs.}$$

Finally, substituting these $H_w$ and $q_o$ values in equation 6-5 results in

$$L_w = \frac{80}{3.2(2.1)^{1.5}} = 8.2 \text{ ft.}$$

In summary, the outlet structure is a 2-stage rectangular weir with first stage crest length of 2.3 ft at elevation 100.0, and second stage crest length of 8.2 ft at elevation 103.6 ft.

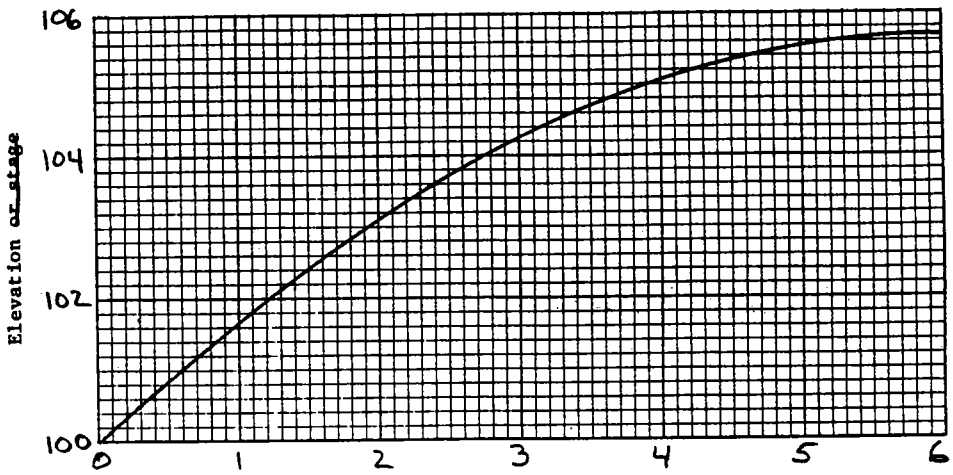
The weir equation used is probably less accurate for the two-stage example than for the single-stage example. The actual second-stage discharge will be slightly more than the one computed, but a discussion of hydraulics of outflow devices is outside the scope of this technical release. Example 6-2 is presented only to illustrate the interrelationship of outflow discharges and storage volume and to show how to develop preliminary estimates of storage requirements for two-stage outlet structures.

## Worksheet 6a: Detention basin storage, peak outflow discharge ($q_o$) known

Project Robbinsville By SWR Date 11/6/85

Location Dyer County, Tennessee Checked RGC Date 11/9/85

Circle one: Present (Developed) 2- stage structure




1. Data:
   Drainage area ..... $A_m$ = 0.117 $mi^2$
   Rainfall distribution type (I, IA, II, III) = II

| | 1st stage | 2nd stage |
|---|---|---|
| 2. Frequency ..... yr | 2 | 25 |
| 3. Peak inflow discharge, $q_i$ .... cfs (From worksheet 4 or 5b) | 91 | 360 |
| 4. Peak outflow discharge, $q_o$ .... cfs 1/ | 50 | 180 |
| 5. Compute $\frac{q_o}{q_i}$ ..... | 0.55 | 0.50 |
| 6. $\frac{V_s}{V_r}$ ..... (Use $\frac{q_o}{q_i}$ with figure 6-1) | 0.26 | 0.28 |
| 7. Runoff, Q ..... in (From worksheet 2) | 1.5 | 3.4 |
| 8. Runoff volume, $V_r$ ..... ac-ft ($V_r = QA_m 53.33$) | 9.4 | 21.2 |
| 9. Storage volume, $V_s$ ..... ac-ft ($V_s = V_r(\frac{V_s}{V_r})$) | 2.4 | 5.9 |
| 10. Maximum stage, $E_{max}$ (From plot) | 103.6 | 105.7 |

1/ 2nd stage $q_o$ includes 1st stage $q_o$.

Figure 6-3.—Worksheet 6a for example 6-2.

**Example 6-3: Estimating $q_o$**

A development is being planned for a 10-acre watershed (0.0156 $mi^2$). A county ordinance requires that the developed-condition outflow from the watershed for a 24-hr, 100-year frequency storm does not exceed the outflow for present conditions. The peak discharge from the watershed for present conditions, 35 cfs, is calculated from procedures in chapter 4. For developed conditions, runoff (Q) is 5.4 inches, peak discharge from the watershed is 42 cfs from procedures in chapter 4, and rainfall distribution is type II.

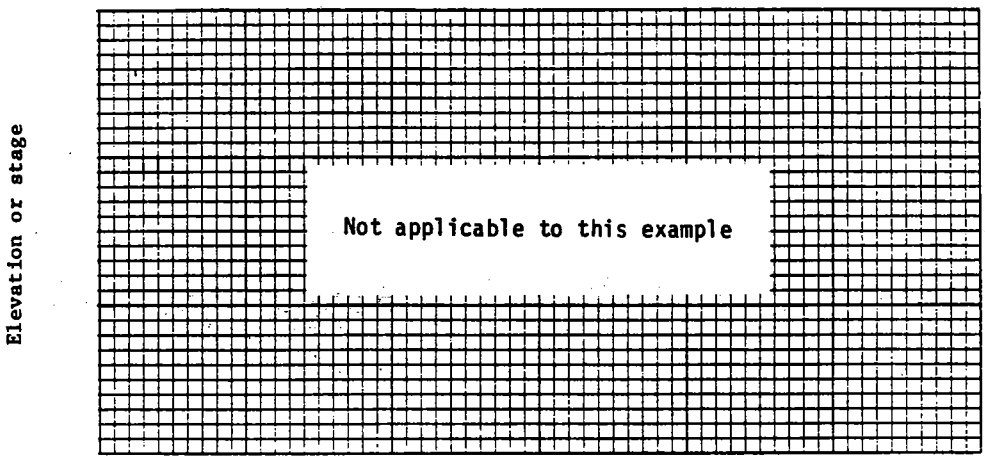
What will be the peak outflow discharge ($q_o$) from a detention basin that is located at the outlet and has maximum allowable storage volume ($V_s$) of 35,000 $ft^3$ and peak inflow discharge ($q_i$) of 42 cfs? Figure 6-4 shows how worksheet 6b is used to estimate $q_o$ as 33 cfs, which is within the 35-cfs limit. An outflow device will be selected to discharge 33 cfs at a stage corresponding to a $V_s$ of 35,000 $ft^3$.

## Worksheet 6b: Detention basin peak outflow, storage volume ($V_s$) known

Project Woods Acres By SWR Date 11/18/85

Location Dyer County, Tennessee Checked RGC Date 11/11/85

Circle one: Present (Developed)




Detention basin storage

| | | 1st stage | 2nd stage |
|---|---|---|---|
| 1. | Data: Drainage area ..... $A_m$ = 0.0156 $mi^2$; Rainfall distribution type (I, IA, II, III) = II | | |
| 2. | Frequency ..... yr | 100 | |
| 3. | Storage volume, $V_s$ ..... ac ft | 0.8 | |
| 4. | Runoff, Q ..... in (From worksheet 2) | 5.4 | |
| 5. | Runoff volume, $V_r$ ..... ac-ft ($V_r = QA_m 53.33$) | 4.5 | |
| 6. | Compute $\frac{V_s}{V_r}$ ..... | 0.18 | |
| 7. | $\frac{q_o}{q_i}$ ..... in ($\frac{V_s}{V_r}$ and figure 6-1) | 0.78 | |
| 8. | Peak inflow discharge, $q_i$ .... cfs (From worksheet 4 or 5b) | 42 | |
| 9. | Peak outflow discharge, $q_o$ .... cfs ($q_o = q_i(\frac{q_o}{q_i})$) 1/ | 33 | |
| 10. | Maximum stage, $E_{max}$ (From plot) | N/A | |

1/ 2nd stage $q_o$ includes 1st stage $q_o$.

Figure 6-4.—Worksheet 6b for example 6-3.

**Example 6-4: Estimating $V_s$, Tabular Hydrograph method**

This example builds on examples 5-1 and 5-2 (pages 5-4 to 5-8). If peak outflow discharge from subarea 7 must not exceed the discharge for present conditions, what will be the storage volume ($V_s$) required in a detention basin at the outlet of subarea 6?

First, compute the outflow hydrograph without subarea 6 as shown in the table below, which presents developed-condition discharges for example 5-2. (The information in the table is from figure 5-4.)

| | Discharge (cfs) at time (hr)— | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Subarea | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 |
| | *cfs* | | | | | | | | |
| 1 | 7 | 9 | 11 | 16 | 24 | 40 | 78 | 122 | 155 |
| 2 | 7 | 9 | 12 | 20 | 33 | 55 | 96 | 132 | 132 |
| 3 | 14 | 29 | 58 | 89 | 106 | 102 | 74 | 46 | 25 |
| 4 | 19 | 32 | 63 | 114 | 169 | 207 | 193 | 143 | 83 |
| 5 | 117 | 167 | 205 | 214 | 202 | 175 | 132 | 99 | 70 |
| 6 omitted | — | — | — | — | — | — | — | — | — |
| 7 | 244 | 167 | 119 | 90 | 72 | 59 | 48 | 40 | 34 |
| Total without subarea 6 | 408 | 413 | 468 | 543 | 606 | 638 | 621 | 582 | 499 |

After computing the outflow hydrograph, determine the maximum permissible outflow discharge from subarea 6. The present condition peak discharge at the outlet of subarea 7 is 720 cfs at 14.3 hr (figure 5-2), and the developed condition peak discharge at the outlet of subarea 7 minus subarea 6 is 638 cfs (table above). The difference between these two discharges, 82 cfs, is the maximum outflow discharge ($q_o$) for the detention basin.

Next, determine the peak discharge for subarea 6 for developed conditions by substituting values in equation 5-1:

$$q = q_t \, A_m Q. \qquad \text{[Eq. 5-1]}$$

From exhibit 5-II, the largest $q_t$ value is 357 csm/in (exhibit 5-II, sheet 7: $T_c$ = 1.0 hr, $T_t$ = 0, and $I_a/P$ = 0.10 at 12.8 hr). From figure 5-4, $A_m Q$ for subarea 6 is 1.31. Therefore,

$$q = (357)\,(1.31) = 468 \text{ cfs.}$$

This q value is, of course, the same as the peak inflow discharge ($q_i$) into the detention basin.

Finally, use worksheet 6a (figure 6-5) to compute $V_s$ as 33.2 acre-ft.

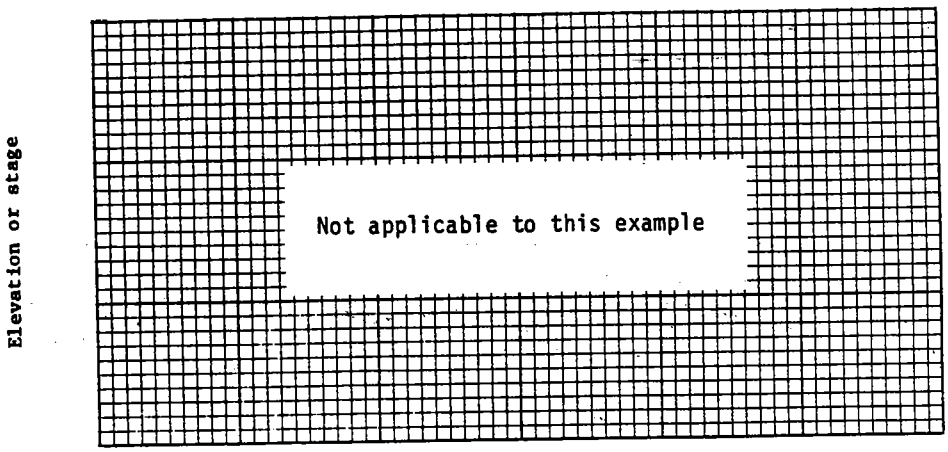
The required storage volume of 33.2 acre-ft is the basis for determining the required stage in the detention basin. This stage is a guide in proportioning a spillway that will discharge 82 cfs or less at that storage. The timing or routing effect is not considered because the outflow hydrograph will discharge at near $q_o$ for a significant period.

## Worksheet 6a: Detention basin storage, peak outflow discharge ($q_o$) known

Project Fallswood By SNR Date 10/8/85

Location Dyer County, Tennessee Checked RGC Date 10/10/85

Circle one: Present Developed




Detention basin storage

1. Data:
   Drainage area ..... $A_m$ = 0.40 $mi^2$
   Rainfall distribution type (I, IA, II, III) = II

| | 1st stage | 2nd stage |
|---|---|---|
| 2. Frequency ..... yr | 25 | |
| 3. Peak inflow discharge, $q_i$ .... cfs (From worksheet 4 or 5b) | 468 | |
| 4. Peak outflow discharge, $q_o$ .... cfs 1/ | 82 | |
| 5. Compute $\frac{q_o}{q_i}$ ..... | 0.175 | |

6. $\frac{V_s}{V_r}$ ..... 0.475 (Use $\frac{q_o}{q_i}$ with figure 6-1)
7. Runoff, Q ..... in 3.28 (From worksheet 2)
8. Runoff volume, $V_r$ ..... ac-ft 69.9 ($V_r = QA_m 53.33$)
9. Storage volume, $V_s$ ..... ac-ft 33.2 ($V_s = V_r(\frac{V_s}{V_r})$)
10. Maximum stage, $E_{max}$ N/A (From plot)

1/ 2nd stage $q_o$ includes 1st stage $q_o$.

Figure 6-5.—Worksheet 6a for example 6-4.

# Appendix A: Hydrologic soil groups

Soils are classified into hydrologic soil groups (HSG's) to indicate the minimum rate of infiltration obtained for bare soil after prolonged wetting. The HSG's, which are A, B, C, and D, are one element used in determining runoff curve numbers (see chapter 2). For the convenience of TR-55 users, exhibit A-1 lists the HSG classification of United States soils.

The infiltration rate is the rate at which water enters the soil at the soil surface. It is controlled by surface conditions. HSG also indicates the transmission rate—the rate at which the water moves within the soil. This rate is controlled by the soil profile. Approximate numerical ranges for transmission rates shown in the HSG definitions were first published by Musgrave (USDA 1955). The four groups are defined by SCS soil scientists as follows:

Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission (greater than 0.30 in/hr).

Group B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15-0.30 in/hr).

Group C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05-0.15 in/hr).

Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-0.05 in/hr).

In exhibit A-1, some of the listed soils have an added modifier; for example, "Abrazo, gravelly." This refers to a gravelly phase of the Abrazo series that is found in SCS soil map legends.

## Disturbed soil profiles

As a result of urbanization, the soil profile may be considerably altered and the listed group classification may no longer apply. In these circumstances, use the following to determine HSG according to the texture of the new surface soil, provided that significant compaction has not occurred (Brakensiek and Rawls 1983):

| *HSG* | *Soil textures* |
|---|---|
| A | Sand, loamy sand, or sandy loam |
| B | Silt loam or loam |
| C | Sandy clay loam |
| D | Clay loam, silty clay loam, sandy clay, silty clay, or clay |

## Drainage and group D soils

Some soils in the list are in group D because of a high water table that creates a drainage problem. Once these soils are effectively drained, they are placed in a different group. For example, Ackerman soil is classified as A/D. This indicates that the drained Ackerman soil is in group A and the undrained soil is in group D.

ODOT Hydraulics Manual Users: the Hydrologic Soil Groups for United States Soils listing is omitted from this publication. Use Hydrologic Soil Groups listed in Appendix A to "Oregon Engineering Handbook - Hydrology Guide."

# Appendix B: Synthetic rainfall distributions and rainfall data sources

The highest peak discharges from small watersheds in the United States are usually caused by intense, brief rainfalls that may occur as distinct events or as part of a longer storm. These intense rainstorms do not usually extend over a large area and intensities vary greatly. One common practice in rainfall-runoff analysis is to develop a synthetic rainfall distribution to use in lieu of actual storm events. This distribution includes maximum rainfall intensities for the selected design frequency arranged in a sequence that is critical for producing peak runoff.

## Synthetic rainfall distributions

The length of the most intense rainfall period contributing to the peak runoff rate is related to the time of concentration ($T_c$) for the watershed. In a hydrograph created with SCS procedures, the duration of rainfall that directly contributes to the peak is about 170 percent of the $T_c$. For example, the most intense 8.5-minute rainfall period would contribute to the peak discharge for a watershed with a $T_c$ of 5 minutes; the most intense 8.5-hour period would contribute to the peak for a watershed with a 5-hour $T_c$.

Different rainfall distributions can be developed for each of these watersheds to emphasize the critical rainfall duration for the peak discharges. However, to avoid the use of a different set of rainfall intensities for each drainage area size, a set of synthetic rainfall distributions having "nested" rainfall intensities was developed. The set "maximizes" the rainfall intensities by incorporating selected short duration intensities within those needed for longer durations at the same probability level.

For the size of the drainage areas for which SCS usually provides assistance, a storm period of 24 hours was chosen for the synthetic rainfall distributions. The 24-hour storm, while longer than that needed to determine peaks for these drainage areas, is appropriate for determining runoff volumes. Therefore, a single storm duration and associated synthetic rainfall distribution can be used to represent not only the peak discharges but also the runoff volumes for a range of drainage area sizes.

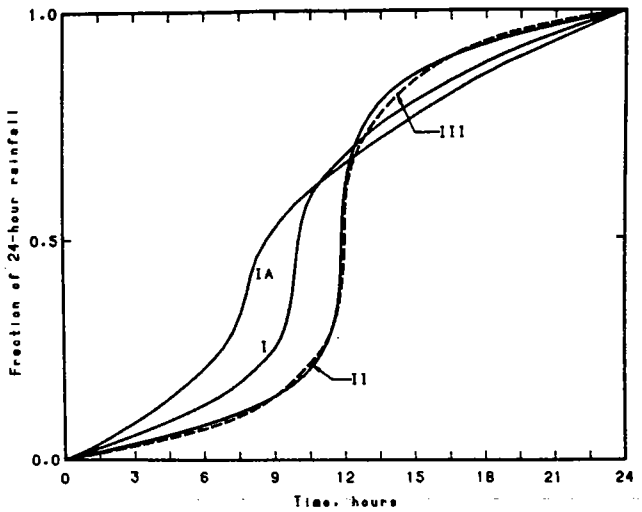



**Figure B-1.—SCS 24-hour rainfall distributions.**

The intensity of rainfall varies considerably during a storm as well as over geographic regions. To represent various regions of the United States, SCS developed four synthetic 24-hour rainfall distributions (I, IA, II, and III) from available National Weather Service (NWS) duration-frequency data (Hershfield 1961; Frederick et al., 1977) or local storm data. Type IA is the least intense and type II the most intense short duration rainfall. The four distributions are shown in figure B-1, and figure B-2 shows their approximate geographic boundaries.

Types I and IA represent the Pacific maritime climate with wet winters and dry summers. Type III represents Gulf of Mexico and Atlantic coastal areas where tropical storms bring large 24-hour rainfall amounts. Type II represents the rest of the country. For more precise distribution boundaries in a state having more than one type, contact the SCS State Conservation Engineer.

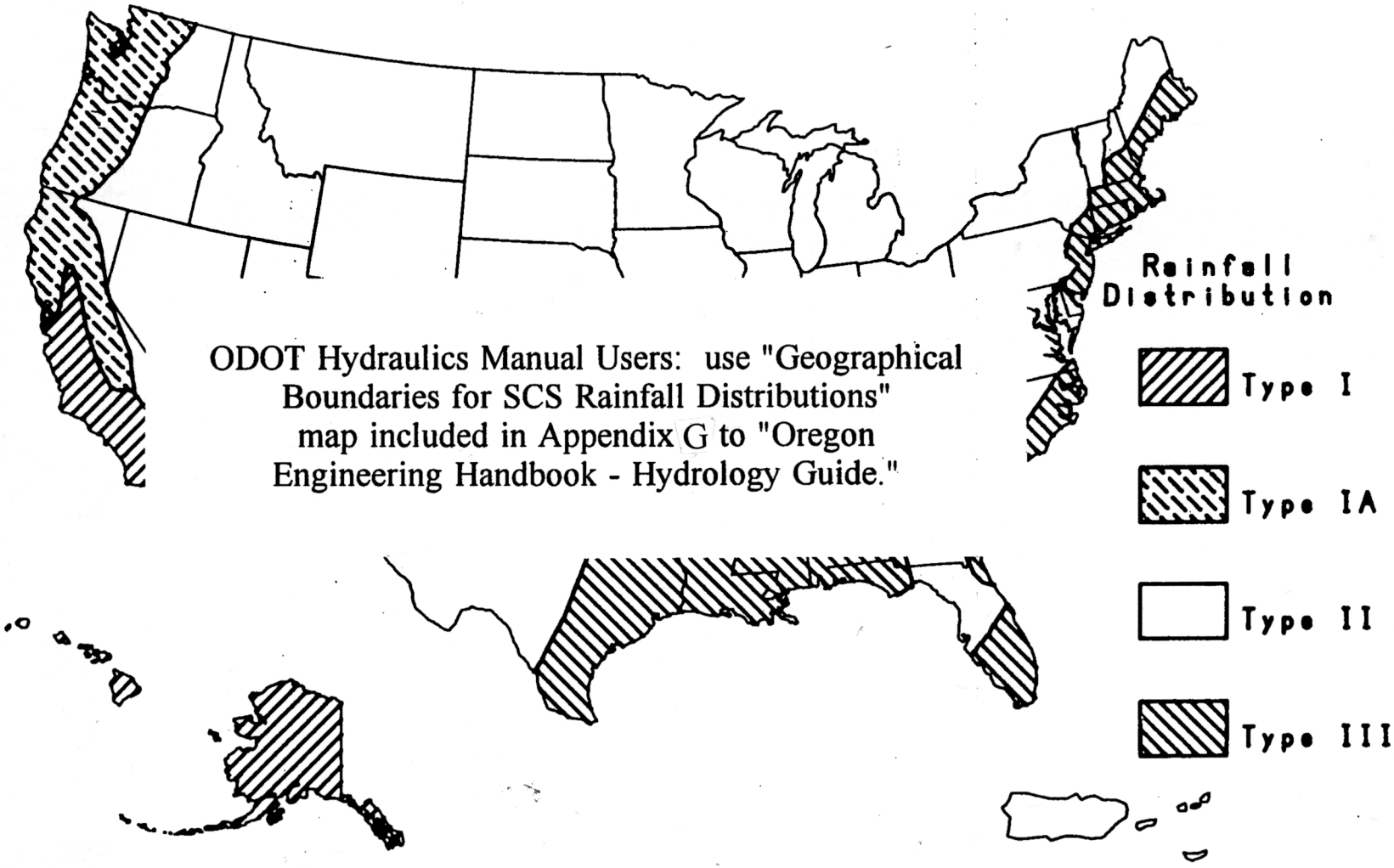



**Figure B-2.—Approximate geographic boundaries for SCS rainfall distributions.**

## Rainfall data sources

This section lists the most current 24-hour rainfall data published by the National Weather Service (NWS) for various parts of the country. Because NWS Technical Paper 40 (TP-40) is out of print, the 24-hour rainfall maps for areas east of the 105th meridian are included here as figures B-3 through B-8. For the area generally west of the 105th meridian, TP-40 has been superseded by NOAA Atlas 2, the Precipitation-Frequency Atlas of the Western United States, published by the National Oceanic and Atmospheric Administration.

### East of 105th meridian

Hershfield, D. M. 1961. Rainfall frequency atlas of the United States for durations from 30 minutes to 24 hours and return periods from 1 to 100 years. U.S. Dep. Commerce, Weather Bur. Tech. Pap. No. 40. Washington, DC. 115 p.

### West of 105th meridian

Miller, J.F., R.H. Frederick, and R.J. Tracey. 1973. Precipitation-frequency atlas of the Western United States. Vol. I, Montana; Vol. II, Wyoming; Vol. III, Colorado; Vol. IV, New Mexico; Vol. V, Idaho; Vol. VI, Utah; Vol. VII, Nevada; Vol. VIII, Arizona; Vol. IX, Washington; Vol. X, Oregon; Vol. XI, California. U.S. Dep. Commerce, National Weather Service, NOAA Atlas 2. Silver Spring, MD.

### Alaska

Miller, John F. 1963. Probable maximum precipitation and rainfall-frequency data for Alaska for areas to 400 square miles, durations to 24 hours and return periods from 1 to 100 years. U.S. Dep. Commerce, Weather Bur. Tech. Pap. No. 47. Washington, DC. 69 p.

### Hawaii

Weather Bureau. 1962. Rainfall-frequency atlas of the Hawaiian Islands for areas to 200 square miles, durations to 24 hours and return periods from 1 to 100 years. U.S. Dep. Commerce, Weather Bur. Tech. Pap. No. 43. Washington, DC. 60 p.

### Puerto Rico and Virgin Islands

Weather Bureau. 1961. Generalized estimates of probable maximum precipitation and rainfall-frequency data for Puerto Rico and Virgin Islands for areas to 400 square miles, durations to 24 hours, and return periods from 1 to 100 years. U.S. Dep. Commerce, Weather Bur. Tech. Pap. No. 42. Washington, DC. 94 p.

ODOT Hydraulics Manual Users:  the 24-hour rainfall maps
For the United States are omitted from this publication.
Use 24-hour precipitation maps in Appendix H to
Chapter 7 in the ODOT Hydraulics Manual

# Appendix C: Computer program

The TR-55 procedures have been incorporated in a computer program. The program, written in BASIC, requires less than 256K memory to operate and was developed for an MS-DOS operating system. Users of the program, however, still need to be familiar with the procedures in this TR. Features of the program include the following:

- The full screen (24 lines, 80 columns) is used to enter data. Flexibility of coding allows movement about the screen for quick data modifications.

- Function keys provide menu power to move to different modules (TR-55 chapters) within the program. Some keys are permanently defined while others vary by module.

- "Help" screens provide pertinent information to the user depending on location in the program. Two types of information are included: (1) define system operation and (2) describe input parameters.

- User files provide for optional entry of local data, such as rainfall-frequency, graphic peak discharge equation coefficients, and tabular hydrographs for other rainfall distributions.

Copies of the program can be obtained from—

National Technical Information Service
U.S. Department of Commerce
5285 Port Royal Road
Springfield, VA 22161
Telephone (703) 487-4650

# Appendix D: Worksheets

This appendix contains seven worksheets that can be reproduced for use with chapters 2 through 6. There is no worksheet for chapter 1.

| *Chapter* | *Worksheet* |
|---|---|
| 2 | 2 |
| 3 | 3 |
| 4 | 4 |
| 5 | 5a, 5b |
| 6 | 6a, 6b |

# Worksheet 2: Runoff curve number and runoff

Project _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed _____

1. Runoff curve number (CN)

| Soil name and hydrologic group (appendix A) | Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio) | CN 1/ Table 2-2 | CN 1/ Fig. 2-3 | CN 1/ Fig. 2-4 | Area ☐ acres ☐ $mi^2$ ☐ % | Product of CN x area |
|---|---|---|---|---|---|---|
| | | | | | | |
| | | | | | | |
| | | | | | | |
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| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | Totals = | | |

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}}$ = _____ = _____; Use CN = ☐

2. Runoff

| | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|
| Frequency ..... yr | | | |
| Rainfall, P (24-hour) ..... in | | | |
| Runoff, Q ..... in (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.) | | | |

# Worksheet 3: Time of concentration ($T_c$) or travel time ($T_t$)

Project _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed _____

Circle one: $T_c$ $T_t$ through subarea _____

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

| Sheet flow (Applicable to $T_c$ only) | | Segment ID | | |
|---|---|---|---|---|
| 1. Surface description (table 3-1) ..... | | | | |
| 2. Manning's roughness coeff., n (table 3-1) .. | | | | |
| 3. Flow length, L (total L $\leq$ 300 ft) ..... | ft | | | |
| 4. Two-yr 24-hr rainfall, $P_2$ ..... | in | | | |
| 5. Land slope, s ..... | ft/ft | | | |
| 6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute $T_t$ ..... | hr | | + | = |

| Shallow concentrated flow | | Segment ID | | |
|---|---|---|---|---|
| 7. Surface description (paved or unpaved) ..... | | | | |
| 8. Flow length, L ..... | ft | | | |
| 9. Watercourse slope, s ..... | ft/ft | | | |
| 10. Average velocity, V (figure 3-1) ..... | ft/s | | | |
| 11. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |

| Channel flow | | Segment ID | | |
|---|---|---|---|---|
| 12. Cross sectional flow area, a ..... | $ft^2$ | | | |
| 13. Wetted perimeter, $p_w$ ..... | ft | | | |
| 14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ..... | ft | | | |
| 15. Channel slope, s ..... | ft/ft | | | |
| 16. Manning's roughness coeff., n ..... | | | | |
| 17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ..... | ft/s | | | |
| 18. Flow length, L ..... | ft | | | |
| 19. $T_t = \frac{L}{3600\ V}$ Compute $T_t$ ..... | hr | | + | = |
| 20. Watershed or subarea $T_c$ or $T_t$ (add $T_t$ in steps 6, 11, and 19) ..... | hr | | | |

# Worksheet 4: Graphical Peak Discharge method

Project _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed _____

1. Data:

   Drainage area ..... $A_m$ = _____ $mi^2$ (acres/640)

   Runoff curve number .... CN = _____ (From worksheet 2)

   Time of concentration .. $T_c$ = _____ hr (From worksheet 3)

   Rainfall distribution type = _____ (I, IA, II, III)

   Pond and swamp areas spread throughout watershed ..... = _____ percent of $A_m$ (____ acres or $mi^2$ covered)

| | | Storm #1 | Storm #2 | Storm #3 |
|---|---|---|---|---|
| 2. Frequency ..... | yr | | | |
| 3. Rainfall, P (24-hour) ..... | in | | | |
| 4. Initial abstraction, $I_a$ ..... (Use CN with table 4-1.) | in | | | |
| 5. Compute $I_a/P$ ..... | | | | |
| 6. Unit peak discharge, $q_u$ ..... (Use $T_c$ and $I_a/P$ with exhibit 4-____) | csm/in | | | |
| 7. Runoff, Q ..... (From worksheet 2). | in | | | |
| 8. Pond and swamp adjustment factor, $F_p$ .... (Use percent pond and swamp area with table 4-2. Factor is 1.0 for zero percent pond and swamp area.) | | | | |
| 9. Peak discharge, $q_p$ ..... (Where $q_p = q_u A_m Q F_p$) | cfs | | | |

## Worksheet 5a: Basic watershed data

Project _____ Location _____ By _____ Date _____

Circle one: Present Developed _____ Frequency (yr) _____ Checked _____ Date _____

| Subarea name | Drainage area $A_m$ ($mi^2$) | Time of concentration $T_c$ (hr) | Travel time through subarea $T_t$ (hr) | Downstream subarea names | Travel time summation to outlet $\Sigma T_t$ (hr) | 24-hr Rainfall P (in) | Runoff curve number CN | Runoff Q (in) | $A_m Q$ ($mi^2$-in) | Initial abstraction $I_a$ (in) | $I_a/P$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | |
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↑ From worksheet 3 ↑ From worksheet 2 ↑ From table 5-1

# Worksheet 5b: Tabular hydrograph discharge summary

Project _____ Location _____ By _____ Date _____

Circle one: Present Developed _____ Frequency (yr) _____ Checked _____ Date _____

| Subarea name | Basic watershed data used 1/ | | | | Select and enter hydrograph times in hours from exhibit 5- 2/ | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Sub-area $T_c$ (hr) | $\Sigma T_t$ to outlet (hr) | $I_a/P$ | $A_m Q$ ($mi^2$-in) | Discharges at selected hydrograph times 3/ (cfs) | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
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| | | | | | | | | | | | | | | | | |
| Composite hydrograph at outlet | | | | | | | | | | | | | | | | |

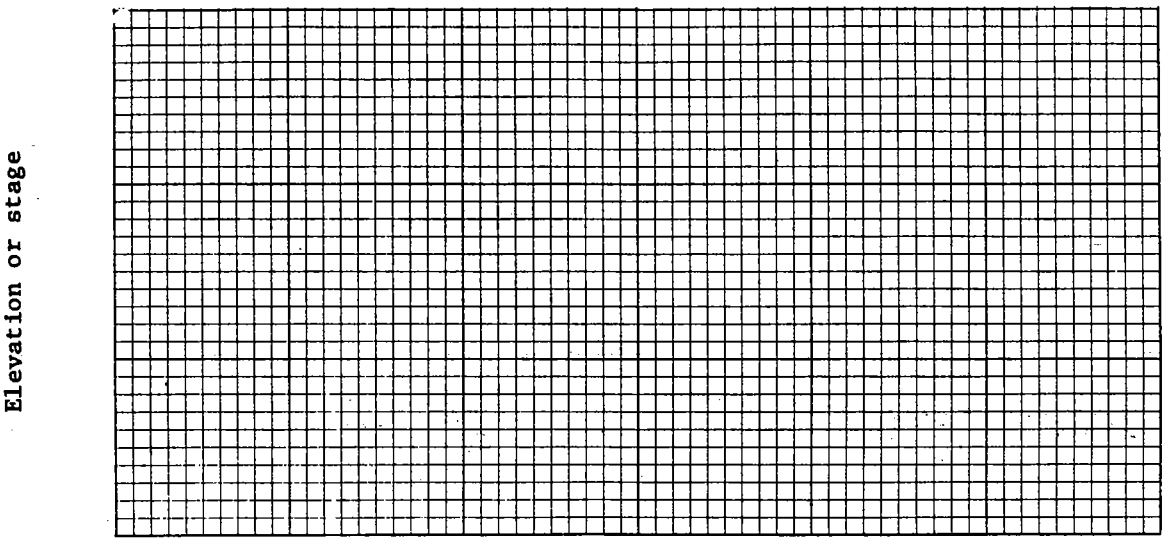
1/ Worksheet 5a. Rounded as needed for use with exhibit 5.
2/ Enter rainfall distribution type used.
3/ Hydrograph discharge for selected times is $A_m Q$ multiplied by tabular discharge from appropriate exhibit 5.

# Worksheet 6a: Detention basin storage, peak outflow discharge ($q_o$) known

Project _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed _____




Detention basin storage

1. Data:
   Drainage area ..... $A_m$ = _____ $mi^2$
   Rainfall distribution type (I, IA, II, III) = _____

| | 1st stage | 2nd stage |
|---|---|---|
| 2. Frequency ..... yr | | |
| 3. Peak inflow discharge, $q_i$ .... cfs (From worksheet 4 or 5b) | | |
| 4. Peak outflow discharge, $q_o$ .... cfs [1/] | | |
| 5. Compute $\frac{q_o}{q_i}$ ..... | | |
| 6. $\frac{V_s}{V_r}$ ..... (Use $\frac{q_o}{q_i}$ with figure 6-1) | | |
| 7. Runoff, Q ..... in (From worksheet 2) | | |
| 8. Runoff volume, $V_r$ ..... ac-ft ($V_r = QA_m 53.33$) | | |
| 9. Storage volume, $V_s$ ..... ac-ft ($V_s = V_r(\frac{V_s}{V_r})$) | | |
| 10. Maximum stage, $E_{max}$ (From plot) | | |

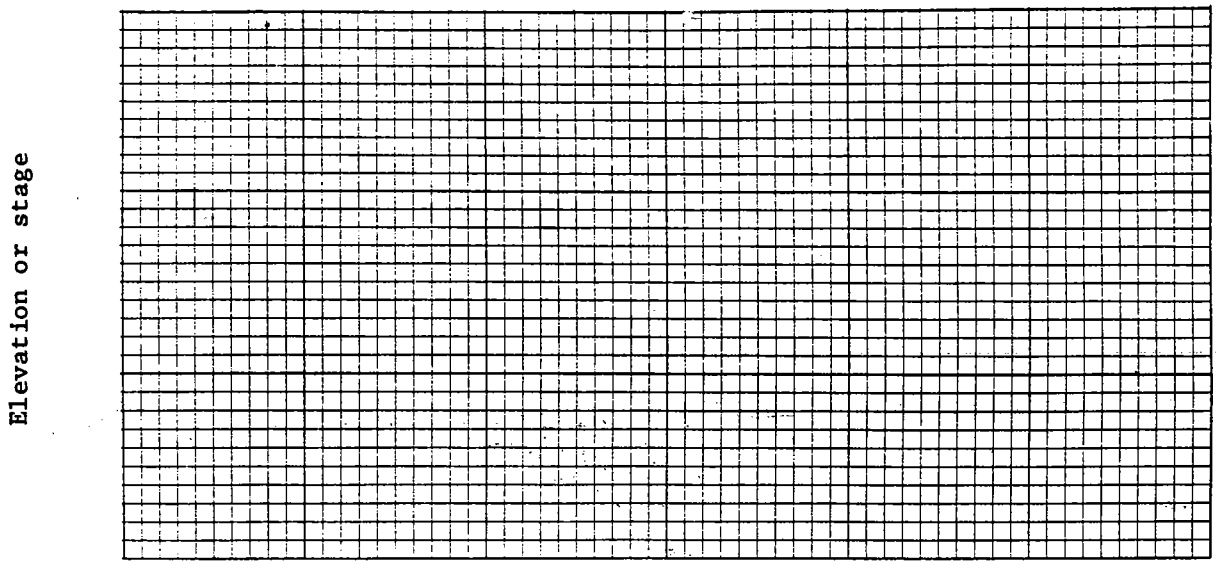
[1/] 2nd stage $q_o$ includes 1st stage $q_o$.

# Worksheet 6b: Detention basin peak outflow, storage volume ($V_s$) known

Project _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed _____




| | | 1st stage | 2nd stage |
|---|---|---|---|
| 1. | Data: Drainage area ..... $A_m$ = _____ $mi^2$ Rainfall distribution type (I, IA, II, III) = _____ | | |
| 2. | Frequency ..... yr | | |
| 3. | Storage volume, $V_s$ ..... ac ft | | |
| 4. | Runoff, Q ..... in (From worksheet 2) | | |
| 5. | Runoff volume, $V_r$ ..... ac-ft ($V_r = QA_m 53.33$) | | |
| 6. | Compute $\frac{V_s}{V_r}$ ..... | | |
| 7. | $\frac{q_o}{q_i}$ ..... in (Use $\frac{V_s}{V_r}$ and figure 6-1) | | |
| 8. | Peak inflow discharge, $q_i$ .... cfs (From worksheet 4 or 5b) | | |
| 9. | Peak outflow discharge, $q_o$ .... cfs ($q_o = q_i(\frac{q_o}{q_i})$) 1/ | | |
| 10. | Maximum stage, $E_{max}$ (From plot) | | |

1/ 2nd stage $q_o$ includes 1st stage $q_o$.

# Appendix E: References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__. 1985. National engineering handbook. Section 4—Hydrology. Washington, DC.

# Appendix F: Equations for figures and exhibits

This appendix presents the equations used in procedure applications to generate figures and exhibits in TR-55.

**Figure 2-1 (runoff equation):**

$$Q = \frac{\left[P - 0.2\left(\frac{1000}{CN} - 10\right)\right]^2}{P + 0.8\left(\frac{1000}{CN} - 10\right)}$$

where

Q = runoff (in),
P = rainfall (in), and
CN = runoff curve number.

**Figure 2-3 (composite CN with connected impervious area):**

$$CN_c = CN_p + (P_{imp}/100)(98 - CN_p)$$

where

$CN_c$ = composite runoff curve number,
$CN_p$ = pervious runoff curve number, and
$P_{imp}$ = percent imperviousness.

**Figure 2-4 (composite CN with unconnected impervious areas and total impervious area less than 30%):**

$$CN_c = CN_p + (P_{imp}/100)(98 - CN_p)(1 - 0.5R)$$

where R = ratio of unconnected impervious area to total impervious area.

**Figure 3-1 (average velocities for estimating travel time for shallow concentrated flow):**

Unpaved $V = 16.1345\ (s)^{0.5}$
Paved $V = 20.3282\ (s)^{0.5}$

where

V = average velocity (ft/s), and
s = slope of hydraulic grade line (watercourse slope, ft/ft).

These two equations are based on the solution of Manning's equation (Eq. 3-4) with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, ft). For unpaved areas, n is 0.05 and r is 0.4; for paved areas, n is 0.025 and r is 0.2.

**Exhibit 4 (unit peak discharges for SCS type I, IA, II, and III distributions):**

$$\log(q_u) = C_0 + C_1 \log(T_c) + C_2 [\log(T_c)]^2$$

where

$q_u$ = unit peak discharge (csm/in),
$T_c$ = time of concentration (hr) (minimum, 0.1; maximum, 10.0), and
$C_0$, $C_1$, $C_2$ = coefficients from table F-1.

**Figure 6-1 (approximate detention basin routing through single- and multiple-stage structures for 24-hour rainfalls of the indicated type):**

$$V_s/V_r = C_0 + C_1 (q_o/q_i) + C_2 (q_o/q_i)^2 + C_3 (q_o/q_i)^3$$

where

$V_s/V_r$ = ratio of storage volume ($V_s$) to runoff volume ($V_r$),
$q_o/q_i$ = ratio of peak outflow discharge ($q_o$) to peak inflow discharge ($q_i$), and
$C_0$, $C_1$, $C_2$, $C_3$ = coefficients from table F-2.

**Table F-1.—Coefficients for the equation used to generate exhibits 4-I through 4-III**

| Rainfall type | $I_a/P$ | $C_0$ | $C_1$ | $C_2$ |
|---|---|---|---|---|
| I | 0.10 | 2.30550 | -0.51429 | -0.11750 |
| | 0.20 | 2.23537 | -0.50387 | -0.08929 |
| | 0.25 | 2.18219 | -0.48488 | -0.06589 |
| | 0.30 | 2.10624 | -0.45695 | -0.02835 |
| | 0.35 | 2.00303 | -0.40769 | 0.01983 |
| | 0.40 | 1.87733 | -0.32274 | 0.05754 |
| | 0.45 | 1.76312 | -0.15644 | 0.00453 |
| | 0.50 | 1.67889 | -0.06930 | 0.0 |
| IA | 0.10 | 2.03250 | -0.31583 | -0.13748 |
| | 0.20 | 1.91978 | -0.28215 | -0.07020 |
| | 0.25 | 1.83842 | -0.25543 | -0.02597 |
| | 0.30 | 1.72657 | -0.19826 | 0.02633 |
| | 0.50 | 1.63417 | -0.09100 | 0.0 |
| II | 0.10 | 2.55323 | -0.61512 | -0.16403 |
| | 0.30 | 2.46532 | -0.62257 | -0.11657 |
| | 0.35 | 2.41896 | -0.61594 | -0.08820 |
| | 0.40 | 2.36409 | -0.59857 | -0.05621 |
| | 0.45 | 2.29238 | -0.57005 | -0.02281 |
| | 0.50 | 2.20282 | -0.51599 | -0.01259 |
| III | 0.10 | 2.47317 | -0.51848 | -0.17083 |
| | 0.30 | 2.39628 | -0.51202 | -0.13245 |
| | 0.35 | 2.35477 | -0.49735 | -0.11985 |
| | 0.40 | 2.30726 | -0.46541 | -0.11094 |
| | 0.45 | 2.24876 | -0.41314 | -0.11508 |
| | 0.50 | 2.17772 | -0.36803 | -0.09525 |

**Table F-2.—Coefficients for the equation used to generate figure 6-1**

| Rainfall distribution (appendix B) | $C_0$ | $C_1$ | $C_2$ | $C_3$ |
|---|---|---|---|---|
| I, IA | 0.660 | -1.76 | 1.96 | -0.730 |
| II, III | 0.682 | -1.43 | 1.64 | -0.804 |

*U.S. GOVERNMENT PRINTING OFFICE:1989-241-851:00249/SCS

# APPENDIX H - OREGON 24-HOUR PRECIPITATION MAPS

The 24-hour precipitation for the 2-year through 1000-year recurrence intervals in Oregon are provided in this appendix. This precipitation data can be viewed on maps in this appendix or using ESRI ArcReader.

A GIS project has been created for use to view Oregon's precipitation data in conjunction with other useful base map information. This is an ESRI ArcReader project and requires ArcReader 9.3.1 software. The following steps are provided to assist with viewing the precipitation data using ArcReader:

**Step 1 -** The required software is ArcReader 10. There is no cost for ArcReader. ODOT staff can contact the Help Desk to request the installation of this program. Others can download this software from the ESRI website:

http://www.esri.com/software/arcgis/arcreader/download.html

**Step 2 -** Open the precipitation project. ODOT staff can use the following link to open the precipitation project: \\sn-salemmill-1\GIS\PROJECTS\Precipitation_GIS. Others can download the raw GIS data from ODOT's ftp site:

ftp://ftp.odot.state.or.us/tdb/trandata/GIS_data/precipitation.zip

**Step 3 -** Select the "Identify" and the following popup window appears (Figure 1). Select the drop down arrow and select "Precip Data" (Figure 2)

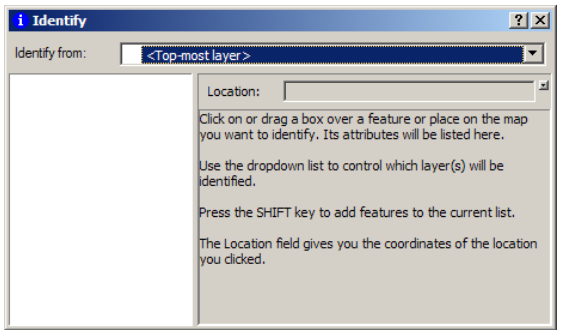



**Figure 1**

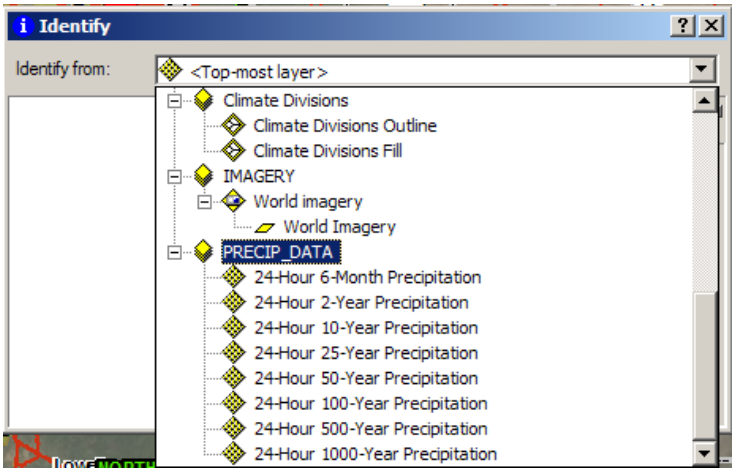



**Figure 2**

**Step 4 -** Locate and select the project site using the "Identify" button. Precipitation data for the selected point will be presented as shown in Figure 3.

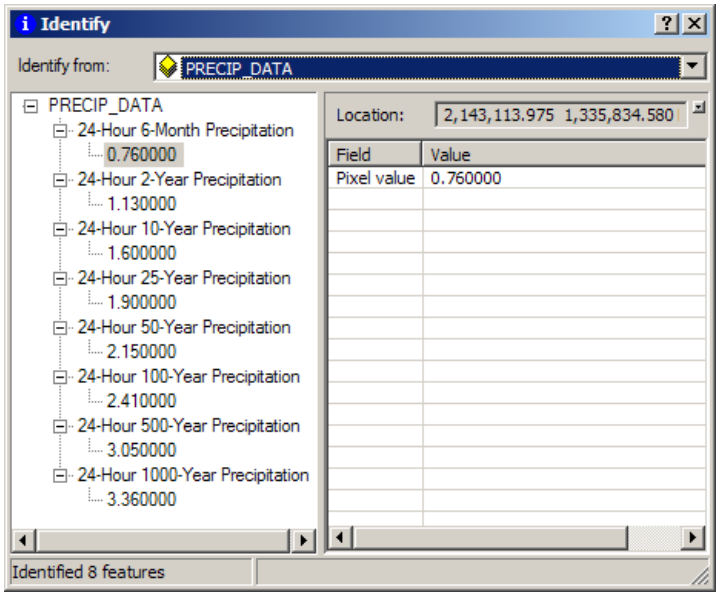
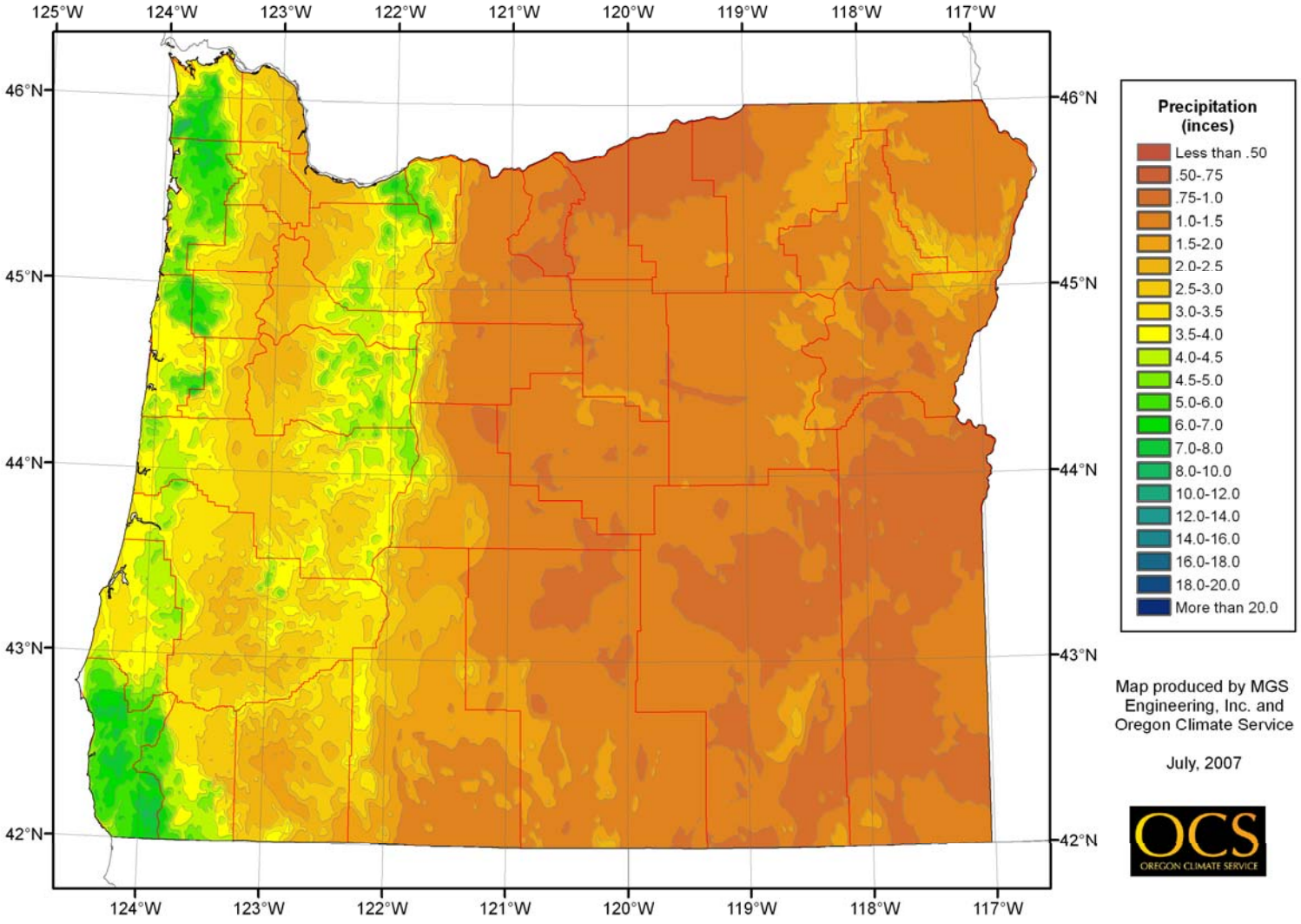



**Figure 3**

# 24-hour 2-year Precipitation, Oregon




Map produced by MGS Engineering, Inc. and Oregon Climate Service

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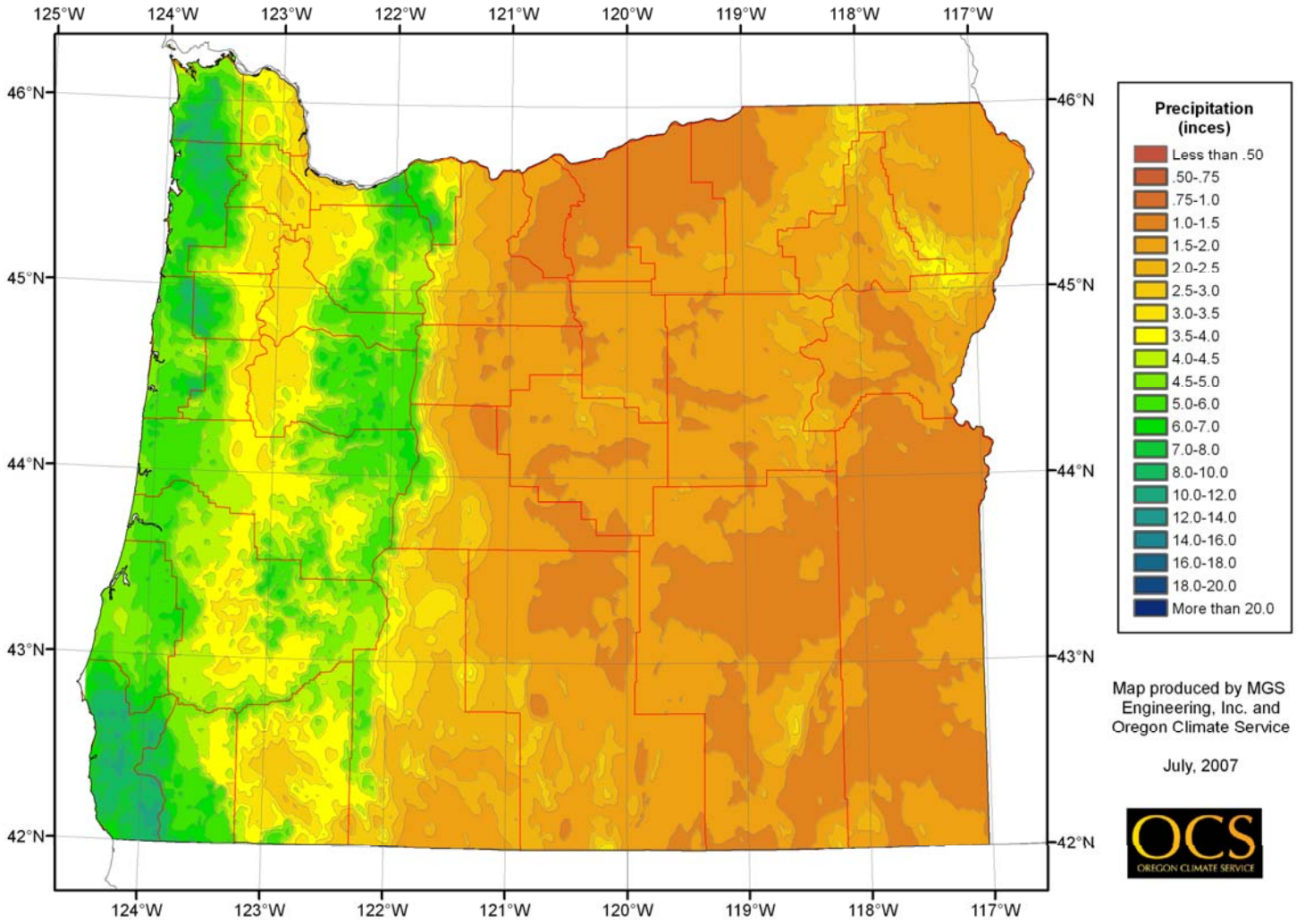
# 24-hour 10-year Precipitation, Oregon

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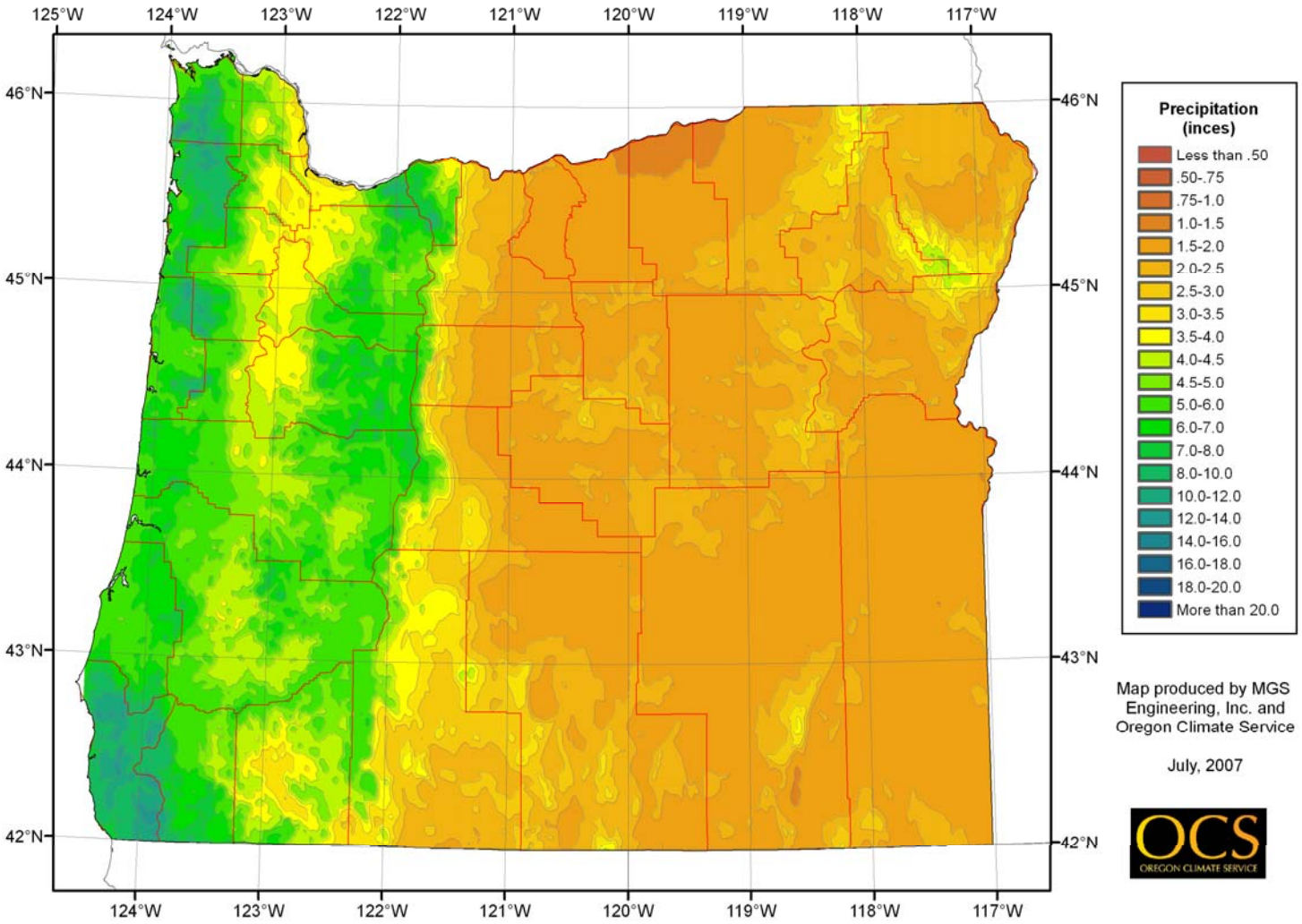
# 24-hour 25-year Precipitation, Oregon




Map produced by MGS Engineering, Inc. and Oregon Climate Service

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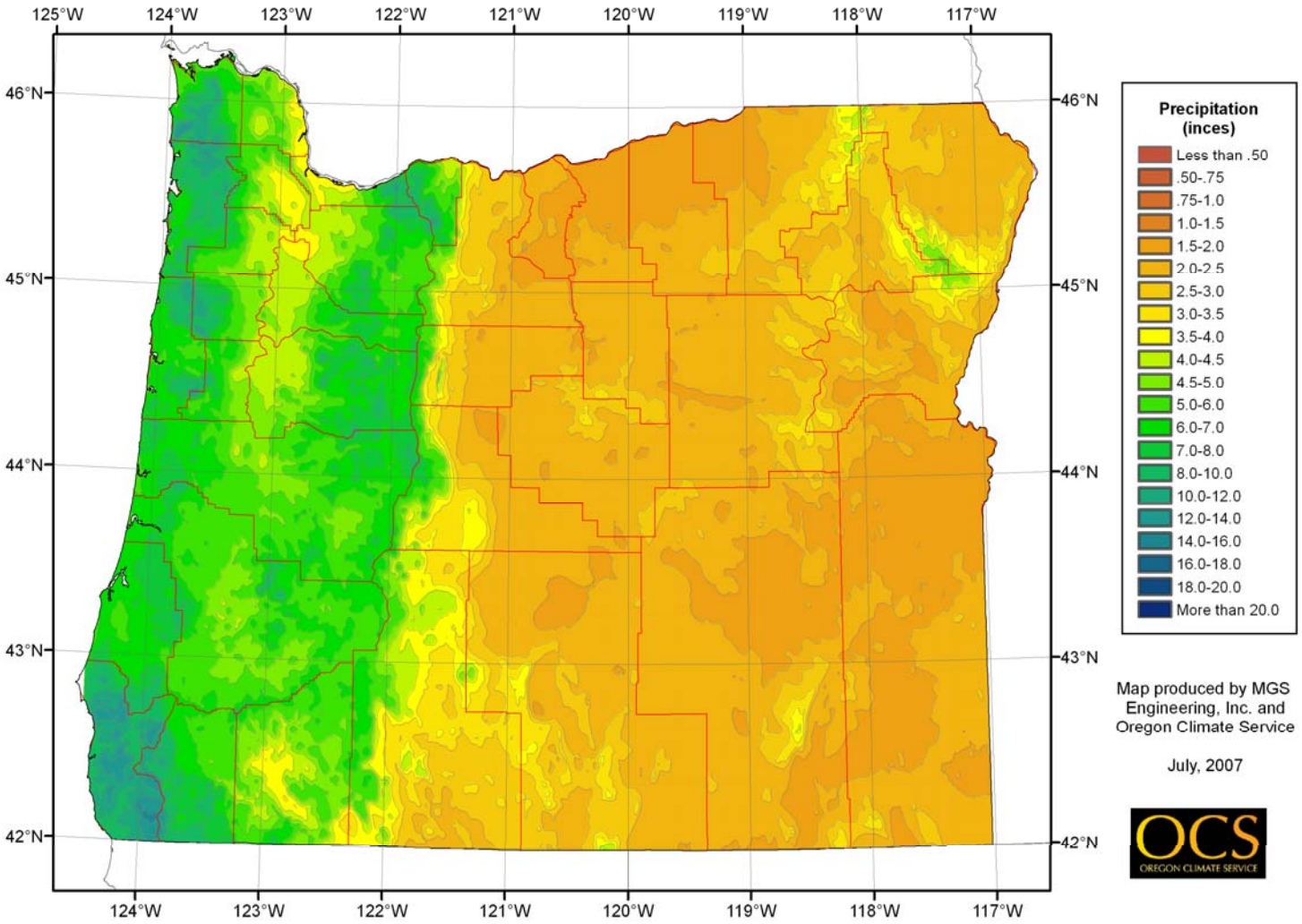
# 24-hour 50-year Precipitation, Oregon

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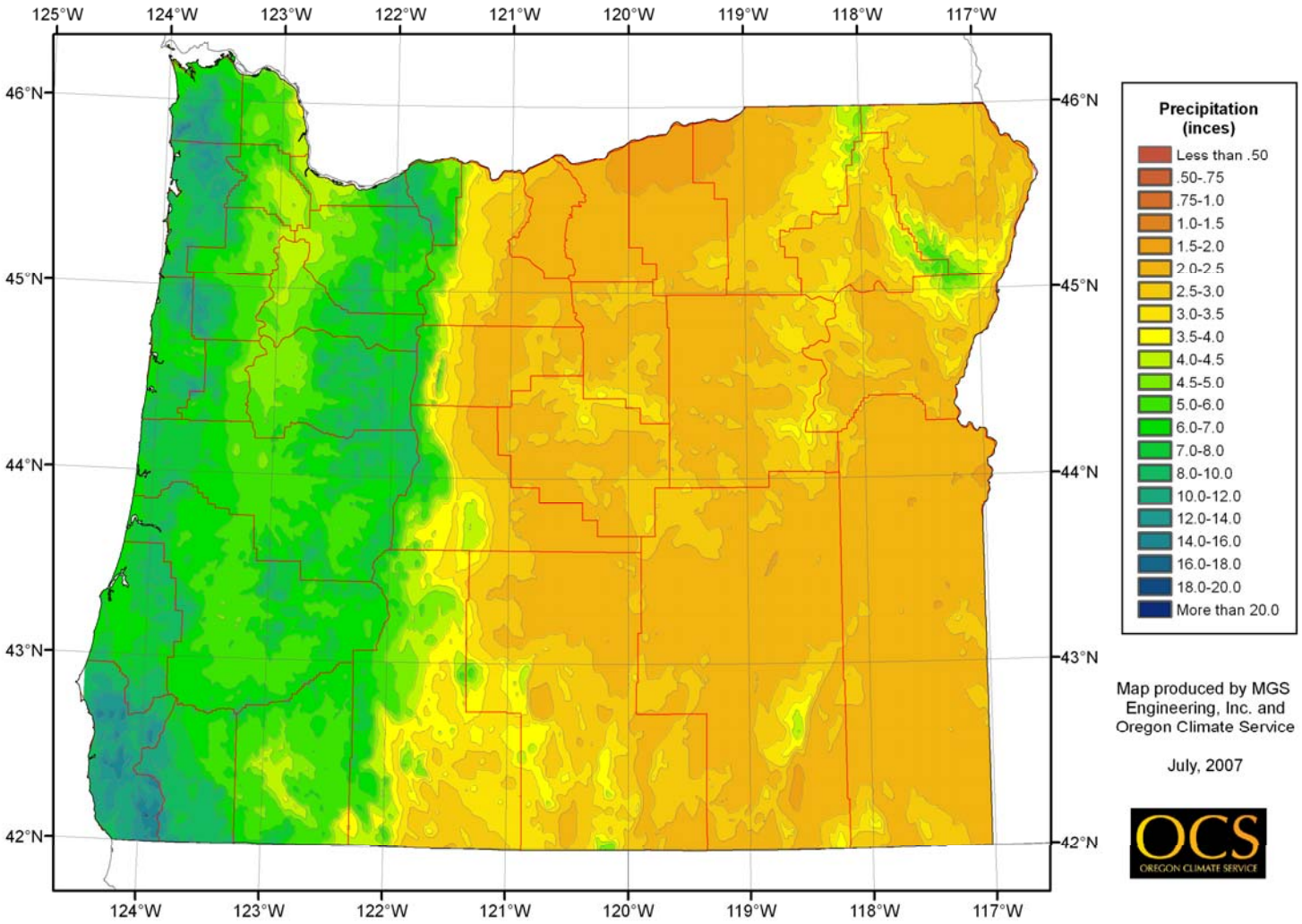
# 24-hour 100-year Precipitation, Oregon

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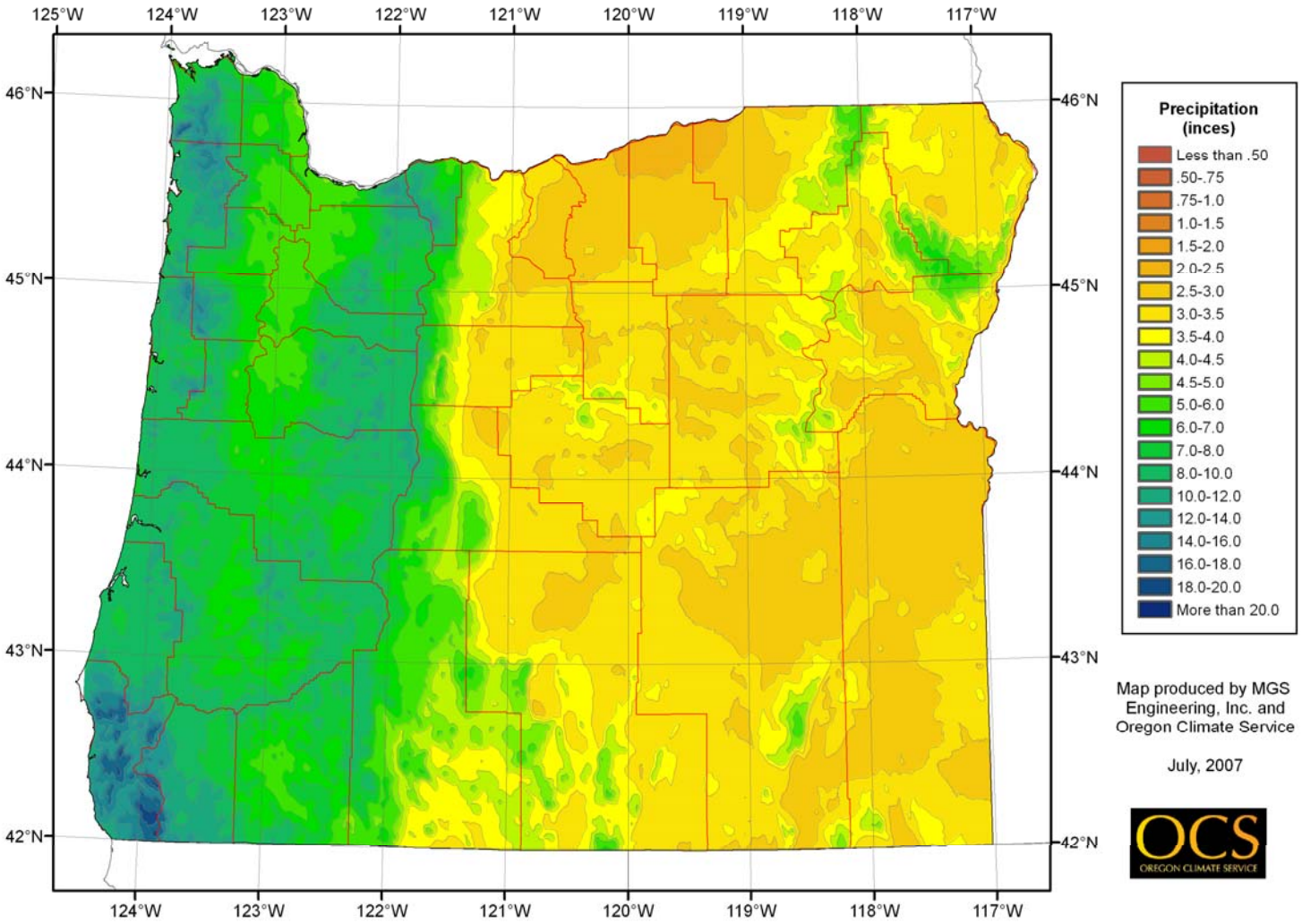
# 24-hour 500-year Precipitation, Oregon

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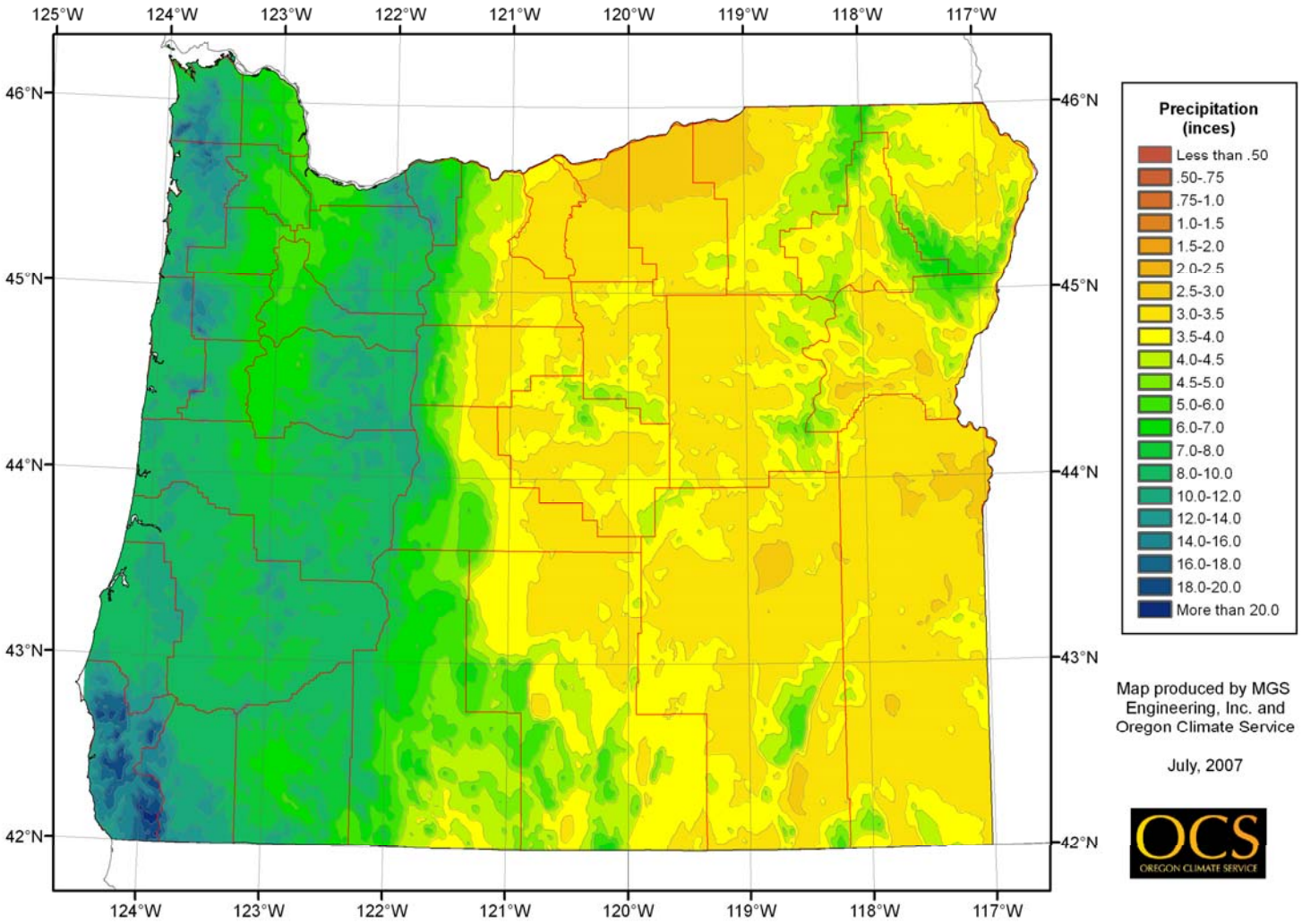
# 24-hour 1000-year Precipitation, Oregon

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# APPENDIX I - 2-YEAR, 7-CONSECUTIVE DAY LOW FLOWS

## 1.0 Introduction

The 2-year 7-consecutive day low flow discharge can be calculated by methods in this Appendix. This is minimum daily discharge expected for at least seven consecutive days during the dry season during an average year. This flow is often used as the low flow design discharge for fish passage design, as discussed in the "Oregon Department of Fish and Wildlife (ODFW) "Guidelines and Criteria for Stream-Road Crossings."

The 2-year 7-consecutive day low flow is one of two low flow design discharges mentioned in the ODFW Guidelines. The 95 percent exceedance flow is the other discharge for this purpose. These two discharges are different in an important aspect. The 2-year 7-consecutive day low flow represents discharges during the driest part of the year, and the 95 percent exceedance discharge can be calculated for any month of the year. As a consequence, exceedance discharges, rather than the 2-year 7-day low flow, are used for facilities designed to pass fish on ephemeral (intermittent) streams.

Two methods to calculate the 2-year 7-day low flow discharge are presented in this Appendix, as follows:

- gage data from nearby basins, and
- USGS regression equations.

Both methods calculate similar discharges at some locations. The two methods may also calculate different discharges at other locations, and it is necessary to determine which calculated discharge, if any, is appropriate and realistic. In addition, neither of the two methods considers consumptive use or water storage in the upstream watershed. Two additional procedures are included to aid the selection of the appropriate discharge and to provide information about upstream consumptive use and storage. They are:

- water use and low flow estimates based on OWRD data, and.
- parol evidence.

Experience has shown that neither of the two calculation methods provides realistic results in all locations. Both procedures should be used during each application.

## 2.0 Limitations of Predictions

Discharges throughout the dry season are extremely difficult to predict using statistics from nearby watersheds or regression equations. Dry season discharges can be strongly influenced by many factors, such as:

- climatic conditions such as rainfall, snowpack, and evaporation,
- the amount and type of vegetation in the riparian areas (vegetated riparian areas tend to sustain dry season flows),
- diversions to or from the subject stream,
- infiltration into exceptionally pervious rock formations or streambed materials,
- water entering the stream from springs and seeps,
- release or retention of streamflow in storage reservoirs, and
- consumptive uses such as livestock watering or irrigation.

The prediction methods can often overestimate dry season discharges in smaller watersheds. The statistical distributions used to develop the regression equations and the low flow statistics are based on gage data. Almost all of these gages are on larger streams that flow throughout the year. As a consequence, discharge estimates based on the equations and statistics often predict there will be discharge throughout the year. This may not be the case in many smaller watersheds.

The streams draining smaller basins may be dry through portions of the year for various reasons. In some instances, streamflow ceases during the winter when the upstream watershed freezes. In other cases, there is limited or no flow during portions of the dry season. Decreased summer flows can be caused by consumptive uses or diversions upstream, or lack of runoff or water percolating into the stream from the aquifer.

Information on water use and parol evidence is used to determine if streamflow ceases during portions of the year. This evidence supplements the calculation results.

## 3.0 Nearby Gage Data Method

Two-year 7-consecutive day low flow discharges have been calculated by the United States Geological Survey (USGS) for many gaged basins in Oregon. The information is presented in the 1993 USGS Open-File Report 93-63 titled "Statistical Summaries of Streamflow Data in Oregon: Volume 2 – Annual Low and High Flow, and Instantaneous Peak Flow." Data from this publication for western Oregon is summarized in Table A. Data for eastern Oregon is summarized in Table B.

Two-year 7-consecutive day low flow discharge information for gaged basins in Washington State near the Oregon border is in USGS Open-File Reports 84-145A and 84-145B. The two reports are Volumes 1 and 2 of the 1985 report titled: "Streamflow Statistics and Drainage-Basin Characteristics for the Southwestern and Eastern Regions, Washington." Volume 1 covers

southwestern Washington, and Volume 2 included the eastern half of the state.

Two-year 7-conecutive day low flow statistics for gaged basins in Idaho near the Oregon border are in the USGS Water Resources Investigations Report 94-4069 titled: "Statistical Summaries of Streamflow Data for Selected Gaging Stations in Idaho and Adjacent States Through September 1990 – Volume 1: Gaging Stations with 10 or More Years of Record. This report was published in 1996.

The listed discharges are valid for the gage locations during the periods of record. These periods are listed in the tables. The discharges will likely represent flows at the gage locations in the future if the upstream watershed characteristics are similar to conditions during the period of record. The listed discharges can also be used to predict flows from nearby basins with similar watershed characteristics.

Many watershed characteristics influence 2-year 7-consecutive day low flow. Several characteristics that significantly affect low flow were determined during the 1970 USGS study by David J. Lystrom titled "Evaluation of the Streamflow-Data Program in Oregon." This is the study that developed the regression equations. The watershed characteristics analyzed in the study that most significantly influenced 2-year 7-consecutive day low flow discharges were:

- Drainage Area (statewide),
- Storage (eastern Oregon),
- Mean Basin Elevation (western Oregon),
- Forest Cover (eastern Oregon),
- Annual Precipitation (statewide),
- Temperature Index (western Oregon), and
- Soils Index (eastern Oregon).

These characteristics are listed in Table A and Table B for gaged watersheds with low flow statistics. This will help determine if the nearby gaged basins resemble the basin to be analyzed. These characteristics are defined in more detail in Subsection 4.1.

## 3.1. Sites Near Gages

The 2-year 7-day low flow statistics from Table A or Table B can be used if the site to be analyzed is on the stream, near the gage, and the upstream watershed characteristics are similar to those during the period of record. The discharges should be adjusted considering water use and storage during the driest month. Water use and storage are listed in the OWRD Water Availability Report. As an example, the gage data period of record extends from 1948 through 1964. The 2-year 7-day low flow discharge listed in Table B is based on conditions during this period of record. A municipal water supply intake was built in 1974 and it uses water from the stream during the driest

month. This water use is expected to occur throughout the dry season with minimal fluctuation. The municipal consumptive use needs to be subtracted from the low flow discharge listed in Table B.

*Note: Intermittent or fluctuating consumptive uses through the dry season such as irrigation or domestic use would not be subtracted from the natural stream flow.*

### 3.2. Sites Away from Gages

Two-year 7-consecutive day low flow statistics from similar nearby watersheds can be used to estimate discharges in the subject watershed with the following procedure.

**Step 1 -** Determine if the watershed is in western or eastern Oregon using Plate 1. Determine the watershed characteristics listed in Section 3.0. These characteristics will also be used in the regression analysis. Procedures to calculate these characteristics are in Section 4.1.

**Step 2 -** Look in Tables A or B to find nearby watersheds with similar characteristics. Read the "Remarks." The Remarks will mention if discharges from the gaged watershed are significantly affected by storage or diversions. Low flow statistics from these gages should not be used for estimates at nearby sites. Note the nearby gaged basins with suitable characteristics, their 2-year 7-day low flows, and their drainage areas.

**Step 3 -** Calculate the 2-year 7-day low flow per square mile of drainage area for suitable nearby basins. This is the "yield" from each basin.

**Step 4 -** Plot the yield determined in Step 3 for each basin versus its drainage area. Estimate the relationship between yield and drainage area for the nearby basins using a line or curve.

**Step 5 -** Estimate the yield from the subject watershed based on the line or curve drawn in Step 4.

**Step 6 -** Calculate the 2-year 7-day low flow by multiplying the estimated yield from Step 5 by the subject basin watershed area determined in Step 1, as follows:

2-year 7-day discharge in cubic feet per second = (2-year 7-day yield in cubic feet per second per square mile) x (basin area in square miles)

**Step 7 -** Consider consumptive uses or storage, as discussed in Section 5.0 of this Appendix. Adjust discharges as needed to account for consumptive uses or storage.

## 3.3. Example of Nearby Gage Data Method

Miner Creek is a small coastal stream in southwest Oregon near Coos Bay. The 2-year 7-consecutive day low flow will be calculated from nearby gage data.

**Step 1 -** The basin is in western Oregon, based on Plate 1. The watershed characteristics are determined by the methods for the regression equations. It has the following characteristics:

- Drainage Area = 1.1 square miles
- Mean Basin Elevation = 216 feet
- Mean Annual Precipitation = 60 inches
- Temperature Index = 39 degrees Fahrenheit

**Step 2 -** Nearby watersheds are identified in Table A, as follows. 2-year 7-day low flows are listed for the gages selected for the remainder of the procedure.

**Gage 14323200 Tenmile Creek near Lakeside**, drainage area = 87 square miles, mean basin elevation = 502 feet, mean annual precipitation = 80 inches, and temperature index = 36 degrees Fahrenheit. Remarks: "Flow affected by natural storage in Tenmile Lake and other lakes …"

**Gage 14324500 West Fork Millicoma River near Allegany**, drainage area = 46.9 square miles, mean basin elevation = 1,220 feet, mean annual precipitation = 91 inches, and temperature index = 34.0 degrees Fahrenheit. Remarks: "No regulation. Only minor diversions upstream from station." 2-year 7-day low flow = 3.9 cubic feet per second

**Gage 14326800 North Fork Coquille River near Fairview,** drainage area = 73.9 square miles, mean basin elevation = 800 feet, mean annual precipitation = 75 inches, and temperature index = 34 degrees Fahrenheit. Remarks: "No regulation. Several diversions for irrigation upstream from station."

**Gage 14324600 South Fork Coquille River above Panther Creek near Illahe,** drainage area = 31.2 square miles, mean basin elevation = 2,860 feet, mean annual precipitation = 85 inches, temperature index = 34 degrees Fahrenheit. Remarks: "No regulation or diversion upstream from station." 2-year 7-day low flow = 1.6 cubic feet per second

**Gage 14234700 South Fork Coquille River near Illahe,** drainage area = 40.6 square miles, mean basin elevation = 2,780 feet, mean annual precipitation = 90 inches, temperature index = 34 degrees. Remarks: "No regulation or diversion upstream from station." 2-year 7-day low flow = 2.3 cubic feet per second

**Gage 14324900 South Fork Coquille River near Powers,** drainage area = 93.2 square miles, mean basin elevation = 2,420 feet, mean annual precipitation = 92 inches,

temperature index = 35 degrees Fahrenheit. Remarks: "No regulation or diversion upstream from station." 2-year 7-day low flow = 13 cubic feet per second.

Not all of the gages provide suitable data to predict discharges at Miner Creek. Gage 14323200 is near the outlet of a chain of large lakes. Gage 14326800 is downstream from irrigation diversions. This may significantly influence the low flow discharges, and they may not represent the Miner Creek Basin. Data from these gages are not used. 2-year 7-day low flow data from the remaining four gages will be used.

*Note: 2-year 7-day low flow statistics for Gages 14323200 and 12326800 in Table A can be used for sites at or near these gages, only.*

**Step 3 -** The 2-year 7-day low flow yield is calculated for the nearby basins as follows:

**Gage 14324500,** $\frac{\text{2 - year 7 - day low flow}}{\text{drainage area}} = \frac{3.9}{46.9} = 0.083$ cubic feet per second per sq mile

**Gage 14324600,** $\frac{1.6}{31.2} = 0.051$ cubic feet per second per square mile

**Gage 14324700,** $\frac{2.3}{40.6} = 0.057$ cubic feet per second per square mile

**Gage 14324900,** $\frac{13}{93.2} = 0.013$ cubic feet per second per square mile

**Step 4 -** The basin yields versus their respective drainage areas are plotted. A straight line linear interpolation is used to determine the 2-year 7-day low flow relationship for the nearby basins, as shown in Figure 1. Use of a straight line interpolation program is recommended, rather than "by hand and eye" manual interpolation estimates.

**Step 5 -** The straight line interpolation equation is shown on the printout. Miner Creek basin yield is calculated using the straight line interpolation equation as follows:

x = Miner Creek basin area = 1.1 square miles

2-year 7-day low flow yield = 0.0014x + 0.0063 = [(0.0014)(1.1)] + 0.0063 = 0.0078 cubic feet per second per square mile

**Step 6 -** Miner Creek 2-year 7-day low flow is calculated as follows:

2-year 7-day low flow = 0.0078 x 1.1 = 0.0086 cubic feet per second.

*Note: The low flow estimate is based on an extrapolation of data from much larger basins. There is potential for considerable error when a prediction is based on an extrapolation of this type. This should be considered when comparing the results of*

*different methods prior to the final discharge estimate.*

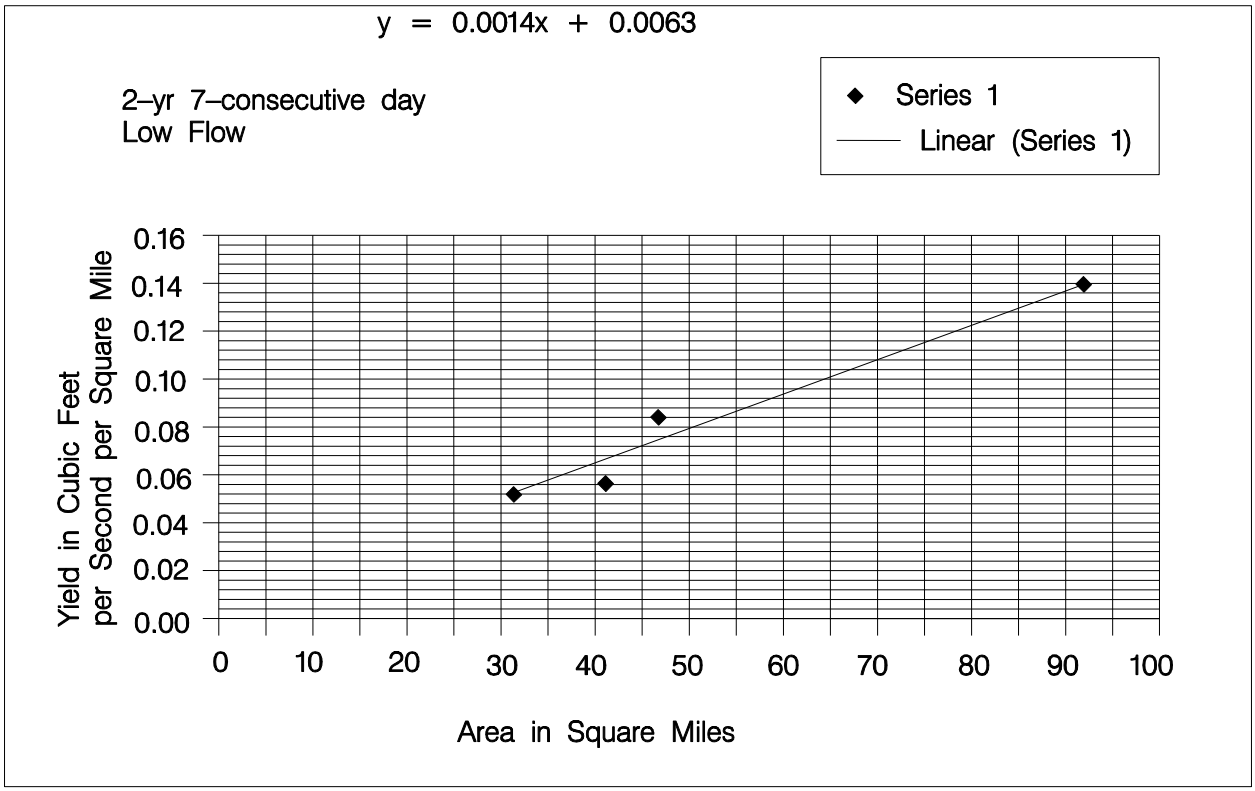


**Figure 1 2-Year 7-Consecutive Day Low Flow Relationship for Basins near Miner Creek**

**Step 7 -** The nearby gages used in the analysis are on streams with little or no consumptive uses or upstream storage. Consumptive uses and storage need to be considered. An OWRD Water Availability Report is available for a location on Miner Creek near the highway crossing. See Section 5.0 of this appendix. It lists a consumptive use of 0.01 cubic feet per second during the driest months. No discharge is reserved for streamflow from this consumptive use. The 2-year 7-day low flow discharge considering consumptive use is:

2-year 7-day low flow considering consumptive use = 0.0086 – 0.01 = -0.0014 cubic feet per second.

*Note: A negative discharge value indicates there will be no stream flow.*

These uses are for domestic consumption and irrigation. They may fluctuate throughout the dry season. As a result, using the nearby gage method, the natural discharge of 0.01 cubic feet per second would be listed as the 2-year 7-consecutive day low flow. Notes

will be included that state the creek may be dry for at least seven consecutive days due to upstream water use.

*Note: This is the result of the nearby gage method. The regression equation method will also be used, and the answers compared to parol evidence.*

## 4.0 USGS Regression Equations

The United States Geological Survey (USGS) has developed low flow regression equations for Oregon. The equations with a detailed description of their derivation and use are in the 1970 USGS Open-File Report by David J. Lystrom titled "Evaluation of the Streamflow-Data Program in Oregon."

These equations are valid for natural streams with basin characteristics similar to typical streams in the area with little or no flow diversion into or from the upstream basin. The equations and a description of the input variables follow.

### 4.1. Equations and Input Variables

There are regression equations for western and eastern Oregon, and the boundary between the two regions is approximately at the crest of the Cascade Mountains. The two regions are shown in Plate 1. The equation for western Oregon is:

$$Q_{2\text{-yr, }7\text{-day}} = 2.82 \times 10^{3} (A^{1.124}) (E^{0.645}) (P^{1.255}) (T^{-4.716}) \quad \text{(Equation 1)}$$

The standard error of estimate is 65 percent.

The equation for eastern Oregon is:

$$Q_{2\text{-yr, }7\text{-day}} = 2.50 \times 10^{-4} (A^{0.745}) (St^{-2.315}) (F^{-2.685}) (P^{5.117}) (S_i^{1.586}) \quad \text{(Equation 2)}$$

The standard error of estimate is 182 percent.

Where:

$Q_{2\text{-yr, }7\text{-day}}$ = Annual minimum 7-day mean daily low flow with a 2-year (average annual) recurrence interval in cubic feet per second (cfs)

A = Drainage area in square miles ($mi^2$). Usually the drainage area is delineated on a USGS 7-1/2 minute quadrangle map and measured with a planimeter.

St = Area of lakes and ponds expressed as a percentage of drainage area plus 1 percent, as shown on the most recent quadrangle maps. For example, drainage basins with lakes and ponds covering 0 percent and 5 percent of their surface area would have St values of 1 and 6, respectively.

E = Mean basin elevation above sea level in feet, determined from a quadrangle map of a practical scale by laying a grid over the map, recording the elevation at each grid intersection, and averaging these elevations. The grid spacing should be selected to give at least 25 intersections within the basin boundary.

F = Forest cover expressed as a percentage of the drainage area covered by forests plus 1 percent, as shown on the most recent quadrangle map. As an example, drainage basins with 0 percent and 66 percent of their surface area covered by forest would have F values of 1 and 67, respectively.

P = Mean annual precipitation, as shown on Plate 2 of this Appendix, in inches.

T = Temperature index, T, is the mean minimum January air temperature as shown on Plate 3 of this Appendix, in degrees Fahrenheit.

$S_i$ = Soil index. The first step in determining the soil index is to calculate the runoff curve number "CN" using methods in NRCS publications Technical Release No. 55 “Urban Hydrology of Small Watersheds" and "Oregon Engineering Handbook - Hydrology Guide." Copies of these publications are included in Appendix G of this chapter. The second step is to calculate the soil index as follows:

$$S_i = \frac{1000}{CN} - 10 \qquad \text{(Equation 3)}$$

Example:

The runoff curve number (CN) was determined from data in the Malheur County Soil Survey using NRCS methods. CN = 63. The Soil Index ($S_i$) was calculated using Equation 3 as follows:

$$S_i = \frac{1000}{63} - 10 = 59$$

## 4.2. Procedure

The regression equation procedure is to determine the input variables, enter the variables into the equation, and calculate the 2-year, 7-consecutive day low flow. The calculated discharge is adjusted to account for consumptive uses and storage, if needed. Example calculations for the Miner Creek basin follow.

## 4.3. Example of USGS Regression Equation Method

The 2-year 7-consecutive day low flow is to be calculated for Miner Creek using the USGS regression equations.

**Step 1 –** The basin is in western Oregon based on Plate 1. The western Oregon regression equation input variables are determined. The drainage area (A) is delineated on a 7.5-minute USGS quadrangle map and it is measured with a planimeter.

A = 1.1 square miles

The mean basin elevation is determined by drawing a grid across the drainage basin outlined on the USGS quadrangle map. There are 32 data points within the basin. The average elevation is 216 feet.

E = (430 + 440 + 360 + 355 + 355 + 400 + 280 + 360 + 245 + 160 + 210 + 320 + 205 + 155 + 285 + 300 + 50 + 110 + 200 + 215 + 220 + 40 + 165 + 70 + 155 + 205 + 85 + 80 + 30 + 130 + 205 + 85) / 32) = 216 feet

E = 216 feet

Mean Annual Precipitation (P) is determined from Plate 2.

P = 60 inches

The Temperature index is determined from Plate 3.

T = 39 degrees Fahrenheit

**Step 2 -** The input variables determined in the first step are entered into the regression equation and the equation is solved. Using Equation 1, the western Oregon regression equation, the 2-year 7-consecutive day low flow is:

$$Q_{2\text{-yr, 7-day}} = 2.82 \times 10^{3} (1.1^{1.124}) (216^{0.645}) (60^{1.255}) (39^{-4.716}) = 0.54 \text{ cfs}$$

The lowest discharge within the standard error of prediction is:

$$0.54 - \left[\left(\frac{65}{100}\right)(0.54)\right] = 0.19 \text{ cubic feet per second}$$

The highest discharge within the standard error of prediction is:

$$0.54 + \left[\left(\frac{65}{100}\right)(0.54)\right] = 0.89 \text{ cubic feet per second}$$

Consumptive use and storage is considered. These uses are based on OWRD information and parol evidence. See Section 5.0 of this Appendix. An OWRD Water Availability Report is available for a location on Miner Creek near the highway crossing. It lists a consumptive use of 0.01 cubic feet per second during the driest month. This use is for domestic consumption and irrigation. No discharge is reserved for streamflow from this use. The 2-year 7-day low flow discharge considering consumptive use is:

2-year 7-day low flow considering consumptive use = 0.54 – 0.01 = 0.53 cubic feet per second.

This upstream consumptive use may fluctuate throughout the dry season. As a result, using the regression equation method, the Miner Creek discharge would be reported as the 0.54 cubic foot per second natural discharge. Notes accompanying this discharge state it could drop to 0.53 cubic feet per second due to upstream consumptive use.

## 5.0 Discharge and Water Use Estimates Based on OWRD Data

The Oregon Water Resources Department (OWRD) calculates water availability, including discharges, at selected locations throughout the state. These discharges are posted on their website: http://www.wrd.state.or.us The OWRD water availability study is described in the August 2002 Technical Report by Richard M. Cooper, P.E. titled “Determining Surface Water Availability in Oregon.” This report is available from the OWRD.

The water availability is calculated at selected sites for the monthly 50 and 80 percent mean daily exceedance discharges. These sites are at the outlets of Water Availability Basins. The 80 percent exceedance discharge is the mean daily flow expected to be exceeded during the subject month, at least 80 percent of the time, on a daily basis. The 80 percent exceedance flow can also be described as the mean daily discharge expected to be exceeded approximately 24 days in a typical month. The 50 percent exceedance flow is larger than the 80 percent exceedance discharge. It is the discharge expected to be exceeded at least 15 days each month.

The 80 percent discharge for the driest month of the year is the more similar of the two exceedance flows to the 2-year 7-consecutive day low flow. It is not identical to the 2-year 7-day daily low flow. It provides a similar discharge to compare to the flows calculated by

methods in Sections 3.0 and 4.0 of this appendix. This will aid the selection of the appropriate calculated discharge.

The OWRD printouts provide data about expected discharges and upstream water use for each month at the outlet of the water availability basin. These discharges are, as follows.

- The "Natural Stream Flow" is the estimated discharge that would occur from the watershed in natural conditions without consumptive uses, diversions, or storage. It is an estimate of "prehistoric" discharges.

- The "Expected Stream Flow" is the calculated discharge if consumptive uses are fully realized. The expected streamflow may be less than the discharge currently occurring because the upstream water rights are not fully utilized.

- The "Reserved Stream Flow" is the discharge reserved from-out-of-stream consumptive uses and storage. It is often reserved for in-stream uses such as navigation, power generation, scenic qualities, or aquatic habitat.

- The "Instream Water Rights" are a summation of the discharge allotted to upstream water users based on existing water rights.

- The "Net Water Available" is the discharge at the outlet of the water availability basin if all upstream users exercise their water rights. A positive value of Net Water Available indicates there would be discharge in the stream, approximately 24 days out of the month, during an average year. A negative value indicates there is insufficient water in the stream to fulfill all upstream water rights during the preceding conditions.

Often the basin used in the OWRD analyses is a different size than the basin upstream from the project site. The 80 percent exceedance discharges listed in the OWRD report can be adjusted to represent the project site as follows:

$$Q_{(adjusted)} = Q_{(in\ report)} \left( \frac{Area_{(project)}}{Area_{(in\ report)}} \right)^{1.02} \qquad \text{(Equation 4)}$$

Where:

$Q_{(adjusted)}$ = 80 percent exceedance discharge at project site in cubic feet per second,
$Q_{(in\ report)}$ = 80 percent exceedance in OWRD report in cubic feet per second,
$Area_{(project)}$ = Drainage area upstream from project in square miles, and
$Area_{(in\ report)}$ = Drainage area in OWRD report in square miles.

*Note: The preceding method is applicable where the watershed upstream from the project site are on the same stream and they both have similar watershed characteristics such as land use and elevation.*

Use of OWRD data to estimate low flow is shown in the following example.

## 5.1. Example of Use of OWRD Estimates

The Miner Creek Basin is selected on the interactive mapping section of the OWRD website. The outlet of the OWRD water availability basin is at the creek's confluence with the Pacific Ocean. This is approximately 500 feet downstream from the highway crossing. The consumptive uses and storage in the report are assumed to represent uses and storage in the watershed upstream from the highway crossing. Three reports are available:

- Peak Discharges for Selected Frequencies,
- Water Availability Table (50 percent exceedance), and
- Water Availability Table (80 percent exceedance).

The basin characteristics are listed in the Peak Discharge report. The characteristics for Miner Creek are shown in Figure 2.

**SELECTED UNGAGED WATERSHED CHARACTERISTICS**

| | | |
|---|---|---|
| Drainage area | (square miles) | 0.67 |
| Mean watershed elevation | (feet) | 169.00 |
| Mean watershed slope | (percent) | 4.36 |
| Precipitation intensity | (inches/hour) | 2.89 |
| January precipitation | (inches) | 10.20 |
| Minimum January Temperatures | (degrees F) | 38.40 |
| Maximum January Temperatures | (degrees F) | 52.80 |
| Depth to Bedrock | (inches) | 55.20 |

**Figure 2 Miner Creek Basin Characteristics From OWRD Website**

The 80 percent exceedance discharges listed in the Water Availability Table for Miner Creek are shown in Figure 3. The months of the year are listed in the first column, and the natural stream flows are listed in the second column. The driest months of the year are September and October

with a discharge of 0.03 cubic feet per second.

The Miner Creek drainage basin upstream from the highway crossing is 1.1 square miles, based on the USGS quadrangle map. The Miner Creek basin used in the OWRD analyses is 0.67 square miles. The natural stream flows in the OWRD report are adjusted to account for the different drainage areas using Equation 4 as follows:

$$Q_{(adjusted)} = 0.03\left(\frac{1.1}{0.67}\right)^{1.03} = 0.05 \text{ cubic feet per second}$$

The OWRD report lists no effects on discharge due to upstream storage. A 0.01 cubic foot per second loss is predicted during the drier months due to consumptive use. The 80 percent exceedance discharge adjusted to account for consumptive use is:

80 percent exceedance discharge with consumptive use = 0.05 – 0.01 = 0.04 cubic feet per second
The 80 percent exceedance discharge, based on OWRD estimates, will fluctuate between 0.05 and 0.04 cubic feet per second, depending on consumptive use.

DETAILED REPORT ON THE WATER AVAILABILITY CALCULATION
Water Availability as of 11/03/2003 for
MINER CR > PACIFIC OCEAN – AT MOUTH

Watershed ID #:72506 Basin: SOUTH COAST Exceedance Level: 80
Time: 12:29 Date: 11/03/2003

| Month | Natural Stream Flow | CU + Stor Prior to 1/1/93 | CU + Stor After 1/1/93 | Expected Stream Flow | Reserved Stream Flow | Instream Water Rights | New Water Available |
|---|---|---|---|---|---|---|---|
| 1 | 1.37 | 0.00 | 0.00 | 1.37 | 0.00 | 3.00 | -1.63 |
| 2 | 1.89 | 0.00 | 0.00 | 1.89 | 0.00 | 3.29 | -1.40 |
| 3 | 1.39 | 0.00 | 0.00 | 1.39 | 0.00 | 2.35 | -0.96 |
| 4 | 0.88 | 0.00 | 0.00 | 0.88 | 0.00 | 1.43 | -0.55 |
| 5 | 0.43 | 0.01 | 0.00 | 0.42 | 0.00 | 0.62 | -0.20 |
| 6 | 0.20 | 0.01 | 0.00 | 0.19 | 0.00 | 0.43 | -0.24 |
| 7 | 0.09 | 0.02 | 0.00 | 0.07 | 0.00 | 0.18 | -0.11 |
| 8 | 0.04 | 0.01 | 0.00 | 0.03 | 0.00 | 0.08 | -0.05 |
| 9 | 0.03 | 0.01 | 0.00 | 0.02 | 0.00 | 0.04 | -0.02 |
| 10 | 0.03 | 0.00 | 0.00 | 0.03 | 0.00 | 0.04 | -0.01 |
| 11 | 0.13 | 0.00 | 0.00 | 0.13 | 0.00 | 0.42 | -0.29 |
| 12 | 0.94 | 0.00 | 0.00 | 0.94 | 0.00 | 2.37 | -1.43 |
| Stor | 862 | 5 | 0 | 857 | 0 | 854 | 5 |

(Discharges for driest months are included in box.)

**Figure 3 Miner Creek 80 Percent Average Daily Exceedance Discharges for Each Month**

## 6.0 Parol Evidence

Calculations provide discharge quantities, but they often do not accurately describe when, why, and how often water flows in intermittent or ephemeral streams, ditches, or canals. Parol evidence is also needed to estimate low flow discharges. It often accurately describes when water is present, the sources of water, and how often flow occurs. Parol evidence can also be the best source for information about irrigation flows and other regulated discharges. Parol evidence is obtained by interviewing people familiar with the waterway, such as:

- ODOT maintenance personnel,
- local watermasters,
- Oregon Department of Fish and Wildlife personnel, and
- local residents.

Typical questions to ask are:

- What are the lowest summer flows during a typical year?
- What is the lowest flow that will occur for a week or so during a typical year?
- What flows through the site during the summer? Rainfall runoff? Irrigation flow? Springwater?
- Is there more or less water flowing through this site than other typical creeks with similar watershed sizes in the area?
- Are there upstream diversions into or out of the creek?

Parol evidence should be recorded along with information such as the contact's address, phone number, etc. It is good practice to obtain information from several sources. Parol evidence is often supported by correspondence, photographs, and in some cases, survey marks. This supporting evidence should be copied or retained as needed.

Parol evidence along with OWRD data should be used to determine if water is reserved during the dry season for in-stream uses such as navigation, scenic qualities, power generation, or fish habitat. Water is often reserved during the dry season for dilution of pollutants downstream from wastewater treatment facilities. This should be investigated when parol data is collected.

## 6.1. Example of Parol Evidence Method

People familiar with Miner Creek are asked about its summer flow characteristics. They state the creek does not normally dry up through the summer. In addition, they describe flows of less than one cubic feet per second when asked about the amount of low flow discharge during the summer.

## 7.0 Determining 2-Year 7-Consecutive Day Low Flow Discharge

The calculation methods sometimes produce similar results. In this case it is necessary to verify the results are reasonable. Often the two calculation methods provide different results. In these instances it is necessary to verify which result is realistic and reasonable. This is done during this step in the hydrology process. It is accomplished by comparing the results of the two calculation methods against each other, and against the estimate based on OWRD data and the parole evidence.

### 7.1. Example of Determining 2-Year 7-Consecutive Day Low Flow Discharge

The 2-year 7-day low flow discharges for Miner Creek based on the two calculation procedures are summarized as follows:

**Nearby gage method:** 0.01 cubic feet per second without consumptive use
No flow with consumptive use

**Regression equation method:** 0.54 cubic feet per second without consumptive use
0.53 cubic feet per second with consumptive use

The similar 80 percent exceedance flow based on OWRD data is:

**Estimate based on OWRD data:** 0.05 cubic feet per second without consumptive use
0.04 cubic feet per second with consumptive use

**Parol evidence method:** Typical discharge less than 1 cubic foot per second at highway crossing, no dry periods.

The results of all the methods do not contradict observed discharges. Selection of discharge value would have to consider the purpose for which the answer would be used.

## 8.0 Reporting Discharge Estimates

Two-year 7-consecutive day low flow estimates are reported in narrative. These discharges are usually reported to two significant figures. Further guidance about rounding figures is in **Chapter 4**.

The first item mentioned in the narrative is the natural discharge from the watershed. This discharge is adjusted to account for diversions into and out of the watershed and consumptive

uses if they will be occurring throughout the dry season and they are not intermittent.
In some instances there is discharge reserved from consumption or diversion for in-stream uses such as navigation, power generation, scenic qualities, or aquatic habitat. The narrative should mention this. The narrative should also mention whether or not the calculated discharges are lower than the flow reserved for in-stream uses. It is not uncommon for this to occur in Oregon streams during the dry season.

The narrative should mention intermittent changes to the 2-year 7-consecutive day low flow due to storage regulation, diversions, or consumption. Estimates of these changes should be noted.

Parol evidence should be mentioned if it is needed to supplement the calculated discharges.

## 8.1. Example of Reporting Discharge Estimates

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The natural discharge is reported as the 2-year 7-consecutive day low flow for Miner Creek. It is not adjusted to account for consumptive use because these uses are intermittent. It is reported as follows:

"The 2-year 7-consecutive day low flow for an average year, during the driest period, is estimated to be 0.05 cubic feet per second."

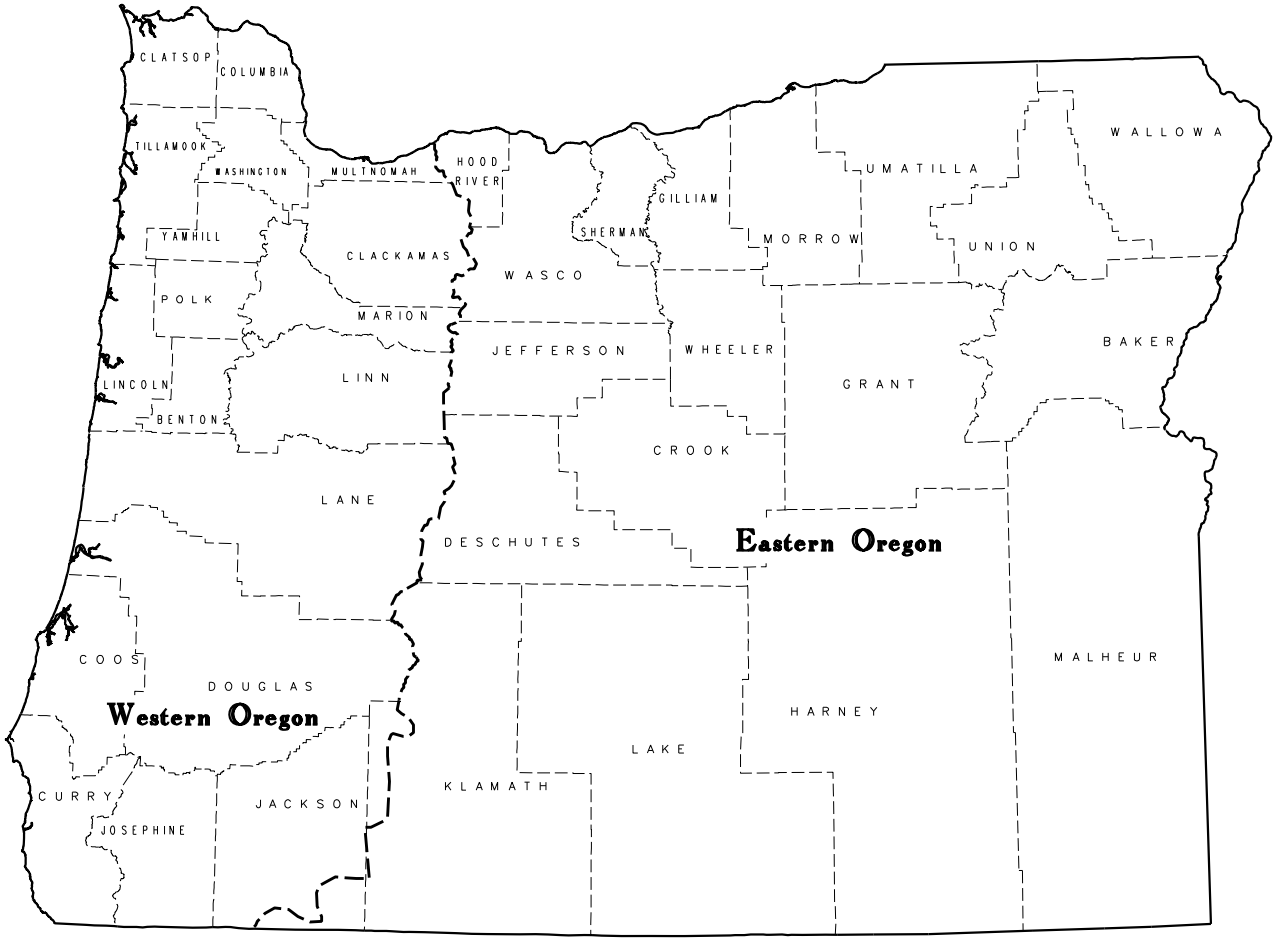
The consumptive uses may reduce this discharge. The following sentences are added to mention this:

"Consumptive uses up to 0.01 cubic feet per second are permitted in the upstream watershed. As a result, 2-year 7-consecutive day low flows are estimated to fluctuate between 0.05 and 0.04 cubic feet per second during the typical year."

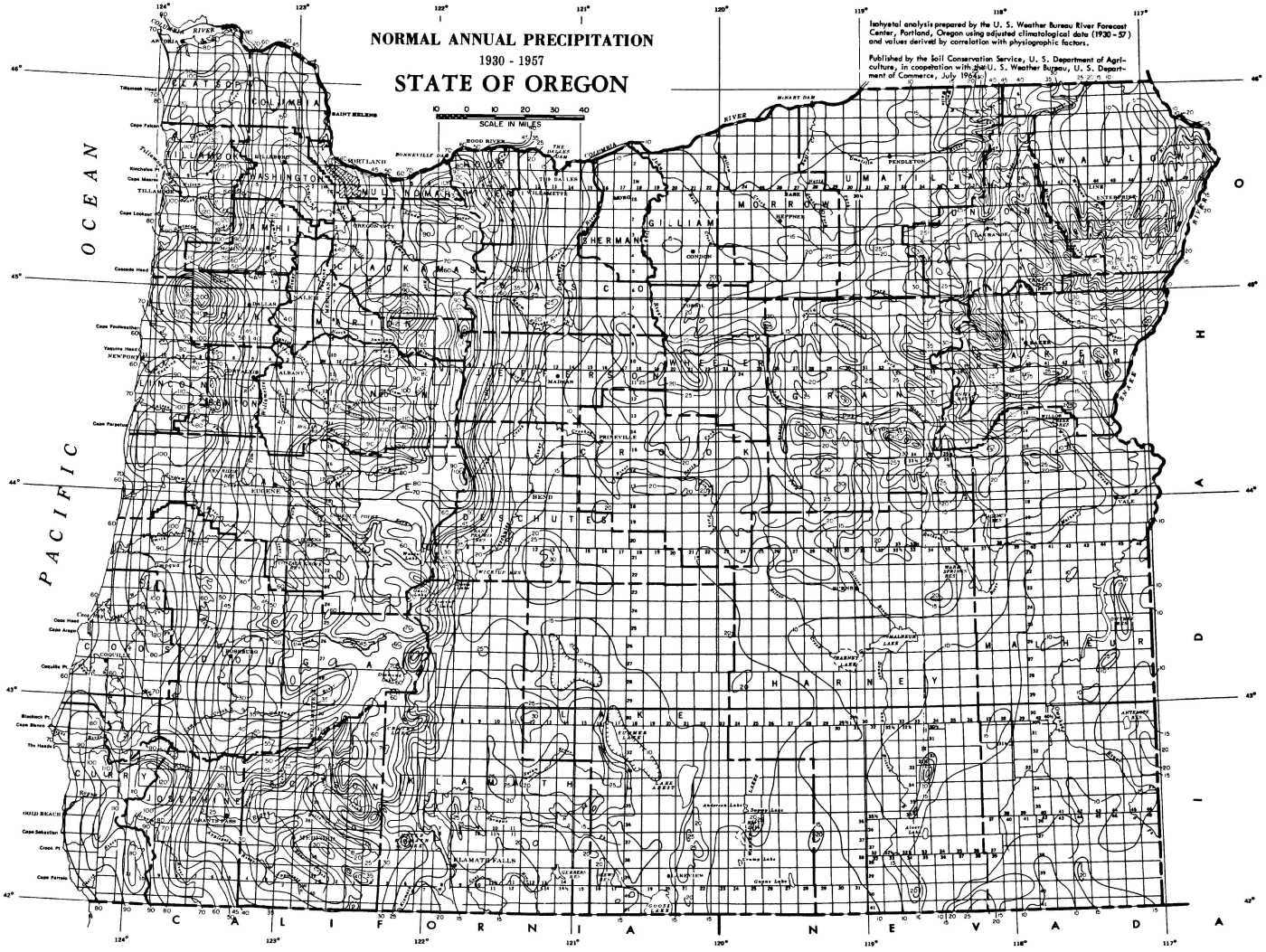
Parol evidence indicates discharge is almost always present. This is mentioned as follows:

"People familiar with the waterway recall that it does not normally dry up during the summer."

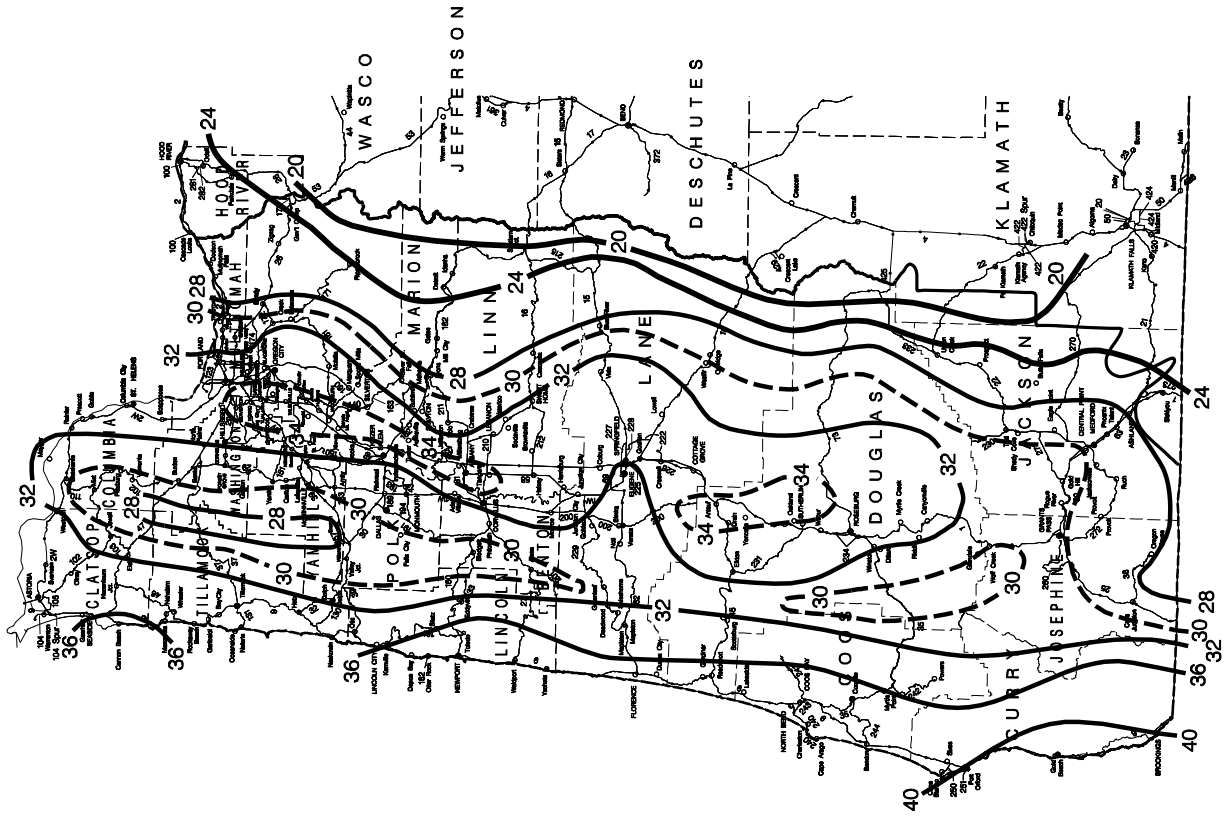
# PLATE 1: BOUNDARY BETWEEN WESTERN AND EASTERN OREGON



# PLATE 2: NORMAL ANNUAL PRECIPITATION 1930 – 1957 STATE OF OREGON



# PLATE 3: MEAN MINIMUM TEMPERATURE (°F.), JANUARY IN WESTERN OREGON




Isolines are drawn through points of approximately equal value. Caution should be used in interpolating on these maps, particulary in mountainous areas.

Based on period 1931 - 1952

# TABLE A: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR WESTERN OREGON

REVISED March 1, 2004

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 = Annual Low and High Flow, and Instantaneous Peak Flow" and USGS Open File Report 79-553 "Magnitude and Frequency of Floods in Western Oregon.")

**(See end pages for gages in adjacent states near the Oregon Border)**

| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (Note 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | MEAN BASIN ELEVATION IN FEET | MEAN ANNUAL PRECIPITATION IN INCHES | TEMPERATURE INDEX IN DEGREES F | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| SANDY RIVER BASIN | | | | | | | | | | | |
| 14134000 | Salmon River near Government Camp, OR | 45°15'55" | 121°43'00" | 61 | 1912-1987 | 8 | 4800 | 88 | 22.5 | 18 | No regulation or diversion upstream from station. |
| 14134500 | Salmon River below Linney Creek, OR | 45°13'20" | 121°51'40" | 22 | 1929-1950- | 54 | 4000 | 85 | 22 | 53 | No diversion or regulation upstream from station. |
| 14135000 | Salmon River at Welches, OR | 45°19'10" | 121°57'10" | 11 | 1926-1936 | 100 | 3610 | 80 | 23 | 72 | No diversion or regulation upstream from station. |
| 14135500 | Salmon River above Boulder Creek, near Brightwood, OR | 45°21'40" | 122°00'40" | 15 | 1938-1952 | 106 | | | | 83 | No regulation or diversion upstream from station. |
| 14137000 | Sandy River near Marmot, OR | 45°23'30" | 122°07'40" | 71 | 1913-1987 | 262 | 3350 | 85 | 26 | 306 | No regulation or diversion upstream from station. |
| 14138800 | Blazed Alder Creek near Rhododendron, OR | 45°27'10" | 121°53'25" | 23 | 1965-1987 | 8.17 | 3300 | 120 | 28 | 2.5 | No diversion or regulation upstream from station. |
| 14138850 | Bull Run River near Multnomah Falls, OR | 45°29'50" | 122°00'50" | 20 | 1968-1987 | 47.9 | 3070 | 125 | 29 | 48 | Regulation at times since 1915 by Bull Run Lake, usable capacity, 12,270 acre-ft., from topographic map. |
| 14138870 | Fir Creek near Brightwood, OR | 45°28'56" | 122°01'36" | 11 | 1977-1987 | 5.46 | | | | 2.7 | No diversion or regulation upstream from station. |
| 14138900 | North Fork Bull Run River near Multnomah Falls, OR | 45°29'40" | 122°02'05" | 21 | 1967-1987 | 8.32 | | | | 14 | Regulation at times since 1958 by North Fork Reservoir, capacity, about 1,030 acre-ft. No diversion upstream from station. |
| 14139500 | Bull Run River below Lake Ben Morrow, OR | 45°29'00" | 122°04'50" | 24 | 1931-1954 | 74 | | | | 80 | Discharge determined by combining discharge through valves near base of dam and discharge over crest of spillway (elevation 1,036 ft.). Leakage at dam is less than 1–$ft^3/s$ and is disregarded. Flow regulated by Bull Run Lake and Lake Ben Morrow (since 1928): flow from Bull Run Lake is not artificially regulated but reaches river through surface and underground channels. |
| 14139700 | Cedar Creek near Brightwood, OR | 45°27'30" | 122°01'50" | 21 | 1967-1987 | 7.93 | | | | 10 | No regulation or diversion upstream from station. |
| 14139800 | South Fork Bull Run River near Bull Run, OR | 45°26'38" | 122°06'20" | 12 | 1976-1987 | 15.4 | | | | 13 | No regulation or diversion upstream from station. |
| 14141500 | Little Sandy River near Bull Run, OR | 45°24'55" | 122°10'20" | 67 | 1921-1987 | 22.3 | 2440 | 96 | 29 | 15 | No regulation or diversion upstream from station. |
| 14142500 | Sandy River below Bull Run River, near Bull Run, OR | 45°26'57" | 122°14'38" | 30 | 1932-1961 | 436 | | | | 360 | Flow regulated since 1915 by Bull Run Lake, since 1929 by Bull Run Reservoir Number One (station 14139000), and since 1961 by Bull Run Reservoir Number Two (station 14139900). Some fluctuation caused by Bull Run power plant of Portland General Electric Company. |
| WILLAMETTE RIVER BASIN | | | | | | | | | | | |
| 14144800 | Middle Fork Willamette River near Oakridge, OR | 43°35'50" | 122°27'20" | 28 | 1960-1987 | 258 | 4380 | 60 | 24 | 245 | No regulation or diversion upstream from station. |
| 14144900 | Hills Creek above Hills Creek Lake, near Oakridge, OR | 43°40'50" | 122°22'10" | 22 | 1960-1981 | 52.7 | 4090 | 58 | 24 | 20 | No regulation or diversion upstream from station. |
| 14145500 | Middle Fork Willamette River above Salt Creek, near Oakridge, OR | 43°43'20" | 122°26'15" | 24 | 1937-1960 | 392 | 4080 | 58 | 24 | 282 | Flow regulated since 1961 by Hills Creek Lake (station 141451000). No diversion upstream from station. |
| 14145500 | Middle Fork Willamette River above Salt Creek, near Oakridge, OR | 43°43'20" | 122°26'15" | 25 | 1963-1987 | 392 | 4080 | 58 | 24 | 211 | |
| 14146000 | Salt Creek near Oakridge, OR | 43°43'45" | 122°25'35" | 17 | 1935-1951 | 113 | 4460 | 60 | 21 | 102 | No regulation. Since spring of 1948 there has been a small intermittent, unmeasured diversion around gage to millpond downstream. |
| 14146500 | Salmon Creek near Oakridge, OR | 43°54'45" | 122°22'18"' | 56 | 1915-1985 | 117 | 4140 | 62 | 24 | 126 | No regulation or diversion upstream from station. |
| 14147000 | Waldo Lake Outlet near Oakridge, OR | 43°46'05" | 122°03'10" | 27 | 1938-1982 | 30.5 | | | | 0.4 | At times seiches from Waldo Lake cause rapid changes in stage at gage many times each hour. No regulation. Diversion tunnel into head of Black Creek, near south end of lake, built about 1914, is sealed off, but there was leakage of 0.51-$ft^3/s$ past control gates, measured Oct. 1, 1981 |

Note 1 Number of values used to compute statistics.

# TABLE A: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR WESTERN OREGON

REVISED March 1, 2004

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 = Annual Low and High Flow, and Instantaneous Peak Flow" and USGS Open File Report 79-553 "Magnitude and Frequency of Floods in Western Oregon.")

**(See end pages for gages in adjacent states near the Oregon Border)**

| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (Note 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | MEAN BASIN ELEVATION IN FEET | MEAN ANNUAL PRECIPITATION IN INCHES | TEMPERATURE INDEX IN DEGREES F | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| WILLAMETTE RIVER BASIN (cont) | | | | | | | | | | | |
| 14147500 | North Fork of Middle Fork Willamette River near Oakridge, OR | 43°45'25" | 122°30'15" | 55 | 1911-1985 | 246 | 3760 | 58 | 25 | 132 | Slight regulation by Waldo Lake: occasional fluctuations during low-water periods caused by log-ponds upstream from station. No diversions upstream from station. |
| 14148000 | Middle Fork Willamette River below North Fork, near Oakridge, OR | 43°48'05" | 122°33'35" | 36 | 1925-1960 | 924 | 3970 | 58 | 25 | 632 | Flow regulated since 1961 by Hills Creek Lake (station 14145100): slight regulation at times by log ponds upstream from station. No diversion upstream from station. |
| 14148000 | Middle Fork Willamette River below North Fork, near Oakridge, OR | 43°48'05" | 122°33'35" | 25 | 1963-1987 | 924 | 3970 | 58 | 25 | 891 | |
| 14150000 | Middle Fork Willamette River near Dexter, OR | 43°56'45" | 122°50'10" | 25 | 1963-1987 | 1,001 | | | | 1140 | Flow regulated since 1953 by Lookout Point Lake (station 14149000), since 1955 by Dexter Lake (re-regulating), and since 1961 by Hills Creek Lake (station 14145100) |
| 14150300 | Fall Creek near Lowell, OR | 43°58'15" | 122°38'15" | 23 | 1965-1987 | 118 | 2400 | 55 | 30 | 26 | No regulation or diversion upstream from station. |
| 14150800 | Winberry Creek near Lowell, OR | 43°54'50" | 122°41'15" | 17 | 1965-1981 | 43.9 | | | | | No regulation or diversion upstream from station. |
| 14151000 | Fall Creek below Winberry Creek, near Fall Creek, OR | 43°56'40" | 122°46'25" | 29 | 1937-1965 | 186 | 2320 | 62 | 31 | 33 | Flow regulated since 1966 by Fall Creek Lake (station 14150900). No diversion upstream from station. |
| 14151000 | Fall Creek below Winberry Creek, near Fall Creek, OR | 43°56'40" | 122°46'25" | 20 | 1968-1987 | 186 | 2320 | 62 | 31 | 41 | |
| 14151500 | Little Fall Creek near Fall Creek, OR | 43°58'10" | 122°45"20" | 12 | 1937-1948 | 52.5 | 2100 | 64 | 32.5 | 15 | No diversion or regulation upstream from station. |
| 14152000 | Middle Fork Willamette River at Jasper, OR | 43°59'55" | 122°54'20" | 20 | 1968-1987 | 1,340 | 3370 | 58 | 28.5 | 1420 | Flow regulated since 1953 by Lookout Point Lake (station 14149000), since 1961 by Hill Creek Lake (station 14145100), and since 1966 by Fall Creek Lake (station 14150900). |
| 14152500 | Coast Fork Willamette River at London, OR | 43°38'30" | 123°05'05" | 51 | 1937-1987 | 72.1 | 2120 | 56 | 33 | 13 | No regulation. Diversions for irrigation upstream from station. |
| 14153500 | Coast Fork Willamette River below Cottage Grove Dam, OR | 43°43'15" | 123°02'55" | 44 | 1944-1987 | 104 | | | | 45 | Flow regulated since 1942 by Cottage Grove Lake (station 14153000). Small diversion for irrigation upstream from station. |
| 14154500 | Row River above Pitcher Creek, near Dorena, OR | 43°44'10" | 122°52'20" | 51 | 1937-1987 | 211 | 2850 | 60 | 31 | 20 | Slight, regulation caused by upstream log ponds. No diversion upstream from station. |
| 14155500 | Row River near Cottage Grove, OR | 43°47'35" | 122°59'25" | 10 | 1940-1949 | 270 | 2630 | 56 | 31.3 | 27 | Flow regulated since October 1949 by Dorena Lake (station 14155000). No diversion upstream from station. |
| 14156500 | Mosby Creek at mouth, near Cottage Grove, OR | 43°46'35" | 122°59'55" | 32 | 1948-1981 | 95.3 | 2030 | 56 | 32.5 | 6.7 | No regulation. Small diversion for irrigation upstream from station. |
| 14157000 | Coast Fork Willamette River at Saginaw, OR | 43°50'05" | 123°02'30" | 16 | 1925-1942 | 529 | 2210 | 55 | 33 | 35 | Small diversion and regulation by log ponds upstream from station: regulation by Cottage Grove Reservoir since Oct. 31, 1942, and Dorena Reservoir since Oct. 11, 1949. |
| 14157500 | Coast Fork Willamette River near Goshen, OR | 43°58'50" | 122°58'55" | 36 | 1952-1987 | 642 | | | | 167 | Flow regulated since 1942 by Cottage Grove Lake (station 14153000) and since 1949 by Dorena Lake (station 14155000). Several small diversions for log ponds and irrigation upstream from station. |
| 14158000 | Willamette River at Springfield, OR | 44°02'45" | 123°01'40" | 23 | 1913-1942 | 2,030 | 2870 | 54 | 31 | 674 | Flow regulated by Cottage Grove (since 1949), Dorena (since 1949), and Lookout Point Reservoir (since 1954). Small Diversions for irrigation upstream from station. |
| 14158500 | McKenzie River at outlet of Clear Lake, OR | 44°21'40" | 121°59'40" | 41 | 1914-1987 | 92.4 | 4120 | 81 | 21 | 215 | Flow regulated by natural storage in lake. At high stages an undetermined flow enters numerous sinkholes in lava rock along south edge of lake upstream from station. Hydrologic drainage boundary uncertain because of interbasin ground-water exchange. |
| 14158790 | Smith River above Smith River Reservoir, near Belknap Springs, OR | 44°20'05" | 122°02'45" | 26 | 1962-1987 | 16.2 | | | | 4.0 | No regulation or diversion upstream from station. |
| 14158850 | McKenzie River below Trail Bridge Dam, near Belknap Springs, OR | 44°16'05" | 122°02'55" | 23 | 1965-1987 | 184 | | | | 60 | Flow regulated since 1963 by Smith River Reservoir (station 14158795). Diurnal fluctuations by powerplants and by Trail Bridge reregulating reservoir upstream. Water is diverted from McKenzie River is SW ¼ sec. 20, T. 14, S. , R. 7E., to Smith River Reservoir and returned to river upstream from station. |
| 14159000 | McKenzie River at McKenzie Bridge, OR | 44°10'45" | 122°07'45" | 51 | 1912-1962 | 348 | 4220 | 86 | 20 | 1040 | Flow regulated since March 1963 by Smith River Reservoir (Carmen-Smith Project) 12 mi. up stream (station 14158795). No diversion upstream from station. |
| 14159000 | McKenzie River at McKenzie Bridge, OR | 44°10'45" | 122°07'45" | 23 | 1965-1987 | 348 | 4220 | 86 | 20 | 1030 | |

Note 1 Number of values used to compute statistics.

# TABLE A: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR WESTERN OREGON

REVISED March 1, 2004

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 = Annual Low and High Flow, and Instantaneous Peak Flow" and USGS Open File Report 79-553 "Magnitude and Frequency of Floods in Western Oregon.")

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| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (Note 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | MEAN BASIN ELEVATION IN FEET | MEAN ANNUAL PRECIPITATION IN INCHES | TEMPERATURE INDEX IN DEGREES F | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| WILLAMETTE RIVER BASIN (cont) | | | | | | | | | | | |
| 14159200 | South Fork McKenzie River above Cougar Lake, near Rainbow, OR | 44°02'50" | 122°13'00" | 29 | 1959-1987 | 160 | 4290 | 65 | 23 | 209 | No regulation or diversion upstream from station. |
| 14159500 | South Fork McKenzie River near Rainbow, OR | 44°08"10" | 122°14'50" | 14 | 1949-1962 | 208 | 4080 | 70 | 24 | 238 | Flow regulated since 1963 by Cougar Lake (station 14159400), usable capacity, 165,000 acre-ft. No diversion upstream from station. |
| 14159500 | South Fork McKenzie River near Rainbow, OR | 44°08'10" | 122°14'50" | 23 | 1965-1987 | 208 | 4080 | 70 | 24 | 236 | |
| 14161100 | Blue River below Tidbits Creek, near Blue River, OR | 44°13'05"' | 122°15'50" | 23 | 1965-1987 | 45.8 | | | | 12 | No regulation or diversion upstream from station. |
| 14161500 | Lookout Creek near Blue River, OR | 44°12'35" | 122°15'20" | 28 | 1951-1987 | 24.1 | 3190 | 104 | 25.7 | 9.8 | No regulation or diversion upstream from station. |
| 14162000 | Blue River near Blue River, OR | 44°10'55" | 122°16'45" | 28 | 1937-1964 | 75 | 3170 | 117 | 26.2 | 21 | No regulation or diversion upstream from station. |
| 14162200 | Blue River at Blue River, OR | 44°09'45" | 122°19'55" | 18 | 1970-1987 | 87.7 | | | | 35 | Flow regulated since October 1968 by Blue River Lake (station 14162100). No diversion upstream from station. Discharge not adjusted for storage or release from Blue River Lake as losses from reservoir at times exceed natural flow. |
| 14162500 | McKenzie River near Vida, OR | 44°07'30" | 122°28'10" | 37 | 1926-1962 | 930 | 3850 | 83 | 23.0 | 1580 | Flow regulated since 1963 by Smith River Reservoir (station 14158795) and Cougar Lake (station 14159400) and since 1968 by Blue River Lake (station 14162100). No diversion upstream from station. |
| 14163000 | Gate Creek at Vida, OR | 44°08'45" | 122°34'15" | 25 | 1953-1987 | 47.6 | 2150 | 92 | 31.0 | 18 | No regulation or diversion upstream from station. |
| 14165000 | Mohawk River near Springfield, OR | 44°05'34" | 122°57'20" | 39 | 1937-1987 | 177 | 1510 | 70 | 32.0 | 22 | Many diversions for irrigation upstream from station. |
| 14165500 | McKenzie River near Coburg, OR | 44°06'45" | 123°02'45" | 17 | 1946-1962 | 1,337 | 3180 | 79 | 30.0 | 1850 | Flow regulated since 1963 by Smith River Reservoir and Cougar Lake and since 1968 by Blue River Lake. Slight diurnal fluctuation caused by logponds and powerplants upstream. Water supply for City of Eugene is diverted 10 mi upstream; small diversions for irrigation upstream from station. |
| 14166000 | Willamette River at Harrisburg, OR | 44°16'14" | 123°10'21" | 18 | 1970-1987 | 3420 | | | | 4510 | Flow regulation by 8 reservoirs upstream from station. Many small diversions upstream from station for irrigation. |
| 14169001 | Long Tom River near Alvadore, OR | 44°07'25" | 123°17'55" | 24 | 1962-1985 | 252 | | | | 32 | Flow regulated since 1941 by Fern Ridge Lake (station 14168000). Several small diversions for irrigation upstream from station. Records include diversion to Coyote Creek Channel. Point of diversion is 500 ft. upstream and point of return, 2.3 mi downstream. Discharge not adjusted for storage or release from Fern Ridge Lake as evaporation from reservoir at times exceeds natural flow and diversions, and beginning in November 1951, most of flow of Amazon Creek has been diverted into Fern Ridge Lake. |
| 14169300 | Amazon Creek at Eugene, OR | 44°00'45" | 123°04'35" | 12 | 1964-1975 | 3.35 | | | | 0.1 | No regulation or diversion upstream from station. |
| 14170000 | Long Tom River at Monroe, OR | 44°18'50"' | 121°17'45" | 12 | 1929-1940 | 391 | | | | 15 | Flow regulated since 1941 by Fern Ridge Lake (station14168000). Several small diversions upstream from station. |
| 14170000 | Long Tom River at Monroe, OR | 44°18'50"' | 121°17'45" | 45 | 1943-1987 | 391 | | | | 31 | |
| 14170500 | Rock Creek near Philomath, OR | 44°30'05" | 123°26'20" | 10 | 1947-1979 | 14.6 | 1510 | 91 | 31.0 | 0.8 | Flow regulated by small storage reservoir operated by City of Corvallis, most low-water flow diverted to City of Corvallis water supply system. |
| 14171000 | Marys River near Philomath, OR | 44°31'35" | 123°20'00" | 44 | 1942-1985 | 159 | 925 | 78 | 31.0 | 11 | Records include flow of Evergreen Creek at Bellfountain road crossing 1.4 mi south of station, with which overflow from Marys River may at times be mingled. Slight regulation by small storage reservoir on Rock Creek from which municipal supply is diverted for City of Corvallis. Other small diversions upstream from station for irrigation. |
| 14172000 | Calapooia River at Holley, OR, | 44°21'05" | 122°47'10' | 51 | 1937-1987 | 105 | 2060 | 89 | 31.0 | 26 | Slight regulation at times during low-water periods by small dam upstream. Diversions for irrigation upstream from station. |
| 14173500 | Calapooia River at Albany, OR | 44°37'15" | 123°07'40" | 40 | 1942-1981 | 372 | 955 | 56 | 33.0 | 23 | Higher flows are affected by backwater from Willamette River at times. Diurnal fluctuation caused by ponds at flourmills near Shedd. Diversions for irrigation upstream from station. |
| 14174000 | Willamette River at Albany, OR | 44°38'20" | 123°06'20" | 18 | 1896-1941 | 4,840 | 2230 | 60 | 31.0 | 2630 | Flow regulated by nine reservoirs upstream from station. Albany power canal diverts water from South Santiam River at Lebanon and discharges into Calapooia River near mouth; small diversions for irrigation and municipal water supply. |
| 14174000 | Willamette River at Albany, OR | 44°38'20" | 123°06'20" | 18 | 1970-1987 | 4,840 | 2230 | 60 | 31.0 | 4800 | |

Note 1 Number of values used to compute statistics.

# TABLE A: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR WESTERN OREGON

REVISED March 1, 2004

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 = Annual Low and High Flow, and Instantaneous Peak Flow" and USGS Open File Report 79-553 "Magnitude and Frequency of Floods in Western Oregon.")

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| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (note 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | MEAN BASIN ELEVATION IN FEEET | MEAN ANNUAL PRECIPITATION IN INCHES | TEMPERATURE INDEX IN DEGREES F | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| WILLAMETTE RIVER BASIN (cont) | | | | | | | | | | | |
| 14178000 | North Santiam River below Boulder Creek, near Detroit, OR | 44°42'25" | 122°06'00" | 59 | 1909-1987 | 216 | 3720 | 80 | 20.0 | 391 | No regulation or diversion upstream from station. |
| 14179000 | Breitenbush River above French Creek, near Detroit, OR | 44°45'10" | 122°07'40" | 54 | 1934-1987 | 108 | 3720 | 77 | 23.0 | 123 | No regulation or diversion upstream from station. |
| 14181500 | North Santiam River at Niagara, OR | 44°45'10" | 122°17'50" | 20 | 1913-1952 | 453 | 3510 | 81 | 22.0 | 567 | Flow regulated since 1953 by Detroit Lake (station 14180500) and Big Cliff Reservoir, usable capacity for re-regulation purposes 2,930 acre-ft. No diversion upstream from stations. |
| 14182500 | Little North Santiam River near Mehama, OR | 44°47'30" | 122°34'40" | 55 | 1933-1987 | 112 | 2640 | 97 | 27.0 | 30 | No regulation or diversion upstream from station. |
| 14183000 | North Santiam River at Mehama, OR | 44°47'20" | 122°37'00" | 34 | 1907-1952 | 655 | 3350 | 86 | 27.0 | 601 | Flow regulated since 1953 by Detroit Lake (station 14180500) and Big Cliff Reservoir, usable capacity for re-regulating purposes, 2,930 acre-ft. No diversion upstream from station. |
| 14183000 | North Santiam River at Mehama, OR | 44°47'20" | 122°37'00" | 33 | 1955-1987 | 655 | 3350 | 86 | 27.0 | 1120 | |
| 14185000 | South Santiam River below Cascadia, OR | 44°23'35" | 122°30'35" | 51 | 1937-1987 | 174 | 2870 | 101 | 27.0 | 51 | No regulation or diversion upstream from station. |
| 14185800 | Middle Santiam River near Cascadia, OR | 44°30'50" | 121°22'15" | 17 | 1965-1981 | 104 | 3300 | 110 | 28.0 | 46 | No regulation or diversion upstream from station. |
| 14185900 | Quartzville Creek near Cascadia, OR | 44°32'25" | 122°26'05" | 21 | 1967-1987 | 99.2 | 3200 | 110 | 28.0 | 30 | No regulation no diversion upstream from station. |
| 14186000 | Middle Santiam River near Foster, OR | 44°27'35" | 122°31'25" | 15 | 1933-1947 | 271 | 2970 | 106 | 26.0 | 80 | No diversion or regulation upstream of station. |
| 14186500 | Middle Santiam River at mouth, near Foster, OR | 44°25'25" | 122°37'25" | 15 | 1952-1966 | 287 | | | | 105 | Slight regulation from construction of Green Peter Dam upstream from station. |
| 14187000 | Wiley Creek near Foster, OR | 44°22'20" | 122°37'20" | 24 | 1949-1972 | 51.8 | 2480 | 71 | 30.0 | 10 | No regulation or diversion upstream from station. |
| 14187100 | Wiley Creek at Foster, OR | 44°23'55" | 122°39'35" | 13 | 1975-1987 | 62.3 | | | | 9 | No regulation or diversion upstream from station. |
| 14187200 | South Santiam River near Foster, OR | 44°24'45" | 122°41'15" | 13 | 1975-1987 | 557 | | | | 670 | Flow regulated since October 1966 by Green Peter Lake (station 14186100) and since December 1966 by Foster Lake (station 14186600). No diversion from upstream. |
| 14187500 | South Santiam River at Waterloo, OR | 44°29'55" | 122°49'20" | 41 | 1925-1965 | 640 | 2560 | 90 | 28 | 155 | Flow regulated since October 1966 by Green Peter Lake (station 141861000) and since December 1966 by Foster Lake (station 14186600). No diversion upstream from station. |
| 14187500 | South Santiam River at Waterloo, OR | 44°29'55" | 122°49'20" | 20 | 1967-1987 | 640 | 2560 | 90 | 28 | 616 | |
| 14188800 | Thomas Creek near Scio, OR | 44°42'42" | 122°45'55" | 24 | 1964-1987 | 109 | 1900 | 55 | 28 | 20 | No regulation. Several small diversions for irrigation upstream from station. |
| 14189000 | Santiam River at Jefferson, OR | 44°42'55" | 122°00'40" | 21 | 1909-1953 | 1,790 | 2470 | 78 | 27 | 508 | Flow regulated since 1953 by Detroit Lake (station 14180500), since 1966 by Green Peter Lake (station 14186100) and by Foster Lake (station 14186600). Salem canal diverts from North Santiam River at Stayton for irrigation and power, most of this water reaches Willamette River by way of Mill Creek at Salem. Stayton Canal diverts from North Santiam River at Stayton for irrigation of lands near town of West Stayton; some return flow reaches North Santiam River upstream from station. Albany power canal diverts from South Santiam River at Lebanon; return flow reaches Willamette River at Albany. |
| 14189000 | Santiam River at Jefferson, OR | 44°42'55" | 122°00'40" | 20 | 1968-1987 | 1,790 | 2470 | 78 | 27.0 | 1350 | |
| 14189500 | Luckiamute River near Hoskins, OR | 44°43'10" | 123°30'10" | 43 | 1936-1978 | 34.3 | 1310 | 91 | 30.0 | 9.7 | Logponds upstream cause diurnal fluctuation at times. Minor diversion upstream from station by pumping for irrigation. |
| 14190000 | Luckiamute River at Pedee, OR | 44°44'35" | 123° 25'25" | 29 | 1940-1970 | 115 | 1010 | 74 | 30.0 | 16 | Some diurnal fluctuation at low flow caused by logponds upstream. Several small diversions for irrigation upstream from station. |
| 14190500 | Luckiamute River near Suver, OR | 44°47'00" | 123°14'00" | 46 | 1941-1987 | 240 | 910 | 72 | 31.0 | 28 | Some diurnal fluctuation during periods of low flow caused by millpond upstream from station. A few small diversions for irrigation upstream from station. |
| 14190700 | Rickreall Creek near Dallas, OR | 44°54'55" | 123°23'02" | 17 | 1962-1978 | 27.4 | | | | 2.4 | Low flow regulated since June 1960 by Aaron Mercer Reservoir, usable capacity, 2,010 acre-ft. Diversion for City of Dallas municipal supply from four tributaries and Rickreall Creek upstream from station. |
| 14191000 | Willamette River at Salem, OR | 44°56'40" | 123°02'30" | 24 | 1911-1941 | 7280 | 2150 | 64 | 31.0 | 3310 | Flow regulated by reservoirs upstream from station. Many small diversions for irrigation upstream from station; part of flow of Salem Canal, which diverts water from North Santiam River, returns to Willamette River downstream from Station, through Mill Creek at Salem |
| 14191000 | Willamette River at Salem, OR | 44°56'40" | 123°02'30" | 18 | 1970-1987 | 7280 | 2150 | 64 | 31.0 | 6520 | |

Note 1 Number of values used to compute statistics.

# TABLE A: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR WESTERN OREGON

REVISED March 1, 2004

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|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| WILLAMETTE RIVER BASIN (cont) | | | | | | | | | | | |
| 14192500 | South Yamhill River near Willamina, OR | 45°02'50" | 123°30'10" | 52 | 1936-1987 | 133 | 950 | 101 | 30.0 | 14 | Slight regulation occasionally at low flows by millpond upstream. No diversion upstream from station. |
| 14193000 | Willamina Creek near Willamina, OR | 45°08'35" | 123°29'35" | 52 | 1936-1987 | 64.7 | 1060 | 88 | 29.0 | 13 | No regulation or diversion upstream from station. |
| 14193300 | Mill Creek near Willamina, OR | 44°58'15" | 123°26'55" | 14 | 1960-1973 | 27.4 | 1720 | 115 | 28.5 | 4.1 | No regulation or diversion upstream from station. |
| 14194000 | South Yamhill River near Whiteson, OR | 45°10'08" | 123°12'25" | 46 | 1942-1987 | 502 | 750 | 78 | 30.0 | 27 | Slight regulation during low-water periods by logpond. Many small diversions for irrigation upstream from station. |
| 14194300 | North Yamhill River near Fairdale, OR | 45°21'55" | 123°22'40" | 26 | 1960-1987 | 9.03 | | | | 3.3 | No regulation or diversion upstream from station. |
| 14195000 | Haskins Creek near McMinnville, OR | 45°18'50" | 123°21'55" | 22 | 1930-1951 | 6.48 | 2320 | 105 | 28.0 | 1.1 | No regulation. Since Sept. 2, 1937, a small amount of water (average, 1.4-ft.$^3$/S) has been diverted at a point 800 ft. upstream for municipal supply of McMinnville. |
| 14196500 | North Yamhill River near Pike, OR | 45°22'15" | 123°17'10" | 10 | 1942-1951 | 47.8 | 1400 | 88 | 29.0 | 9.1 | Occasional diurnal fluctuations caused by small dams upstream; no seasonal regulation. Water supply for City of McMinnville is diverted from Haskins Creek upstream from station |
| 14197000 | North Yamhill River at Pike, OR | 45°22'10" | 123°15'15" | 24 | 1950-1973 | 66.8 | 1270 | 80 | 29.0 | 8.2 | Seasonal regulation by Haskins Creek Reservoir (station 14195500): occasional diurnal fluctuation caused by Haskins Creek Dam and smaller dams upstream. Water supply for City of McMinnville is diverted from Haskins Creek from station and for City of Yamhill is diverted from Turner Creek upstream from station. Smaller diversions upstream from station for irrigation. |
| 14198000 | Willamette River at Wilsonville, OR | 45°17'57" | 122°45'00" | 20 | 1954-1973 | 8,400 | | | | 6160 | Flow regulated by reservoirs upstream from station. Many diversions for irrigation upstream from station. |
| 14198500 | Molalla River above Pine Creek, near Wilhoit, OR | 45°00'35" | 122°28'45" | 51 | 1937-1987 | 97 | 2910 | 91 | 27.0 | 31 | No regulation or diversion upstream from station. |
| 14200000 | Molalla River near Canby, OR | 45°14'40" | 122°41'10" | 44 | 1930-1978 | 323 | 1910 | 68 | 31.0 | 61 | No regulation. Numerous small diversions for irrigation upstream from station. |
| 14200300 | Silver Creek at Silverton, OR | 45°00'34" | 122°47'15" | 12 | 1965-1979 | 47.9 | | | | 6.6 | No regulation. Several small diversions for irrigation and municipal use upstream from station. |
| 14201000 | Pudding River near Mount Angel, OR | 45°03'47" | 122°49'45" | 26 | 1941-1966 | 204 | 1280 | 60 | 31.0 | 16 | No regulation. Many small diversions for irrigation upstream from station. |
| 14201500 | Butte Creek at Monitor, OR | 45°06'06" | 122°44'42" | 29 | 1942-1985 | 58.7 | 1670 | 73 | 31.0 | 5.7 | No regulation. Diversions for irrigation by pumping upstream from station. |
| 14202000 | Pudding River at Aurora, OR | 45°14'00" | 122°44'56" | 35 | 1930-1964 | 479 | 859 | 55 | 33.0 | 51 | Slight regulation at high stages by pumping plant at mouth of Little Pudding River and at times, in summer by mills on tributaries. Small diversions upstream from station. |
| 14202500 | Tualatin River near Gaston, OR | 45°26'11" | 123°10'07" | 13 | 1952-1984 | 48.5 | 1260 | 76 | 29.0 | 6.4 | Slight diurnal fluctuation caused by logponds upstream. Small diversions for irrigation upstream from station. In 1949 City of Hillsboro began diverting about 5-ft$^3$/s for municipal supply. Some water is diverted from Roaring Creek upstream for Forest Grove municipal supply. |
| 14202980 | Scoggins Creek below Henry Hagg Lake near Gaston, OR | 45°28'10" | 123°11'56" | 11 | 1977-1987 | 38.8 | | | | 14 | Flow completely regulated by Henry Hagg Lake since January 1975. Discharge not adjusted for storage or release from Henry Hagg Lake as evaporation from reservoir at times exceeds natural flow. |
| 14203000 | Scoggins Creek near Gaston, OR | 45°27'32" | 123°09'16" | 16 | 1959-1974 | 43.3 | 1090 | 66 | 29.0 | 1.9 | Some diurnal fluctuation caused by logponds upstream from station. Diversions by pumping for irrigation upstream from station. Part of domestic water supply for Hillsboro is diverted from Sain Creek upstream from station. |
| 14203500 | Tualatin River near Dilley, OR | 45°28'30" | 123°07'23" | 25 | 1950-1974 | 125 | 1260 | 59 | 30.0 | 4.5 | Diurnal fluctuation caused by operation of millpond on Scoggins Creek upstream from station and regulation by Henry Hagg Lake since January 1975. Diversions upstream from station of approximately 3,000 acre-ft. from J. W. Barney Reservoir on the Middle Fork of North Fork Trask River for municipal water supply and irrigation in Wapato Lake area. |
| 14203500 | Tualatin River near Dilley, OR | 45°28'30" | 123°07'23" | 11 | 1977-1987 | 125 | 1260 | 59 | 30.0 | 48 | |
| 14204000 | Gales Creek near Gales Creek, OR | 45°38'30" | 123°15'55" | 15 | 1937-1970 | 33.2 | 1310 | 70 | 28.0 | 6.2 | No regulation or diversion upstream. |
| 14204500 | Gales Creek near Forest Grove, OR | 45°33'20" | 123°11'10" | 25 | 1942-1981 | 66.1 | 1080 | 63 | 29.0 | 8.8 | No regulation. Small diversions for irrigation upstream from station. |
| 14205500 | East Fork Dairy Creek at Mountaindale, OR | 45°38'05" | 123°02'35" | 10 | 1942-1951 | 43 | 1120 | 48 | 31.0 | 11 | Diurnal fluctuation at low stages caused by logpond upstream. Probably some pumping for irrigation upstream from station. |
| 14206500 | Tualatin River at Farmington, OR | 45°26'50" | 122°56'58" | 16 | 1952-1984 | 568 | 750 | 48 | 31.0 | 37 | Flow regulated by Henry Hagg Lake since January 1975. |

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|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| WILLAMETTE RIVER BASIN (cont) | | | | | | | | | | | |
| 14207500 | Tualatin River at West Linn, OR | 45°21'03" | 121°40'30" | 22 | 1930-1951 | 706 | 655 | 46 | 32.0 | 10 | October1951 to September 1970, all records published for this station included the daily flow in Oswego Canal. Oswego Canal diverts at point 5.0 mi upstream from station for development of power between outlet of Lake Oswego and Willamette River. Some regulation in low-water season by flashboards on crest of diversion dam for Oswego Canal and regulation by Henry Hagg Lake since January 1975. Several diversions upstream from station. |
| 14207500 | Tualatin River at West Linn, OR | 45°21'03" | 121°40'30" | 18 | 1953-1970 | 706 | 655 | 46 | 32.0 | 3.4 | |
| 14208000 | Clackamas River at Big Bottom, OR | 45°01'00" | 121°55'10" | 49 | 1922-1970 | 136 | 3900 | 91 | 20.0 | 242 | Nor regulation or diversion upstream from station. |
| 14208500 | Oak Grove Fork at Timothy Meadows, OR | 45°07'00" | 121°48'00" | 13 | 1914-1928 | 54 | 3740 | 70 | 21.0 | 125 | No diversion upstream from station. No regulation except natural storage in Clackamas Lake and in meadows upstream from station. |
| 14208700 | Oak Grove Fork near Government Camp, OR | 45°06'50" | 121°48'50" | 30 | 1958-1987 | 54.4 | | | | 40 | Flow regulated since 1956 by Timothy Lake (station 14208600). No diversion upstream from station. |
| 14209000 | Oak Grove Fork above Powerplant Intake, OR | 45°04'20" | 121°57'00" | 37 | 1911-1955 | 126 | 3750 | 69 | 22.0 | 308 | Flow regulated since 1956 by Timothy Lake (station 14208600). No diversion upstream from station. |
| 14209000 | Oak Grove Fork above Powerplant Intake, OR | 45°04'20" | 121°57'00" | 30 | 1958-1987 | 126 | 3750 | 69 | 22.0 | 264 | |
| 14209500 | Clackamas River above Three Lynx Creek, OR | 45°07'30" | 122°04'20" | 69 | 1910-1987 | 479 | 3570 | 79 | 23.0 | 663 | Minor regulation since May 1956 by Timothy Lake (station 14208600). |
| 14210000 | Clackamas River at Estacada, OR | 45°18'00" | 122°21'10" | 48 | 1910-1957 | 671 | 3350 | 77 | 25.0 | 794 | Large diurnal fluctuations and some regulation caused by powerplants at River Mill Dam and, since 1958 North Fork Dam. Minor regulation since 1956 by Timothy Lake (station 14208600). Two small diversions upstream from station for Oregon City and Estacada municipal water supply. |
| 14210000 | Clackamas River at Estacada, OR | 45°18'00" | 122°21'10" | 28 | 1960-1987 | 671 | 3350 | 77 | 25.0 | 742 | |
| 14211000 | Clackamas River near Clackamas, OR | 45°23'36" | 122°31'54" | 20 | 1964-1983 | 930 | | | | 784 | Diurnal fluctuations and some regulation by powerplants and several storage dams upstream, operated by Portland General Electric Co. Small diversions upstream from station for Estacada municipal water supply |
| 14211500 | Johnson Creek at Sycamore, OR | 45°28'40" | 122°30'24" | 46 | 1942-1987 | 26.5 | 520 | 43 | 32.0 | 0.8 | Slight diurnal fluctuation at low flow caused by recreational ponds upstream. Small diversions for irrigation upstream from station. |
| 14211720 | Willamette River at Portland, OR | 45°31'07" | 122°40'00" | 14 | 1974-1987 | 11,100 | | | | 7290 | Flow regulated by many reservoirs upstream. Many diversions for irrigation upstream from station. |
| YOUNGS RIVER BASIN | | | | | | | | | | | |
| 14251500 | Youngs River near Astoria, OR | 46°04'02" | 123°47'20" | 13 | 1929-1941 | 40.1 | 930 | 97 | 34.0 | 5.5 | No regulation, Youngs River-Lewis and Clark Water District has diverted water 4 mi upstream from station for domestic use downstream from station since 1941. |
| 14251500 | Youngs River near Astoria, OR | 46°04'02" | 123°47'20" | 16 | 1943-1958 | 40.1 | 930 | 97 | 34.0 | 6.2 | |
| NEHALEM RIVER BASIN | | | | | | | | | | | |
| 14301000 | Nehalem River near Foss, OR | 45°42'15" | 123°45'15" | 47 | 1941-1987 | 667 | 1180 | 82 | 30.0 | 93 | No regulation. Several small diversions for irrigation and domestic use upstream from station. |
| WILSON RIVER BASIN | | | | | | | | | | | |
| 14301500 | Wilson River near Tillamook, OR | 45°29'05" | 123°41'20" | 55 | 1933-1987 | 161 | 1640 | 103 | 29.0 | 72 | No regulation. Small diversions for domestic use upstream from station. |
| TRASK RIVER BASIN | | | | | | | | | | | |
| 14302500 | Trask River near Tillamook, OR | 45°26'25" | 123°43'00" | 33 | 1933-1972 | 145 | 1740 | 103 | 29.0 | 75 | No regulation. Water diverted from the J. W. Barney Reservoir since July 1, 1972. |
| NESTUCCA RIVER BASIN | | | | | | | | | | | |
| 14302900 | Nestucca River near Fairdale, OR | 45°18'40" | 123°25'05" | 17 | 1971-1987 | 6.18 | | | | 2.2 | Flow regulated since March 1969 by McGuire Lake about 1 mi upstream from station (station 14302800); during winter months lake is empty except when inflow exceeds capacity of outlet tunnel. |
| 14303000 | Nestucca River near McMinnville, OR | 45°19'30" | 123°27'00" | 15 | 1930-1944 | 12 | 2070 | 100 | 28.0 | 1.9 | No diversion upstream from station. Flow slightly regulated by dam at outlet of Meadow Lake. In summer, all of flow leaks under earth fill dam. (Note: dam removed in 1965) |
| 14303600 | Nestucca River near Beaver, OR | 45°16'00" | 123°50'45" | 22 | 1966-1987 | 180 | 1300 | 110 | 31.0 | 72 | No regulation. Small diversions for irrigation upstream from station. |
| SILETZ RIVER BASIN | | | | | | | | | | | |
| 14305500 | Siletz River at Siletz, OR | 45°42'55" | 123°53'10" | 65 | 1907-1987 | 202 | 1260 | 118 | 31.0 | 81 | Slight regulation from logponds. Small diversions upstream from station for irrigation. |
| YAQUINA RIVER BASIN | | | | | | | | | | | |
| 14306030 | Yaquina River near Chitwood, OR | 44°39'29" | 123°50'15" | 14 | 1974-1987 | 71 | | | | 7.6 | No regulation or diversion upstream from station. |

Note 1 Number of values used to compute statistics.

# TABLE A: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR WESTERN OREGON

REVISED March 1, 2004

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 = Annual Low and High Flow, and Instantaneous Peak Flow" and USGS Open File Report 79-553 "Magnitude and Frequency of Floods in Western Oregon.")

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| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (Note 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | MEAN BASIN ELEVATION IN FEET | MEAN ANNUAL PRECIPITATION IN INCHES | TEMPERATURE INDEX IN DEGREES F | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| YAQUINA RIVER BASIN (cont) | | | | | | | | | | | |
| 14306036 | Mill Creek near Toledo, OR | 44°34'35" | 123°54'25" | 12 | 1962-1973 | 4.18 | | | | 0.5 | Except for average discharge, figures not adjusted for diversion for City of Toledo municipal supply. Occasional fluctuation caused by City of Toledo diversion dam, capacity, 250 acre-ft. |
| ALSEA RIVER BASIN | | | | | | | | | | | |
| 14306100 | North Fork Alsea River at Alsea, OR | 44°22'45" | 123°35'40" | 29 | 1959-1987 | 63 | 1410 | 98 | 31.0 | 18 | No regulation. Some diversion by pumping upstream from station. |
| 14306400 | Five Rivers near Fisher, OR | 44°20'15" | 123°49'35" | 21 | 1962-1987 | 114 | 900 | 100 | 30.0 | 28 | No regulation or diversion upstream from station. |
| 14306500 | Alsea River near Tidewater, OR | 44°23'10" | 123°49'50" | 46 | 1941-1987 | 334 | 1150 | 93 | 31.0 | 79 | No regulation. Diversion for irrigation upstream from station. |
| 14306700 | Needle Branch near Salado, OR | 44°30'55" | 123°51'20" | 14 | 1960-1973 | 0.27 | 650 | 100 | 34.0 | 0.0 | No regulation or diversion upstream from station. |
| 14306800 | Flynn Creek near Salado, OR | 44°32'20" | 123°51'05" | 14 | 1960-1973 | .78 | 947 | 98 | 34.0 | 0.2 | No regulation or diversion upstream from station. |
| 14306810 | Deer Creek near Salado, OR | 44°32'05" | 123°52'35" | 14 | 1960-1973 | 1.17 | 1030 | 95 | 35.0 | 0.3 | No regulation or diversion upstream from station. |
| BIG CREEK RIVER BASIN | | | | | | | | | | | |
| 14306900 | Big Creek near Roosevelt Beach, OR | 44°10'05" | 124°03'55" | 14 | 1974-1985 | 11.9 | | | | 6.4 | No regulation or diversion upstream from station. |
| SIUSLAW RIVER BASIN | | | | | | | | | | | |
| 14307500 | Lake Creek at Triangle Lake, OR | 44°09'40" | 123°34'10" | 23 | 1933-1955 | 52.5 | 1430 | 83 | 31.0 | 9.9 | Flow regulated by natural storage in Triangle Lake. No diversion upstream from station. |
| 14307580 | Lake Creek near Deadwood, OR | 44°04'58" | 123°47'05" | 19 | 1969-1987 | 174 | | | | 27 | Flow slightly regulated by natural storage in Triangle Lake. Several diversions for irrigation upstream from station. |
| 14307620 | Siuslaw River near Mapleton, OR | 44°03'45" | 123°52'55" | 19 | 1967-1987 | 588 | | | | 95 | No regulation or diversions upstream from station. |
| 14307645 | North Fork Siuslaw River near Minerva, OR | 44°02'50" | 124°00'10" | 17 | 1969-1985 | 41.2 | | | | 16 | No regulation. Small diversions for irrigation upstream from station. |
| UMPQUA RIVER BASIN | | | | | | | | | | | |
| 14307700 | Jackson Creek near Tiller, OR | 42°57'15" | 122°49'40" | 30 | 1957-1986 | 152 | 3570 | 49 | 30.0 | 18 | No regulation or diversion upstream from station. |
| 14308000 | South Umpqua River at Tiller, OR | 42°55'50" | 122°56'50" | 47 | 1941-1987 | 449 | 3130 | 45 | 31.0 | 47 | No regulation. Small diversions for irrigation upstream from station. |
| 14308500 | Elk Creek near Drew, OR | 42°53'25" | 122°55'00" | 27 | 1956-1982 | 54.4 | 2900 | 52 | 31.0 | 0.6 | No regulation. Several diversions for irrigation upstream from station. |
| 14308600 | South Umpqua River at Days Creek, OR | 42°58'05" | 123°09'60" | 12 | 1976-1987 | 641 | | | | 50 | No regulation. Many small diversions for irrigation upstream from station. |
| 14308700 | Days Creek at Days Creek, OR | 42°58'55" | 123°08'55" | 16 | 1957-1972 | 55.3 | 1680 | 37 | 32.5 | 0.5 | No regulation. Several diversions for irrigation upstream from station. |
| 14309000 | Cow Creek near Azalea, OR | 42°49'30" | 123°10'40" | 55 | 1930-1985 | 78 | 2920 | 54 | 32.0 | 8.8 | Flow regulated since Oct. 7, 1985 by Galesville Reservoir (station 14308995). Diversions for irrigation upstream from station. |
| 14309500 | West Fork Cow Creek near Glendale, OR | 42°48'15" | 123°36'35" | 31 | 1957-1987 | 86.9 | 1990 | 83 | 30.0 | 7.4 | No regulation or diversion upstream from station. |
| 14310000 | Cow Creek near Riddle, OR | 42°55'25" | 123°25'40" | 30 | 1956-1985 | 456 | 683 | 59 | 31.0 | 27 | Regulated since Oct. 7, 1985 by Galesville Reservoir (station 14308995). Many small diversions for irrigation upstream from station. |
| 14310700 | South Myrtle Creek near Myrtle Creek, OR | 43°01'55" | 123°11'30" | 16 | 1957-1972 | 43.9 | 1960 | 37 | 33.0 | 1.6 | No regulation. Several diversions for irrigation upstream from station. |
| 14311000 | North Myrtle Creek near Myrtle Creek, OR | 43°02'30" | 123°15'30" | 30 | 1957-1986 | 54.2 | 2050 | 35 | 33.0 | 2 | No regulation. Several diversions for irrigation upstream from station. |
| 14311200 | Olalla Creek near Tenmile, OR | 43°02'20" | 123°32'35" | 16 | 1958-1973 | 61.3 | 1740 | 48 | 30.5 | 0.1 | No regulation. Some diversions for irrigation upstream from station. |
| 14312000 | South Umpqua River near Brockway, OR | 43°08'00" | 123°23'50" | 37 | 1943-1979 | 1,670 | 2230 | 46 | 32.0 | 82 | Regulation from Ben Irving Reservoir, since January 1980, on Berry Creek during summer months. Many small diversions from irrigation upstream from station. |

Note 1 Number of values used to compute statistics.

# TABLE A: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR WESTERN OREGON

REVISED March 1, 2004

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 = Annual Low and High Flow, and Instantaneous Peak Flow" and USGS Open File Report 79-553 "Magnitude and Frequency of Floods in Western Oregon.")

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|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| UMPQUA RIVER BASIN (cont) | | | | | | | | | | | |
| 14312200 | Deer Creek near Roseburg, OR | 43°13'10" | 123°16'35" | 17 | 1957-1973 | 53.2 | 1080 | 33 | 33.0 | 0.4 | No regulation. Diversions upstream from station from logponds and many small diversions by pumping for irrigation upstream from station |
| 14312500 | Lake Creek near Diamond Lake, OR | 43°11'10" | 122°09'55" | 30 | 1935-1984 | 54.9 | | | | 9.4 | Flow regulated by gates and fish racks at lake outlet. No diversion upstream from station. |
| 14315500 | North Umpqua River at Toketee Falls, OR | 43°15'50" | 122°25'20" | 20 | 1927-1948 | 339 | 5130 | 53 | 24.0 | 581 | No diversion or regulation upstream from station. |
| 14316500 | North Umpqua River above Copeland Creek, near Toketee Falls, OR | 43°17'45" | 122°32'10" | 37 | 1951-1987 | 475 | 4870 | 52 | 20.0 | 757 | Considerable fluctuation caused by powerplants upstream; flow slightly regulated by Diamond Lake and by Lemolo Lake (station 14313000). No diversion upstream from station. |
| 14316700 | Steamboat Creek near Glide, OR | 43°21'00" | 122°43'40" | 30 | 1958-1987 | 227 | 2640 | 57 | 30.0 | 38 | No regulation or diversion upstream from station. |
| 14317500 | North Umpqua River above Rock Creek, near Glide, OR | 43°19'40"' | 123°00'00" | 19 | 1926-1945 | 886 | 3980 | 52 | 27.0 | 684 | No diversion or regulation upstream from station. |
| 14317600 | Rock Creek near Glide, OR | 43°20'45" | 122°59'30" | 15 | 1959-1973 | 97.4 | 2800 | 56 | 32.0 | 21 | No diversion or regulation upstream from station. |
| 14318000 | Little River at Peel, OR | 45°15'10" | 123°01'30" | 32 | 1956-1987 | 177 | 2590 | 51 | 32.5 | 20 | No regulation. Small diversions for rural domestic use and irrigation upstream from station. |
| 14318500 | North Umpqua River near Glide, OR | 43°18'20" | 123°07'00" | 12 | 1917-1938 | 1,210 | 3520 | 50 | 30.0 | 730 | No diversion or regulation upstream from station. |
| 14319500 | North Umpqua River at Winchester, OR | 43°16'20" | 123°24'40" | 41 | 1910-1987 | 1,344 | 3480 | 49 | 33.5 | 833 | Diurnal fluctuation caused by upstream powerplants; slight regulation by Lemolo Lake and Diamond Lake. Several small diversions upstream from station. |
| 14319900 | Calapooya Creek at Nonpareil, OR | 43°25'04" | 123°09'13" | 10 | 1978-1987 | 88.6 | | 45 | 34.0 | 9.4 | Only minor diversions by pumping for irrigation upstream from station. |
| 14320700 | Calapooya Creek near Oakland, OR | 43°24'10" | 123°21'45" | 17 | 1957-1973 | 210 | 1310 | 46 | 33.7 | 6.4 | Diversion upstream from station for municipal supply of cities of Sutherlin and Oakland. Small diversions by pumping for irrigation upstream from station. |
| 14321000 | Umpqua River near Elkton, OR | 43°35'10" | 123°33'15" | 81 | 1907-1987 | 3,683 | 2480 | 47 | 32.0 | 1000 | Regulation by powerplants on North Umpqua River ordinarily does not affect discharge at this station. Diversions for irrigation upstream from station. |
| 14322000 | Elk Creek near Drain, OR | 43°38'30" | 123°17'50" | 18 | 1957-1979 | 104 | 1020 | 43 | 34.0 | 0.5 | No regulation. Small diversions by pumping for irrigation upstream from station. Municipal supply for town of Yoncalla is diverted from Wilson Creek upstream from station. |
| TENMILE CREEK BASIN | | | | | | | | | | | |
| 14323200 | Tenmile Creek near Lakeside, OR | 43°34'40" | 124°11'30" | 18 | 1959-1976 | 87 | 502 | 80 | 36.0 | 8.5 | Flow affected by natural storage in Tenmile Lake and other tributaries to Eel and Saunders Creeks. No diversion upstream from station. |
| COOS RIVER BASIN | | | | | | | | | | | |
| 14324500 | West Fork Millicoma River near Allegany, OR | 43°28'35" | 124°03'20" | 26 | 1956-1981 | 46.9 | 1220 | 91 | 34.0 | 3.9 | No regulation. Only minor diversions for irrigation upstream from station. |
| COQUILLE RIVER BASIN | | | | | | | | | | | |
| 14324600 | South Fork Coquille River above Panther Creek, near Illahe, OR | 42°45'30" | 123°59'10" | 13 | 1958-1970 | 31.2 | 2860 | 85 | 34.0 | 1.6 | No regulation or diversion upstream from station. |
| 14324700 | South Fork Coquille River near Illahe, OR | 42°43'30" | 124°00'40" | 17 | 1958-1974 | 40.6 | 2780 | 90 | 34.0 | 2.3 | No regulation or diversion upstream from station |
| 14324900 | South Fork Coquille River near Powers, OR | 42°47'05" | 124°02'25" | 13 | 1958-1970 | 93.2 | 2420 | 92 | 35.0 | 13 | No regulation or diversion upstream from station. |
| 14325000 | South Fork Coquille River at Powers, OR | 42°53'30" | 124°04'10" | 67 | 1918-1987 | 169 | 2200 | 77 | 35.0 | 20 | No regulation. Small diversions for irrigation upstream from station. |
| 14326500 | Middle Fork Coquille River near Myrtle Point, OR | 43°01'30" | 124°05'20" | 15 | 1932-1946 | 305 | 1460 | 76 | 32.0 | 13 | No diversion upstream from station. Flow regulated during winter and spring months by operation of log ponds. |
| 14326800 | North Fork Coquille River near Fairview, OR | 43°11'03" | 124°04'33" | 17 | 1965-1981 | 73.9 | 800 | 75 | 34.0 | 4.5 | No regulation. Several diversions for irrigation upstream from station. |
| 14327000 | North Fork Coquille River near Myrtle Point, OR | 43°04'15" | 124°06'20" | 21 | 1930-1968 | 282 | 1180 | 72 | 34.0 | 24 | No regulation. Small diversions for irrigation upstream from station. |

Note 1 Number of values used to compute statistics.

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REVISED March 1, 2004

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|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| ROGUE RIVER BASIN | | | | | | | | | | | |
| 14327500 | Rogue River above Bybee Creek, near Union Creek, OR | 42°56'05" | 122°25'15" | 22 | 1931-1952 | 156 | 5200 | 56 | 23.0 | 264 | No regulation or diversion upstream from station. |
| 14328000 | Rogue River above Prospect, OR | 42°46'30" | 122°29'55" | 65 | 1909-1987 | 312 | 4900 | 54 | 23.0 | 379 | No regulation or diversion upstream from station. |
| 14330000 | Rogue River below Prospect, OR | 42°43'50" | 122°30'55" | 18 | 1970-1987 | 379 | | | | 889 | Fluctuations caused by powerplant 600 ft. upstream from station. Small diversions for irrigation upstream from station. |
| 14330500 | South Fork Rogue River above Imnaha Creek, near Prospect, OR | 42°42'25" | 122°23'20" | 17 | 1933-1949 | 52 | 5230 | 50 | 20.0 | 46 | No diversion or regulation upstream from station. |
| 14331000 | Imnaha Creek near Prospect, OR | 42°41'20" | 122°23'00" | 15 | 1935-1949 | 26 | 4850 | 58 | 21.0 | 19 | No diversion or regulation upstream from station. |
| 14333000 | Middle Fork Rogue River near Prospect, OR | 42°44' 05" | 122°24'05" | 22 | 1934-1955 | 56.5 | 5310 | 56 | 21.0 | 113 | All records given herein include flow in Middle Fork power canal which diverts 850 ft. upstream from station for hydroelectric power and returns water to the Rogue River above South Fork Rogue River; practically no storage upstream from diversion dam. |
| 14333500 | Red Blanket Creek near Prospect, OR | 42°46'40" | 122°25'35" | 47 | 1935-1981 | 45.5 | 5290 | 54 | 20.0 | 58 | No regulation. Small diversions for irrigation upstream from station. |
| 14334700 | South Fork Rogue River, south of Prospect, OR | 42°42'45" | 122°30'20" | 18 | 1970-1987- | 246 | | | | 88 | Some regulation by South Fork canal dam upstream. Power diversions upstream from station from South Fork Rogue River, Middle Fork Rogue River, and Red Blanket Creek divert water to Rogue River via Main Canal. During summer base flow all of streamflow is diverted for power except that for fish life. Base flow at station is principally from springs downstream from power diversions. |
| 14335000 | Rogue River below South Fork Rogue River, near Prospect, OR | 42°42'00" | 122°35'40" | 36 | 1930-1965 | 650 | 4700 | 50 | 23.0 | 913 | Considerable diurnal fluctuation caused by powerplant 5.5 mi upstream from station. Small diversions for irrigation upstream from station. |
| 14335500 | South Fork Big Butte Creek near Butte Falls, OR | 42°32'25" | 122°33'15" | 59 | 1929-1987 | 138 | 3950 | 36 | 22.5 | 67 | Flow slightly regulated since 1952 by Willow Creek Reservoir, capacity, 7,320 acre-ft. Diversions for irrigation upstream from station and for municipal water supply for Medford (since 1927) and Butte Falls. |
| 14337500 | Big Butte Creek near McLeod, OR | 42°39'05" | 122°41'25" | 30 | 1947-1987 | 245 | 3520 | 36 | 24.0 | 60 | Slight regulation by fish hatchery 600 ft. upstream from station. Several diversions in the vicinity of Butte Falls, the Two largest being the City of Medford diversion and Eagle Point Irrigation District Canal. |
| 14337600 | Rogue River near McLeod, OR | 42°39'20" | 122°42'50" | 10 | 1967-1976 | 938 | | | | 1060 | Flow regulated since February 1977 by Lost Creek Lake (station 14335040. Diversions for irrigation upstream from station; most of low flow of Big Butte Creek is diverted near Butte Falls. |
| 14337800 | Elk Creek near Cascade Gorge, OR | 42°46'25" | 122°40'15" | 13 | 1975-1987 | 78.8 | | | | 2.8 | No regulation. Many diversions upstream from station for irrigation. |
| 14337870 | West Branch Elk Creek near Trail, OR | 42°42'40" | 122°44'55" | 11 | 1975-1987 | 14.2 | | | | 1.3 | No regulation or diversions upstream from stations. |
| 14338000 | Elk Creek near Trail, OR | 42°39'50" | 122°44'50" | 41 | 1947-1987 | 133 | 3100 | 39 | 28.0 | 2.9 | Low flow regulation resulting from construction of Elk Creek Dam 1.3 mi upstream. Diversions for irrigation and dam construction upstream from station. |
| 14339000 | Rogue River at Dodge Bridge, near Eagle Point, OR | 42°31'30" | 122°50'30" | 37 | 1940-1976 | 1,215 | 3930 | 43 | 25.0 | 1090 | Flow regulated since February 1977 by Lost Creek Lake (station 14335040. Diversions for irrigation upstream from station; most of low flow of Big Butte Creek (station 14337500) is diverted near Butte Falls. |
| 14339500 | South Fork Little Butte Creek at Big Elk Ranger Station, OR | 42°20'40" | 122°21'30" | 23 | 1928-1950 | 17 | 5350 | 26 | 22.0 | 7.6 | No diversion or regulation upstream from station. |
| 14341500 | South Fork Little Butte Creek near Lakecreek, OR | 42°24'30" | 122°36'00" | 35 | 1923-1957 | 138 | 4440 | 22 | 25.0 | 15 | No regulation. Diversions for irrigation upstream from station; also, in December 1958 Dead Indian collection canal began diverting upstream from station from Code Creek and Dead Indian Creek and in December 1959 South Fork Little Butte collection canal began diverting upstream from station from South Fork Little Butte Creek, Daley Creek, and Beaver Dam Creek. These are transbasin diversions to Howard Prairie Reservoir in Klamath River basin, but eventually this creek water is diverted back to Rogue River basin for irrigation of lands in the Ashland-Medford area and power development enroute. |
| 14341500 | South Fork Little Butte Creek near Lakecreek, OR | 42°24'30" | 122°36'00" | 21 | 1962-1982 | 138 | 4440 | 22 | 25.0 | 11 | |
| 14342500 | North Fork Little Butte Creek at Fish Lake, near Lakecreek, OR | 42°22'35" | 122°21'20" | 62 | 1925-1987 | 20.8 | 6520 | 34 | 22.0 | 3.1 | Since 1915, Fish Lake has stored water for irrigation by Medford Irrigation District. Cascade Canal diverts from Fourmile Lake in Klamath River basin and discharges into lava bed 1.0 mi upstream from Fish Lake; diversion began August 1923. No diversion from creek upstream from station. |
| 14343000 | North Fork Little Butte Creek near Lakecreek, OR | 42°24'10" | 122°32'10" | 56 | 1913-1985 | 43.8 | 4820 | 31 | 24.0 | 28 | Flow partly regulated since 1915 by Fish Lake (published with station 14342500). Diversions for irrigation upstream from station; some water diverted into Fish Lake from Fourmile Lake, in Klamath River Basin, since 1923. |

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|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| ROGUE RIVER BASIN (cont) | | | | | | | | | | | |
| 14350000 | Emigrant Creek near Ashland, OR | 43°09'50" | 122°36'15" | 24 | 1963-1986 | 64.3 | | | | 0.1 | Flow regulated since 1924 by Emigrant Lake. Several diversions upstream from station for irrigation; the principal diversion canals are Ashland lateral and East lateral. Records for Ashland lateral and inflow to the basin from Green Springs powerplant can be obtained from the Oregon Water Resources Department. From June 1923 to August 1960, water diverted by Keene Creek Canal from Klamath River basin into Emigrant Creek upstream from station. Beginning in May 10960, water from Klamath basin diverted to Emigrant Creek upstream from station via Green Springs powerplant diversion. |
| 14353000 | West Fork Ashland Creek near Ashland, OR | 42°08'55" | 122°42'55" | 14 | 1926-1982 | 10.5 | 5120 | 21 | 27.0 | 2.2 | No regulation or diversion upstream from station. |
| 14353500 | East Fork Ashland Creek near Ashland, OR | 42°09'10" | 122°42'30" | 14 | 1926-1982 | 8.14 | 5040 | 22 | 26.0 | 2.2 | No regulation or diversion. |
| 14357500 | Bear Creek at Medford, OR | 42°19'40" | 122°52'10" | 63 | 1922-1987 | 289 | | | | 12 | Flow partly regulated since 1924 by Emigrant Lake (published with station 14350000). Numerous diversions for irrigation and municipal use upstream from station. |
| 14359000 | Rogue River at Raygold, near Central Point, OR | 42°26'15" | 122°59'10" | 70 | 1907-1976 | 2,053 | 3560 | 35 | 25.0 | 1130 | Flow regulated since February 1977 by Lost Creek Lake (station 14335040). Slight regulation by Fish Lake (published with station 14342500) and Emigrant Lake (published with station 14350000). Many diversions for irrigation upstream from station. |
| 14361500 | Rogue River at Grants Pass, OR | 42°25'50" | 123°19'00" | 37 | 1940-1976 | 2,459 | | | | 1010 | Flow regulated since February 1977 by Lost Creek Lake (station 14355040), slight regulation by Fish Lake and Emigrant Lake. Large fluctuation at times caused by Savage Rapids Dam 5.5 mi upstream from station. Many diversions from Rogue River and tributaries upstream from station, the largest of which is at Savage Rapids Dam or Grants Pass Irrigation District, 5.5 miles upstream from station. |
| 14362000 | Applegate River near Copper, OR | 42°03'50" | 123°06'37" | 41 | 1940-1980 | 225 | 4280 | 35 | 27.0 | 37 | Flow regulated since December 1980 by Applegate Lake (station 14361900). Some storage during winter in Squaw Lakes Reservoir, capacity, 1,100 acre-ft. on Squaw Creek upstream from station. Diversions upstream from station from Carberry Creek for irrigation in Thompson Creek basin. |
| 14363000 | Applegate River near Ruch, OR | 42°10'40" | 123°02'40" | 26 | 1927-1953 | 302 | 3900 | 32 | 28.0 | 24 | Diversions for irrigation of about 700 acres upstream from station. Cameron (Comstock). Ditch diverts as much as 14-$ft^3/s$ around station on left bank. An average of about 8-$ft^3/s$ is diverted upstream from station for irrigation in Thompson Creek basin. Several hundred acre-feet stored in Squaw Lake (capacity, 1,100 acre-ft) each spring for irrigation the following summer. |
| 14366000 | Applegate River near Applegate, OR | 42°14'30" | 123°08'20" | 41 | 1940-1980 | 483 | 3660 | 29 | 28.5 | 19 | Flow regulated since December 1980 by Applegate Lake (station 14361900). Many diversions for irrigation upstream from station. McDonald Creek Canal diverts from McDonald Creek upstream from station for irrigation in Bear Creek basin. Thompson Creek Irrigation Association ditch diverts upstream from station for irrigation in Thompson Creek basin. Fowler-Keeler and Berryman ditches divert upstream from station for irrigation downstream. |
| 14368500 | Powell Creek near Williams, OR | 42°16'00" | 123°17'40" | 11 | 1948-1958 | 8.17 | 3220 | 28 | 28.0 | 1.4 | No regulation or diversions upstream from station. |
| 14369500 | Applegate River near Wilderville, OR | 42°21'15" | 123°24'20" | 17 | 1940-1950 | 698 | | | | 10 | Flow regulated since December 1980 by Applegate Lake (station 14361900). Many diversions for irrigation upstream from station. Wilderville ditch diverts up to 16-$ft^3/s$ 0.3 mi upstream and at the mouth of Jackson Creek. |
| 14370000 | Slate Creek at Wonder, OR | 42°21'40" | 123°31'10" | 11 | 1947-1957 | 31.4 | 2160 | 45 | 31.0 | 1.2 | No regulations. Several small diversions upstream for irrigation. |
| 14371500 | Grave Creek at Pease Bridge, near Placer, OR | 42°38'30" | 123°12'40" | 41 | 1947-1987 | 22.1 | 3480 | 53 | 31.0 | 0.8 | No regulations. One small diversion upstream from station. Prior to 1945, Columbia upper ditch diverted water about 2 mi upstream from station, bypassing station. |
| 14372300 | Rogue River near Agness, OR | 42°34'50" | 124°03'30" | 15 | 1962-1976 | 3,939 | 3100 | 40 | 27.0 | 1130 | Flow regulated since February 1977 by Lost Creek Lake (station 14335040), since December 1980 by Applegate Lake (station 14361900), slight regulation by Fish Lake and Emigrant Lake. Many diversions for irrigation and mining. |
| 14372500 | East Fork Illinois River near Takilma, OR | 42°00'10" | 123°37'30" | 45 | 1943-1987 | 42.3 | 3900 | 74 | 29.0 | 8.6 | No regulation. Two small diversions for irrigation upstream from station. |
| 14375000 | Sucker Creek near Holland, OR | 42°09'00" | 123°27'50" | 19 | 1947-1965 | 76.2 | 3910 | 57 | 27.0 | 26 | No regulation. Grayback Canal and 3 small diversions from Grayback and Cave Creeks divert water for domestic use and irrigation upstream from station. Most of return flow from these diversions enters creek upstream from station. |
| 14375100 | Sucker Creek below Little Grayback Creek, near Holland, OR | 42°09'35" | 123°28'40" | 21 | 1967-1987 | 83.9 | | | | 25 | Grayback canal and 3 small diversions from Grayback and Cave Creeks divert water for domestic use and irrigation upstream from station. Return flow from these diversions enters creek upstream from station. |

Note 1 Number of values used to compute statistics.

# TABLE A: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR WESTERN OREGON

REVISED March 1, 2004

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 = Annual Low and High Flow, and Instantaneous Peak Flow" and USGS Open File Report 79-553 "Magnitude and Frequency of Floods in Western Oregon.")
(Data for California supplies by USGS Sacramento Office.)
(Data for Washington from USGS Open-File 84-145-1 "Streamflow Statistics and Drainage-Basin Characteristics for the Southwestern and Eastern Regions, Washington, Volume I. Southwestern Washington.")

**(See end pages for gages in adjacent states near the Oregon Border)**

| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (Note 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | MEAN BASIN ELEVATION IN FEET | MEAN ANNUAL PRECIPITATION IN INCHES | TEMPERATURE INDEX IN DEGREES F | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | |
| ROGUE RIVER BASIN (cont) | | | | | | | | | | | |
| 14375500 | West Fork Illinois River below Rock Creek near O'Brien, OR | 42°02'20" | 123°44'50" | 30 | 1956-1985 | 42.4 | 2500 | 80 | 31.0 | 4.6 | Three small diversions from Elk Creek for irrigation upstream from station. |
| 14377000 | Illinois River at Kerby, OR | 42°11'50" | 123°39'30" | 34 | 1928-1961 | 364 | 2930 | 66 | 29.0 | 29 | No regulation. Diversions for irrigation of 5,500 acres upstream from station. Some diversions for mining during winter months. |
| 14677100 | Illinois River at Kerby, OR | 42°13'55" | 123°39'45" | 25 | 1963-1987 | 380 | | | | 32 | No regulation. Diversions for irrigation upstream from station. |
| 14377500 | Deer Creek near Dryden, OR | 42°15'50" | 123°27'00" | 10 | 1947-1956 | 22 | 3370 | 43 | 28.4 | 1.5 | No regulation. Small diversions upstream from station for irrigation. |
| 14378000 | Illinois River near Selma, OR | 42°22'45" | 123°48'40" | 10 | 1958-1967 | 665 | 2780 | 58 | 29.0 | 76 | No regulation. Many diversions upstream from station for irrigation, mining, and logpond operation. Records include flow of Panther Creek. |
| 14378200 | Illinois River near Agness, OR | 42°31'15" | 124°02'35" | 20 | 1962-1981 | 988 | | | | 165 | No regulation. Many diversions upstream from station for irrigation, mining, and logpond operation. Records include flow of Fox Creek. |
| CHETCO RIVER BASIN | | | | | | | | | | | |
| 14400000 | Chetco River near Brookings, OR | 42°07'25" | 124°11'10" | 17 | 1971-1987 | 271 | | | | 69 | No regulation or diversion upstream from station. |
| GAGES IN CALIFORNIA NEAR OREGON BORDER | | | | | | | | | | | |
| 11533000 | Lopez Creek near Smith River, California | 41°57'36" | 124°12'08" | 5 | 1962-1966 | 0.92 | 700 | 80 | 39.0 | 0.01 | No regulation or diversion above station. |
| 11532620 | Mill Creek near Crescent City, California | 41°44'32" | 124°06'06" | 7 | 1975-1981 | 28.6 | | | | 3.6 | Minor regulation and diversion above station for lumber mill and park campground use. |
| 11531000 | Middle Fork Smith River at Gasquet, California | 41°50'40" | 123°57'35" | 13 | 1912-1965 | 130 | 1500 | 104 | 38.0 | 53 | No regulation or diversion above station. |
| GAGES IN WASHINGTON STATE NEAR COLUMBIA RIVER BETWEEN BONNEVILLE DAM AND PORTLAND | | | | | | | | | | | |
| 14143500 | Washougal River near Washougal, WA | 45°37'20" | 122°18'00" | 34 | 1946-1979 | 108 | 1610 | 100 | 28.0 | 65.3 | Minor regulation or diversion caused by fish hatchery above station |
| 14212000 | Salmon Creek near Battle Ground, WA | 45°46'25" | 122°26'35" | 31 | 1945-1975 | 18.3 | 3010 | 73 | 29.0 | 2.6 | No regulation or diversion above station. |
| GAGES IN WASHINGTON STATE NEAR COLUMBIA RIVER, DOWNSTREAM FROM PORTLAND | | | | | | | | | | | |
| 14223000 | Kalama River near Kalama, WA | 46°01'10" | 122°43'50" | 15 | 1918-1932 | 179 | 2100 | 96 | 29.0 | 202 | Slight fluctuations from powerplant above station. Practically no storage at intakes. |
| 14223500 | Kalama River below Italian Creek Near Kalama, WA | 46°02'42" | 122°48'51" | 28 | 1948-1975 | 198 | 1880 | 82 | 29.0 | 235 | No regulation. Small diversion for fish hatchery which is returned to stream above gage station. |
| 14243500 | Delameter Creek near Castle Rock, WA | 46°15'49" | 122°57'58" | 19 | 1951-1969 | 19.6 | 906 | 72 | 32.0 | 3.8 | Some diversions for domestic use. No regulation. |
| 14245000 | Coweman River near Kelso, WA | 46°07'42" | 122°50'14" | 28 | 1952-1979 | 119 | 1390 | 69 | 31.0 | 37.7 | No regulation or diversion upstream from station. |
| 14247500 | Elochoman River near Cathlamet, WA | 46°13'17" | 123°20'18" | 30 | 1942-1971 | 65.8 | 1190 | 94 | 32.0 | 28.8 | Minor diversions for irrigation and domestic use. Some diversion and minor regulation by Washington State Fish Hatchery 0.8 mi upstream. Most of the water diverted by the hatchery returned to river above the gage. |
| 14248200 | Jim Crow Creek near Grays River, WA | 46°16'37" | 123°33'37" | 9 | 1966-1974 | 5.48 | 575 | 85 | 34.0 | 0.6 | No regulation or diversion above station. |
| 14249000 | Grays River above South Fork near Grays River, WA | 46°23'36" | 123°28'39" | 19 | 1957-1975 | 39.9 | 1350 | 116 | 31.0 | 26.3 | No regulation or diversion above station. |
| 14250500 | West Fork Grays River near Grays River, WA | 46°23'10" | 123°33'30" | 20 | 1950-1969 | 15.2 | 1180 | 103 | 30.0 | 9.2 | Some regulation and diversion by fish hatchery above station. Diversions returned to stream above station. |

Note 1 Number of values used to compute statistics

# TABLE B: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR EASTERN OREGON

(Revised March 1, 2004)

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 – Annual Low and High Flow, and Instantaneous Peak Flow," USGS Water Resources Investigations Report 82-4078" Magnitude and Frequency of Floods in Eastern Oregon".)

**(See end pages for gages in adjacent states near the Oregon border)**

| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (NOTE 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | AREA OF LAKES AND PONDS, % (NOTE 2) | FOREST COVER, PERCENT (NOTE 2) | MEAN ANNUAL PRECIPITATION IN INCHES | SOILS INDEX | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | | |
| WARNER LAKE BASIN | | | | | | | | | | | | |
| 10366000 | Twentymile Creek near Adel, OR | 42°04'20" | 119°57'42" | 42 | 1946-1987 | 194 | 3.0 | 20.0 | 15 | 3.90 | 2.4 | Some regulation by pumpage from Cowhead Lake. Diversions in Oregon for irrigation upstream from station; considerable diversions for irrigation in Cowhead Lake area in California. |
| 10370000 | Camas Creek near Lakeview, OR | 42°12'59" | 120°06'05" | 23 | 1914-1973 | 63 | 0.0 | 68.0 | 20 | 7.00 | 3.6 | No regulation. Diversions for irrigation upstream from station. |
| 10371000 | Drake Creek near Adel, OR | 42°12'00" | 120°00'41" | 20 | 1952-1973 | 67 | 0.7 | 10.0 | 15 | 3.50 | 4.2 | Some regulation by two reservoirs upstream from station with combined capacity of 436 acre-ft. Diversions for irrigation upstream from station. |
| 10371500 | Deep Creek above Adel, OR | 42°11'21" | 120°00'02" | 54 | 1934-1987 | 249 | 1.7 | 44.9 | 17 | 5.60 | 10 | No regulation. Diversions for irrigation upstream from station. |
| 10378500 | Honey Creek near Plush, OR | 42°25'33" | 119°55'23" | 40 | 1912-1987 | 170 | 0.3 | 20.4 | 20 | 4.60 | 0.4 | Slight regulation by five small reservoirs, combined capacity, 870 acre-ft. Diversions for irrigation upstream from station. |
| ABERT LAKE BASIN | | | | | | | | | | | | |
| 10384000 | Chewaucan River near Paisley, OR | 42°41'05" | 120°34'08" | 33 | 1955-1987 | 275 | 0.1 | 82.9 | 18 | 9.5 | 25 | No regulation. Diversions for irrigation upstream from station. |
| SUMMER LAKE BASIN | | | | | | | | | | | | |
| 10388001 | Ana River near Summer Lake, OR | 42°59'42" | 120°44'54" | 15 | 1967-1987 | | | | | | 68 | Indeterminate drainage area. Ana River Springs, source of the stream located ¼ mile upstream, is flooded by pondage behind diversion dam. All records presented herein include flow in Summer Lake Canal which diverts 300 feet upstream from station for irrigation of lands along west side of Summer Lake. Flow regulated by gates at diversion dam. |
| SILVER LAKE BASIN | | | | | | | | | | | | |
| 10390001 | Silver Creek near Silver Lake, OR | 43°06'50" | 121°03'59" | 21 | 1967-1987 | 180 | | | | | 3.4 | Flow regulated by reservoir, capacity, 800 acre-ft, 1.5 mi upstream from station and by Thompson Valley Reservoir, capacity, 17,400 acre-ft, 11 mi upstream from station. Records given herein include flow in Silver Lake Irrigation District Canal which diverts 1.5 mi upstream from station. No record of diversion October 1943 to September 1965. |
| MALHEUR AND HARNEY LAKES BASIN | | | | | | | | | | | | |
| 10393500 | Silvies River near Burns, OR | 43°42'55" | 119°10'35" | 64 | 1924-1987 | 934 | 0.0 | 68.6 | 19 | 5.80 | 7.1 | No regulations. Diversion for irrigation upstream from station during periods of high flow only. |
| 10396000 | Donner und Blitzen River near Frenchglen, OR | 42°47'28" | 118°52'00" | 49 | 1938-1987 | 200 | 0.1 | 10.7 | 14 | 3.50 | 31 | No regulation or diversion upstream from station. |
| 10403000 | Silver Creek near Riley, OR | 43°41'30" | 119°39'30" | 28 | 1953-1980 | 228 | 0.1 | 71.4 | 20 | 7.10 | 1.4 | No regulation. Diversions for irrigation upstream from station. |
| ALVORD LAKE BASIN | | | | | | | | | | | | |
| 10406500 | Trout Creek near Denio, OR | 42°09'20" | 118°27'14" | 54 | 1934-1987 | 88 | 0.0 | 8.3 | 14 | 3.30 | 2.6 | No regulation. Diversions for irrigation upstream from station. |
| GOOSE LAKE BASIN | | | | | | | | | | | | |
| 11339500 | Drews Creek near Lakeview, OR | 42°07'10" | 120°34'45" | 38 | 1939-1981 | 212 | | | | | 0.9 | Record herein, except average discharge, not adjusted for diversion by North Drews Canal. Since 1912, flow regulated by Drews Reservoir, capacity, 62,550 acre-ft. Diversion for irrigation upstream from station, and since March 1914, North Drews Canal has diverted upstream from station for irrigation of lands west of Lakeview. Records subsequent to September 1981 in files of Oregon Water Resources Department. |
| 11340500 | Cottonwood Creek near Lakeview, OR | 42°14'14" | 120°30'16" | 10 | 1910-1919 | 32.9 | 0.3 | 92.0 | 15 | 2.90 | 1.6 | Flow regulated since 1923 by Cottonwood Reservoir, capacity, 7,540 acre-ft. Since October 1961, 240 acre-ft unregulated storage in Cottonwood Meadows, 9 mi upstream. Diversions for irrigation upstream from station. Records subsequent to September 1981 in files of Oregon Water Resources Department. |
| 11340500 | Cottonwood Creek near Lakeview, OR | 42°41'14" | 120°30'16" | 49 | 1926-1981 | 32.9 | 0.3 | 92.0 | 15 | 2.90 | 0.4 | |
| KLAMATH LAKE BASIN | | | | | | | | | | | | |
| 11493500 | Williamson River near Klamath Agency, OR | 42°44'25" | 121°50'00" | 32 | 1956-1987 | 1,290 | 6.1 | 74.3 | 24 | 13.8 | 0.0 | Flow affected by natural storage in Klamath Marsh. Small diversions upstream from station for irrigation in vicinity of marsh. Statistics uncertain due to excessive zero events. |
| 11497500 | Sprague River near Beatty, OR | 42°26'50" | 121°14'15" | 33 | 1955-1987 | 513 | 0.6 | 65.2 | 17 | 9.60 | 103 | No regulation. Diversions for irrigation upstream from station in the vicinity of Bly. |
| 11501000 | Sprague River near Chiloquin, OR | 42°35'05" | 121°50'55" | 56 | 1932-1987 | 1,580 | 1.5 | 64.8 | 20 | 10.90 | 194 | Minor regulation from irrigation diversions upstream from station. |
| 11502500 | Williamson River below Sprague River, near Chiloquin, OR | 42°34'15" | 121°52'35" | 63 | 1925-1987 | 3,000 | 3.4 | 69.7 | 22 | 12.20 | 497 | Some regulation by diversion dams and logpond operations of Sprague River. Diversions for irrigation upstream from station. |
| 11504000 | Wood River at Fort Klamath, OR | 42°42'05" | 121°59'20" | 15 | 1915-1936 | 90 | 0.0 | 84.2 | 28 | 11.30 | 148 | Many diversions for irrigation upstream from station. Regulation by diversion dams. |

Notes 1 Number of values used to compute statistics.
2 Percent of drainage area.

# TABLE B: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR EASTERN OREGON

(Revised March 1, 2004)

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 – Annual Low and High Flow, and Instantaneous Peak Flow," USGS Water Resources Investigations Report 82-4078" Magnitude and Frequency of Floods in Eastern Oregon".)

**(See end pages for gages in adjacent states near the Oregon border)**

| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (NOTE 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | AREA OF LAKES AND PONDS, % (NOTE 2) | FOREST COVER, PERCENT (NOTE 2) | MEAN ANNUAL PRECIPITATION IN INCHES | SOILS INDEX | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | | |
| KLAMATH LAKE BASIN (cont) | | | | | | | | | | | | |
| 11509500 | Klamath River at Keno, OR | 42°08'00" | 121°57'40" | 57 | 1931-1987 | 3,920 | | | | | 340 | Flow regulated since 1919 by Upper Klamath Lake (Station 11507001). Fluctuation by Keno powerplant 0.9 mi upstream. Diversions for irrigation upstream from station. |
| 11510700 | Klamath River below John C. Boyle Powerplant, near Keno, OR | 42°05'05" | 122°04'20" | 26 | 1962-1987 | 4,080 | | | | | 480 | Flow regulated by Upper Klamath Lake (Station 11507001). Large diurnal fluctuation caused by Keno and John C. Boyle powerplants. Diversions for irrigation upstream from station. |
| 11514500 | Keene Creek near Ashland, OR | 42°10'15" | 122°28'40" | 16 | 1950-1965 | 12.1 | | | | | 0.2 | Flow regulated since December 1922 by Hyatt Reservoir. No diversion upstream from station. Practically entire flow diverted downstream from station by Green Springs Powerplant diversion. |
| SNAKE RIVER BASIN | | | | | | | | | | | | |
| 13181000 (NOTE 3) | Owyhee River near Rome, OR | 42°52'02" | 117°38'52" | | 1951-1991 | 8,000 | | | | | 102 | Flow regulated by Antelope Reservoir, capacity, 70,000 acre-ft, increased in 1970, and Wild Horse Reservoir, capacity, 32,690 acre-ft, and numerous small reservoirs. Diversions upstream from station for irrigation. |
| 13213100 (NOTE 3) | Snake River at Nyssa, OR | 43°52'34" | 116°58'53" | | 1976-1991 | 58,700 | | | | | 6,560 | Flow regulated by many reservoirs above station. |
| 13290450 (NOTE 3) | Snake River at Hells Canyon Dam, Idaho-Oregon State Line | 45°15'05" | 116°41'50" | | 1967-1991 | 73,300 | | | | | 8,770 | Flow regulated by many reservoirs above station, with a total usable capacity of more than 10,000,000 acre-ft, the most effective of which is Brownlee Reservoir 38 miles upstream (see sta 13289700). Diurnal fluctuations caused by Hells Canyon powerplant. Diversions above station for irrigation of about 3,820,000 acres, of which 742,000 acres are irrigated by withdrawals from ground water (1966 determination). |
| OWYHEE RIVER BASIN | | | | | | | | | | | | |
| 13178000 (NOTE 3) | Jordan Creek above Lone Tree Creek, near Jordan Valley, OR | 42°52'27" | 116°57'12" | | 1947-1971 | 440 | | 38.9 | 26.1 | | 2.0 | Diversions upstream from station for irrigation. |
| 13182000 | Owyhee River above Lake Owyhee, OR | 43°13'34" | 117°29'47" | 20 | 1932-1951 | 10,400 | | | | | 161 | Flow regulated by Antelope Reservoir, Wild Horse Reservoir, and numerous small reservoirs. Diversions upstream from station for irrigation. |
| 13183000 | Owyhee River below Owyhee Dam, OR | 43°39'17" | 117°15'16" | 54 | 1934-1987 | 11,160 | | | | | 3.3 | Flow regulation since October 1932 by Lake Owyhee (station 13182500), and by many smaller reservoirs. Diversion of up to 457,000 acre-ft from Lake Owyhee during the year for irrigation of lands downstream from station and outside the basin. Many smaller diversions upstream from Lake Owyhee for irrigation upstream from station. |
| 13184000 | Owyhee River at Owyhee, OR | 43°46'57" | 117°03'30" | 17 | 1896-1927 | 11,300 | 0.4 | 3.5 | 11 | 4.40 | 5.6 | Flow regulated since October 1932 by Lake Owyhee, and smaller reservoirs. Diversions from Lake Owyhee for irrigation of lands upstream from station and outside the basin. Many smaller diversions upstream from Lake Owyhee for irrigation. |
| MALHEUR RIVER BASIN | | | | | | | | | | | | |
| 13214000 | Malheur River near Drewsey, OR | 43°47'05" | 118°19'50" | 60 | 1928-1987 | 910 | 0.2 | 29.4 | 16 | 5.3 | 3.9 | Slight regulation by small reservoirs upstream from station. Diversions for irrigation upstream from station. |
| 13215000 | Malheur River below Warmsprings Reservoir, near Riverside, OR | 43°34'29" | 118°12'31" | 65 | 1921-1987 | 1,100 | | | | | 0.1 | Flow completely regulated since November 1919 by Warmsprings Reservoir (station 13214500). Diversions for irrigation upstream from station. |
| 13216500 | N FK Malheur River above Beulah Reservoir, near Beulah, OR | 43°56'54" | 118°10'24" | 50 | 1938-1987 | 355 | 0.0 | 49.8 | 19 | 4.90 | 35 | No regulation. Diversions for irrigation upstream from station. |
| 13217500 | N Fk Malheur River at Beulah, OR | 43°54'28" | 118°09'08" | 50 | 1937-1987 | 440 | | | | | 0.1 | Flow regulated since 1935 by Beulah Reservoir (station 13217000). Diversions for irrigation upstream from station. Low flow statistics uncertain due to excessive zero events. |
| 13220000 | Malheur River at Little Valley, near Hope, OR | 43°53'08" | 117°30'25" | 29 | 1951-1979 | 3,010 | | | | | 23 | Flow regulated by Warmsprings and Beulah Reservoirs. Vale-Oregon Canal diverted 163,700 acre-ft upstream from station at Namorf in sec 31, T.20 S., R.41 E., for supplying Bully Creek Reservoir and for irrigation. Many small diversions for irrigation upstream from station. |
| 13220500 | Malheur River near Hope, OR | 43°56'40" | 117°28'50" | 13 | 1937-1949 | 3,030 | | | | | 27 | Since 1930, Vale-Oregon Canal has diverted at Namorf for irrigation upstream and downstream from station. Many small diversions for irrigation upstream from station. Flow regulated by Warmsprings Reservoir and, since December 1935, by Agency Valley Reservoir. |
| 13226500 | Bully Creek at Warmsprings, near Vale, OR | 44°01'10" | 117°27'35" | 21 | 1965-1985 | 539 | 0.0 | 1.1 | 17 | 5.00 | 0.7 | No regulation. Many diversions for irrigation upstream from station. |
| 13227000 | Bully Creek near Vale, OR | 43°57'30" | 117°20'30" | 24 | 1939-1962 | 570 | | | 17.8 | | 6.1 | Occasional fluctuations caused by releases from Vale-Oregon Canal which diverts water from Malheur River for irrigation of lands ; Considerable return flow at times enters Bully Creek upstream from station. Diversions for irrigation of about 7,000 acres upstream from station west of Vale. |

Notes 1 Number of values used to compute statistics.
2 Percent of drainage area.
3 Data from USGS Open-File Report 94-4069 and USGS Water-Resources Investigations Report 01-4093. See Idaho gage page for more complete reference listing.

# TABLE B: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR EASTERN OREGON

(Revised March 1, 2004)

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 – Annual Low and High Flow, and Instantaneous Peak Flow," USGS Water Resources Investigations Report 82-4078" Magnitude and Frequency of Floods in Eastern Oregon".)

**(See end pages for gages in adjacent states near the Oregon border)**

| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (NOTE 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | AREA OF LAKES AND PONDS, % (NOTE 2) | FOREST COVER, PERCENT (NOTE 2) | MEAN ANNUAL PRECIPITATION IN INCHES | SOILS INDEX | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | | |
| BURNT RIVER BASIN | | | | | | | | | | | | |
| 13269300 | N Fk Burnt River near Whitney, OR | 44°36'00" | 118°15'10" | 13 | 1966-1978 | 110 | 0.0 | 90.0 | 24 | 5.30 | 0.6 | Some regulations from irrigation and mining operations upstream. A transmountain diversion from headwaters of Middle Fork John Day River delivers as much as 12 $ft^3/s$ to North Fork Burnt River upstream from station. |
| 13270800 | S Fk Burnt River above Barney Creek, near Unity, OR | 44°24'25" | 118°18'01" | 18 | 1964-1981 | 38.5 | 0.0 | 92.0 | 23 | 6.90 | 18 | No regulation or diversion upstream from station. |
| 13273000 | Burnt River near Hereford, OR | 44°30'14" | 118°10'35" | 48 | 1940-1987 | 309 | | | | | 2.2 | Flow regulated since 1938 by Unity Reservoir (station 13272500). Diversions for irrigation upstream from station. |
| 13274200 | Burnt River near Bridgeport, OR | 44°32'27" | 117°41'10" | 23 | 1958-1980 | 650 | | | | | 17 | Flow regulated since 1938 by Unity Reservoir. Many diversions for irrigation upstream from station. |
| 13275000 | Burnt River at Huntington, OR | 44°21'30" | 117°16'20" | 19 | 1958-1980 | 1,093 | | | | | 30 | Flow regulated since 1938 by Unity Reservoir. Diversions for irrigation upstream from station. |
| POWDER RIVER BASIN | | | | | | | | | | | | |
| 13275300 | Powder River near Sumpter, OR | 44°40'20" | 117°59'40" | 19 | 1969-1987 | 168 | | | | | 5.6 | Flow completely regulated since Oct. 31, 1967, by Phillips Lake, active capacity, 90,540 acre-ft. Many small diversions for irrigation upstream from station. |
| 13275500 | Powder River near Baker, OR | 44°39'20" | 117°52'30" | 48 | 1905-1967 | 219 | 0.0 | 74.7 | 25 | 5.8 | 5.1 | Flow regulated since Oct. 31, 1967, by Phillips Lake (active capacity, 90,540 acres-ft). Many small diversions for irrigation upstream from station. At times Auburn ditch diverts water into basin upstream from station. Discharge Before Phillips Lake Regulation |
| 13286700 | Powder River near Richland, OR | 44°46'40" | 117°17'30" | 19 | 1969-1987 | 1,310 | | | | | 31 | Flow regulated by Phillips Lake since October 1967, usable capacity, 90,540 acre-ft., Wolf Creek Reservoir since April 1975, usable capacity, 10,400 acre-ft., Thief Valley Reservoir since February 1932, usable capacity, 5,560 acre-ft. Diversions for irrigation upstream and downstream from station. Low flow discharge represents conditions with Phillips Dam regulation. |
| 13288200 | Eagle Creek above Skull Creek, near New Bridge, OR | 44°52'50" | 117°15'10" | 29 | 1959-1987 | 156 | 0.2 | 59.2 | 45 | 3.40 | 71 | No regulation. Some diversions upstream from station for irrigation and one small interbasin diversion for irrigation supply. All diversions are small compared to flow at station during irrigation season. |
| 13289500 | Powder River near Robinette, OR | 44°46'10" | 117°04'10" | 28 | 1930-1957- | 1,660 | | | | | 73 | Flow partly regulated by several reservoirs, the largest being Thief Valley Reservoir (capacity 17,000 acres-ft). Many diversions for irrigation upstream from station. One Canal with capacity of about 5 $ft^3/s$ diverts around station on left bank. |
| PINE CREEK BASIN | | | | | | | | | | | | |
| 13290190 (NOTE 3) | Pine Creek near Oxbow, OR | 44°57'13" | 116°52'21" | | 1968-1991 | 230 | | 50.2 | 33.7 | | 35 | Diversions above station for irrigation of about 19,000 acres (1966 determination). |
| IMNAHA RIVER BASIN | | | | | | | | | | | | |
| 13292000 | Imnaha River at Imnaha, OR | 45°33'45" | 116°50'00" | 58 | 1930-1987 | 622 | 0.0 | 50.8 | 28 | 3.70 | 99 | No regulation. Diversions for irrigation upstream from station. Water is diverted from Big Sheep Creek and tributaries upstream from station for irrigation in Wallowa River Basin. |
| GRANDE RONDE RIVER BASIN | | | | | | | | | | | | |
| 13318500 | Grande Ronde River near Hilgard, OR | 45°19'06" | 118°16'15" | 19 | 1938-1956 | 505 | 0.0 | 86.0 | 24 | 5.90 | 15 | Several small diversions for irrigation upstream from station. Since 1909, City of La Grande has diverted about 3 $ft^3/s$ for municipal use at Beaver Creek Reservoir (capacity, about 900 acres-ft). |
| 13318800 | Grande Ronde River at Hilgard, OR | 45°20'21" | 118°14'35" | 14 | 1968-1981 | 555 | | | | | 21 | Slight regulation by City of La Grande Reservoir on Beaver Creek, capacity, about 900 acre-ft. Diversions for irrigation upstream from station. Since 1909, City of La Grande has diverted about 3 $ft^3/s$ from Beaver Creek upstream from station for municipal use. |
| 13319000 | Grande Ronde River at La Grande, OR | 45°20'47" | 118°07'26" | 66 | 1919-1987 | 678 | 0.0 | 84.5 | 24 | 5.60 | 18 | Since 1915, slight regulation by City of La Grande reservoir on Beaver Creek, capacity, about 900 acre-ft. Diversions for irrigation upstream from station. Since 1909, City of La Grande has diverted about 3 $ft^3/s$ from Beaver Creek upstream from station for domestic water supply. |
| 13320000 | Catherine Creek near Union, OR | 45°09'20" | 117°46'26" | 51 | 1937-1987 | 105 | 0.0 | 88.3 | 25 | 2.80 | 22 | No regulation. Several small diversions for irrigation upstream from station. Since 1937, diversion to Big Creek in Powder River basin provides a small part of the water used for irrigation in that basin. |
| 13323500 | Grande Ronde River near Elgin, OR | 45°30'45" | 117°55'35" | 25 | 1957-1981 | 1,250 | 0.1 | 62.5 | 23 | 4.20 | 13 | No regulation. Many diversions for irrigation upstream from station. |
| 13323600 | Indian Creek near Imbler, OR | 45°26'00" | 117°49'20" | 12 | 1939-1950 | 22 | 0.0 | 98.7 | 32 | 4.60 | 3.1 | No diversion or regulation upstream from station. |
| 13325001 | E Fk Wallowa River near Joseph, OR | 45°16'20" | 117°12'35" | 30 | 1954-1983 | 10.3 | | | | | 10 | All records present herein include flow in Wallowa Falls powerplant tailrace of Pacific Power & Light CO. Most of low flow is diverted at dam 1.5 mi upstream into a conduit 1.0 mi above Wallowa Falls Powerhouse and discharged into West Fork 0.4 mi below powerhouse. |
| 13327500 | Wallowa River at Joseph, OR | 45°20'15" | 117°13'35" | 58 | 1905-1987 | 50.9 | | | | | 14 | Silver Lake ditch, diverts at Wallowa Lake dam for irrigation northeast of Joseph. City of Joseph diverts less than 1.0 $ft^3/s$ from Wallowa Lake for municipal use. |
| 13329500 | Hurricane Creek near Joseph, OR | 45°20'15" | 117°17'30" | 53 | 1926-1978 | 29.6 | 0.2 | 53.0 | 43 | 2.30 | 14 | No diversion or regulation upstream from station. |

Notes 1 Number of values used to compute statistics.
2 Percent of drainage area.
3 Data from USGS Open-File Report 94-4069 and USGS Water-Resources Investigations Report 01-4093. See Idaho gage page for more complete reference listing.

# TABLE B: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR EASTERN OREGON

(Revised March 1, 2004)

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 – Annual Low and High Flow, and Instantaneous Peak Flow," USGS Water Resources Investigations Report 82-4078" Magnitude and Frequency of Floods in Eastern Oregon".)

**(See end pages for gages in adjacent states near the Oregon border)**

| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (NOTE 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | AREA OF LAKES AND PONDS, % (NOTE 2) | FOREST COVER, PERCENT (NOTE 2) | MEAN ANNUAL PRECIPITATION IN INCHES | SOILS INDEX | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | | |
| GRANDE RONDE RIVER BASIN (cont) | | | | | | | | | | | | |
| 13330000 | Lostine River near Lostine, OR | 45°26'20" | 117°25'35" | 62 | 1914-1987 | 70.9 | 0.4 | 80.3 | 38 | 2.80 | 26 | Minam Lake Reservoir, capacity 440 acre-ft, has stored and diverted flow from Minam River since 1917 for irrigation in Lostine River basin. Diversions for irrigation upstream from station. |
| 13330500 | Bear Creek near Wallowa, OR | 45°31'37" | 117°33'05" | 61 | 1925-1985 | 68 | 0.0 | 84.8 | 38 | 2.50 | 9.7 | No regulation. Diversion for irrigation upstream from station. Water for irrigation in Lostine River basin diverted from Little Bear Creek, a tributary upstream from station, in sec. 32, T.1 S., R.43 E. |
| 13331500 | Minam River at Minam, OR | 45°37'12" | 117°43'32" | 22 | 1914-1987 | 240 | 0.7 | 80.0 | 40 | 1.60 | 65 | No regulation. Minam Lake, capacity 440 acre-ft, has stored and diverted flow from Minam River since 1917 for irrigation in Lostine River basin. |
| 13332500 | Grande Ronde River at Rondowa, OR | 45°43'36" | 117°46'59" | 60 | 1982-1987 | 2,555 | 0.2 | 60.5 | 26 | 3.90 | 416 | Flow slightly regulated by Wallowa Lake (station 13326000) and small reservoirs. Diversions for irrigation upstream from station, chiefly in vicinity of La Grande, Enterprise, and Wallowa; one transbasin diversion from Sheep Creek in Imnaha River basin for irrigation in Wallowa Valley. |
| 13333000 | Grande Ronde River at Troy, OR | 45°56'47" | 117°26'54" | 42 | 1946-1987 | 3,275 | 0.1 | 66.2 | 25 | 4.30 | 661 | Flow slightly regulated by Wallowa Lake (station 13326000) and small reservoirs. Diversions for irrigation upstream from station, chiefly in vicinity of La Grande, Enterprise, and Wallowa; one transbasin diversion from Big Sheep Creek and tributaries in Imnaha River basin for irrigation in Wallowa Valley. |
| WALLA WALLA RIVER BASIN | | | | | | | | | | | | |
| 14010000 | S Fk Walla Walla River near Milton-Freewater, OR | 45°49'48" | 118°10'08" | 64 | 1908-1987 | 63 | 0.0 | 86.3 | 33 | 5.60 | 100 | No regulation or diversion upstream from station. |
| 14010500 | S Fk Walla Walla River below Pacific Power & Light Co.'s plant, near Milton, OR | 45°53'00" | 118°16'25" | 17 | 1905-1945 | 80 | | | | | 83 | Small diversion for irrigation of about 300 acres upstream from station. Since 1905, some diurnal fluctuation caused by powerplant upstream from station. |
| 14010800 | N Fk Walla Walla River near Milton-Freewater, OR | 45°53'06" | 118°11'06" | 17 | 1971-1987 | 34.4 | 0.0 | 75.0 | 34 | 3.70 | 6.8 | No regulation; one diversion upstream from station. |
| 14011000 | N Fk Walla Walla River near Milton, OR | 45°54'08" | 118°16'55" | 39 | 1931-1969 | 43.8 | 0.0 | 67.1 | 31 | 3.40 | 2.5 | No regulation. Diversions upstream from station for irrigation. |
| UMATILLA RIVER BASIN | | | | | | | | | | | | |
| 14020000 | Umatilla River above Meacham Creek, near Gibbon, OR | 45°43'11" | 118°19'20" | 54 | 1934-1987 | 131 | 0.0 | 68.7 | 19 | 5.60 | 43 | No regulation or diversion upstream from station. |
| 14020300 | Meacham Creek at Gibbon, OR | 45°41'20" | 118°21'20" | 11 | 1977-1987 | 176 | | | | | 9.5 | No regulation or diversion upstream from station. |
| 14021000 | Umatilla River at Pendleton, OR | 45°40'20" | 118°47'30" | 52 | 1936-1987 | 637 | 0.0 | 37.4 | 18 | 3.30 | 31 | No regulation. Many diversions for irrigation upstream from station. |
| 14022000 | Umatilla River above McKay Creek, near Pendleton, OR | 45°40'20" | 118°50'00" | 10 | 1952-1934 | 700 | | | | | 20 | Many diversions for irrigation upstream from station. Slight regulation caused by mills upstream. |
| 14022200 | N Fk McKay Creek near Pilot Rock, OR | 45°30'24" | 118°36'57" | 13 | 1975-1987 | 48.6 | | | | | 0.7 | No regulation or diversion upstream from station. |
| 14022500 | McKay Creek near Pilot Rock, OR | 45°32'57" | 118°46'24" | 58 | 1928-1987 | 180 | 0.0 | 39.3 | 19 | 2.30 | 0.1 | No regulation. Many small diversions for irrigation upstream from station. |
| 14025000 | Birch Creek at Rieth, OR | 45°39'10" | 118°52'45" | 42 | 1931-1976 | 291 | 0.0 | 18.8 | 18 | 1.90 | 0.0 | No regulation. Many diversions for irrigation upstream from station. |
| 14026000 | Umatilla River at Yoakum, OR | 45°40'38" | 119°02'09" | 22 | 1905-1926 | 1,280 | 0.0 | 29.1 | 17 | 2.80 | 27 | Slight regulation by Furnish Reservoir, capacity 3,900 acre-ft, beginning in 1910 and continuing until 1934 when reservoir filled with silt. Flow regulated to some extent since 1927 by McKay Reservoir (station 14023000). Many diversions for irrigation upstream from station. |
| 14026000 | Umatilla River at Yoakum, OR | 45°40'38" | 119°02'09" | 59 | 1929-1987 | 1,280 | 0.0 | 29.1 | 17 | 2.80 | 53 | |
| 14032000 | Butter Creek near Pine City, OR | 45°32'48" | 119°18'14" | 52 | 1932-1987 | 291 | 0.0 | 10.7 | 15 | 2.30 | 0.2 | No regulation. Several small diversions for irrigation upstream from station. Water is diverted into headwaters of Butter Creek from Fivemile Creek, a tributary of Camas Creek in John Day River basin, for irrigation downstream from station. |
| 14033500 | Umatilla River near Umatilla, OR | 45°54'11" | 119°19'33" | 26 | 1962-1987 | 2,290 | | | | | 2.0 | Some regulation since 1927 by McKay Reservoir (station 14023000). Many diversions upstream from station for irrigation of lands upstream and downstream from station; Brownell Canal diverts downstream from station. Diversions since 1908 to Cold Springs Reservoir, on off-channel reservoir, capacity. 52,380 acres-ft. |
| WILLOW CREEK BASIN | | | | | | | | | | | | |
| 14034500 | Willow Creek at Heppner, OR | 45°21'02" | 119°32'56" | 30 | 1953-1982 | 96.8 | 0.0 | 24.8 | 16 | 2.50 | 0.1 | Flow regulated by Willow Creek Lake, 00.2 mi upstream, since Feb. 16, 1983. Many diversions for irrigation upstream from station. Part of flow of Ditch Creek (John Day basin) is diverted to Willow Creek upstream from station. |
| 14034800 | Rhea Creek near Heppner, OR | 45°15'46" | 119°36'51" | 26 | 1962-1987 | 120 | 0.0 | 30.0 | 16 | 2.00 | 0.1 | No regulation. Many diversions for irrigation upstream from station. |

Notes 1 Number of values used to compute statistics.
2 Percent of drainage area.

# TABLE B: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR EASTERN OREGON

(Revised March 1, 2004)

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|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | | |
| WILLOW CREEK BASIN (cont) | | | | | | | | | | | | |
| 14036000 | Willow Creek near Arlington, OR | 45°45'12" | 120°00'35" | 18 | 1962-1979 | 850 | 0.0 | 6.8 | 14 | 2.30 | 0.0 | No regulation. Many diversions for irrigation upstream from station. |
| JOHN DAY RIVER BASIN | | | | | | | | | | | | |
| 14037500 | Strawberry Creek above Slide Creek, near Prairie City, OR | 44°20'30" | 118°39'20" | 56 | 1932-1987 | 7 | 1.1 | 67.1 | 37 | 5.60 | 2.1 | Flow affected by natural storage in Strawberry Lake. No diversion upstream from station. |
| 14038500 | John Day River at Prairie City, OR | 44°27'15" | 118°43'00" | 23 | 1928-1951 | 231 | | | | | 9.6 | No regulation. Several diversions upstream including Prairie Power Canal, (not used for power since February 1952) which diverts upstream from station in SE ¼ sec.7, T.13 S., R.34 E., for irrigation upstream and downstream from station. |
| 14038500 | John Day River at Prairie City, OR | 44°27'15" | 118°43'00" | 15 | 1954-1968 | | | | | | 17 | |
| 14038530 | John Day River near John Day, OR | 44°25'07" | 118°54'19" | 18 | 1970-1987 | 386 | 0.0 | 55.0 | 25 | 1.50 | 16 | No regulation upstream. Many diversions upstream from station for irrigation. |
| 14040500 | John Day River at Picture Gorge, near Dayville, OR | 44°31'15" | 119°37'30" | 60 | 1928-1987 | 1,680 | 0.0 | 48.3 | 22 | 5.20 | 17 | No regulation. Many diversions for irrigation upstream from station. |
| 14041500 | N Fk John Day River near Dale, OR | 44°59'55" | 118°56'25" | 28 | 1931-1958 | 525 | 0.1 | 97.0 | 27 | 4.70 | 40 | Flow regulated by Olive Lake (capacity, about 5,500 acre-ft) and Upper Reservoir on Lake Creek (capacity, about 700 acre-ft). Some diurnal fluctuation at low flow caused by logging operations upstream from station. Several small diversions for irrigation and mining upstream from station. Since 1865 water has been diverted upstream from station at times to North Fork Burnt River. |
| 14042000 | Camas Creek near Lehman, OR | 45°10'16" | 118°43'53" | 19 | 1952-1970 | 60.7 | 0.0 | 73.8 | 24 | 2.70 | 0.9 | No regulation. A few small diversions for irrigation upstream from station. |
| 14042500 | Camas Creek near Ukiah, OR | 45°09'25" | 118°49'10" | 51 | 1916-1987 | 121 | 0.0 | 77.7 | 24 | 2.90 | 3.2 | No regulation. Diversions for irrigation upstream from station. |
| 14044000 | Middle Fk John Day River at Ritter, OR | 44°53'20" | 119°08'25" | 57 | 1931-1987 | 515 | 0.0 | 80.5 | 23 | 4.60 | 17 | No regulation. Diversions for irrigation upstream from station. |
| 14044500 | Fox Creek at Gorge, near Fox, OR | 44°37'30" | 119°15'10" | 27 | 1932-1958 | 90.2 | 0.0 | 40.0 | 21 | 3.50 | 0.0 | Diversions for irrigation of 4,800 acres upstream from station. |
| 14046000 | N Fk John Day River at Monument, OR | 44°48'50" | 119°25'20" | 58 | 1930-1987 | 2,520 | 0.0 | 70.1 | 22 | 4.90 | 76 | Very slight regulation by small reservoirs upstream. Many small diversions for irrigation upstream from station. |
| 14046500 | John Day River at Service Creek, OR | 44°47'38" | 120°00'50" | 57 | 1931-1987 | 5,090 | 0.0 | 57.3 | 21 | 4.90 | 95 | Slight regulation by several small reservoirs upstream from station. Many small diversions for irrigation upstream from station. |
| 14047390 | Rock Creek above Whyte Park, near Condon, OR | 45°15'53" | 120°01'15" | 11 | 1977-1987 | 297 | | | | | 0.7 | No regulation or diversion upstream from station. |
| 14048000 | John Day River at McDonald Ferry, OR | 45°35'16" | 120°24'30" | 82 | 1906-1987 | 7,580 | 0.0 | 41.8 | 19 | 4.20 | 96 | No regulation. Many diversions for irrigation upstream from station. |
| DESCHUTES RIVER BASIN | | | | | | | | | | | | |
| 14050000 | Deschutes River below Snow Creek, near La Pine, OR | 43°48'51" | 121°46'33 " | 49 | 1939-1987 | 132 | 1.4 | 74.0 | 49 | 11.90 | 84 | No regulation. Crater Creek Canal diverts water to Tumalo Creek basin from tributaries of Soda Creek. Stream is spring fed and peak discharge may occur several months after the precipitation which caused it. Hydrologic drainage boundary uncertain because of interbasin ground-water exchange. |
| 14050500 | Cultus River above Cultus Creek, near La Pine, OR | 43°49'06" | 121°47'40" | 51 | 1924-1987 | 16.5 | 1.0 | 98.2 | 40 | 9.60 | 44 | No regulation or diversions upstream from station. Hydrologic drainage boundary uncertain because of interbasin ground-water exchange. |
| 14051000 | Cultus Creek above Crane Prairie Reservoir, near La Pine, OR | 43°49'17" | 121°49'22" | 23 | 1939-1961 | 33.2 | 7.2 | 92.2 | 45 | 13.80 | 0.1 | Some regulation by fish screens at Cultus Lake since 1962. No diversion upstream from station. Hydrologic boundary uncertain because of interbasin ground-water exchange. |
| 14051000 | Cultus Creek above Crane Prairie Reservoir, near La Pine, OR | 43°49'17" | 121°49'22" | 24 | 1964-1987 | 33.2 | 7.2 | 92.2 | 45 | 13.80 | 0.1 | |
| 14052000 | Deer Creek above Crane Prairie Reservoir, near La Pine, OR | 43°48'48" | 121°50'18" | 31 | 1957-1987 | 21.5 | 2.8 | 94.4 | 31 | 13.80 | 0.0 | No regulation or diversion upstream from station. Hydrologic drainage boundary uncertain because of interbasin ground-water exchange. |
| 14052500 | Quinn River near La Pine, OR | 43°47'03" | 121°50'06" | 49 | 1939-1987 | | | | | | 15 | Drainage area indeterminate, normal flow is entirely from springs 150 feet upstream. No regulation or diversion upstream from station. |
| 14054000 | Deschutes River below Crane Prairie Reservoir, near La Pine, OR | 43°45'13" | 121°46'57" | 64 | 1924-1987 | 254 | | | | | 32 | Flow regulated since 1922 by Crane Prairie Reservoir (station 14053500). No diversion upstream from station. Hydrologic drainage boundary uncertain because of interbasin ground-water exchange. |
| 14054500 | Brown Creek near La Pine, OR | 43°42'57" | 121°48'10" | 50 | 1924-1987 | 21 | 0.2 | 99.2 | 35 | 13.80 | 30 | No regulation. No diversion upstream from station. Hydrologic drainage uncertain because of interbasin ground-water exchange. |

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|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | | |
| DESCHUTES RIVER BASIN (cont) | | | | | | | | | | | | |
| 14055500 | Odell Creek near Crescent, OR | 43°32'51" | 121°57'41" | 42 | 1935-1976 | 39 | 14.1 | 83.3 | 60 | 13.80 | 31 | Flow affected occasionally in winter by ice jams at outlet of Odell Lake, and slightly affected at times by seiches in Odell Lake. No diversion upstream from station. |
| 14056500 | Deschutes River below Wickiup Reservoir, near La Pine, OR | 43°41'10" | 121°41'13" | 44 | 1944-1987 | 483 | | | | | 38 | Flow regulated by Crane Prairie Reservoir (station 14053500), and since 1942 by Wickiup Reservoir (station 14056000). Some leakage from Crane Prairie and Wickiup Reservoirs does not pass station. Some spill bypassed station in 1955. Crater Creek canal diverts water upstream from station to Tumalo Creek basin. Hydrologic drainage boundary uncertain because of interbasin ground-water exchange. |
| 14057000 | Deschutes River at Pringle Falls, near La Pine, OR | 43°44'20" | 121°36'50" | 18 | 1924-1941 | 507 | | | | | 550 | Flow regulated since 1922 by Crane Prairie Reservoir and since 1942 by Wickiup Reservoir. Crater Creek Canal diverts water above station to Tumalo Creek basin. Hydrologic drainage uncertain because of interbasin ground-water exchange. |
| 140507000 | Deschutes River at Pringle Falls, near La Pine, OR | 43°44'20" | 121°36'50" | 17 | 1944-1960 | 507 | | | | | 69 | |
| 14057500 | Fall River near La Pine, OR | 43°47'48" | 121°34'18" | 48 | 1940-1987 | 45.1 | 0.0 | 98.2 | 36 | 13.10 | 134 | Diversion only to ponds at fish hatchery 50 ft upstream from station, from which water returns to river upstream from station. Stream is spring fed and momentary extremes are caused by operation of fish hatchery. Hydrologic drainage boundary uncertain because of interbasin ground-water exchange. |
| 14060000 | Crescent Creek at Crescent Lake, near Crescent, OR | 43°30'11" | 121°58'20" | 58 | 1930-1987 | 60.7 | | | | | 1.4 | Flow regulated since 1922 by Crescent Lake (station 14059500). No diversion upstream from station. Hydrologic drainage uncertain because of interbasin ground-water exchange. |
| 14063000 | Little Deschutes River near La Pine, OR | 43°41'21" | 121°30'06" | 62 | 1926-1987 | 859 | | | | | 53 | Flow regulated since 1922 by Crescent Lake (station 14059500). Many diversions for irrigation upstream from station. Hydrologic drainage uncertain because of interbasin ground-water exchange. |
| 14064500 | Deschutes River at Benham Falls, near Bend, OR | 43°55'49" | 121°24'39" | 17 | 1926-1942 | 1,759 | | | | | 917 | Flow regulated by Crane Prairie Reservoir, Crescent Lake, and Wickiup Reservoir. Many diversions for irrigation near Bend. |
| 14064500 | Deschutes River at Benham Falls, near Bend. OR | 43°55'49" | 121°24'39" | 17 | 1945-1987 | 1,759 | | | | | 636 | |
| 14066000 | Deschutes River below Lava Island, near Bend, OR | 44°00'00" | 121°22'30" | 14 | 1927-1941 | 1,829 | | | | | 815 | Flow regulated by Crescent Lake and Crane Prairie Reservoir and, since 1942 by Wickiup Reservoir. Small Diversions for irrigation upstream from station. Arnold Canal diverts upstream from station for irrigation near Bend. |
| 14066000 | Deschutes River below Lava Island, near Bend, OR | 44°00'00" | 121°22'30" | 22 | 1944-1965 | 1,829 | | | | | 621 | |
| 14070500 | Deschutes River below Bend, OR | 44°04'59" | 121°18'24" | 30 | 1958-1987 | 1,899 | | | | | 28 | Flow regulated by powerplant at Bend, Crescent Lake, Crane Prairie Reservoir, and Wickiup Reservoir. Six large canals and several small ditches divert water upstream from station for irrigation. |
| 14073001 | Tumalo Creek near Bend, OR | 44°05'16" | 121°22'18" | 63 | 1925-1987 | 57.8 (see remarks) | 0.1 | 60.9 | 20 | 15.8 | 50 | All records given herein include flow in Columbia Southern Canal, which diverts 8 mi upstream from station for irrigation of land near Tumalo. Crater Creek Canal diverts flow of tributaries of Soda Creek into head of Tumalo Creek. Diversion upstream from station for municipal supply of Bend since Dec. 15, 1926. |
| 14075000 | Squaw Creek near Sisters, OR | 44°14'02" | 121°33'57" | 72 | 1908-1987 | 45.2 | 0.2 | 77.8 | 48 | 7.00 | 41 | No regulation. A canal near mouth of Pole Creek has diverted the entire flow of that creek since 1885. Prior to Oct. 1, 1982, drainage area of 57.8 $mi^2$ included that of Pole Creek. Water is diverted from Snow Creek, a tributary upstream from station, for irrigation in Three Creek basin. |
| 14076500 | Deschutes River near Culver, OR | 44°29'56" | 121°19'12" | 34 | 1954-1987 | 2,705 | | | | | 492 | Flow regulated by Crescent Lake and Crane Prairie and Wickiup Reservoirs. Many diversions for irrigation upstream from station. |
| 14077500 | N Fk Beaver Creek near Paulina, OR | 44°10'00" | 119°44'00" | 11 | 1944-1954 | 64.4 | 0.0 | 42.9 | 20 | 8.40 | 0.2 | Several small reservoirs upstream from station store water for irrigation and stock watering. Most of summer flow is directed for irrigation of 1,000 acres upstream from station. |
| 14078500 | N Fk Crooked River above Deep Creek, OR | 44°19'55" | 120°04'55" | 10 | 1945-1954 | 159 | 0.0 | 70.7 | 21 | 7.00 | 1.1 | No regulation. Several diversions for irrigation of about 3,600 acres upstream from station. |
| 14079500 | Crooked River near Post, OR | 44°07'00" | 120°15'00" | 24 | 1942-1972 | 2,160 | 0.3 | 22.6 | 20 | 5.50 | 9.8 | No regulation. Many diversions for irrigation upstream from station. Approximately 500 $mi^2$ of drainage area is probably noncontributing. |
| 14078000 | Beaver Creek near Paulina, OR | 44°09'50" | 119°55'20" | 30 | 1946-1975 | 450 | 0.4 | 21.2 | 20 | 2.50 | 0.5 | No regulation. Diversions for irrigation upstream from station. Two small ditches divert upstream from station for irrigation below. |
| 14080500 | Crooked River near Prineville, OR | 44°06'50" | 120°47'40" | 17 | 1943-1959 | 2,700 | 0.3 | 26.0 | 20 | 5.10 | 4.9 | Flow completely regulated since December 1960 by Prineville Reservoir (station 14080400). Diversions for irrigation upstream from station. Discharge not adjusted for storage or release from Prineville Reservoir as evaporation from reservoir at times exceeds natural flow. Approximately 500 $mi^2$ of drainage area is probably non-contributing. |
| 14080500 | Crooked River near Prineville, OR | 44°06'50" | 120°47'40" | 26 | 1943-1959 | 2,700 | | | | | 25 | |
| 14087400 | Crooked River below Opal Springs, near Culver, OR | 44°29'33" | 121°17'50" | 25 | 1963-1987 | 4,300 | | | | | 25 | Flow regulated since December 1960 by Prineville Reservoir (station 14080400) and Ochoco Reservoir, capacity 47,500 acres-ft. Dam and powerplant 500 ft upstream, completed in 1985, causes brief fluctuations in flow. Many diversions for irrigation upstream from station. Practically all of the summer flow comes from Opal Springs and other springs within 15 mi upstream from station. Simultaneous records (1961-63) at former gaging station 5.6 mi downstream indicated over 15 percent increase to summer flow from springs downstream from this station. Approximately 500 $mi^2$ of drainage area is probably non-contributing. |

Notes 1 Number of values used to compute statistics.
2 Percent of drainage area.

# TABLE B: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR EASTERN OREGON

(Revised March 1, 2004)

(Data for Oregon from USGS Open-File Report 93-63 "Statistical Summaries of Streamflow Data in Oregon: Volume 2 – Annual Low and High Flow, and Instantaneous Peak Flow," USGS Water Resources Investigations Report 82-4078" Magnitude and Frequency of Floods in Eastern Oregon".)

**(See end pages for gages in adjacent states near the Oregon border)**

| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (NOTE 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | AREA OF LAKES AND PONDS, % (NOTE 2) | FOREST COVER, PERCENT (NOTE 2) | MEAN ANNUAL PRECIPITATION IN INCHES | SOILS INDEX | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | | |
| DESCHUTES RIVER BASIN (cont) | | | | | | | | | | | | |
| 14087500 | Crooked River near Culver, OR | 44°33'40" | 121°16'10" | 15 | 1946-1960 | 4,330 | | | | | 1310 | Flow slightly regulated (since 1919) by Ochoco Reservoir (capacity, 47,500 acres-ft) and since December 1960, by Prineville Reservoir (capacity 152,800 acre-ft); occasional diurnal fluctuation caused by powerplant 1.2 mi upstream from station. Water is diverted for irrigation of land upstream from station. The area served increased from about 30,000 acres in 1918 to 37,000 acres in 1946. Several hundred cubic feet per second of water diverted from Deschutes River for irrigation of other lands upstream from station. Opal Springs and several other springs within about 17 mi upstream from station contribute about 1,000 $ft^3/s$ to flow. Approximately 500 $mi^2$ of drainage area is probably non-contributing. |
| 14088000 | Lake Creek near Sisters, OR | 44°25'35" | 121°43'30" | 68 | 1919-1987 | 22.2 | 4.9 | 93.7 | 65 | 13.80 | 28 | Flow occasionally regulated by Suttle Lake 150 ft upstream from station. |
| 14091500 | Metolius River near Grandview, OR | 44°37'33" | 121°28'55" | 66 | 1913-1987 | 316 | 0.7 | 91.4 | 68 | 4.70 | 1290 | No Regulation. Many small diversions for irrigation upstream from station. Stream is spring fed. Hydrologic drainage boundary uncertain because of interbasin ground-water exchange. |
| 14092500 | Deschutes River near Madras, OR | 44°43'34" | 121°14'45" | 31 | 1926-1956 | 7,820 | | | | | 3420 | Diurnal fluctuation caused by Lake Simtuatus and reregulating reservoir since 1957, combined capacity for normal operation, 6,500 acre-ft. Some winter and spring runoff stored in Ochoco Reservoir, capacity, 47,500 acre-ft, in Crescent Lake, Crane Prairie and Wickiup Reservoirs, combined capacity, 354,600 acre-ft, and since 1960, in Prineville Reservoir, capacity, 152,800 acre-ft, and since 1964, in Lake Billy Chinook, capacity, 534,700 acre-ft. Large diversions in upper basin for irrigation. |
| 14092500 | Deschutes River near Madras, OR | 44°43'34" | 121°14'45" | 23 | 1926-1956 | 7,820 | | | | | 3,530 | |
| 14092885 | Shitike Creek below Wolford Canyon, near Warm Springs, OR | 44°46'20" | 121°18'15" | 12 | 1976-1987 | 75.8 | | | | | 40 | No regulation. Some diversion for irrigation and Warm Springs water supply. |
| 14097100 | Warm Springs River near Kahneeta Hot Spring, OR | 44°51'24" | 121°08'55" | 14 | 1974-1987 | 526 | | | | | 244 | No regulation. Small diversions for irrigation upstream from station. |
| 14101500 | White River below Tygh Valley, OR | 45°14'30" | 121°05'38" | 69 | 1919-1987 | 417 | 0.2 | 69.7 | 45 | 2.20 | 110 | Diversions upstream from station for irrigation, and prior to 1963 for power generation. |
| 14103000 | Deschutes River at Moody, near Biggs, OR | 45°37'20" | 120°54'05" | 13 | 1899-1919 | 10,500 | | | | | 4,740 | Some fluctuation caused by regulation at Lake Simustus since 1957. Some winter and spring runoff stored in Ochoco Reservoir since 1919, capacity 46,420 acre-ft, in Crescent Lake, Crane Prairie since 11922, and Wickiup Reservoirs since 1944, combined capacity, 323,390 acre-ft, and since 1960, in Prineville Reservoir (station 14080400), and since 1964 in Lake Billy Chinook (station 14092100). Large diversions in upper river basin for irrigation. |
| 14103000 | Deschutes River at Moody, near Biggs, OR | 45°37'20" | 120°54'05" | 22 | 1966-1987 | 10,500 | | | | | 40,080 | |
| MOSIER CREEK BASIN | | | | | | | | | | | | |
| 14113200 | Mosier Creek near Mosier, OR | 45°38'55" | 121°22'35" | 17 | 1965-1981 | 41.5 | 0.3 | 91.0 | 28 | 1.50 | 1.2 | No regulation. Several small pumping diversions for irrigation upstream from station. |
| HOOD RIVER BASIN | | | | | | | | | | | | |
| 14113400 | Dog River near Parkdale, OR | 45°24'30" | 121°31'10" | 10 | 1962-1971 | 4.50 | 0.0 | 95.0 | 45 | 1.20 | 1.7 | No regulation or diversion upstream from station. |
| 14118500 | West Fork Hood River near Dee, OR | 45°35'55" | 121°38'05" | 54 | 1934-1987 | 95.6 | 0.6 | 85.5 | 100 | 3.20 | 135 | No regulation. Dee Irrigation District canal diverts from right bank about 6 mi upstream from station for irrigation upstream from station and in Middle Fork Basin. Diversions from Green Point Creek basin upstream from station for irrigation near Oak Grove; water from two of these diversions is carried in Hood River Irrigation District canal. |
| 14120000 | Hood River at Tucker Bridge, near Hood River, OR | 45°39'20" | 121°32'50" | 24 | 1899-1987 | 279 | 0.2 | 89.0 | 79 | 3.30 | 285 | Some daily fluctuation caused by diversion dam upstream from station and sawmill at Dee. Diversions for irrigation upstream from station. |

Notes 1 Number of values used to compute statistics.
2 Percent of drainage area.

# TABLE B: 2-YEAR 7-CONSECUTIVE DAY LOW FLOW DISCHARGES FOR EASTERN OREGON

(Revised March 1, 2004)

(Data for California supplied by USGS Sacramento Office)

(Data for Idaho from USGS Open-File Report 94-4069 "Statistical Summaries of Streamflow Data for Selected Gaging Stations in Idaho and Adjacent States Through September 1990 – Volume I: Gaging Station with 10 or More years of Record." and USGS Water-Resources Investigations Report 01-4093 "Estimating Monthly and Annual Streamflow Statistics at Ungaged Sites in Idaho.")

(Data for Washington from USGS Open-File Report 84-145A and 84-145B "Streamflow Statistics and Drainage-Basin Characteristics for the Southwestern and Eastern Regions, Washington".)

| BASIN | | LATITUDE IN DEG, MIN, AND SEC | LONGITUDE IN DEG, MIN, AND SEC | YEARS OF RECORD (NOTE 1) | PERIOD OF RECORD | DRAINAGE AREA IN SQUARE MILES | AREA OF LAKES AND PONDS, % (NOTE 2) | FOREST COVER, PERCENT (NOTE 2) | MEAN ANNUAL PRECIPITATION IN INCHES | SOILS INDEX | 2-YEAR - 7 DAY LOW FLOW IN CUBIC FEET PER SECOND | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION NUMBER | STATION NAME | | | | | | | | | | | |
| GAGES IN CALIFORNIA NEAR OREGON BORDER | | | | | | | | | | | | |
| 10360900 | Bidwell Creek below Mill Creek, near Fort Bidwell California | 41°52'57" | 120°10'26" | 22 | 1961-1982 | 25.6 | | | | | 3.4 | Less than 2 $ft^3/s$ diverted upstream for irrigation. No storage above station. |
| 11488700 | Dry Lake Tributary at Perez, California | 41°40'20" | 121°15'25" | 3 | 1963-1966 | 1.74 | 0.0 | 5.0 | 17 | -- | 0.0 | No storage or diversion above station. |
| 11517800 | Beaver Creek near Klamath River, California | 41°53'40" | 122°49'20" | 5 | 1960-1964 | 106 | | | | | 21 | Some small diversions for irrigation above station. |
| 11516900 | Little Shasta River near Montague, California | 41°45'11" | 122°17'42" | 21 | 1958-1978 | 48.2 | | | | | 3.2 | No regulation or diversion above station. |
| 11516600 | Cottonwood Creek at Hornbrook, California | 41°55'06" | 122°33'45" | 7 | 1965-1971 | 89.8 | | | | | 0.26 | Some diversion above station for irrigation. |
| 11521500 | Indian Creek near Happy Camp, California | 41°50'07" | 123°22'55" | 46 | 1958-2003 | 120 | | | | | 39 | Small diversion above station for irrigation. |
| GAGES IN IDAHO NEAR OREGON BORDER | | | | | | | | | | | | |
| 13250600 | Big Willow Creek near Emmett, ID | 44°04'25" | 116°29'10 | | 1963-1982 | 47.4 | | 4.8 | 15.9 | | 2.9 | None |
| 13267000 | Mann River near Weiser, ID | 44°23'30" | 116°53'40" | | 1912-1961 | 56 | | 55.4 | 22.1 | | 1.2 | One diversion above station for irrigation. |
| 13289960 | Wildhorse River at Brownlee Dam, ID | 44°51'08" | 116°53'41" | | 1980-1991 | 177 | | 62.2 | 27.5 | | 21 | Small diversions upstream for irrigation. |
| 13315500 | Mud Creek near Tamarack, ID | 45°00' | 115°21' | | 1947-1959 | 15.8 | | 93.0 | 35.4 | | 1.3 | No regulation or diversion above station. |
| 13316800 | North Fork Skookumchuck Creek near White Bird, ID | 45°43'34" | 116°12'16" | | 1962-1971 | 15.3 | | 69.3 | 30.2 | | 0.9 | Ditch bypassing station diverts from left bank 0.2 mi upstream from station. |
| GAGES IN WASHINGTON STATE NEAR COLUMBIA RIVER, BETWEEN BONNEVILLE DAM AND MCNARY DAM | | | | | | | | | | | | |
| 14121300 | White Salmon River below Cascades Creek near Trout Lake, WA | 46°06'06" | 121°36'14" | 20 | 1959-1978 | 32.4 | 0.00 | 77 | 106 | | 74.4 | Probably some small diversions above station for irrigation. No regulation. |
| 14121400 | White Salmon River above Trout Creek near Trout Lake, WA | 46°01'47" | 121°31'45" | 9 | 1961-1969 | 64.9 | | 83 | 97 | | 140.1 | No regulation or diversion above station. |
| 14121500 | Trout Lake Creek near Trout Lake, WA | 46°00'15" | 121°32'13" | 10 | 1911-1969 | 69.3 | 0.50 | 92 | 82 | | 39.7 | No regulation or diversion above station. |
| 14122000 | White Salmon River near Trout Lake, WA | 45°59'29" | 121°29'39" | 11 | 1930-1967 | 185 | | 84 | 82 | | 86.7 | No regulation. Diversions above for irrigation of about 3,100 acres. |
| 14127000 | Wind River above Trout Creek near Carson, WA | 45°48'31" | 121°54'27" | 24 | 1946-1979 | 108 | 3 | 98 | 103 | | 79.1 | Slight regulation by fish hatchery above station. Upstream diversions all returned to stream above station. |
| 14128000 | Panther Creek near Carson, WA | 45°48'00" | 121°52'00" | | | 30.1 | | 95 | 90 | | | No regulation or diversion above station. |
| 14128500 | Wind River near Carson, WA | 45°48'10" | 121°44'12" | 43 | 1934 | 225 | 3 | 52 | 99 | | 180.0 | Low flow occasionally affected by pondage at Forest Service powerplant on Trout Creek. No diversions above stations. |

Notes 1 Number of values used to compute statistics.
2 Percent of drainage area.

# APPENDIX J – MEAN DAILY EXCEEDANCE DISCHARGES

## 1.0 Introduction

The mean daily exceedance discharges for any month can be calculated by methods in this Appendix. These discharges are used in temporary water management and fish passage design. The discharges most often used are the 5, 10, 25, 50, or 95 percent exceedances.

**5 Percent Exceedance -** The mean daily discharge, during an average year, is expected to exceed this flow on 5 percent of the days during the specified month. This is 1 or 2 days a month. This discharge is used to design temporary water management facilities providing a low risk of failure. An example would be a roadway detour culvert on a major highway, where there would be considerable cost and inconvenience if the pipe would wash out or the road overtopped. This is the largest of the five exceedance flows.

**10 Percent Exceedance -** The mean daily discharge during an average year is expected to exceed this flow on 10 percent of the days during the specified month. This is 3 days a month. This discharge is used as a high flow design discharge in fish passage design, as discussed in the Oregon Department of Fish and Wildlife (ODFW) "Guidelines and Criteria for Stream-Road Crossings."

**25 Percent Exceedance -** The mean daily discharge, during an average year, is expected to exceed this flow on 25 percent of the days during the specified month. This is 7 or 8 days a month. This discharge is used to design temporary water management facilities having a normal risk of failure. An example would be a dam diverting water into a bypass pipe around a project. Occasional overtopping of the dam may be tolerable, and the riparian impacts of a larger structure may not be desired. This is also the typical discharge used to estimate facility sizes in engineer's cost estimates.

**50 Percent Exceedance -** The mean daily discharge, during an average year, is expected to exceed this flow on half of the days during the specified month. This is about 15 days a month. This discharge is used in cost estimates more often than design. For example, a pump may be selected that is capable of conveying the 5 or 25 percent exceedance discharge, but its estimated monthly fuel costs may be based on conveying the 50 percent exceedance flow.

**95 Percent Exceedance -** The mean daily discharge during an average year is expected to exceed this flow on 95 percent of the days during the specified month. This is 29 days a month. This discharge is occasionally used in temporary water management. It is most often used as low

flow design discharge in fish passage design, as discussed in the ODFW Guidelines. This is the smallest of the five exceedance flows.

The 95 percent exceedance is one of two low flow design discharges recommended in the Guidelines. The other is the 2-year, 7-consecutive day low flow. These two low flows are different in an important way. The 95 percent exceedance discharge can be calculated for any month of the year. The 2-year 7-consecutive day low flow represents discharges during the driest part of the year.

Exceedance discharges, both 10 and 95 percent, are often used for analysis of fish passage during migration because they can be calculated for specific months of the year. These are the only discharges that can be practically used to design installations on ephemeral streams with insufficient water for fish passage during the dry season.

## 2.0 Limitations of Method

Daily discharges are often extremely difficult to accurately predict using statistics from nearby watersheds. Local watershed characteristics can strongly influence the discharge magnitude, and these characteristics are often unique to the subject basin. In addition, there are often substantial differences in discharges from year-to-year due to variability in climate and water use. Some factors that influence daily flows are:

- climatic conditions such as rainfall, snowpack, and evaporation,
- the amount and type of vegetation in the riparian areas (vegetated riparian areas tend to sustain dry season flows),
- diversions to or from the subject stream,
- infiltration into exceptionally pervious rock formations or streambed materials,
- water entering the stream from springs and seeps,
- release or retention of streamflow in storage reservoirs, and
- consumptive uses such as livestock watering or irrigation.

A prediction method based on gage data can often overestimate daily flows in smaller watersheds. Almost all of these gages used to develop the statistics are on larger streams that flow throughout the year. As a consequence, discharge estimates based on gage statistics will often predict discharge throughout the year. This may not be the case in many smaller ephemeral watersheds.

The streams draining smaller basins may be dry through portions of the year for various reasons. In some instances, streamflow ceases during the winter when the upstream watershed freezes. In other cases, there is limited or no flow during portions of the dry season. Decreased summer flows can be caused by consumptive uses or diversions upstream, or a lack of runoff or water percolating into the stream from the aquifer.

Water use information and parol evidence is used to determine if streamflow ceases during portions of the year. This evidence is used to supplement the calculation results.

## 3.0 Exceedance Discharge Statistics

Exceedance statistics for Oregon gaged basins are available from many sources. Statistics for many basins have been calculated by the United States Geological Survey (USGS). The information is presented in the 1990 USGS Open-File Report 90-118 titled "Statistical Summaries of Streamflow Data in Oregon: Volume 1 – Monthly and Annual Streamflow, and Flow-Duration Values."

Exceedance statistics for Washington gaged basins near the Oregon border are in the USGS Open-File Reports 84-145-A and 84-145 B. The two reports are Volumes 1 and 2 of the 1985 report titled: "Streamflow Statistics and Drainage-Basin Characteristics for the Southwestern and Eastern Regions, Washington. Volume 1 covers southwestern Washington, and Volume 2 includes the eastern half of the state.

Exceedance statistics for stations in Idaho near the Oregon border are in the USGS Water-Resources Investigations Report 01-4093 titled "Estimating Monthly and Annual Streamflow Statistics at Ungaged Sites in Idaho." This report was published in 2001.

Exceedance statistics are also available on the Water Availability Report System (WARS) section of the OWRD website: http://www.wrd.state.or.us/. This is the most complete compilation of Oregon exceedance statistics.

Exceedance statistics selected specifically for ODOT low flow discharge estimates are summarized in publications on the ODOT Geo-Environmental Section's Engineering and Asset Management Unit website. These statistics were compiled by OWRD or ODOT using WARS. There are booklets for each of the eighteen OWRD administrative drainage basins in Oregon. Use of this data is recommended. Remarks are included to describe the utility of the data. It is essential to review the remarks to verify that the statistics for a gaged basin are suitable to estimate discharges in a nearby watershed. The Remarks also note if the statistics are based on natural or measured stream flows.

The distinction between these two discharges is discussed in the following sections.

### 3.1 Natural Stream Flow Exceedance Statistics

Natural stream exceedance flows are derived from statistical distributions of mean daily discharge measurements at gage sites. The initial distribution of the measured discharges is normalized to account for wet or dry climatic cycles that may have occurred during the gage

period of record. Further adjustments are made to account for upstream consumptive water use, storage, or diversions. The resulting natural stream flows are intended to represent discharges from the watershed in its natural prehistoric or predevelopment conditions. They can also be used to estimate natural discharges in other similar nearby watersheds. These adjustments are made by the OWRD. This process is discussed in detail in the August 2002 OWRD Open File Report SW 02-002 by Richard M. Cooper, P.E. titled "Determining Surface Water Availability in Oregon."

Upstream diversions, storage, or consumptive use need to be added or subtracted from the natural discharges to calculate discharges expected at the site. The remarks on the data tables, OWRD water use information, and/or parol evidence is used to estimate these additions or reductions.

### 3.2 Measured Stream Flow Exceedance Statistics

Measured stream exceedance flows are also derived from statistical distributions of mean daily discharge measurements at gages. These distributions are not normalized or adjusted. This data can be used to estimate discharges from nearby watersheds if natural flow data is not available. Care should be used when this is done. Measured stream flow statistics in the ODOT Mean Daily Flow Data books include the following.

- Exceedance statistics for gages on large and extensively regulated rivers such as the Columbia and Lower Willamette. This data is intended for use on these rivers near the gage locations. In most cases these statistics are based on long periods of record and they give good estimates of expected future conditions.

- Exceedance statistics for gages in neighboring states near the Oregon border. The selected sites are on streams with little or no upstream regulation, diversion, or consumptive use. Statistical distributions from these gages may be useful when sites near the state border are analyzed.

- Miscellaneous sites throughout the state. These distributions should be used with care. In many locations they are the best available data.

Storage, diversions, or consumptive uses also need to be considered when using measured stream flow data. This is done in a slightly different way than the procedure for natural stream flows. The differences in water use between the subject and nearby basins need to be added or subtracted from the discharges estimated from measured flow statistics. The remarks on the data tables, OWRD water use information, and/or parol evidence can used to estimate these additions or reductions.

## 4.0 Procedure

There are four parts to the exceedance discharge calculation procedure, and all are discussed in detail in the following sections. The first part is to determine natural or measured flow discharges for the desired exceedances. The next part is to determine discharge additions or reductions due to diversions or storage, and losses due to consumptive use. The last part is to use parol evidence to verify the calculated discharge is realistic, or to adjust the results of the calculations. In many instances the parol evidence will also provide information to be used in the second part. The fourth part is to report the discharges.

### 4.1 Estimating Exceedance Discharges Based on Natural or Measured Flow Statistics

The first part of the procedure is to calculate exceedance statistics for the subject watershed based on gaged basin data.

**Step 1 -** Outline the subject watershed on a map and measure the drainage area. Look in the appropriate ODOT Mean Daily Flow Data book to find gaged basins in the vicinity with similar watershed and climatic characteristics. The statistics from a gage in the subject or a single nearby basin can be adjusted and used if the watershed and climate characteristics are very similar. This is done in the first example.

Often there are no very similar basins nearby. In these instances, the statistics from at least three or more basins should be used. The basins should be as similar as possible to the subject basin. This is illustrated in the second example.

Selecting appropriate "similar nearby" basins requires considerable judgment and knowledge of Oregon climate. A similar nearby basin may be a hundred or more miles away if climatic and watershed conditions are similar to the subject basin. This occurs at many locations in Oregon, such as the Willamette Valley, the east side of the Cascade Range, the grasslands in north central Oregon, and the desert in southeast Oregon. Conversely, a watershed which is quite close to the subject drainage may not be a "similar nearby" basin. Examples are the basins around Mount Hood and the Columbia River Gorge. Drainage basins as close as 20 to 50 miles apart can have greatly different climates, terrain, and land use. References to aid in the selection of similar nearby drainage basins are as follows.

- Drainage basin characteristics listed in the **Chapter 7** Appendices tables.
- Climatic conditions shown on the maps in the **Chapter 7** appendices.
- The "Oregon Atlas" published by the University of Oregon.
- Experience gained from observation while traveling around the state.

**Step 2 -** Read the "Remarks" for the gaged basin in the flow data book. The Remarks will mention if discharges from the gaged watershed are significantly affected by storage, diversions, or consumptive use. The remarks will also state if the statistics are based on natural or measured stream flows. Natural flow statistics are preferred.

Measured flow statistics can be used if natural statistics are not available. Measured flow statistics from basins with little upstream storage, diversions, or consumptive uses are preferred to estimate discharges in other similar basins. These basins most closely resemble basins in natural conditions. Measured flow statistics from streams with significant storage, diversions, or consumptive use should only be used for sites on those streams near the gages.

**Step 3 -** Calculate the exceedance discharges per square mile of drainage area from nearby similar basins. This is the "yield" from each basin.

**Step 4 -** Plot the yield determined in Step 3 for each basin versus its drainage area. Estimate the relationship between yield and drainage basin area using a line or curve.

**Step 5 -** Estimate the yield from the subject watershed based on the line or curve drawn in Step 4.

**Step 6 -** Calculate the exceedance discharge by multiplying the estimated yield from Step 5 by the subject basin watershed area determined in Step 1, as follows:

Exceedance discharge in cubic feet per second = (exceedance discharge yield in cubic feet per second per square mile) x (basin area in square miles)

The exceedance flows calculated in this step may need to be adjusted to account for storage, diversions, or consumptive use. This is done by examining water use data and obtaining parol evidence, as shown in Parts 2 and 3.

### 4.2 Discharge Adjustments for Water Storage, Diversions, and Use

The second part of the procedure is to adjust the exceedance discharges calculated in the first part to account for storage, diversions, or consumptive use. The adjustments are based on remarks in the ODOT Mean Daily Flow Data publications and information from the Oregon Water Resources Department (OWRD). The steps are as follows.

**Step 1 -** Review remarks on data sheets in the ODOT Mean Daily Flow Data publications for the subject and adjacent basins, if available. These remarks are copied from USGS and OWRD publications containing gage data, and they address storage, regulation, and consumptive use. These remarks are an aid to the user, and they should not be considered as a definitive and comprehensive description of storage and water use.

The Remarks were collected during Step 2 of Part 1.

**Step 2 -** Review water availability information from the OWRD. The OWRD calculates water availability, including discharges, at selected locations throughout the state. These discharges are posted on their website: http://www.wrd.state.or.us/ The OWRD water availability study is described in the August 2002 Technical Report by Richard M. Cooper, P.E. titled "Determining Surface Water Availability in Oregon." This report is available from the OWRD.

The OWRD printouts provide data about storage, diversions, and expected water use for each month at the outlet of the water availability basin. This information is in the Detailed Report on Water Availability. These reports are shown in the examples.

**Step 3 -** Review OWRD information about the locations of the water diversions or withdrawal. Diversions or withdrawals upstream from the site affect discharges through the site. Conversely, withdrawals or diversions downstream from the site do not influence discharges at the site. Diversion and withdrawal locations are listed in the Water Rights Point of Diversion Details table, along with information about the water source, maximum permitted withdrawal, and use. These tables are shown in the examples.

The Point of Diversion Details table lists the maximum diversions or water use allowed by existing permits. Estimates based on these maximum permitted flows are sufficient for most hydraulic design purposes. In reality, seldom are all of the permitted water diversions or withdrawals in full use at the same instant. A certified professional, such as a water rights examiner, should be consulted about actual water use. This professional should also be consulted if water rights and use are critical aspects of the project.

**Step 4 -** Estimate any adjustments to the flows determined in Part 1 based on information obtained in Part 2. Adjust discharges, if needed, using guidelines from Part 4.

### 4.3 Parol Evidence

The third part of the procedure is to consider parol evidence. Parol evidence is "word of mouth" testimony about the hydraulic history of the site. Often it is supported by other documentation such as written correspondence and photographic evidence. This information supplements the calculated discharges. Calculations provide discharge quantities, but they often do not accurately describe when, why, and how often water flows in intermittent or ephemeral streams, ditches, or canals. Parol evidence can also be the best source for information about storage, diversions, and consumptive use. This is illustrated in the second example.

Parol evidence is obtained by interviewing people familiar with the waterway, such as:

- maintenance personnel,
- local watermasters,
- Oregon Department of Fish and Wildlife personnel, and
- local residents.

Typical questions to ask are:

- What are the usual discharges during a typical year?
- What is the lowest flow that will occur for a week or so during a typical year?
- What flows through the site during the various seasons? Rainfall runoff? Irrigation flow? Springwater?
- Is there more or less water flowing through this site than other typical creeks with similar watershed sizes in the area?
- Are there upstream diversions into or out of the creek?

Parol evidence should be recorded along with information such as the contact's address, phone number, etc. It is good practice to obtain information from several sources. A site visit is also useful to verify all needed information has been obtained and is correct.

### 4.4 Reporting Discharge Estimates

Exceedance discharges are reported in tables or narrative. The temporary water management discharges are reported as both the percent exceedance and the number of days a month the discharge is exceeded. The 10 and 95 percent exceedance flows for fish passage design are reported as percent exceedance discharges. This is shown in the examples. Exceedance discharges are usually reported to two significant figures. Further guidance about rounding figures is in **Chapter 4**.

**Temporary Water Management –** The 5, 25, and 50 percent exceedance discharges for temporary water management are reported on a table. The table lists natural discharges adjusted to account for storage, diversions, or consumptive use. These adjustments are based on storage, diversions, or uses that can be predicted with certainty in both timing and quantity. Adjustments are made for uses that significantly influence discharges. Minor uses are ignored.

The table notes mention storage, diversions, or withdrawals during the temporary water management period if their timing and quantity cannot be predicted. These items are mentioned if they significantly influence discharges. Minor uses are ignored.

**High and Low Flows for Fish Passage Design –** The 10 and 95 percent exceedance discharges for fish passage design are reported, if needed. The natural discharges are listed

after they are adjusted to account for storage, diversions, or consumptive use. These adjustments are based on storage, diversions, or uses that can be predicted with certainty and will not vary throughout the project design life. Adjustments are made for uses that significantly influence discharges. Minor uses are ignored.

The narrative accompanying the fish passage exceedance discharge mentions storage, diversions, or withdrawals if they may fluctuate throughout the project design life. These items are mentioned if they significantly influence the discharge. Minor uses are ignored.

## 5.0 Example 1 – Exceedance Discharges Based on a Single Nearby Gaged Basin

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Rock Creek is a small creek flowing northward in the Columbia River Gorge. It drains from the Cascade Mountains. The confluence of Rock Creek and the Columbia River is at the west side of the small town of Mosier. The total drainage area is 14.1 square miles.

Five, 25, and 50 percent exceedance discharges are needed for the months of July, August, and September. These are the months TWM will be in place. Fish are not expected to migrate during the TWM period, and they may migrate during the other months. The 10 percent discharge is needed for January, the wettest month. The 95 percent discharge is needed for October, the driest month outside of the TWM period.

**Part 1** is to determine natural or measured flow exceedance statistics.

**Step 1 -** The subject basin is outlined on a 7.5 – minute USGS quadrangle map and measured with a planimeter. The drainage area for the site under investigation is 13.8 square miles. Nearby basins are identified in the ODOT Mean Daily Flow Data book for the Hood Basin. The adjacent Mosier Creek basin has similar climatic conditions and land use. It flows into the Columbia River at the east side of Mosier. The page in the ODOT Mean Daily Flow Data book for Mosier Creek is shown in Figure 1.

**Step 2 -** The Remarks for Mosier Creek in the ODOT data book say “No regulation. Several small pumping diversions for irrigation upstream from station.” The Remarks also say the exceedance statistics are natural flow data. It is assumed the exceedance statistics for Mosier Creek have been adjusted to account for storage, diversions, and consumptive use, since this is natural flow data. It is also assumed the natural flow data represents the similar Rock Creek watershed in pre-development conditions. The exceedance discharges from the 41.5 square mile Mosier Creek basin are:

Discharge from Mosier Creek basin
in cubic feet per second

| | Month | | | | |
|---|---|---|---|---|---|
| Exceedance | January | July | August | September | October |
| 5 percent | | 4.1 | 2.9 | 3.7 | |
| 10 percent | 223 | | | | |
| 25 percent | | 3.1 | 2.3 | 2.7 | |
| 50 percent | | 2.4 | 2.0 | 2.3 | |
| 95 percent | | | | | 2.2 |

**Step 3 –** The yield from the Mosier Creek basin is calculated in this step. It is calculated by dividing each exceedance discharge by the 41.5 square mile drainage area, as follows:

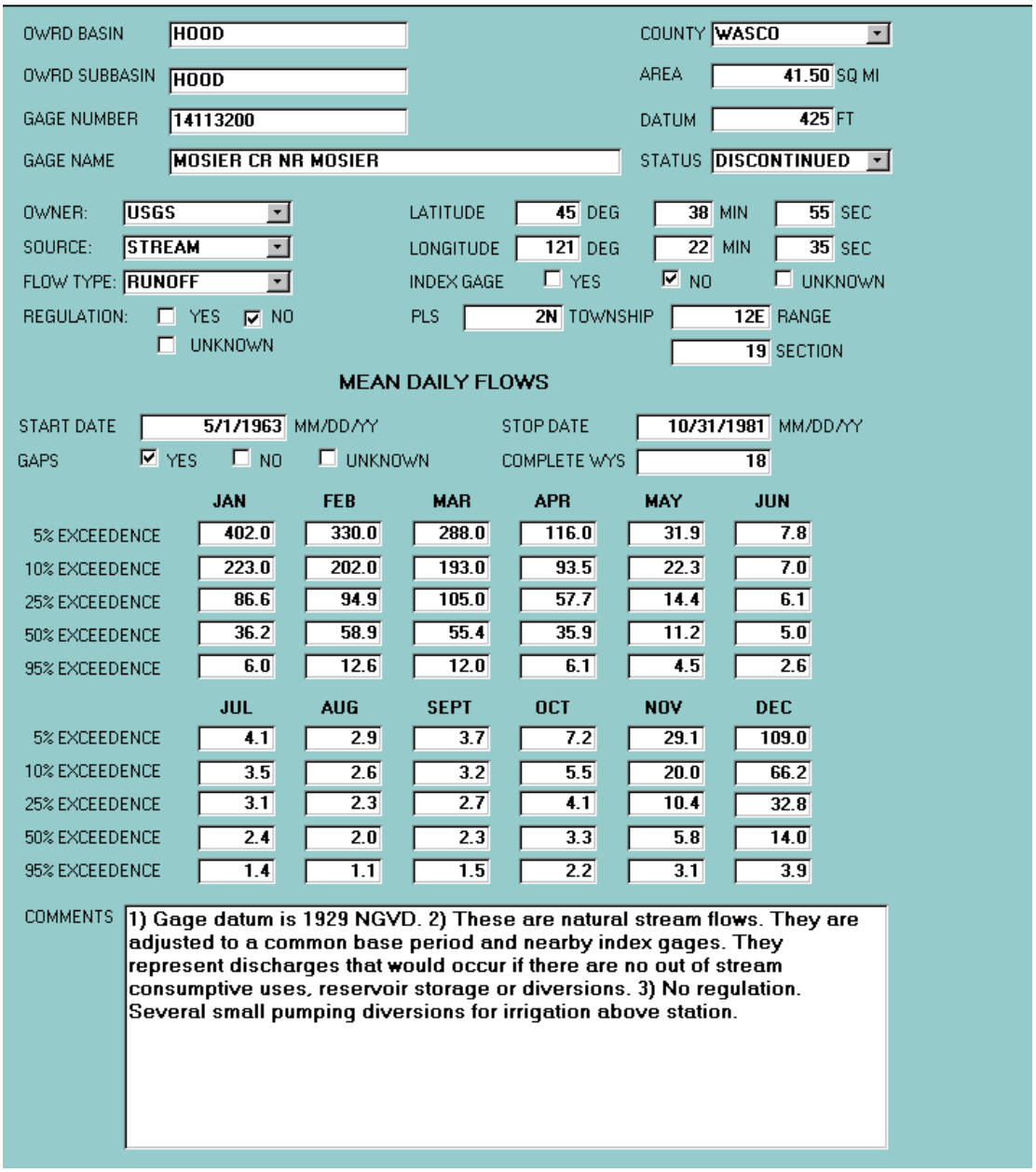
Yield from Mosier Creek basin in
cubic feet per second per square mile

| | Month | | | | |
|---|---|---|---|---|---|
| Exceedance | January | July | August | September | October |
| 5 percent | | 0.099 | 0.070 | 0.089 | |
| 10 percent | 5.4 | | | | |
| 25 percent | | 0.075 | 0.055 | 0.065 | |
| 50 percent | | 0.058 | 0.048 | 0.055 | |
| 95 percent | | | | | 0.053 |

**Steps 4, 5, and 6 –** The exceedance discharges for the subject stream are calculated by multiplying the yields from Step 3 by the 13.8 square mile Rock Creek drainage area, as follows:

Discharge from Rock Creek basin
in cubic feet per second

| | Month | | | | |
|---|---|---|---|---|---|
| Exceedance | January | July | August | September | October |
| 5 percent | | 1.3 | 0.96 | 1.2 | |
| 10 percent | 75 | | | | |
| 25 percent | | 1.0 | 0.76 | 0.90 | |
| 50 percent | | 0.80 | 0.67 | 0.76 | |
| 95 percent | | | | | 0.73 |



OWRD BASIN: HOOD
COUNTY: WASCO
OWRD SUBBASIN: HOOD
AREA: 41.50 SQ MI
GAGE NUMBER: 14113200
DATUM: 425 FT
GAGE NAME: MOSIER CR NR MOSIER
STATUS: DISCONTINUED
OWNER: USGS
LATITUDE: 45 DEG 38 MIN 55 SEC
SOURCE: STREAM
LONGITUDE: 121 DEG 22 MIN 35 SEC
FLOW TYPE: RUNOFF
INDEX GAGE: YES NO UNKNOWN
REGULATION: YES NO UNKNOWN
PLS: 2N TOWNSHIP 12E RANGE 19 SECTION

MEAN DAILY FLOWS

START DATE: 5/1/1963 MM/DD/YY
STOP DATE: 10/31/1981 MM/DD/YY
GAPS: YES NO UNKNOWN
COMPLETE WYS: 18

| | JAN | FEB | MAR | APR | MAY | JUN |
|---|---|---|---|---|---|---|
| 5% EXCEEDENCE | 402.0 | 330.0 | 288.0 | 116.0 | 31.9 | 7.8 |
| 10% EXCEEDENCE | 223.0 | 202.0 | 193.0 | 93.5 | 22.3 | 7.0 |
| 25% EXCEEDENCE | 86.6 | 94.9 | 105.0 | 57.7 | 14.4 | 6.1 |
| 50% EXCEEDENCE | 36.2 | 58.9 | 55.4 | 35.9 | 11.2 | 5.0 |
| 95% EXCEEDENCE | 6.0 | 12.6 | 12.0 | 6.1 | 4.5 | 2.6 |

| | JUL | AUG | SEPT | OCT | NOV | DEC |
|---|---|---|---|---|---|---|
| 5% EXCEEDENCE | 4.1 | 2.9 | 3.7 | 7.2 | 29.1 | 109.0 |
| 10% EXCEEDENCE | 3.5 | 2.6 | 3.2 | 5.5 | 20.0 | 66.2 |
| 25% EXCEEDENCE | 3.1 | 2.3 | 2.7 | 4.1 | 10.4 | 32.8 |
| 50% EXCEEDENCE | 2.4 | 2.0 | 2.3 | 3.3 | 5.8 | 14.0 |
| 95% EXCEEDENCE | 1.4 | 1.1 | 1.5 | 2.2 | 3.1 | 3.9 |

COMMENTS: 1) Gage datum is 1929 NGVD. 2) These are natural stream flows. They are adjusted to a common base period and nearby index gages. They represent discharges that would occur if there are no out of stream consumptive uses, reservoir storage or diversions. 3) No regulation. Several small pumping diversions for irrigation above station.

**Figure 1 Page in ODOT Mean Daily Flow Data Book for Mosier Creek**

**Part 2** of the procedure is to adjust the natural flow exceedance discharges calculated in Part 1 to account for storage, diversions, and consumptive use.

**Step 1 -** The first step is to look at the remarks on the data sheet for the adjacent Mosier Creek basin. Diversions into or out of the Rock Creek basin will be investigated if they are mentioned in the Mosier Creek data. The remarks do not mention any diversions to or from the Mosier basin. The remarks are in the table shown on Figure 1.

**Step 2 -** The second step is to examine OWRD data to determine storage, diversions, or consumptive use. The OWRD has calculated water availability and consumptive use for the entire Rock Creek watershed upstream from the Columbia River. The Water Availability Table can be found on the interactive mapping section of the OWRD website. The table for the 50 percent exceedance discharge is shown in Figure 2. Consumptive uses of 0.00, 0.02, 0.01, 0.01, and 0.00 cubic feet per second are listed for January, July, August, September, and October, respectively. No storage or diversions are noted.

*Note: The consumptive uses listed on the OWRD 80 percent exceedance table are identical, and they could also be used.*

**Step 3 -** The point of withdrawal locations are determined during this step. This will help determine whether or not the consumptive uses reduce discharges at the project site. The project site is 0.35 miles upstream from the mouth of the creek on the Columbia River. It is possible the withdrawals could occur within the 0.35 mile reach downstream from the crossing, and they would not affect discharges at the site.

DETAILED REPORT ON THE WATER AVAILABILITY CALCULATION
Water Availability as of 11/14/2003 for
ROCK CR > COLUMBIA RIVER - AT MOUTH

Watershed ID #: 30410526 Basin: Hood Exceedance Level : 50
Time: 15:06 Date: 11/14/2003

| Month | Natural Stream Flow | CU + Stor Prior 1/1/93 | CU + Stor After 1/1/93 | Expected Stream Flow | Reserved Stream Flow | Instream Water Rights | Net Water Available |
|---|---|---|---|---|---|---|---|
| 1 | 5.07 | 0.00 | 0.00 | 5.07 | 0.00 | 0.00 | 5.07 |
| 2 | 10.50 | 0.00 | 0.00 | 10.50 | 0.00 | 0.00 | 10.50 |
| 3 | 13.20 | 0.00 | 0.00 | 13.20 | 0.00 | 0.00 | 13.20 |
| 4 | 3.07 | 0.03 | 0.00 | 3.04 | 0.00 | 0.00 | 3.04 |
| 5 | 0.71 | 0.09 | 0.00 | 0.63 | 0.00 | 0.00 | 0.63 |
| 6 | 0.16 | 0.07 | 0 | 0.09 | 0.00 | 0.00 | 0.09 |
| 7 | 0.06 | 0.02 | 0.00 | 0.04 | 0.00 | 0.00 | 0.04 |
| 8 | 0.04 | 0.01 | 0.00 | | 0.00 | 0.00 | 0.03 |
| 9 | 0.02 | 0.01 | 0.00 | | 0.00 | 0.00 | 0.01 |
| 10 | 0.03 | 0.00 | 0.00 | 0.03 | 0.00 | 0.00 | 0.03 |
| 11 | 0.17 | 0.00 | 0.00 | 0.17 | 0.00 | 0.00 | 0.17 |
| 12 | 3.20 | 0.00 | 0.00 | 3.20 | 0.00 | 0.00 | 3.20 |
| Stor | 2160 | 14 | 0 | 2150 | 0 | 0 | 2150 |

Permitted surface water uses.

**Figure 2 Water Availability Table for Rock Creek**

The points of withdrawal are determined from the "Water Rights Point of Diversion Details" table for the Rock Creek water availability basin. This table is obtained from the OWRD website. An abbreviated table is shown in Figure 3. These parameters are specified when the table is requested to assure all needed data is provided:

- "All Uses" as the water use to be printed, and
- a time period between 1 January 1800 and the present (14 November 2003).

The table lists two permitted surface water diversions in the Rock Creek basin, as follows.

- Permit # S 6322, surface water withdrawal from Rock Creek, up to 0.08 cubic feet per second, at any time of the year, for irrigation.

- Permit # S 3102, springwater withdrawal, up to 0.01 cubic feet per second, at any time of the year, for irrigation and domestic use.

The point of withdrawal locations are described in the detail table for both permitted uses. Both are in Township 2 North, Range 11 East, Section 11. These withdrawal points are within the watershed upstream from the project site. These consumptive uses, if utilized, will reduce discharges at the site.

| STREAM NAME | APPL | PERMIT | LOCATION | USE | SEASON | SOURCE | RATE(CFS) | RATE(AFT) | P/S | STAT |
|---|---|---|---|---|---|---|---|---|---|---|
| ROCK CREEK > COLUMBIA RIVER | S 9567 | S 6322 | 0.200N 11.00E 11 NENW | IR | 1/1 - 12/31 | ROCK CREEK | 0.8 (EST) | | P | NC |
| ROCK CREEK > COLUMBIA RIVER | G 13322 | G 12690 | 0.200N 11.00E 02 SENE | GD | 1/1 - 12/31 | A WELL | 0.09 (EST) | | P | NC |
| ROCK CREEK > COLUMBIA RIVER | G 13322 | G 12690 | 0.200N 11.00E 02 SENE | CM | 1/1 - 12/31 | A WELL | 0.14 (EST) | | P | NC |
| ROCK CREEK > COLUMBIA RIVER | 322 | G 12690 | 0.200N 11.00E 02 SENE | IM | 1/1 - 12/31 | A WELL | 0.14 (EST) | | P | NC |
| UNN STR > ROCK CREEK | S 5149 | G 3102 | 0.200N 11.00E 11 SENE | ID | 1/1 - 12/31 | SPRS | 0.1 (EST) | | P | NC |
| UNN STR > ROCK CRE | R 75526 | | 0.200N 11.00E 14 SWNW | LW | 1/1 - 12/31 | RUNOFF/RES 1 | 0.0 (EST) | 0.5 (EST) | P | NC |
| UNN STR > RO R | R 75526 | | 0.200N 11.00E 14 SWNW | FP | 1/1 - 12/31 | RUNOFF/RES 1 | 0.0 (EST) | 0.0 (EST) | P | NC |
| UNN STR CK CREEK | R 75526 | | 0.200N 11.00E 15 SENE | LW | 1/1 - 12/31 | RUNOFF/RES 3 | 0.0 (EST) | 0.2 (EST) | P | NC |
| UNN STR > ROCK CREEK | R 75526 | | 0.200N 11.00E 15 SENE | LW | 1/1 - 12/31 | RUNOFF/RES 2 | 0.0 (EST) | 0.2 (EST) | P | NC |
| UNN STR > ROCK CREEK | R 75526 | | 0.200N 11.00E 15 SENE | FP | 1/1 - 12/31 | RUNOFF/RES 3 | 0.0 (EST) | 0.0 (EST) | P | NC |
| UNN STR > ROCK CREEK | R 75526 | | 0.200N 11.00E 15 SENE | FP | 1/1 - 12/31 | RUNOFF/RES 2 | 0.0 (EST) | 0.0 (EST) | P | NC |
| UNN STR > ROCK CREEK | G 14773 | G 13633 | 0.200N 11.00E 11 NWSE | IR | 1/1 - 12/31 | A WELL | 0.134 (EST) | | P | NC |

Permitted surface water uses.

**Figure 3 OWRD Water Rights Points of Diversion Details Table**

**Part 3 –** Parol evidence is collected and considered during this step. People are interviewed who have as much as 20 years of experience with the stream. They say the stream dries up during the spring, and it almost always dries up before late June. They also say the stream stays dry throughout the summer until the area receives heavy precipitation in the fall.

**Part 4 –** The discharges are reported during this step. The Temporary Water Management (TWM) Discharges are summarized in a table with notes. The fish passage design discharges are mentioned separately with associated notes.

**Temporary Water Management Discharges –** The temporary water management exceedance flows vary from 0.67 cubic feet per second (August 50 percent) to 1.3 cubic feet per second (July 5 percent). The expected consumptive uses vary from 0.01 cubic feet per second (August and September) to 0.02 cubic feet per second (July). The consumptive uses are a very small portion of the predicted discharges. They are considered “minor uses” and are not reported in the notes or used in any adjustments. The natural discharges calculated in Step 6 are reported in the table shown in Figure 4. Parol evidence indicates actual discharges are likely to be lower than those listed in the table. This is mentioned in the table notes.

**Fish Passage Design Discharges** - The 10 and 95 percent exceedance discharges for the wettest and driest months of the fish migration period are reported. The 10 percent exceedance discharge of 75 cubic feet per second is reported for January, and the 95 percent discharge of 0.73 cubic feet per second is listed for October. These flows are based on the natural discharges calculated in Step 6 of Part 1, and they are presented as follows:

"Fish are expected to migrate through the crossing during the months of the year outside of the TWM period. These are the months of October through June. The wettest month of this migration period is January, and the 10 percent average daily exceedance discharge for this month is 75 cubic feet per second. This is the high flow design discharge for fish passage. The driest month of the migration period is October. The 95 percent average daily exceedance discharge for October 0.73 cubic feet per second. This is the low flow design discharge for fish passage.

These calculated discharges are based on measured flow data from an adjacent gaged basin. Actual stream flows may be less than this discharge, and surface flow may cease for extended periods during the summer and early fall."

**ROCK CREEK**
ESTIMATED DISCHARGES FOR
TEMPORARY WATER MANAGEMENT

| | AVERAGE DAILY DISCHARGE IN CUBIC FEET PER SECOND (GALLONS PER MINUTE) | | |
|---|---|---|---|
| **NOTE** | **1** | **2** | **3** |
| JULY | 1.3 (580) | 1.0 (450) | 0.80 (360) |
| AUGUST | 0.96 (430) | 0.76 (340) | 0.67 (300) |
| SEPTEMBER | 1.2 (540) | 0.90 (400) | 0.76 (340) |

1) 5 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 2 days each month.)
2) 25 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 8 days each month.)
3) 50 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 16 days each month.)

1.1.1

1.1.2 In-water work period extends from 1 July through 30 September. Temporary water management shown on plans recommended throughout in-water work period. Listed discharges are surface water from the upstream watershed. The estimated discharges are based on an adjacent gaged basin. Discharges in the subject watershed may differ.

Actual discharges are expected to be lower than estimated discharges. Surface water discharge may cease for extended periods.

**Figure 4 Temporary Water Management Discharge Table for Rock Creek**

## 6.0 Example 2 – Exceedance Discharges Based on Several Nearby Gaged Basins

Unnamed Creek is a small stream draining fruit orchards in the Columbia River Gorge. The confluence of the creek and the Hood River is north of the small town of Odell. 5, 25, and 50 percent exceedance discharges are needed for the months of July, August, and September. This is the TWM period. Fish passage is not needed so the 10 and 95 percent discharges will not be calculated.

**Part 1** of the analysis is to determine natural or measured exceedance flow statistics.

**Step 1 -** The subject basin is outlined on a 7.5 – minute USGS quadrangle map. The small basin is difficult to accurately delineate using only the map. The drainage boundaries are verified by a field inspection. The drainage area is measured by a planimeter, and it is 0.108 square miles.

Nearby gaged basins are identified in the ODOT Mean Daily Flow Data book for Hood Basin. Unlike the first example, there is no basin in the immediate area with similar climatic conditions and land use. It is necessary to calculate discharge estimates with data from basins out of the immediate area. Usually at least three basins are used when this is done.

The subject basin is in the Hood River Valley on the north side of Mount Hood. Four gaged basins are in this valley or surrounding mountains, and natural discharge statistics are available for all. The basins are as follows.

**Gage 14113400 Dog River near Parkdale**, drainage area = 4.50 square miles, gage elevation = 4,347 feet. The basin is in the mountains south of the nearby Upper Hood River Valley.

**Gage 14118000 Green Point Creek below North Fork near Dee**, drainage area = 20.00 square miles, gage elevation = 1100 feet. The basin is in the mountains west of the Hood River Valley.

**Gage 14118500 West Fork Hood River near Dee,** drainage area = 95.6 square miles, gage elevation = 802 feet. The basin is in mountains to the southwest of Hood River Valley.

**Gage 14120000 Hood River at Tucker Bridge near Hood River,** drainage area = 279 square miles, gage elevation = 383 feet. The basin is in the mountains surrounding the Hood River Valley and Upper Hood River Valley.

Not all nearby gaged basins could be used. Several gaged basins are located to the east of Hood River Valley near the towns of Mosier, The Dalles and Dufur. The monthly discharge data was examined, and it appears these basins have distinctly different discharge characteristics than those around the Hood River Valley. The summer flows are much less compared to the winter discharges, and there is less discharge per square mile of drainage area. Data from these basins are not used.

Several gaged basins are located to the west of Hood River Valley in the Sandy River Basin. The monthly data indicates these basins also have different discharge characteristics. The September discharges, in particular, are considerably higher than those in the Hood River Valley. Data from these basins are not used.

Several gaged basins are north of Hood River Valley in Washington. These basins have significantly higher summer discharges than the Hood River Valley area basins. Statistics from these basins are not used.

**Step 2 -** The Remarks for the four basins in Hood River Valley area mention the following about storage, regulation, or diversion.

**Gage 14113400 Dog River near Parkdale,** No regulation or diversion above station.

**Gage 14118000 Green Point Creek below North Fork near Dee,** No regulation. Water is diverted above station in NW1/4 S10, T1N, R9E, and from North Fork in SE1/4 SW1/4 S30, T2N, R9E, and in SW1/4 S3, T1N, R9E, for irrigation near Oak Grove outside of Green Point Creek Basin.

**Gage 14118500 West Fork Hood River near Dee,** No regulation. Dee Irrigation District canal diverts from right bank about 6 miles upstream from station for irrigation upstream from station and in Middle Fork basin. Diversions from Green Point basin upstream from station for irrigation near Oak Grove: water from two of these diversions is carried in Hood River Irrigation District canal.

**Gage 14120000 Hood River at Tucker Bridge near Hood River,** Some daily fluctuation possibly caused by diversion dam upstream from station and Sawmill at Dee. Diversions for irrigation above station.

The Remarks for all four gages also mention: "These are natural stream flows. They are adjusted to a common base period. They represent discharges that would occur if there are no out of stream consumptive uses, reservoir storage or diversions." This means the statistical distributions have been adjusted to account for the diversions, and they represent the watersheds in natural conditions prior to development. Discharges from similar nearby basins in their natural state can be directly estimated using these flows. The discharges are:

| | Mean Daily Low Flows for Nearby Basins in cubic feet per second | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Month** | July | | | August | | | September | | |
| **Percent Exceedance** | 5 | 25 | 50 | 5 | 25 | 50 | 5 | 25 | 50 |
| **Basin** | | | | | | | | | |
| Dog R near Parkdale | 32.2 | 11.0 | 7.7 | 7.5 | 5.1 | 4.0 | 5.1 | 3.8 | 3.1 |
| Green Point Cr below North Fork near Dee | 50.8 | 33.5 | 26.8 | 27.7 | 21.9 | 16.5 | 31.0 | 20.6 | 6.2 |
| West Fork Hood R near Dee | 413 | 291 | 242 | 269 | 213 | 183 | 325 | 204 | 175 |
| Hood R at Tucker Br near Hood River | 1080 | 729 | 599 | 701 | 538 | 455 | 660 | 489 | 423 |

**Step 3 -** The basin yields are calculated by dividing the exceedance discharges by the drainage areas, as follows:

| | | Basin Yield in cubic feet per second per square mile | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Month** | | July | | | August | | | September | | |
| **Percent Exceedance** | | 5 | 25 | 50 | 5 | 25 | 50 | 5 | 25 | 50 |
| **Basin** | **Drainage Area in Square Miles** | | | | | | | | | |
| Dog R near Parkdale | 4.5 | 7.16 | 2.44 | 1.71 | 1.67 | 1.13 | 0.89 | 1.13 | 0.84 | 0.69 |
| Green Point Cr below North Fork near Dee | 20.0 | 2.54 | 1.68 | 1.34 | 1.39 | 1.10 | 0.83 | 1.55 | 1.03 | 0.81 |
| West Fork Hood R near Dee | 95.6 | 4.32 | 3.04 | 2.53 | 2.81 | 2.23 | 1.91 | 3.40 | 2.13 | 1.83 |
| Hood R at Tucker Br near Hood River | 279 | 3.87 | 2.61 | 2.15 | 2.51 | 1.93 | 1.63 | 2.37 | 1.75 | 1.52 |

**Step 4 -** The yield versus discharge curves are determined by linear interpolation. This was automatically done by a computer program. The printouts are shown in Figure 5.

**Step 5 -** The yields for the 0.108 square mile subject basin are calculated during this step. This is done automatically when using the program. The equations and yields are:

| Month | Exceedance | Equation | Mean Daily Low Flow Yield in cubic feet per second per square mile |
|---|---|---|---|
| July | 5 percent | -0.0044x + 4.9092 | 4.91 |
| July | 25 percent | 0.0019x + 2.2573 | 2.26 |
| July | 50 percent | 0.0022x + 1.7137 | 1.71 |
| August | 5 percent | 0.0036x + 1.7395 | 1.74 |
| August | 25 percent | 0.003x + 1.3006 | 1.30 |
| August | 50 percent | 0.0028x + 1.0342 | 1.03 |
| September | 5 percent | 0.0037x + 1.7385 | 1.74 |
| September | 25 percent | 0.003x + 1.1412 | 1.14 |
| September | 50 percent | 0.0028x + 0.9307 | 0.93 |

x = basin area in square miles

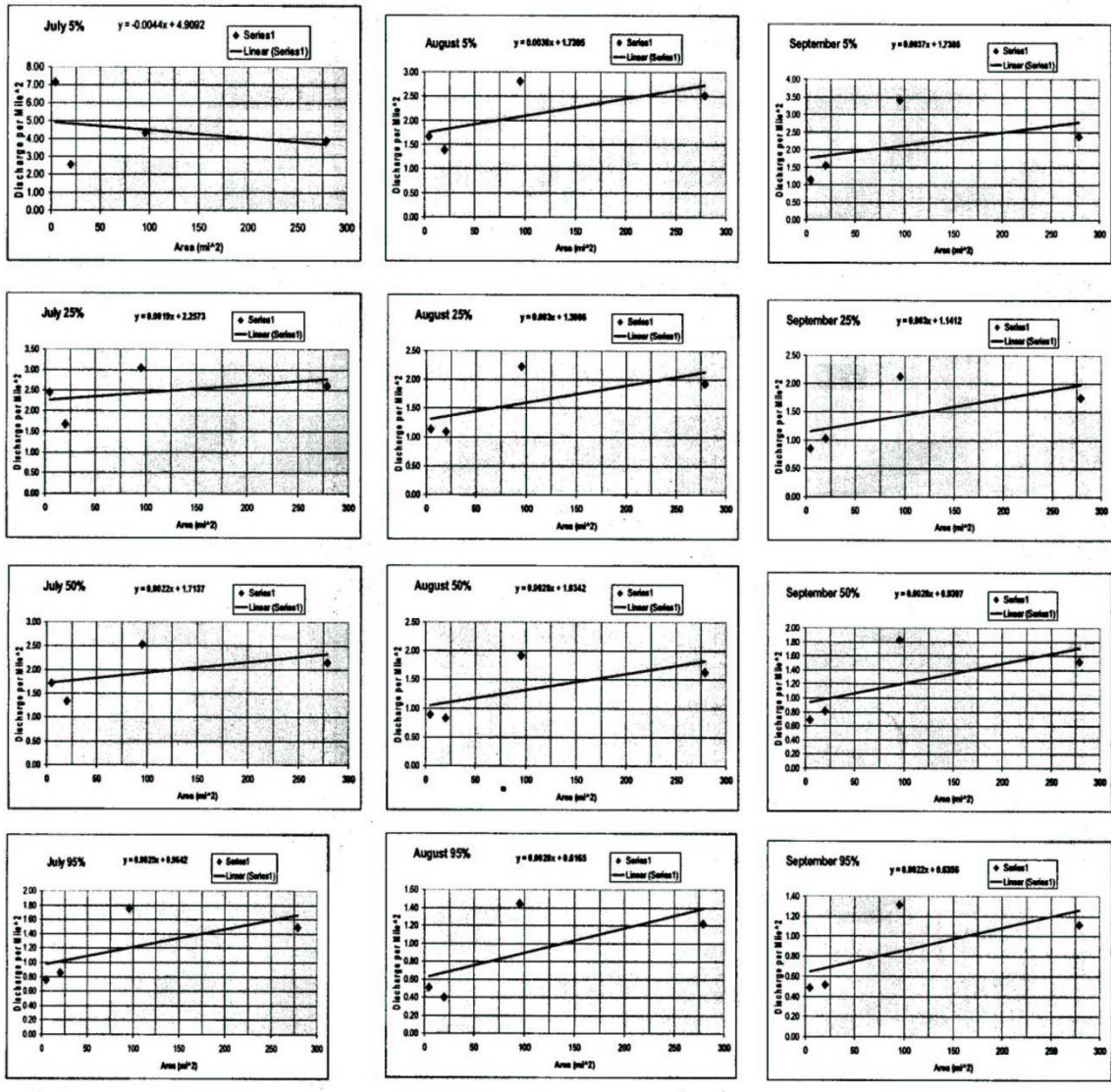



**Figure 5 Linear Interpolation of Natural Flow Data from Nearby Basins**

**Step 6 -** The exceedance discharges for the subject basin are calculated by multiplying the yields by the 0.108 square mile drainage area. The discharges are as follows:

| | **Mean Daily Low Flow in cubic feet per second** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Month** | July | | | August | | | September | | |
| **Percent Exceedance** | 5 | 25 | 50 | 5 | 25 | 50 | 5 | 25 | 50 |
| **Basin** | | | | | | | | | |
| Unnamed Creek | 0.53 | 0.24 | 0.19 | 0.19 | 0.14 | 0.11 | 0.19 | 0.12 | 0.10 |

*Note: The less frequent exceedance discharges should be greater than the more frequent discharges, for any given month. For example, during June the 5 percent discharge should be greater than the 25 percent discharge, and the 25 percent flow should be bigger than the 50 percent discharge, etc. Data from additional nearby stations may be needed if this does not occur.*

**Part 2 –** The discharges calculated in Part 1 are natural flows. They represent runoff from the watershed in pre-development conditions when it was covered with forest and grasslands. Now the watershed has been developed into orchards with a few residences. This development can significantly alter discharges. OWRD data is reviewed in this step to determine water use.

**Step 1 -** The first step of Part 2 is to look at the Remarks in the ODOT mean daily flow data books for nearby gaged basins. None of the basins have any remarks pertaining to the subject basin.

**Step 2 -** Storage, diversions, or water uses are determined during this step using OWRD information. Unlike the first example, the subject drainage is part of a much larger OWRD Water Availability Basin, and it is difficult to determine water use from a "Detailed Report on the Water Availability Calculation." Water use will be determined from the OWRD website for the two public land survey sections containing the subject basin, Township 2 North, Range 10 East, Sections 15 and 22, Willamette Meridian. Details are listed in the "Water Rights Point of Diversion Details" tables for the sections. Abbreviated tables are shown in Figure 6.

| Stream Name | Application | Permit | Location | Use | Season | Priority | Source | Rate (CFS) | Rate (AFT) | P/S | Status |
|---|---|---|---|---|---|---|---|---|---|---|---|
| HOOD R > COLUMBIA R | | | 02.00N 10.00E 15 NENE | IL | 1/1 ~ 12/31 | 12/31/1890 | UNN STR | 0.0 (est) | | P | NC |
| HOOD R > COLUMBIA R | | | 02.00N 10.00E 15 NENE | IL | 1/1 ~ 12/31 | 12/31/1890 | A SPR | 0.125 (est) | | S | NC |
| HOOD R > COLUMBIA R | | | 02.00N 10.00E 15 NWSW | IS | 1/1 ~ 12/31 | 12/31/1909 | SMALL SEEPAGE STR | 0.0 (est) | | S | NC |
| HOOD R > COLUMBIA R | S 10821 | S 7354 | 02.00N 10.00E 15 NWNE | IS | 1/1 ~ 12/31 | 06/03/1926 | A SPR | 0.0 (est) | | P | NC |
| HOOD R > COLUMBIA R | S 10821 | S 7354 | 02.00N 10.00E 15 NENW | IS | 1/1 ~ 12/31 | 06/03/1926 | A SPR | 0.25 (est) | | S | NC |
| HOOD R > COLUMBIA R | S 16307 | S 12120 | 02.00N 10.00E 15 NESW | ID | 1/1 ~ 12/31 | 04/06/1936 | SPR 1 | 0.035 (est) | | S | NC |
| HOOD R > COLUMBIA R | S 16307 | S 12120 | 02.00N 10.00E 15 NENW | ID | 1/1 ~ 12/31 | 04/06/1936 | SPR 2 | 0.015 (est) | | S | NC |
| HOOD R > COLUMBIA R | S 56053 | S 42328 | 02.00N 10.00E 15 SESW | IC | 1/1 ~ 12/31 | 06/10/1977 | UNN STR/RES | 0.06 (est) | | P | NC |
| HOOD R > COLUMBIA R | S 56053 | S 42328 | 02.00N 10.00E 15 SESW | IC | 1/1 ~ 12/31 | 10/25/1977 | UNN STR/RES | 0.11 (est) | | P | NC |
| HOOD R > COLUMBIA R | R 56795 | R 6702 | 02.00N 10.00E 15 SESW | IR | 1/1 ~ 12/31 | 10/25/1977 | UNN STR | 0.0 (est) | 0.29 (est) | P | NC |
| HOOD R > COLUMBIA R | S 61263 | S 45643 | 02.00N 10.00E 15 SESW | IS | 1/1 ~ 12/31 | 02/19/1981 | RESERVOIR | 0.45 (est) | | S | NC |
| HOOD R > COLUMBIA R | S 61263 | S 45643 | 02.00N 10.00E 15 SESW | LV | 1/1 ~ 12/31 | 02/19/1981 | RESERVOIR | 0.0 (est) | 0.05 (est) | P | NC |
| UNN STR > HOOD R | S 8528 | S 5544 | 02.00N 10.00E 15 NENW | IS | 1/1 ~ 12/31 | 07/24/1922 | A SPR | 0.18 (est) | | S | NC |
| UNN STR > HOOD R | R 33118 | R 2253 | 02.00N 10.00E 15 NENW | IS | 1/1 ~ 12/31 | 05/21/1959 | WW IN UNN SWALE | 0.0 (est) | 0.4 (est) | S | NC |
| UNN STR > HOOD R | S 10735 | S 7284 | 02.00N 10.00E 15 NESW | IS | 1/1 ~ 12/31 | 04/26/1926 | UNN STR | 0.11 (est) | | S | NC |
| UNN STR > HOOD R | S 33119 | S 26225 | 02.00N 10.00E 15 NENW | IS | 1/1 ~ 12/31 | 05/21/1959 | UNN STR | 0.15 (est) | | S | NC |
| UNN STR > HOOD R | G 1701 | G 1561 | 02.00N 10.00E 15 NENW | IS | 1/1 ~ 12/31 | 03/23/1960 | A WELL | 0.25 (est) | | S | NC |
| UNN STR > HOOD R | R 61455 | R 8252 | 02.00N 10.00E 15 SESW | LV | 1/1 ~ 12/31 | 03/27/1981 | UNN STR | 0.0 (est) | 0.005 (est) | P | NC |
| UNN STR > HOOD R | R 61455 | R 8252 | 02.00N 10.00E 15 SESW | IS | 1/1 ~ 12/31 | 03/27/1981 | UNN STR | 0.0 (est) | 0.45 (est) | S | NC |

SECTION 15

| Stream Name | Application | Permit | Location | Use | Season | Priority | Source | Rate (CFS) | Rate (AFT) | P/S | Status |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ODELL CR > HOOD R | | | 02.00N 10.00E 22 NESE | IR | 1/1 ~ 12/31 | 12/31/1885 | ODELL CR | 0.025 (est) | | P | NC |
| ODELL CR > HOOD R | | | 02.00N 10.00E 22 NENE | IR | 1/1 ~ 12/31 | 12/31/1887 | ODELL CR | 0.125 (est) | | P | NC |
| ODELL CR > HOOD R | | | 02.00N 10.00E 22 NENE | IR | 1/1 ~ 12/31 | 12/31/1887 | ODELL CR | 0.125 (est) | | P | NC |
| UNN STR > ODELL CR | S 7483 | S 4748 | 02.00N 10.00E 22 NESW | IS | 1/1 ~ 12/31 | 08/20/1920 | UNN STR | 0.31 (est) | | S | NC |

SECTION 22

**Figure 6 Water Rights Points of Diversion Details**

**Step 3 -** The details table lists many permitted uses throughout the section. The uses within the subject watershed are determined during this step. The contour map in the "Interactive Mapping" section of the OWRD website covering the subject basin is examined. This is shown in Figure 7.

*Note: The drainage basin boundaries are not shown on the map displayed by the OWRD. These basin boundaries are added by the author to make the map easier to understand.*

**Figure 7 Map Showing Diversion Points**

Two points of diversion are shown by purple triangles on the OWRD basin map. Both are adjacent to small reservoirs. Two permitted surface water uses corresponding to these diversions are listed in details table shown in Figure 6. They are:

- Permit # S 42328, surface water withdrawal from the subject Unnamed Stream and a reservoir on the stream, up to 0.17 cubic feet per second, for irrigation.

- Permit # S 45643, surface water withdrawal from another reservoir on the subject stream, up to 0.45 cubic feet per second, for irrigation.

The total water use can be 0.62 cubic feet per second if all of these rights are fully used at the same time. The discharges in the creek due to natural runoff vary from 0.10 to 0.51 cubic feet per second, based on the various 50 percent exceedances determined in Part 1 Step 6. The total water rights for withdrawal from the reservoirs are greater than the discharge from the creek into the reservoirs. As a result, the creek at the project site, which is downstream from the reservoirs, may be dry during the months of June through September due to either upstream storage or water use.

*Note: Public land survey locations for water storage, diversion, or use are listed on the various OWRD tables. This information can also be used to locate diversion points.*

**Part 3 –** Parol evidence is considered during this step. Several people familiar with the drainage are interviewed. There is a system of irrigation canals, gates, pipes, and ditches in the area. Irrigation district personnel say the irrigation system is in operation throughout the summer to water the local pear orchards. They say the farmers turn off the flow of water to their fields immediately before the pear harvest. The canals overflow during the interval between the cessation of water use and the time when the irrigation district closes the canal headgates. During this interval a surge of overflow water up to 3 to 4 cubic feet per second is expected through the drainage at the crossing site.

A site visit was made to verify and supplement the parol evidence. An irrigation weir is noted in the immediate vicinity of the project, as shown in Figure 8. Water impounded by the weir during the summer is used to irrigate a small nearby orchard. Proposed construction will occur in this area. Care will be needed during the project temporary water management to allow the orchard to be irrigated.

**Figure 8 Irrigation Weir on Project Site**

**Part 4 –** The final part is to compile the discharge estimates. The natural discharges are reported in the table. They are expected to occur throughout the construction period due to storm runoff and discharge from the aquifer. The lack of water due to consumptive use, or the surge of water caused by irrigation system overflow may occur at unpredictable times throughout the project. These varying discharges are mentioned in the notes to the table. The table is shown in Figure 9. The need to provide access to water for irrigating the field where the gate may be removed is mentioned to the designer in correspondence.

UNNAMED CREEK
ESTIMATED DISCHARGES FOR
TEMPORARY WATER MANAGEMENT

| | AVERAGE DAILY DISCHARGE IN CUBIC FEET PER SECOND (GALLONS PER MINUTE) | | |
|---|---|---|---|
| **NOTE** | **1** | **2** | **3** |
| JULY | 0.53 (240) | 0.24 (110) | 0.19 (85) |
| AUGUST | 0.19 (85) | 0.14 (63) | 0.11 (49) |
| SEPTEMBER | 0.19 (85) | 0.12 (54) | 0.10 (45) |

1) 5 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 2 days each month.)
2) 25 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 8 days each month.)
3) 50 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 16 days each month.)

1.1.3 In-water work period extends from 15 July through 31 August. Temporary water management shown on plans recommended throughout in-water work period. Listed discharges are surface water runoff from the upstream watershed. Irrigation discharges not included. The estimated discharges are based on nearby gaged basins. Discharges in the subject watershed may differ.

The creek could go dry during the in-water work period due to water use upstream or natural conditions. An additional 3 to 4 cubic feet per second irrigation flow can be expected during portions of the in-water work period. The dry periods and flow surges can occur without prior warning.

**Figure 9 Temporary Water Management Discharge Table for Unnamed Creek**

# APPENDIX K – MAXIMUM PREDICTED DISCHARGE BASED ON STATION RECORDS AND ENVELOPE CURVE

## 1.0 Introduction

Many temporary structures and the vast majority of temporary water management (TWM) facilities are in place during the dry season, only. Mean daily exceedance discharges can be used to design most of these dry season facilities. The largest exceedance discharge is the 5 percent exceedance flow. This discharge, during an average year, is expected to be exceeded on 5 percent of the days during the specified month. This is one or two days during a typical month.

Some TWM and temporary construction features may be of too critical importance to use the 5 percent exceedance flow as either a design or check discharge. In these cases, the risk of damage or failure caused by a discharge exceeded one or two days a month is too great. Examples of these facilities could be cofferdams, water diversion pipes or temporary bridges on critical roadways, or work bridges.

These critical dry season facilities can be designed using the 5-year event as the design discharge and the 10 or 25-year event as the check discharge. This is done for TWM facilities and temporary construction to be in place all year or through the flood season. A dry season facility or structure designed using 5-year event discharges will almost always have enough hydraulic capacity for the dry season discharges. In most cases, it would have far too much hydraulic capacity and be unduly expensive.

The procedure described in this appendix is an alternative to using the 5-year discharge as a design or check flow for dry season structures and TWM facilities. The method is based on an envelope curve and the maximum mean daily discharges recorded at nearby stream gages during the months the TWM or temporary structure will be in place. It is intended to provide a maximum predicted discharge that is not expected to be exceeded during the dry season. The maximum predicted discharge is typically less than the 5-year event discharge and greater than the 5 percent mean daily exceedance flow.

*Note: The maximum discharge predicted by the envelope curve is not necessary for many TWM or temporary structure applications. It is only calculated and reported if needed.*

## 2.0 Limitations of Method

This procedure is based on daily discharges recorded at nearby gaging stations. Rarely are these nearby watersheds identical to the subject drainage to be studied. Local watershed characteristics can strongly influence the discharge magnitude, and these characteristics are often unique to the subject basin. In addition, there are often substantial differences in discharges from year-to-year due to variability in climate and water use. Some factors that influence daily flows are:

- climatic conditions such as rainfall, snowpack, and evaporation,
- the amount and type of vegetation in the riparian areas (vegetated riparian areas tend to sustain dry season flows),
- diversions to or from the subject stream,
- infiltration into exceptionally pervious rock formations or streambed materials,
- water entering the stream from springs and seeps,
- release or retention of streamflow in storage reservoirs, and
- consumptive uses such as livestock watering or irrigation.

Many of these factors are investigated when the exceedance discharges are calculated and water use and parol information is collected. This is discussed in **Chapter 7** Appendix J. The water use information and parol evidence should also be used to verify the results calculated by this procedure are reasonable. This is especially critical for streams in agricultural areas where considerable discharge could be routed through the project site during the irrigation season.

## 3.0 Gage Data Statistics

This procedure is based on the maximum mean daily recorded discharges at streamflow gages during the months the TWM or temporary construction will be in place. Daily statistics for gaged basins in Oregon and nearby states are available in tabular form on the United States Geological Service (USGS) website http://water.usgs.gov/ Daily discharge statistics are also summarized in the USGS "Water Resources Data – Oregon" books issued for each water year. A typical page is shown in the example.

The daily streamflow statistic used in this procedure is the maximum daily mean discharge for the water year during the months the TWM or temporary structure will be in place. These maximums are circled on Figure 2 in the example. The remarks from the stream gage records are also used, and they are inside the box shown on the figure.

*Note: The remarks mentioning upstream storage, consumption, or diversions are not listed on the USGS website with the statistical data. They are included in the USGS annual books, as shown in Figure 1. The remarks pertaining to upstream storage, consumption, or diversions are also listed with the mean daily exceedance flow statistics on the ODOT Geo-Environmental Sections' Engineering and Asset Management Unit website. Remarks for many gages are also listed in the tables to* ***Chapter 7*** *Appendix I.*

## 4.0 Procedure

There are six steps to the envelope curve discharge calculation procedure, and all are discussed in detail in this section and shown in a subsequent example. In summary, the process is as follows.

- Collect station statistics and determine the maximum discharges of record at suitable nearby gages.
- Calculate the yields from the maximum discharges of record, yields from the 5 percent exceedance discharges, and yields from the 5-year events.
- Plot the yields from the maximum discharges of record against the corresponding drainage areas. Draw an envelop curve that includes the maximum yields of record.
- Plot the yields from the 5 percent exceedance discharges and the 5-year events. Compare the envelope curve against these yields. This is done to verify that the envelope curve is reasonable.
- Calculate the discharge from the subject basin based on the envelope curve. This is the maximum discharge expected at the site based on data from nearby gages.
- Compare the discharge calculated in the previous step against water use and parole evidence.

**Step 1 -** Locate gages on the stream or in nearby similar watersheds. Locate the watersheds using guidelines in Appendix J for the exceedance discharges. Tabulate the maximum mean daily discharges during the months the TWM or temporary structure will be in place for each year of record. This is shown in Table 1 of the example.

*Note: The gages used for the exceedance analysis are almost always suitable for this procedure. An exception is when the gage is on a watershed with significant upstream storage, consumptive use, or diversions, and the exceedance data is based on "natural"*

*stream flows that were adjusted to account for these factors. The monthly maximum daily flow data from the gage data will be based on "measured" stream flows and it will not have been adjusted. Data from this gage may be acceptable for the exceedance discharge calculations and it should not be used for the envelope curve analysis. A discussion of "natural" and "measured" stream flows is in Appendix J.*

**Step 2 -** Determine the maximum discharge of record for each gage during the period of the year when the TWM or temporary structure is in place in cubic feet per second. Use the data summarized in the previous step. Divide this maximum discharge by the gaged basin drainage area in square miles. This will calculate the highest recorded daily yield in cubic feet per second per square mile.

Determine the 5 percent exceedance yield for each basin by dividing the 5 percent exceedance discharge by the basin area. The 5 percent exceedance discharges are determined using procedures in Appendix J.

Determine the 5-year flood yields by dividing the 5-year event discharges by the drainage basin areas. The 5-year event discharges can be calculated by the log-Pearson method described in **Chapter 7** or they can be looked up in a suitable reference.

*Note: The previous analyses should provide a single maximum recorded yield, 5 percent exceedance yield, and 5-year event yield for each gaged drainage basin. This is shown in Table 2 of the example.*

**Step 3 -** Select a graph paper with a vertical scale large enough to include all of the yields ranging from the 5 percent exceedance yields up to the 5-year event yields. Assure the horizontal scale is large enough to encompass all of the drainage basin areas. Usually this is graph paper with logarithmic scales on both axes.

Plot the maximum yields of record determined in Step 2 against the drainage areas. Draw an envelope curve that encloses all of the yields of record that are not considered to be outliers. This is shown in Figure 3 of the example.

*Note: Usually four to six gages are used in these analyses. Occasionally the maximum recorded yield from one of these basins is significantly larger than the corresponding yields from the others. It may be possible that this high yield was caused by a freak storm or a torrent due to a debris dam collapse, beaver or man-made dam failure, or other cause. This large flow may be considered to be a "high outlier" and not considered when the envelope curve is drawn.*

**Step 4 -** Plot the yields from the 5 percent exceedance discharges determined in Step 3 for each basin on the graph. In general, the 5 percent exceedance yields are less than the peak discharge of record yields. All calculations and data should be carefully reviewed if this does not occur. An error is likely.

Plot the yields from the 5-year event on the graph. They should be significantly higher than the peak discharge of record yields if the following occurs:

- the TWM or temporary structures will be in place during the dry season, and
- the dry season has significantly less runoff than the flood season.

All calculations and supporting data should be reviewed if the period of the year used for the envelope curve is the dry season and the curve predicts yields similar to or greater than the 5-year yields.

**Step 5 -** Calculate the highest yield expected from the subject basin based on the envelope curve. This is shown in Figure 3 of the example. Estimate the highest expected discharge in cubic feet per second by multiplying the subject basin yield in cubic feet per second per square mile by the subject basin area in square miles.

**Step 6 -** Compare the calculated discharge from Step 5 against parole evidence collected during the exceedance discharge calculation procedure. Consider water use. The discharge calculated from the envelope curve should be greater than discharges routinely observed at the site. The cause should be investigated if this does not occur. One or more of the following may be possible:

- there is an error in the envelope curve procedure,
- the subject basin has drainage characteristics unlike nearby gaged basin, such as an underground water source (spring or seep),
- there is water diversion into the upstream watershed,
- the dry season flows are augmented by discharge from a wastewater treatment facility or other industrial source, or
- water stored during the flood season is released into the stream during the dry season.

The flow calculated in Step 5 may need to be adjusted using water use data and parol evidence.

## 5.0 Reporting Discharge Estimate

The maximum predicted discharge based on station records and envelope curve is listed in the TWM section of the Hydraulics Report and shown on the TWM plans. It may also be shown on the plan sheets for the temporary structures it applies to, such as cofferdams or workbridges. The following data should accompany the listed discharge:

- the starting and ending dates of the period of the year to which the discharge applies,
- the name or other description of the drainage,
- a statement saying the listed discharge is based on surface water runoff from the upstream watershed, if this is the case,
- a statement describing the discharge source if it is not surface runoff, and
- a statement saying the listed discharge is based on data from nearby gaged basins and discharges in the subject basin may differ.

The maximum predicted discharge can be included in the table accompanying the exceedance discharges. These notes do not need to be repeated if this is the case.

## 6.0 Example – Maximum Predicted Discharge Based on Station Records and Envelope Curve

Honey Creek is a small tributary to the North Umpqua River draining a forested mountain watershed. TWM and temporary construction may be in place during the in-water work period of July 1 through September 15. The watershed area is measured on a USGS 7.5 minute quadrangle map with a planimeter. It is 5.35 square miles. The mouth of the drainage basin is at elevation 850 feet.

One temporary feature will be a water diversion pipe under the North Umpqua Highway. This pipe will have minimal fill cover. It is critical that this pipe pass all discharges expected during the in-water work period with minimal upstream headwater elevation. This will keep the highway from overtopping and the pipe from washing out. The risk of potential damage and delay to traffic justifies use of the maximum predicted discharge as a diversion pipe check flow rather than the 5-percent discharge. The 5-percent discharge will be used as the design flow.

**Step 1 -** This watershed is in undeveloped condition. The basin has not been gaged. There are several gaged basins in the surrounding area on rural mountainous forested watersheds.

Gage data from six nearby basins will be used to estimate discharges from the Honey Creek basin. These basins are shown in Figure 1. They are:

**Gage 14316495 Boulder Creek near Tokatee Falls,** drainage area = 30.4 square miles, gage elevation = 1,640 feet. Boulder Creek is a tributary to the North Umpqua River to the east of Honey Creek.

**Gage 14316700 Steamboat Creek near Glide,** drainage area = 227 square miles, gage elevation = 1,128 feet. Steamboat Creek is another tributary to the North Umpqua River east of Honey Creek.

**Gage 14317600 Rock Creek near Glide,** drainage area = 97.4 square miles, gage elevation = 940 feet. Rock Creek is a tributary to the North Umpqua River to the west of Honey Creek.

**Gage 14318000 Little River at Peel,** drainage area = 177 square miles, gage elevation = 828 feet. Little River is another tributary to the North Umpqua River west of Honey Creek.

**Gage 14319850 Gassy Creek near Nonpareil,** drainage area = 9.19 square miles, gage elevation = 790 feet. Gassy Creek is a tributary to Calapooya Creek. The Calapooya drainage is adjacent to, and immediately to the north of, the North Umpqua River drainage.

**Gage 14319900 Calapooya Creek at Nonpareil,** drainage area = 88.6 square miles, gage elevation = 699 feet.

Daily discharge data is available for all of these watersheds from the USGS website or annual USGS "Water Resources Data – Oregon" publications. A page from the Water Year 1988 publication listing Calapooya Creek data is shown in Figure 2. The "Remarks" tell whether or not the stream was regulated or had major diversions during the water year. The remarks for Calapooya Creek are enclosed in a box drawn on Figure 2. The remarks state there are minor diversions, only. Regulation is not mentioned. No reservoirs are shown on the USGS 7-1/2 minute quadrangle map. It is assumed the watershed is unregulated. This step was repeated for each of the other five watersheds to verify they were unregulated.

The maximum recorded daily mean discharges in Calapooya Creek during July, August, and September of the 1988 Water Year period are circled. This step is repeated for the other five gages for each year of record. The maximum discharges are tabulated for each gage. The tabulation for the Calapooya Creek gage is shown in Table 1.

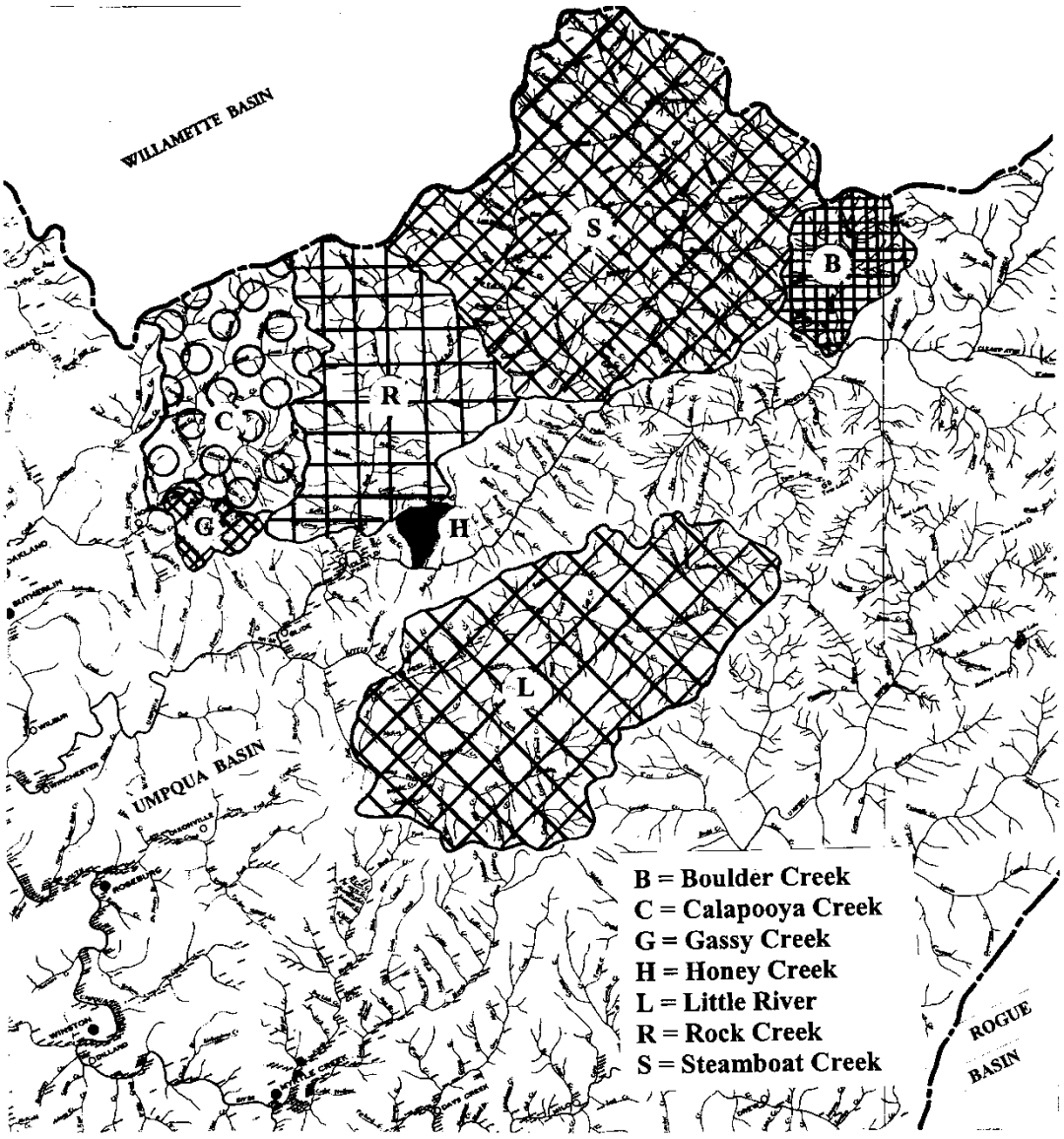



**Figure 1 Location of Nearby Watersheds**

236 NORTH UMPQUA RIVER BASIN

14319900 CALAPOOYA CREEK AT NONPAREIL, OR

LOCATION.--Lat 43°25'04", long 123°09'13", in SW 1/4 SE 1/4 sec.3, T.25 S., R.4 W., Douglas County, Hydrologic Unit 17100303, on left bank 0.3 mi upstream from county road bridge, 0.9 mi northeast of Nonpareil, and at mile 26.7.

DRAINAGE AREA.--88.6 $mi^2$.

PERIOD OF RECORD.--July 1976 to September 1988 (discontinued).

GAGE.--Water-stage recorder and crest-stage gage. Datum of gage is 699.22 ft above National Geodetic Vertical Datum of 1929 (Douglas County Survey bench mark).

REMARKS.--Records good except those for estimated daily discharges and periods of backwater Oct. 1 to Dec. 1 and July 19 to Sept. 30, which are fair. Only minor diversions by pumping for irrigation upstream from station. Several measurements of water temperature were made during the year.

AVERAGE DISCHARGE.--12 years, 201 $ft^3/s$, 30.81 in/yr, 145,600 acre-ft/yr.

EXTREMES FOR PERIOD OF RECORD.--Maximum discharge, 7,640 $ft^3/s$ Dec. 6, 1981, gage height, 11.16 ft; minimum discharge, 3.7 $ft^3/s$ Sept. 23-25, 1987.

EXTREMES FOR CURRENT YEAR.--Peak discharges greater than base discharge of 2,700 $ft^3/s$ and maximum (*):

| Date | Time | Discharge ($ft^3/s$) | Gage height (ft) | Date | Time | Discharge ($ft^3/s$) | Gage height (ft) |
|---|---|---|---|---|---|---|---|
| Jan. 10 | 0730 | *3,040 | *7.49 | No other peak greater than base discharge. | | | |

Minimum discharge, 4.1 $ft^3/s$ Oct. 2.

DISCHARGE, CUBIC FEET PER SECOND, WATER YEAR OCTOBER 1987 TO SEPTEMBER 1988
MEAN VALUES

| DAY | OCT | NOV | DEC | JAN | FEB | MAR | APR | MAY | JUN | JUL | AUG | SEP |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 4.8 | 11 | 112 | 142 | e300 | 111 | 159 | 276 | 538 | 46 | 17 | 9.6 |
| 2 | 4.5 | 17 | 194 | 128 | e250 | 100 | 164 | 281 | 472 | 44 | 18 | 9.0 |
| 3 | 4.5 | 18 | 460 | 126 | e210 | 113 | 313 | 374 | 404 | 46 | 16 | 7.7 |
| 4 | 5.2 | 13 | 534 | 129 | e190 | 132 | 406 | 397 | 320 | 47 | 14 | 7.2 |
| 5 | 6.5 | 12 | 362 | 128 | e170 | 195 | 377 | 356 | 266 | 43 | 13 | 7.0 |
| 6 | 6.8 | 12 | 427 | 123 | e160 | 238 | 321 | 340 | 221 | 42 | 13 | 7.2 |
| 7 | 6.5 | 12 | 482 | 120 | e150 | 242 | 341 | 570 | 200 | 38 | 13 | 7.9 |
| 8 | 5.9 | 12 | 345 | 141 | e140 | 231 | 299 | 528 | 178 | 36 | 12 | 8.2 |
| 9 | 6.8 | 14 | 313 | 480 | e200 | 513 | 259 | 476 | 183 | 34 | 10 | 8.3 |
| 10 | 7.5 | 15 | 1020 | 2340 | e550 | 493 | 220 | 385 | 259 | 32 | 9.9 | 7.5 |
| 11 | 7.7 | 14 | 634 | 1410 | e400 | 345 | 186 | 304 | 225 | 31 | 11 | 7.4 |
| 12 | 7.1 | 37 | 362 | 833 | e330 | 262 | 162 | 274 | 187 | 32 | 13 | 7.5 |
| 13 | 6.1 | 46 | 253 | 553 | e280 | 213 | 146 | 315 | 160 | 30 | 12 | 6.9 |
| 14 | 5.4 | 51 | 199 | 717 | e230 | 181 | 156 | 268 | 141 | 30 | 13 | 6.3 |
| 15 | 6.8 | 31 | 185 | e1450 | e200 | 159 | 140 | 234 | 124 | 30 | 13 | 5.5 |
| 16 | 7.3 | 32 | 283 | 976 | e190 | 142 | 130 | 252 | 111 | 28 | 13 | 5.9 |
| 17 | 7.7 | 28 | 234 | 665 | e170 | 126 | 128 | 245 | 101 | 26 | 13 | 6.7 |
| 18 | 7.4 | 22 | 180 | 478 | e140 | 115 | 123 | 221 | 93 | 25 | 13 | 7.4 |
| 19 | 7.4 | 17 | 151 | 358 | 128 | 107 | 122 | 195 | 87 | 24 | 12 | 17 |
| 20 | 7.6 | 14 | 131 | e280 | 123 | 100 | 125 | 174 | 81 | 23 | 11 | 21 |
| 21 | 7.3 | 16 | 159 | e240 | 118 | 97 | 432 | 158 | 73 | 22 | 10 | 12 |
| 22 | 7.0 | 18 | 211 | e220 | 113 | 93 | 538 | 144 | 68 | 21 | 9.9 | 9.3 |
| 23 | 6.8 | 29 | 226 | e210 | 108 | 133 | 387 | 133 | 65 | 21 | 9.1 | 8.2 |
| 24 | 6.8 | 42 | 200 | e220 | 103 | 142 | 283 | 124 | 62 | 20 | 8.0 | 7.7 |
| 25 | 6.8 | 68 | 172 | e200 | 97 | 148 | 227 | 117 | 59 | 20 | 8.9 | 7.9 |
| 26 | 6.5 | 38 | 152 | e190 | 94 | 161 | 190 | 110 | 58 | 20 | 9.0 | 12 |
| 27 | 6.6 | 27 | 137 | e180 | 90 | 225 | 164 | 104 | 56 | 19 | 9.2 | 26 |
| 28 | 6.8 | 21 | 133 | e190 | 87 | 231 | 150 | 122 | 54 | 18 | 9.5 | 17 |
| 29 | 6.4 | 17 | 145 | e220 | 84 | 217 | 178 | 138 | 56 | 17 | 9.6 | 12 |
| 30 | 6.8 | 19 | 142 | e340 | --- | 199 | 215 | 137 | 50 | 16 | 9.7 | 9.9 |
| 31 | 8.7 | --- | 155 | e400 | --- | 174 | --- | 142 | --- | 17 | 9.5 | --- |
| TOTAL | 206.0 | 723 | 8693 | 14187 | 5405 | 5938 | 7041 | 7894 | 4952 | 898 | 362.3 | 293.2 |
| MEAN | 6.65 | 24.1 | 280 | 458 | 186 | 192 | 235 | 255 | 165 | 29.0 | 11.7 | 9.77 |
| MAX | 8.7 | 68 | 1020 | 2340 | 550 | 513 | 538 | 570 | 538 | 47 | 18 | 26 |
| MIN | 4.5 | 11 | 112 | 120 | 84 | 93 | 122 | 104 | 50 | 16 | 8.0 | 5.5 |
| AC-FT | 409 | 1430 | 17240 | 28140 | 10720 | 11780 | 13970 | 15660 | 9820 | 1780 | 719 | 582 |
| CFSM | .08 | .27 | 3.17 | 5.17 | 2.10 | 2.16 | 2.65 | 2.87 | 1.86 | .33 | .13 | .11 |
| IN. | .09 | .30 | 3.65 | 5.96 | 2.27 | 2.49 | 2.96 | 3.31 | 2.08 | .38 | .15 | .12 |

CAL YR 1987 TOTAL 46310.5 MEAN 127 MAX 1580 MIN 3.7 AC-FT 91860 CFSM 1.43 IN. 19.44
WTR YR 1988 TOTAL 56592.5 MEAN 155 MAX 2340 MIN 4.5 AC-FT 112300 CFSM 1.75 IN. 23.76

e Estimated

**Figure 2 Page from USGS Annual "Water Resources Data – Oregon" Publication**

**HIGHEST MEAN DAILY FLOW FOR MONTHS**
**TWM OR TEMPORARY STRUCTURES WILL BE IN-PLACE**

| CALAPOOYA CREEK GAGE # 14319900 | | | |
|---|---|---|---|
| WATER YEAR | JULY HIGHEST DISCHARGE[1] | AUGUST HIGHEST DISCHARGE[1] | SEPTEMBER HIGHEST DISCHARGE[1] |
| 1976 | 36 | 101 | 23 |
| 1977 | 26 | 39 | 135 |
| 1978 | 59 | 56 | 188 |
| 1979 | 30 | 25 | 56 |
| 1980 | 173 | 54 | 20 |
| 1981 | 51 | 14 | 207 |
| 1982 | 50 | 23 | 74 |
| 1983 | 234 | 94 | 56 |
| 1984 | 69 | 23 | 18 |
| 1985 | 51 | 14 | 207 |
| 1986 | 50 | 13 | 373[2] |
| 1987 | 189 | 33 | 11 |
| 1988 | 47 | 18 | 26 |

[1] All discharges in cubic feet per second.
[2] Highest recorded daily discharge for period of record during TWM work period. This discharge is used in subsequent basin yield calculation.

**Table 1 Highest Monthly Mean Daily Discharge Tabulation for Period of Record**

**Step 2 -** The highest daily mean discharge during the period of record is determined for each of the six gages. This discharge for Calapooya Creek is 373 cubic feet per second. This occurred in September 1986 during the 1976 through 1988 period of record, as shown in Table 1.

The highest mean daily discharges for all six gages, in cubic feet per second, are divided by their respective drainage areas in square miles. This calculates the highest yields of record during the in-water work period in cubic feet per second per square mile. Table 2 shows these yields for all six gages. The yield from the Calapooya Creek basin is 4.2 cubic feet per second per square mile.

The 5 percent exceedance discharges were calculated using procedures in **Chapter 7** Appendix J. These discharges are divided by their respective drainage areas to get the 5 percent exceedance yields shown in Table 2. The largest 5 percent yield for the Calapooya Creek basin is predicted to occur in July and it is 0.73 cubic feet per second per square mile.

The 5-year event discharges were calculated using log-Pearson Type III procedures as discussed in **Chapter 7**. These discharges are divided by their respective drainage areas to get the 5-year event yields listed in Table 2. The 5-year event yield from the Calapooya Creek basin is estimated to be 67 cubic feet per second per square mile.

**YIELDS FOR MONTHS TWM WILL BE IN PLACE**
**BASINS NEAR HONEY CREEK**

| GAGE | DRAINAGE AREA (Square miles) | WATER YEAR | MONTH OF HIGHEST DAILY DISCHARGE | HIGHEST DAILY DISCHARGE[1] | HIGHEST YIELD[2,3] | 5 PERCENT EXCEEDENCE YIELD[2,4] | 5-YEAR FLOOD YIELD[2,5] |
|---|---|---|---|---|---|---|---|
| 14319850 | 9.19 | 1993 | JULY | 12 | 1.3 | .24 | 110 |
| 14319900 | 88.6 | 1986 | SEPTEMBER | 373 | 4.2 | .73 | 67 |
| 14316495 | 30.4 | 1999 | JULY | 79 | 2.6 | 1.6 | -- [6] |
| 14316700 | 227.0 | 1986 | SEPTEMBER | 2220 | 9.8 | .83 | 93 |
| 14317600 | 97.4 | 1971 | SEPTEMBER | 334 | 3.4 | .96 | 106 |
| 14318000 | 177.0 | 1986 | SEPTEMBER | 1360 | 7.6 | .73 | 76 |

[1] All discharges in cubic feet per second.
[2] All yields in cubic feet per second per square mile.
[3] Highest yield based on highest recorded daily discharge during TWM work period.
[4] Largest of the monthly 5% exceedence discharges calculated for the TWM work period.
[5] 5- year floods for gaged watersheds based on log-Pearson Type III analyses.
[6] Log-Pearson analysis not done: Insufficient years of record.

**Table 2 Yield Tabulations for Each Gage**

**Step 3 -** The highest yields for the periods of record are plotted on logarithmic graph paper versus the drainage areas. An envelope curve is drawn to include all yields of record except high outliers. The six data points and an example curve is shown in Figure 3.

None of the six points appear to be an outlier. An outlier would be a data point that did not fit the pattern established by the other data points, and it would indicate an exceptionally high or low yield from a basin. The cause of an exceptional event would be investigated. Possible causes are discussed in the previous section. Professional judgment would be used to decide whether or not to include this exceptional yield within the envelope curve.

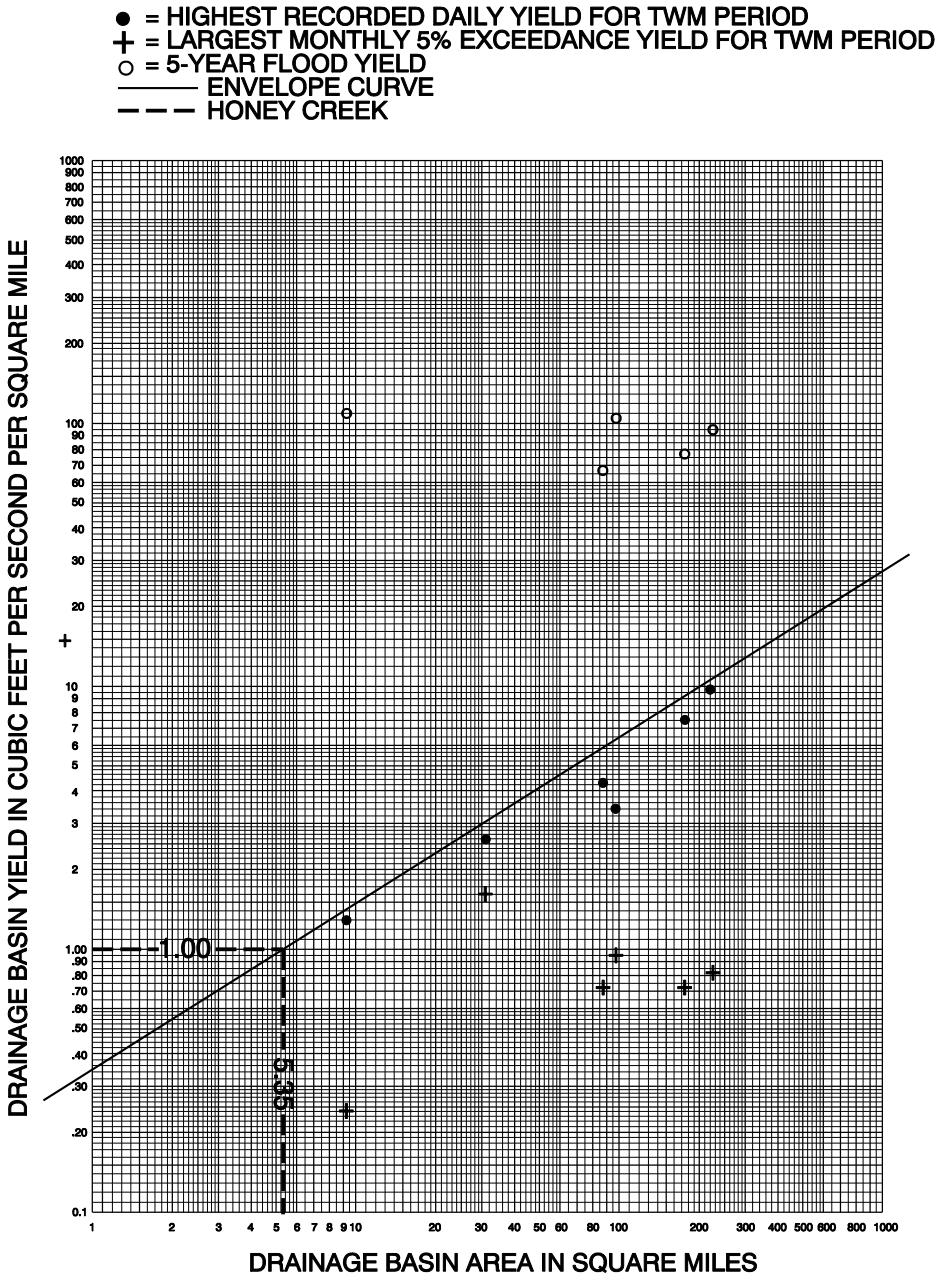



**Figure 3 Envelope Curve for Highest Expected Discharge During TWM Period**

**Step 4 -** Both the 5 percent exceedance discharges and 5 year event yields are plotted on the graph versus their respective drainage areas. The envelope curve is compared to these yields. The curve is lower than the 5-year yields and higher than the 5 percent exceedance discharges, as shown in Figure 3. This is a typical case. The cause would be investigated if it was otherwise. There may be errors in the calculation procedures.

**Step 5 -** The highest yield expected from the subject basin is determined from the envelope curve, as shown in Figure 3. It is 1.0 cubic feet per second per square mile. This is multiplied by the 5.35 square mile Honey Creek drainage basin area to get a maximum predicted discharge of 5.3 cubic feet per second.

**Step 6 -** Water use and parol evidence are considered in this step. Permitted water use in the upstream watershed was determined from information on the OWRD website, as discussed in **Chapter 7** Appendix J. The OWRD Water Rights Point of Diversion Summary lists total permitted withdrawls of 0.76 cubic feet per second. Most of these withdrawls are from small ponds, reservoirs, or tributary creeks on the ridges surrounding the Honey Creek drainage. They are not expected to significantly influence the maximum predicted discharge. One permitted withdrawl is directly from Honey Creek. It is 0.01 cubic feet per second for irrigation. This loss is considered to be minor, and it is not included in the maximum predicted discharge calculations.

Parol evidence was collected by interviewing people familiar with Honey Creek. Nothing in the parol evidence indicates the calculated 5.3 cubic foot per second maximum predicted discharge is not realistic.

**Step 7 -** The maximum predicted discharge is reported as shown in Figure 4. It is reported along with the exceedance discharges.

**HONEY CREEK**
**ESTIMATED DISCHARGES**
**FOR TEMPORARY WATER MANAGEMENT**

| | AVERAGE DAILY DISCHARGE IN CUBIC FEET PER SECOND (GALLONS PER MINUTE) | | |
|---|---|---|---|
| **NOTE** | **1** | **2** | **3** |
| JULY | 4.7 (2100) | 2.1 (940) | 1.4 (630) |
| AUGUST | 1.7 (760) | 0.93 (420) | 0.65 (290) |
| SEPTEMBER | 1.4 (630) | 0.81 (360) | 0.49 (220) |
| MAXIMUM PREDICTED DISCHARGE | | | |
| JULY 1 – SEPTEMBER 30 | | | 5.3 ( 2400 ) |

1) 5 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 2 days each month.)
2) 25 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 8 days each month.)
3) 50 Percent Exceedance Discharge (Average daily discharge expected to be exceeded 16 days each month.)

Discharges are not expected to exceed the maximum predicted discharge.

In-water work period extends from 1 July through 15 September. Temporary water management shown on plans recommended throughout in-water work period. Listed discharges are surface water runoff from the upstream watershed. The estimated discharges are based on nearby gaged basins. Discharges in the subject watershed may differ.

**Figure 4 Table Reporting Maximum Predicted Discharge**

# Chapter 8

## CHANNELS

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## 8.1 Introduction

Open channels, whether natural or man-made, must have a free surface that is exposed to atmospheric pressure. For any closed conduit (pipe, box, or arch) to operate as an open channel, there must be an air space between the water surface and the inside top of the conduit. In other words, a closed conduit operating as an open channel can only flow partially full. Open channel hydraulics are always used to design these highway facilities:

- pavement gutters,
- roadside ditches,
- water quality swales,
- stream or other channel changes and reconstruction, and
- bridges.

The following facilities function under either pressure or open channel flow. Open channel flow principles are often used to evaluate the hydraulic performance of:

- storm drainage systems, and
- culverts.

Open channels are of two types, natural or artificial. Natural channels are:

- usually stream channels with their size and shape determined by natural forces,
- typically compound in cross-section with a main channel for conveying low flows and a floodplain to transport flood flows, and
- usually shaped geomorphologically by the long term effects of the sediment load and water discharge which they convey.

Artificial channels include roadside channels and ditches, irrigation canals and ditches, drainage canals and ditches, water quality swales, and closed conduits. Artificial channels are:

- constructed channels with regular geometric cross-sections, and
- unlined, or lined with artificial or natural material to protect against erosion.

## 8.2 Policy and Practice

General policies of the Federal government and ODOT pertaining to hydraulics are discussed in **Chapter 3**. Agency practices specific to channels include:

- coordination with other Federal, State, and local agencies concerned with water resources

planning and management will have high priority in the design of channels,
- safety of the general public is an important consideration in the selection of the cross-sectional geometry of artificial drainage channels,
- the design of artificial drainage channels or other facilities shall consider the frequency and type of maintenance expected and make allowance for access of maintenance equipment and personnel,
- a stable channel is the goal for all channels that are located on highway right-of-way or that impact highway facilities,
- environmental impacts of channel modifications, including disturbance of fish habitat, wetlands, and riparian areas must be minimized as much as practicable, and
- the channel design shall be checked using a range of discharges based on class of roadway, consequences of traffic interruption, flood hazard risks, economics, potential damage to private property, and local site conditions.

Exceptions to design practices should be approved by the ODOT Geo-Environmental Section's Engineering and Asset Management Unit.

## 8.3 Design Criteria

Policies and practice define a definite course of action to guide and determine present and future decisions. Design criteria are developed to implement the specific policies and practice. Design criteria for channels follow.

### 8.3.1 Large Natural and Artificial Channels

The following criteria apply to natural and artificial channels designed to convey more than 50 cubic feet per second. They apply to both new channels and revisions to existing channels.

- Channels conveying stormwater runoff are designed and checked using the discharge recurrence intervals listed in **Chapter 3**. Higher design or check discharges may be justified for critical installations where considerable danger to the public or expense could occur as a result of a failure.
- Channels conveying regulated flow are designed to convey the operating discharges. These discharges are provided by irrigation districts, the U.S. Bureau of Reclamation, etc. These channels are checked using the discharge recurrence intervals listed in **Chapter 3** if they convey stormwater runoff.
- Channels are evaluated for bankfull flow if it can occur.
- The cross-sectional shape, meander, pattern, roughness, sediment transport, and slope of a relocated channel should mimic to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.

- Bank stabilization should be provided, where appropriate, as a result of any channel encroachment or realignment. This may include both the upstream and downstream banks as well as the local site.
- Features such as dikes and levees associated with channel modifications should have a 16-foot minimum top width to allow access for maintenance equipment. Turnaround points should be provided no further than 900 feet apart and at the end of any such feature. Concurrence on the minimum top width and turnaround point spacing should be obtained from the jurisdiction responsible for maintaining the dikes and levees.
- Dikes and levees should have a minimum of 1 foot, or two velocity heads, of freeboard above the design flow elevations.
- Levee, dike, or roadway embankment side slopes acting as channel banks should not have side slopes in excess of the angle of repose of the soil and/or lining. Side slopes should be 1V:1-1/2H or flatter in the case of loose riprap lining.
- The effects of wave action, increased flow velocities around bends, etc. should be considered in the channel design, if applicable.

Regulatory agency standards and criteria may also apply to the channel, and the design should satisfy these requirements. Irrigation ditch and canal crossings are almost always subject to U.S. Bureau of Reclamation or irrigation district standards and criteria. Additional guidance on large channel design is included in the remainder of this chapter and **Chapter 15**, Bank Protection.

### 8.3.2 Small Natural and Artificial Channels

Small natural and artificial channels are designed to convey less than 50 cubic feet per second. This includes roadside and most drainage ditches, water quality swales, channel reconstruction upstream and downstream from small culverts and bridges, and gutters. Gutters are the smallest artificial channels. Most gutters are designed to convey less than 2 cubic feet per second.

Small natural channels should match, as much a practicable, the natural channel curvature, slope, hydraulic roughness, and cross-section. This is not required for small artificial channels if they are properly protected from erosion or scour. The channel should be stable, both in vertical and horizontal alignment. Biological elements should be incorporated where possible to enhance habitat functions. The biological elements may also be used to provide channel stability if they will provide adequate performance and durability.

Gutters and roadside ditches are almost always within the clear zone alongside the highway. Ditches within the clear zone typically are shallow with relatively flat side slopes. These facilities are designed using criteria based on traffic safety as well as hydraulic characteristics. These facilities are designed to standards and criteria in the current ODOT "Highway Design Manual", the most recent ODOT "Standard Drawings," guidance in this chapter, and criteria in **Chapter 13**, Storm Drainage.

Roadside ditches, drainage ditches, and most small channels outside of the clear zone are usually designed with deeper sections and steeper side slopes than facilities in the clear zone. Hydraulic characteristics often govern the design of these ditches. Design criteria for smaller artificial channels are listed below. Additional guidelines for small channel design are in the remainder of this chapter.

- Roadside ditches and permanent ditch linings should be designed using the design discharge recurrence intervals listed in **Chapter 3**.
- Temporary ditch linings should be designed for the 2-year flow.
- If a ditch is relocated or realigned, the cross-sectional shape, roughness, and slope should conform to the existing conditions insofar as practicable.
- Side slopes of ditches outside of the clear zone should not exceed the angle of repose of the soil and/or lining. Side slopes should be 1V:1-1/2H or flatter in the case of loose riprap lining.
- Flexible linings should be designed according to the method of permissible unit shear stress.
- The design flow should be confined within the banks of the ditch. In cases where there are no defined ditch banks, such as flow alongside the toe of an embankment fill, the design flow should be confined to ODOT right-of-way.

Water quality swales are designed to the criteria and guidelines in **Chapter 14**, Water Quality, supplemented by the guidelines in this chapter.

## 8.4 Documentation - Channel Design Studies

The results from typical ODOT channel studies are summarized in this subsection. Documentation may need to be sealed by a professional engineer registered in Oregon, as discussed in **Chapter 3**.

### 8.4.1 Documentation – Large Natural and Artificial Channels

The Preliminary Hydraulics Recommendations, Hydraulics Reports, and distribution list for large channel studies generally follow the bridge and large culvert documentation guidelines in **Chapter 4**. The difference is, the recommendations and report addresses a large channel instead of a bridge or large culvert. Specific information related to channels includes the following:

- Hydrologic and hydraulic data for design, and check floods if the channel conveys stormwater, as listed in **Chapter 3**.
- Hydrologic and hydraulic data for the operating flow, if applicable.
- Hydrologic and hydraulic data for the bankfull flow, if it occurs.
- A summary of the design standards from other agencies which influence the design, and documentation showing the proposed design complies with the standards.

- Cross-sections of the new, existing, or realigned channels, showing dimensions, Manning's n values, and flow depths.
- Channel bottom profiles and water surface profiles of the new, existing, or realigned channels.
- Hydraulic data tables for the new, existing, or realigned channels listing flows, water depths, water surface elevations at critical locations, and velocities. Sample hydraulic data tables for bridges and culverts are shown in **Chapter 4**. A similar data table is recommended for large channels.
- A description of recommended highway embankment and channel erosion protection.
- Recommendations about maintenance and access for maintenance.
- The printout from a step-backwater analysis, showing velocity and flow depth, energy, and channel bottom shear at sections of both the modified channel and the adjacent natural channels both upstream and downstream.

### 8.4.2 Documentation - Small Natural and Artificial Channels

The documentation and supporting data guidelines for small culverts in **Chapter 4** are recommended as a model for small channel studies. The small channel study will address a small channel instead of a small culvert. Specific information related to small channels includes the following.

- Applicable hydrology.
- Manning's "n" value used in design.
- Design and check flow depths and velocities.
- A description or drawing of the channel, including location, critical elevations, slope, bottom width, depth, side slopes, and recommended lining.

## 8.5 Definitions

The Glossary in this manual provides definitions of many terms used in open channel hydraulics.

## 8.6 Equations

Analysis of open channel flow in both natural and artificial channels uses the basic principles of fluid mechanics -- energy, continuity, and momentum. These principles are described by mathematical relationships. This section discusses the equations expressing these relationships.

### 8.6.1 Energy Equations

Energy relationships in open channel hydraulics are often described using the term "head." Head

represents energy per unit weight of fluid which has dimensions of length and is therefore a linear measurement. Commonly used heads are defined in the Glossary.

**Energy Equation -** The energy equation expresses conservation of energy in open channel flow. The energy equation should only be applied between two cross-sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected. Written between an upstream open channel Cross-section 1 and a downstream Cross-section 2, the energy equation is:

$$h_1 + \alpha_1 \left( \frac{V_1^2}{2g} \right) = h_2 + \alpha_2 \left( \frac{V_2^2}{2g} \right) + h_L \qquad \text{(Equation 8-1)}$$

Where:

$h_1$ and $h_2$ are the upstream and downstream stages, respectively in feet
$\alpha$ = velocity distribution coefficient
V = mean velocity in feet per second
g = gravitational acceleration, 32.2 feet per second squared
$h_L$ = energy head loss between Cross-sections 1 and 2 due to minor losses and/or friction loss, in feet

The terms in the energy equation are illustrated graphically in Figure 8-1. The energy equation states that the total energy head at the upstream cross-section is equal to the total energy head at the downstream cross-section plus the intervening energy head loss. The terms used in the energy equation are as follows:

**Stage -** The stage (h) is the sum of the elevation head (z) at the channel bottom and the pressure head, y.

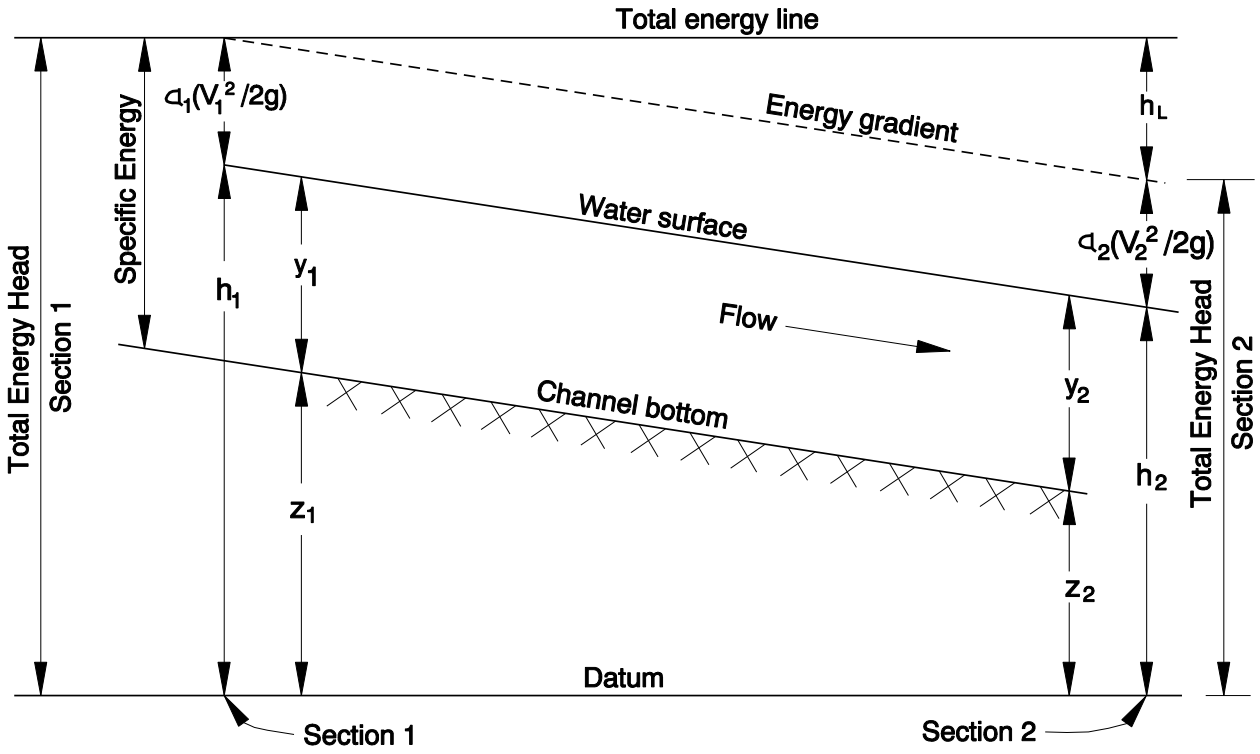
$$h = z + y \qquad \text{(Equation 8-2)}$$

**Velocity Head -** The velocity head is calculated as follows:

$$h_{velocity} = \alpha \frac{V^2}{2g} \qquad \text{(Equation 8-3)}$$

Where:

$h_{velocity}$ = velocity head in feet
$\alpha$ = velocity distribution coefficient (see Equation 8-5)
V = mean velocity in feet per second
G = gravitational acceleration, 32.2 feet per second squared




$\alpha$ = Velocity distribution coefficient
$g$ = Gravitational acceleration
$h$ = Stage elevation above datum
$h_L$ = Energy head loss
$y$ = Flow depth, typically called pressure head
$z$ = Channel bottom elevation above datum, typically called elevation head

**Figure 8-1 Terms in the Energy Equation**

**Specific Energy -** Specific energy (E) is typically expressed as a linear measurement relative to the channel bottom called specific energy head. If the channel is not too steep (slope less than 10 percent) and the streamlines are nearly straight and parallel, E becomes the sum of the pressure and velocity heads:

$$E = y + h_{velocity} \qquad \text{(Equation 8-4)}$$

Where:

E = specific energy head in feet
y = pressure head in feet (this is flow depth in an open channel)
$h_{velocity}$ = velocity head (see Equation 8-3)

**Velocity Distribution Coefficient -** Velocities are not uniformly distributed in a channel section because the top of the flow is bounded by a free surface causing negligible friction drag, and the wetted perimeter is bounded by the channel sides or the overbanks which may impart considerable friction drag. Velocity distributions for typical cross-section shapes are shown in the popular textbook by Ven Te Chow titled "Open-Channel Hydraulics."

As a result of this nonuniform velocity distribution, the actual velocity head of an open channel is usually greater than the average velocity head computed as $(Q / A_t)^2 / 2g$ where $A_t$ is total waterway cross-section area. The average velocity head calculated by the preceding equation is multiplied by the velocity distribution coefficient ($\alpha$) to more accurately estimate the actual velocity head.

Velocity distribution coefficients should be considered in the water surface profile calculations when they significantly affect the velocity head. This most often occurs in waterways having hydraulically efficient main channels and relatively constricted overbank areas. In addition, this coefficient should be considered when the water surface profile is to be calculated with a high degree of accuracy. Almost all modern step-backwater analysis programs calculate and use this coefficient.

Velocity distribution coefficients are often ignored or considered to be 1.0 when analyzing channels without overbank areas, such as conduits, ditches, canals, etc. They are also ignored in applications where accurate hydraulic estimates are not necessary.

The velocity distribution coefficient is also useful when determining the highest probable velocity of waterborne debris or ice. The velocity of these materials in the center of the main channel may be significantly greater than the average velocity in the cross-section. These higher velocities can be estimated by multiplying the average velocity in the section by the velocity distribution coefficient. Conversely, the velocity distribution coefficient can also be used to estimate the slower velocities debris or ice will have in less hydraulically efficient overbank areas.

There are several methods to determine the velocity distribution coefficient. The coefficient for natural channels can be calculated using the method in the US Corps of Engineers HEC-RAS step

backwater program. It is based on the conveyance in three flow elements: left overbank, right overbank, and channel. It can be written in terms of conveyance and area as follows:

The velocity coefficient (α) is computed based on the conveyance in the three flow elements: left overbank, right overbank, and channel. It can also be written in terms of conveyance and area as in the following equation:

$$a = \frac{(A_1)\left[\frac{K_{lob}^3}{A_{lob}^2} + \frac{K_{ch}^3}{A_{ch}^2} + \frac{K_{rob}^3}{A_{rob}^2}\right]}{K_1^3} \qquad \text{(Equation 8-5)}$$

Where:

$A_1$ = total flow areas of cross-section in square feet

$A_{lob}$, $A_{ch}$, $A_{rob}$ = flow areas of left overbank, main channel, and right overbank, respectively, in square feet

$K_1$ = total conveyance of cross-section in cubic feet per second

$K_{lob}$, $K_{ch}$, $K_{rob}$ = conveyance of left overbank, main channel, and right overbank, respectively, in cubic feet per second

Calculating the velocity distribution coefficient using Equation 8-5 can be time consuming if done by hand. An approximate coefficient based on typical velocity distributions is adequate for most hand calculation purposes. Typical velocity distribution coefficients, based on the January 1956 publication by Steponas Kolupaila "Methods of determination of the kinetic energy factor" in The Port Engineer, Calcutta, India, Volume 5, Number 1, are:

- regular channels, flumes, spillways: 1.10 minimum, 1.15 average, 1.20 maximum,
- natural streams and torrents: 1.15 minimum, 1.30 average, 1.50 maximum,
- rivers under ice cover: 1.20 minimum, 1.50 average, 2.00 maximum, and
- river valleys, overflooded: 1.50 minimum, 1.75 average, 2.00 maximum.

**Minor Losses -** Flow contraction or expansion between cross-sections results in energy losses. These losses are often called “minor losses” and they are a function of the difference in velocity heads between upstream Cross-Section 1 and downstream Cross-Section 2, as follows:

$$h_L = C_c \Delta h_{velocity} = C_c \left[\left(\frac{\alpha_2 V_2^2}{2g}\right) - \left(\frac{\alpha_1 V_1^2}{2g}\right)\right] \qquad \text{(Contraction head loss, Equation 8-6)}$$

$$h_L = C_e \Delta h_{velocity} = C_e \left[\left(\frac{\alpha_1 V_1^2}{2g}\right) - \left(\frac{\alpha_2 V_2^2}{2g}\right)\right] \qquad \text{(Contraction head loss, Equation 8-7)}$$

Where:

$h_L$ = minor loss between bounding cross-sections in feet

$\Delta h_{velocity}$ = change in velocity heads between bounding cross-sections

Contraction and expansion coefficients are as follows;

- No transition: $C_c = 0.0$ and $C_e = 0.0$
- Gradual transition: $C_c = 0.1$ and $C_e = 0.3$
- Typical transition at bridge: $C_c = 0.3$ and $C_e = 0.5$
- Abrupt transition: $C_c = 0.6$ and $C_e = 0.8$

*Note: Contraction and expansion losses are always zero or a positive value.*

**Friction Loss -** Friction between the flow and the wetted perimeter causes friction loss. Between two cross-sections, the friction loss is calculated as follows:

$$h_F = LS_f \qquad \text{(Equation 8-8)}$$

Where:

$h_F$ = friction loss between bounding cross-sections in feet

L = reach length in feet. Sometimes weighted reach lengths are used (see Equations 8-22 and 8-23).

$S_f$ = friction slope (see Equations 8-9 or 8-10)

*Note: Friction loss is always a positive value.*

**Friction Slope -** The friction slope is also the energy grade line slope. It is expressed as the friction loss per unit length of channel, and it can be calculated by several methods. Two commonly used methods are:

- Average conveyance equation

$$S_f = \left( \frac{Q_u + Q_d}{K_u + K_d} \right)^2 \qquad \text{(Equation 8 - 9)}$$

- Geometric mean friction slope equation

$$S_f = (S_{fu} S_{fd})^{1/2} \qquad \text{(Equation 8-10)}$$

Where:

$S_f$ = friction slope is feet per foot

$Q_u$, $Q_d$ = discharge at the upstream and downstream cross-sections, respectively, in cubic feet per second

$K_u$, $K_d$ = conveyance at the upstream and downstream cross-sections, respectively, in cubic

feet per second

$S_{fu}$, $S_{fd}$ = friction slope at the upstream and downstream cross-sections, respectively, in foot per foot

*Note: The friction slope is equal to the bed slope during relatively uniform flow in nearly uniform channels where expansion and contraction losses are negligible.*

### 8.6.2 Continuity Equation

The concept of continuity results from the principle of conservation of mass. The mass of fluid is the same passing through all sections of a stream, per unit of time, if the rate of flow is constant.

**Continuity Equation -** The continuity relationship is expressed by the continuity equation as follows:

$$Q = A_1 V_1 = A_2 V_2 \qquad \text{(Equation 8-11)}$$

Where:

Q = discharge in cubic feet per second
A = cross-sectional area of flow in square feet
V = mean cross-sectional velocity in feet per second
Subscripts 1 and 2 refer to successive cross-sections along the flow path.

### 8.6.3 Manning's Equations

Normal flow occurs in an open channel when the energy gradient, water surface, and bed are parallel. In nature, normal flow typically occurs in long prismatic channels such as canals. Normal flow rarely occurs in natural channels. In the analysis of natural waterways, however, it is assumed that normal flow occurs in the more uniform reaches of the channel. As a result, normal flow equations such as the Manning's formula are used extensively in hydraulic modeling.

**Manning's Equation -** The mean velocity, V, can be computed with Manning's equation for a given depth of flow in an open channel with a steady, uniform flow:

$$V = \left(\frac{1.486}{n}\right) R^{2/3} S^{1/2} \qquad \text{(Equation 8-12)}$$

Where:

V = mean velocity in feet per second
n = Manning's roughness coefficient
R = hydraulic radius = A / P, in feet
A = cross-sectional area in square feet
P = wetted perimeter in feet

S = slope of the energy gradeline in feet per feet

Note: For steady uniform flow, S = channel slope

The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n values. Selecting Manning's n values for natural channels is discussed in Subsections 8.10.2 and 8.10.4. Ranges of n values for various channels and floodplains are listed in Appendix A. Manning's n can also be calculated for normal flow if the water surface elevation, and either flow or velocity are known, using the following equations:

$$n = \frac{1.486\,(R^{2/3}\,S^{1/2})}{V}$$ , if velocity is known (Equation 8-13)

$$n = \frac{1.486\,(A\,R^{2/3}\,S^{1/2})}{Q}$$ ,if flow is known (Equation 8-14)

**Manning's Equation for Discharge -** The continuity equation can be combined with Manning's equation to obtain Manning's equation for discharge:

$$Q = \left(\frac{1.486}{n}\right) A\,R^{2/3}\,S^{1/2}$$ (Equation 8-15)

Where:

Q = discharge in cubic feet per second

A unique flow depth occurs during steady, uniform flow for a given channel geometry, slope, roughness, and discharge Q. It is called normal depth, and it can be calculated for natural stream channels and many artificial channel shapes using the procedures in the remainder of this chapter.

**Conveyance Equation -** The first terms of Equation 8-15 can be combined to determine the channel conveyance, K. The following equation is used:

$$K = \left(\frac{1.486}{n}\right) A\,R^{2/3}$$ (Equation 8-16)

Where:

K = conveyance in cubic feet per second

Equation 8-15 can then be written as:

$$Q = K\,S^{1/2}$$ (Equation 8-17)

The conveyance represents the carrying capacity of a stream cross-section based upon its geometry

and roughness characteristics alone, and it is independent of the streambed slope. The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross-section and the flow distribution through the opening in a proposed stream crossing. It is also used to determine the velocity distribution coefficient, $\alpha$ (see Equation 8-5).

## 8.7 Supercritical, Subcritical, and Critical Flows

Open channel flow can be classed as supercritical, subcritical, or critical. Supercritical flows:

- are influenced mainly by inertial forces, and are often described as torrents or rapids,
- have flow depths less than critical depth,
- occur on slopes steeper than the critical slope,
- have Froude numbers greater than one, and
- small water surface disturbances are always swept downstream in supercritical flow, and the flow characteristics at a cross-section are controlled by flow characteristics at upstream cross-sections.

Subcritical flows:

- are mainly influenced by gravity forces, and are often called tranquil,
- have flow depths greater than critical depth,
- occur on slopes flatter than the critical slope,
- have Froude numbers less than one, and
- small water surface disturbances such as waves can travel both upstream and downstream, and flow characteristics at a cross-section are controlled by flow characteristics at downstream cross-sections.

Critical flows:

- are an unstable condition rarely encountered in nature,
- flow is almost always in either the super or subcritical regime, and it passes through critical flow when it makes a transition from one regime to the other.

Relative to subcritical flows, supercritical flow occurs on steeper slopes, has shallower flow depths, and faster velocities.

### 8.7.1 Froude Number

The Froude number ($F_r$) is an important dimensionless parameter in open channel flow. It represents the ratio of inertial forces to gravitational forces. This expression for Froude number applies to channels of any cross-sectional shape.

$$F_r = \frac{V}{\left(\frac{gd}{\alpha}\right)^{0.5}}$$ (Equation 8-18)

Where:

$\alpha$ = velocity distribution coefficient (see Equation 8-5)
V = mean velocity = Q / A, in feet per second
g = acceleration of gravity (32.2 feet per second squared)
d = hydraulic depth = A / T, in feet
A = cross-sectional area of flow in square feet
T = channel top width at the water surface in feet
Q = total discharge in cubic feet per second

*Note: The hydraulic depth is equal to the flow depth for rectangular channels.*

### 8.7.2 Critical Depth

At a constant discharge, flows at different depths have different specific energy heads. The minimum specific energy head occurs at a depth called critical depth at which the Froude number is one. Critical depth is also the depth of maximum discharge when the specific energy head is held constant. When flow is at critical depth, the velocity head is equal to half the hydraulic depth. The general expression for flow at critical depth is:

$$\frac{\alpha Q^2}{g} = \frac{A^3}{T}$$ (Equation 8-19)

Where:

$\alpha$ = velocity distribution coefficient (see Equation 8-5)
Q = total discharge in cubic feet per second
g = gravitational acceleration, 32.2 feet per second squared
A = cross-sectional area of flow in square feet
T = channel top width at the water surface in feet

Equation 8-19 must be satisfied, no matter what the shape of the channel, when flow is at critical depth. The slope is classified as a mild slope if the normal depth is greater than critical depth, and the slope is called a steep slope if normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

Critical depths for closed conduits and many artificial channel shapes are discussed Subsection 8.14.6. Critical depth charts for common prismatic shapes are in Appendix B.

## 8.8 Transition Sections and Rapidly Varied Flow

Transition sections occur when there are changes in discharge, roughness, cross-section, or slope and the flow regime does not change as water passes through the transition section. In other words, there is subcritical flow upstream, through, and downstream of the transition. Transition sections with supercritical flow are similar. There is supercritical flow downstream, through and upstream of the transition.

Rapidly varied flow occurs when there are changes in discharge, roughness, cross-section, or slope and the flow changes from one regime to another. The most common occurrences are hydraulic drops and hydraulic jumps. Hydraulic drops occur when subcritical flow turns into supercritical flow. Typical locations are the upstream edge of constrictive bridge openings, sections of channel where a flatter slope upstream breaks to a steeper slope downstream, and sections where a hydraulically rough upstream channel changes to a hydraulically smooth downstream channel.

Hydraulic jumps occur when supercritical flow turns into subcritical flow. Locations of hydraulic jumps can be at the downstream edge of constrictive bridge openings, channels where a steeper upstream section changes to a flatter downstream section, and channels where a hydraulically smooth upstream channel changes to a hydraulically rough downstream channel.

Understanding transition sections and rapidly varied flow is important in channel design because drops and jumps often cause energy loss and create turbulence. Special features may be needed in the turbulent areas to protect from scour or erosion.

Transition sections and areas of rapidly varied flow cannot be analyzed by single-section methods. Energy losses other than friction loss also need to be considered. The analysis can be done by hand methods as discussed in Section 8.16. One, two, or three dimensional computer programs can also be used. The step-backwater programs HEC-RAS and WSPRO mentioned in the following subsection have the ability to analyze channel reaches with transition sections and rapidly varied flow. Hydraulic analysis methods are discussed in the following section.

## 8.9 Hydraulic Analysis Methods

The hydraulic analysis typically determines the depth and velocity that given discharge will have in a channel of known geometry, roughness, and slope. The flow depths and velocities are necessary for the design or analysis of channels, channel linings, and highway drainage structures.
Flow in open-channels is complex. The velocity vector of flow at any point in the channel typically has three components, as follows:

- a longitudinal component horizontal and parallel to the channel centerline,
- a lateral component horizontal and transverse to the channel centerline, and

- a vertical component.

Three dimensional models analyze flow velocity in all three of these directions with respect to time. These methods are highly complex and seldom used. Two dimensional models analyze flow velocity and water surface elevation in the longitudinal and lateral directions with respect to time. These models are typically used in special situations such as:

- wide floodplains with multiple openings, particularly where embankments are skewed to the flow direction,
- floodplains with significant variations in roughness or complex geometry such as ineffective flow areas, flow around islands, or multiple channels,
- sites where more accurate flow patterns and velocities are needed to design better and more cost-effective countermeasures such as riprap along embankments and/or abutments,
- tidally affected river crossings and crossings of tidal inlets, bays, and estuaries,
- high risk or sensitive locations where potential losses and liability costs are high, and
- analyzing the effects of sedimentation and scour due to channel changes.

Two-dimensional hydraulic models are complex and directions on their use are beyond the scope of this manual. It is recommended that computer programs such as US Corps of Engineers RMA-2V or the FHWA's Finite Element Surface Water Modeling System: Two Dimensional Flow in a Horizontal Plane - FESWMS-2DH (FESWMS) be used. FESWMS is recommended for crossings of rivers and floodplains it supports both super and subcritical flow analysis and can analyze weirs (roadway overtopping), culverts, and bridges.

The Surface Water Modeling System (SMS) developed by the Engineering Computer Graphics Laboratory at Brigham Young University in cooperation with the U.S. Army Corps of Engineers Waterways Experiment Station and the FHWA can be used to develop the finite element mesh and associated boundary conditions necessary for RMA-2V and FESWMS. The solution files from RMA-2V or FESWMS, which contain water surface elevation, velocity, or other functional data at each node of the mesh, can be read into SMS to generate vector plots, color-shaded contour plots, time variant curve plots, and dynamic animation sequences.

Open channel flow hydraulic analysis is simplified by assuming one-dimensional flow. Lateral and vertical velocity vector components are ignored and flow is assumed to occur only in the longitudinal direction. This assumption is the basis of many of the most frequently used hydraulic modeling methods. The two hydraulic modeling methods presented in this manual, the single-section analysis and the step-backwater method are one-dimensional models.

### 8.9.1 Single-Section Method

The single-section method (slope-area method) is based on the cross-sectional geometry, hydraulic roughness, and energy gradeline slope of a single waterway cross-section. This method is simply a solution of Manning's equation for the normal depth of flow given the discharge. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either

artificial or natural channels. The results obtained by assuming steady uniform flow are understood to be approximate and general, but they provide a satisfactory solution to many practical problems. As a result, this method is often used to determine hydraulic characteristics of waterways with relatively uniform cross-section, roughness, and slope. Examples are canals, ditches, gutters, and closed-conduits such as culverts and storm drains. The single-section method can also be used to estimate highwater elevations for bridges that do not constrict the flow on streams with relatively uniform cross-section. The step-backwater method or other procedures that consider energy losses in addition to friction are preferred in situations where uniform or nearly uniform flow does not occur.

Use of the single-section method to determine a stage-discharge curve is discussed in Section 8.11. Use of the single-section method to analyze flow in artificial channels is discussed in the remainder of this chapter.

### 8.9.2 Step-Backwater Method

The step-backwater method is used to compute water surface profiles. It is often used in bridge hydraulic design and flood studies. The method is particularly useful to determine how far upstream or downstream water surface elevations are affected by a bridge, culvert, or other encroachment into the floodplain.

The step-backwater method is generally more accurate than the single section method. It is seldom done by a pencil-and-paper analysis because it requires extensive iterative calculations. Computer programs such as the U.S. Corps of Engineers "Hydraulic Engineering Center River Analysis System" (HEC-RAS), or the Federal Highway Administration's "WSPRO - Step Backwater and Bridge Hydraulics" program in the HYDRAIN package are often used. This chapter contains a general discussion of step-backwater modeling. Specific discussion of step-backwater models, instructions, and example problems are included in the user's manuals for the various programs.

### 8.9.3 Calibration

Regardless of the method, the model should be calibrated using historical highwater marks and/or gaged streamflow data to ensure that it accurately represents local conditions. The following parameters, in order of preference, may be adjusted to calibrate the hydraulic model: Manning's n, slope, discharge, and cross-section. Proper calibration is essential if accurate results are to be obtained.
It is good practice to find two or more people who have observed the site for at least twice as many years as the calibration flood recurrence interval. Typically two or more people can be found who have seen a site for at least ten years, and they can be asked about the 2 and 5-year events. The two-year event occurs one out of every two years, on the average. The five year event occurs one in five years, on the average.

Witnesses give conflicting accounts in some instances. Additional people need to be interviewed or

other information should be used to verify the hydraulic history of the site. An example of a question to these observers follows.

"My hydraulic model predicts water will just cover the top of the culvert inlet once a year during one of two years, on the average. It also predicts water will overtop the driveway at least once during one of every 5 years, on the average. Does this match the flooding you observed?"

The model would be adjusted depending on the observer statements. The model "n" value, discharge, or other characteristics could be adjusted if the predicted flooding occurs more or less often than the observed flooding.

Sometimes a gage station is on the stream and the peak discharges during notable floods can be estimated fairly accurately. Witnesses can often provide detailed accounts of water surface elevations during these floods. The hydraulic model can be adjusted so it produces a water surface elevation similar to the observed during the discharge estimated for the flood.

## 8.10 Hydraulic Modeling

One of the first steps in hydraulic analysis is to model the waterway. The choice of analysis method depends on the type of information needed, the importance of the hydraulic structure, and the sensitivity of the area to flooding. As examples, a roadside ditch design may only need a single-section analysis if it had a fairly uniform slope, roughness, and cross-section, a large bridge that constricts a floodplain in a rural area may justify a step-backwater flow analysis, and a large bridge that affects flood elevations in a city may justify a two-dimensional hydraulic study. Elements of typical single-section and step-backwater hydraulic models are described in this section.

### 8.10.1 Cross-Sections

Stream cross-sectional geometry is defined by lateral distance and ground elevation coordinates which locate individual ground points. The cross-section is taken normal to the flow direction along a single straight line where possible. It may be necessary to use a cross-section with intersecting straight lines which are perpendicular to the flow path, i.e. a "dog-leg" section, in wide floodplains or bends where flow lines are not parallel. It is especially important to plot the cross-section on paper or electronically to reveal any inconsistencies or errors.

Cross-sections should be located to be representative of the subreaches between them. Major breaks in bed profile, abrupt changes in roughness or shape, control sections such as waterfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require that cross-sections are taken at shorter intervals. Cross-sections representing changes in channel shape and roughness are shown in Figure 8-2. Additional discussion about cross-section location is in **Chapter 6**.

### 8.10.2 Manning's n Value Selection

Manning's n is affected by many factors and its selection in natural channels depends heavily on engineering experience. Pictures of channels and flood plains for which the discharges and channel properties have been measured and Manning's n values have been calculated are in USGS Water-Supply Paper 1849, published in 1967, by Harry H. Barnes, Jr. titled "Roughness Characteristics of Natural Channels." Additional pictures and formulae for calculating roughness values are in FHWA Report No. FHWA-TS-84-204, published in 1984, by G. J. Arcement, Jr. and V. R. Schneider, titled "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains." These publications are available from the FHWA website: http://www.fhwa.dot.gov/bridge/elibrary.htm Manning's n values for artificial channels are more easily defined than n values for natural channels. Appendix A lists typical n values for natural and artificial channels.

### 8.10.3 Cross-Section Subdivision

The cross-section may need to be divided into subsections with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness. The subdivision points should be carefully chosen so that the distribution of flow or conveyance is nearly uniform within each subdivision. Dividing the cross-section will allow the conveyance of each subdivision to be calculated separately. This is necessary to calculate the velocity distribution factor (Equation 8-5) and the weighted reach lengths (Equations 8-22 and 8-23). It is also needed to prevent “switchback”, as discussed in Subsection 8.10.5. The three cross-sections shown in Figure 8-2 are shown in more detail in Figure 8-3 to illustrate waterway subdivision.

Cross-Section 1 is shown in Figure 8-3a. One side of this channel is deep and has no vegetation. The other side is almost as deep and has light vegetation that does not retard flood flows. Flow velocities are expected to be fairly uniform throughout the section, and as a result, the section will not be subdivided and a single n value will be used. A Manning's n value from Appendix A could be used for this natural stream channel.

Cross-Section 2 is shown in Figure 8-3b. There are no significant changes in geometry or roughness which would cause one section of the waterway to have significantly different flow velocities than the other sections. As a result, the section will not be subdivided and a single composite Manning's n value will be calculated by method shown in the next subsection.
Cross-Section 3 is shown in Figure 8-3c. There are changes in waterway geometry and roughness in different sections of the channel that cause differences in flow velocities. Flow in the deep smooth main channel is expected to have higher velocities than flow over the wide, hydraulically rough, and flat floodplains. As a result, the cross-section is subdivided at the channel banks. It should be noted that Cross-Section 3 would also be subdivided at the channel banks due to changes in the waterway geometry even if there was no difference in the n values or velocity between the overbanks and the main channel.

Guidance on the subdivision of cross-sections, calculation of the velocity distribution coefficient, as well as many other aspects of open-channel hydraulics are described in the text by Jacob Davidian titled "Computation of Water-Surface Profiles in Open Channels." The text comprises Chapter A-15 of Book 3 of the 1984 USGS publication titled "Techniques of Water-Resources Investigations of the United States Geological Survey."

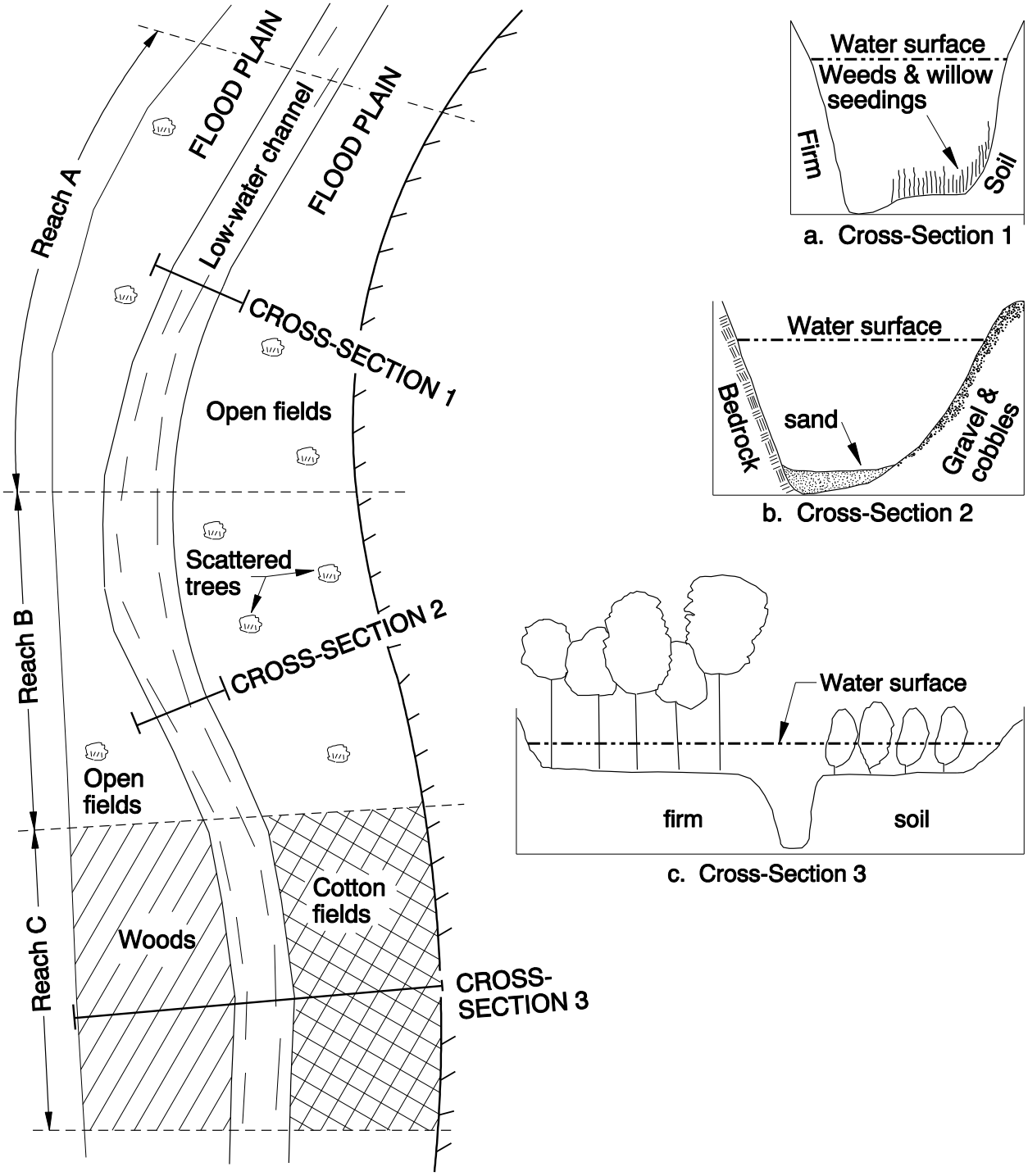



**Figure 8-2 Reach and Cross-Section Location**

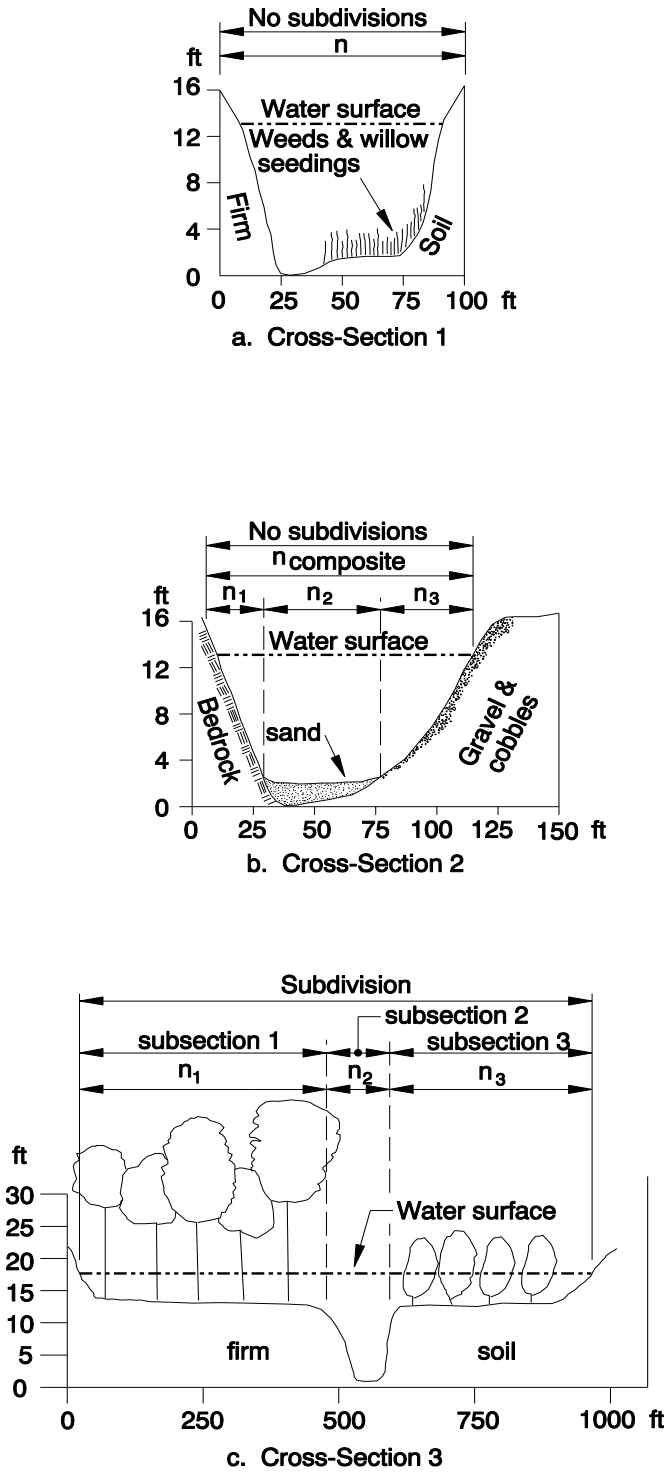



**Figure 8-3 Waterway Subdivision**

### 8.10.4 Composite Roughness Coefficients

Occasionally, portions of the cross-section wetted perimeter will have different roughness coefficients, and the differences in roughness will not cause a significant variation in channel velocities. This often occurs in channels with the bottom and sides of different materials. In these cases, the waterway is not subdivided and a single composite roughness coefficient is calculated. The composite Manning's n value can be calculated by the following equation:

$$n_c = \left( \frac{\sum (n_i^{3/2} P_i)}{P} \right)^{2/3} \qquad \text{(Equation 8-20)}$$

i=1

Where:

$n_c$ = composite roughness coefficient, n
P = wetted perimeter of entire waterway in feet
$n_i$ = roughness coefficient for subsection i
$P_i$ = wetted perimeter of subsection i in feet

Cross-Section 2 shown in Figure 8-3b is an example of this type of waterway. The channel banks are composed of sections with bedrock, gravel, and cobbles, and the channel bottom is sand. These materials have different Manning's n values $n_1$, $n_2$, and $n_3$ which correspond to sections of the wetted perimeter $P_1$, $P_2$, and $P_3$. Based on Equation 8-20, the composite Manning's n value is calculated by the following equation:

$$n_{composite} = \left[ \frac{(n_1^{3/2} P_1) + (n_2^{3/2} P_2) + (n_3^{3/2} P_3)}{(P_1 + P_2 + P_3)} \right]^{2/3} \qquad \text{(Equation 8-21)}$$

### 8.10.5 Switchback Phenomenon

A plot of the stage discharge curve may have the irregular "switchback" shape if a cross-section is improperly subdivided, as shown in Figure 8-4a. A switchback results when the calculated discharge decreases with an associated increase in elevation. This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross-sectional area which causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. A discharge is computed which is lower than the discharge based on the lower water depth due to the combination of the slightly larger cross-sectional area and the significantly lower hydraulic radius. More subdivisions within such cross-sections should be used to avoid the switchback.

This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross-section being used in a step-backwater program, and it is possible an erroneous water surface profile could be calculated. For this reason, the cross-section should always be subdivided with respect to both roughness and geometric changes. Note that the actual n value, itself, may be the same in adjacent subsections, as shown in Figure 8-4b. In this figure, overbank areas are subdivided to prevent switchback.

### 8.10.6 Reach Lengths

Curving stream channels may have different reach lengths between cross-sections in the main channel and overbanks, as shown in Figure 8-5. To simplify the hydraulic analysis, a weighted reach length (L) is used in the hydraulic equations. Two methods of calculating L are as follows:

- Conveyance-weighted reach length, L:

$$L = \frac{\left(L_{lob}K_{lob} + L_{ch}K_{ch} + L_{rob}K_{rob}\right)}{\left(K_{lob} + K_{ch} + K_{rob}\right)} \qquad \text{(Equation 8-22)}$$

Where:

$L_{lob}$, $L_{ch}$, $L_{rob}$ = flow distance between sections in the left overbank, main channel, and right overbank, respectively, in feet

$K_{lob}$, $K_{ch}$, $K_{rob}$ = conveyance in the left overbank, main channel, and right overbank, respectively, of the section with the unknown water surface elevation, in cubic feet per second

- Discharge-weighted reach length, L:

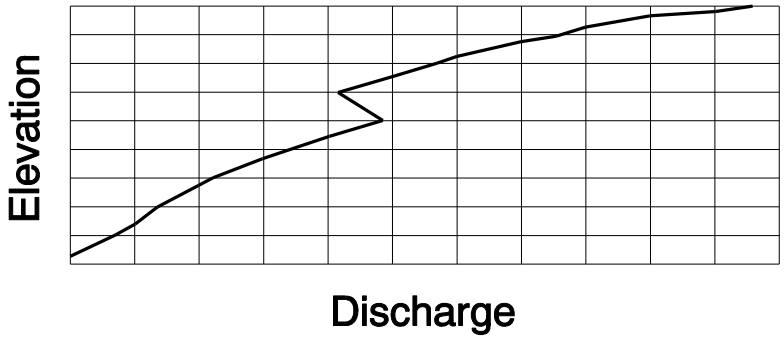
$$L = \frac{\left(L_{lob}Q_{lob} + L_{ch}Q_{ch} + L_{rob}Q_{rob}\right)}{\left(Q_{lob} + Q_{ch} + Q_{rob}\right)} \qquad \text{(Equation 8-23)}$$

Where:

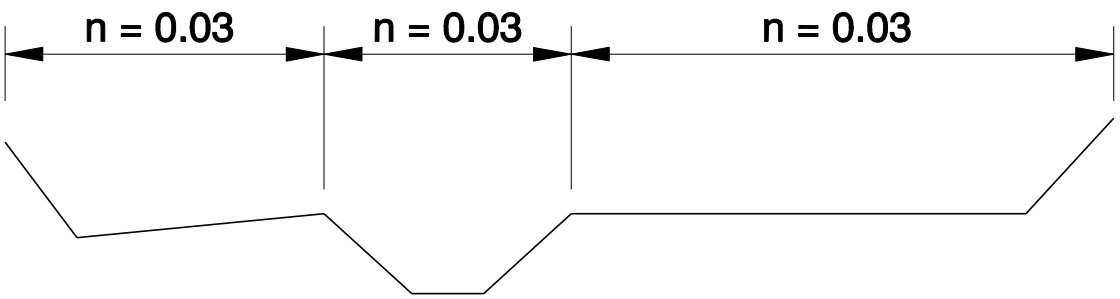
$L_{lob}$, $L_{ch}$, $L_{rob}$ = flow distance between cross-sections in the left overbank, main channel, and right overbank, respectively, in feet

$Q_{lob}$, $Q_{ch}$, $Q_{rob}$ = arithmetic averages of flows in the left overbanks, main channels, and right overbanks, respectively, of the bounding cross-sections in cubic feet per second. For example: $Q_{lob} = (Q_{lob\ (Section\ 1)} + Q_{lob\ (Section\ 2)}) / 2$

*Note: An iterative solution is used. The conveyances or discharges are assumed at the unknown section, the weighted reach length is calculated based on the previous assumption, and the*




a. Switchback




b. Cross-section subdivision to avoid swithcback

**Figure 8-4 Switchback**

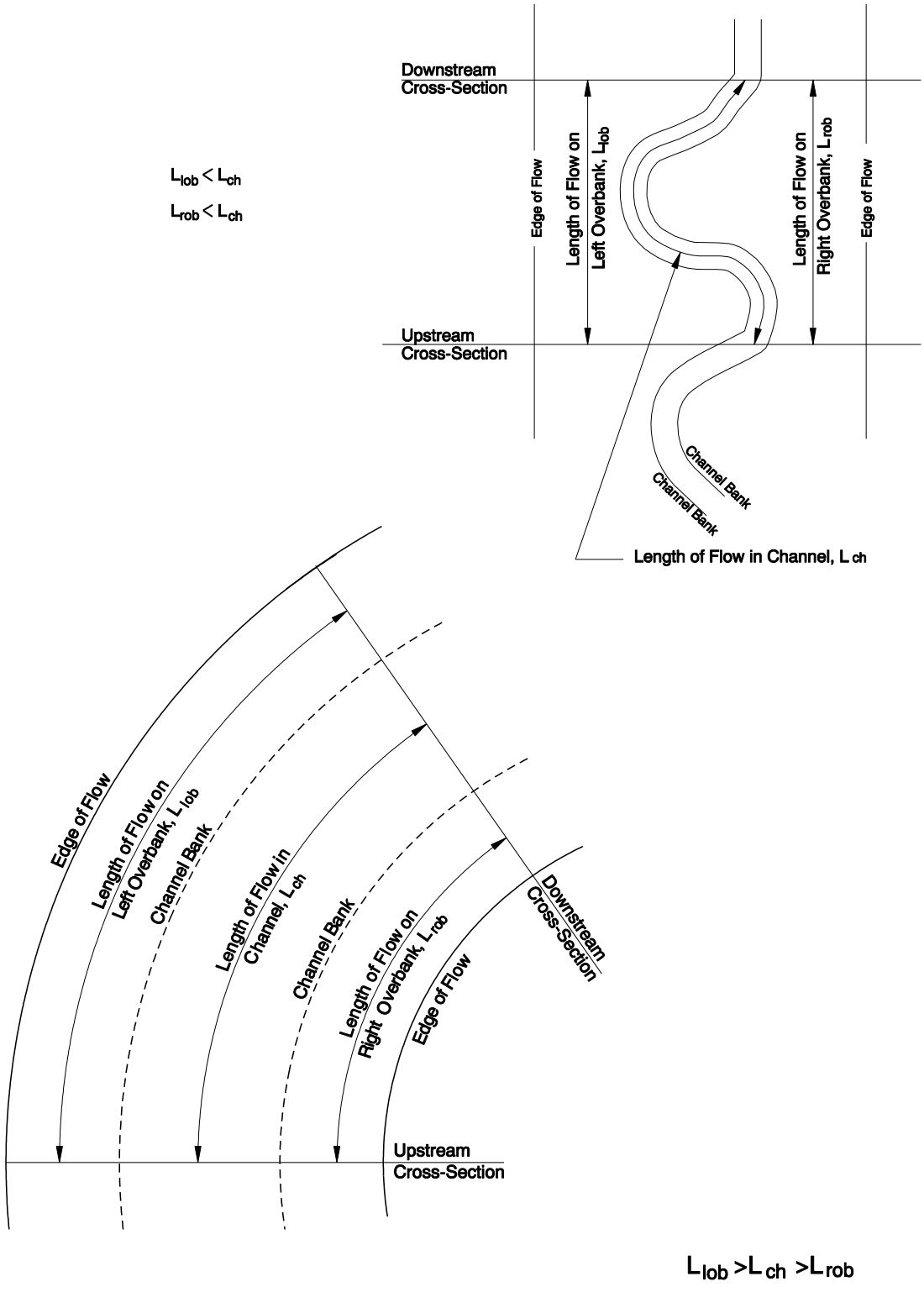



**Figure 8-5 Varying Reach Lengths Between Sections**

*conveyances or discharges are recalculated using the adjusted reach length. These steps are repeated until the assumed and calculated conveyances or discharges match each other. These calculations are cumbersome if done by hand. Use of a step-backwater computer program is recommended.*

## 8.11 Stage-Discharge Curve

A stage-discharge curve is a graphical relationship of streamflow depth or elevation versus discharge at a specific cross-section on a stream. A stage-discharge curve is shown in Figure 8-8 to illustrate the example problem. The stage-discharge curve can be determined by single-section analysis if uniform or nearly uniform flow is present. The step-backwater method should be used if non-uniform or varied flow occurs. The stage-discharge curve can be determined as follows.

**Step 1 -** Select the typical cross-section at or near the location where the stage-discharge curve is needed. This is the only cross-section for a single-section analysis. Additional sections will be needed upstream or downstream from the location where the stage-discharge curve is needed in a step-backwater analysis, as follows:

- at least three sections are needed downstream if subcritical flow occurs, or,
- at least three sections are needed upstream if supercritical flow occurs.

**Step 2 -** Subdivide cross-section(s) and assign n-values to subsections as described in Subsection 8.10.3.

**Step 3 -** Estimate the energy grade line slope at the section for a single-section analysis, or at the starting section for a step-backwater analysis. The slope of the surveyed streambed or water surface can be used since uniform flow is assumed.

**Step 4 -** Select a range of incremental water surface elevations at the cross-section and calculate the discharge at each elevation using Manning's equation for discharge if a single-section analysis is made. Total discharge at each elevation is the sum of the discharges through each subsection if the cross-section is subdivided. The wetted perimeter should be measured along the solid boundary of the cross-section and not along the vertical water interface between subsections when determining hydraulic radius for subdivided cross-sections.

Calculate the water surface at the cross-section in question for several incremental discharges if a step-backwater analysis is used. Start the analysis at the furthest downstream section if there is subcritical flow. Do the opposite if there is supercritical flow.

**Step 5 -** Plot the stage-discharge curve after the discharge has been calculated at several incremental elevations, or the elevations have been calculated for incremental discharges. This plot can be used to determine the water surface elevation corresponding to the discharges of interest.

Single-section stage-discharge curves can be calculated by hand tabulation, as shown in the example. Stage-discharge curves calculated by the step-backwater method, especially in the case of stream channels, typically use output from a computer program such as HEC-RAS or WSPRO.

The transverse velocity variation in any channel cross-section is a function of subsection geometry and roughness, and it may vary considerably from one stage and discharge to another. It is important to know this variation for purposes of designing scour control measures and locating relief openings in highway fills, for example. The best method of establishing transverse velocity variations is by current meter measurements. If this is not possible, the single-section or step-backwater method can be used by dividing the cross-section into subsections of relatively uniform roughness and geometry. It is assumed that the energy grade line slope is the same across the cross-section so that the total conveyance $K_t$ of the cross-section is the sum of the subsection conveyances. The total discharge is then $K_tS^{1/2}$ and the discharge in each subsection is proportional to its conveyance. The velocity in each subsection is obtained from the continuity equation, V = Q / A. HEC-RAS or WSPRO also list velocities for multiple subsections or the total cross-section.

There may be locations where a stage-discharge relationship has already been determined using measured flows and stages, such as at gaging stations. These stage-discharge relationships are commonly called “rating curves.” They are available for most stations from the operators. Measured stage-discharge curves will yield more accurate estimates of water surface elevation and should take precedence over the analytical methods described above.

### 8.11.1 Example Problem - Stage-Discharge Curve

This example illustrates use of a stage-discharge curve to determine the tailwater elevation at a culvert outlet. The single-section method will be used.

Given:

The culvert outlet is at Channel Station 0+98.4, as shown in Figure 8-6a. The invert elevation is 729.00 feet.

A typical channel cross-section was surveyed at Channel Station 4+46 and it is shown in Figure 8-6b. It is Cross-Section A.
The 25-year peak discharge ($Q_{25}$) is 175 cubic feet per second.

The stream profile is surveyed. The streambed slope through the culvert site is 0.0027 foot per foot.

Find:

Tailwater elevation at the culvert outlet during the 25-year event.

**Step 1 -** The downstream channel geometry is fairly uniform and it is at a constant slope. Near uniform flow occurs. The single cross-section at Station 4+46 represents the downstream channel.

**Step 2 -** The section at Station 4+46 is divided into subsections using guidelines in Subsection 8.10.3. The subdivisions are due to both geometry and varying roughness. The subsections are given roughness values based on vegetation and terrain using values in Appendix A. Subsection 1 is an overbank area with light brush and trees: n = 0.06. Subsection 2 is the main channel, and it is clean, straight, and it has a few weeds and rocks: n = 0.035. Subsection 3 is an overbank area with scattered brush and dense weeds: n = 0.05. The cross-section data is tabulated as follows:

**Step 3 -** The energy grade line slope is assumed to be the slope of the stream bottom as represented by a line drawn through the high points on the channel bottom profile. There is a 2.16 foot drop in this 800 foot long stream reach, and the slope (S) is calculated to be 0.0027 feet per foot.

**Step 4 -** A range of flow elevations are arbitrarily chosen. In this example, the selected elevations are at increments of approximately 1 foot. The area (A) and wetted perimeter (P) are calculated from geometric formulae or measured from a cross-section plot drawn to scale for each subsection at each elevation increment. The wetted perimeter is considered to be the perimeter of the subsection where water contacts a solid surface. The hydraulic radius (R) is calculated for each subsection at each elevation increment as follows: $R = A / P$.

The flow (Q) and velocity (V) in each subsection is calculated by Manning's Equations 8-15 and 8-12, respectively. The flows in each of the subsections are added together to get the total flow in the waterway. The areas of each subsection are added together to get the total waterway area. The total discharge is divided by the total waterway area to calculate the average flow velocity in the waterway. Typically, the calculations are tabulated. Tabular calculations for three of the five elevation increments are shown in Figure 8-7.

**Step 5 -** The tabulated discharge and elevation values are plotted to determine the stage-discharge curve. The stage discharge curve at Cross-Section A is shown in Figure 8-8. The water surface elevation at the section is 730.81 feet during the 50-year event.

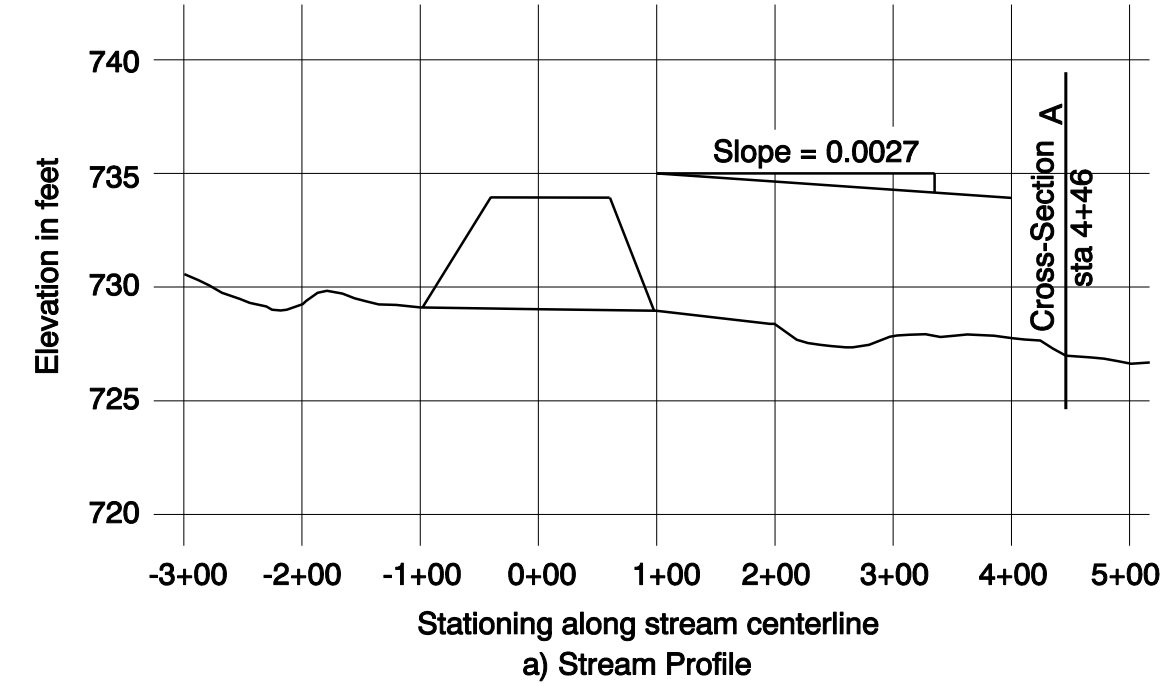
Cross-Section A is 348 feet downstream from the culvert outlet, and the estimated water surface slope is 0.0027 feet per foot. The water surface elevations at Cross-Section A are adjusted to represent the tailwater elevations at the outlet as follows:

$TW_{Outlet\ 50\text{-}yr}$ = (0.0027 x 348 feet) + 730.81 = 731.75 feet ← Answer

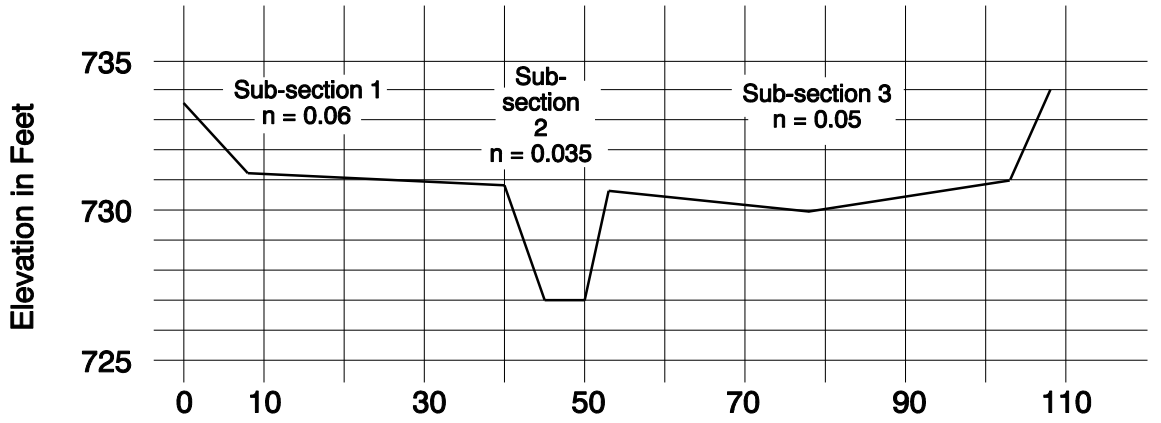
---

Table of Cross-Section Data (Section "A" at Sta. 4+46)

| Distance (feet) | Elevation (feet) | n-value |
|---|---|---|
| 0.00 | 733.50 | 0.06 |
| 8.0 | 731.20 | 0.06 |
| 40.0 | 730.81 | 0.035 |
| 45.0 | 727.00 | 0.035 |
| 50.0 | 727.00 | 0.035 |
| 53.0 | 730.61 | 0.05 |
| 78.0 | 729.40 | 0.05 |
| 103.0 | 731.00 | 0.05 |
| 108.0 | 734.00 | 0.05 |




a) Stream Profile




b) Cross-Section A

**Figure 8-6 Site Data**

| Elevation = 728.0 | | | Slope = 0.0027 | | | | |
|---|---|---|---|---|---|---|---|
| Sub-section ID | I | II | III | IV | V | VI | Totals/Average |
| Area (square feet) | | 6.0 | | | | | 6.0 |
| Wetted Perimeter (feet) | | 7.9 | | | | | |
| Hydraulic Radius (feet) | | 0.76 | | | | | |
| $R^{2/3}$ | | 0.83 | | | | | |
| n | 0.060 | 0.035 | 0.050 | | | | |
| Q = (cubic feet a second) | | 11 | | | | | 11 |
| Sub-section Velocity (feet per second) | | 1.8 | | | | | 1.8 |

| Elevation = 729.0 | | | Slope = 0.0027 | | | | |
|---|---|---|---|---|---|---|---|
| Sub-section ID | I | II | III | IV | V | VI | Totals/Average |
| Area (square feet) | | 14 | | | | | |
| Wetted Perimeter (feet) | | 11 | | | | | |
| Hydraulic Radius (feet) | | 1.3 | | | | | |
| $R^{2/3}$ | | 1.2 | | | | | |
| n | 0.060 | 0.035 | 0.050 | | | | |
| Q = (cubic feet a second) | | 37 | | | | | 37 |
| Sub-section Velocity (feet per second) | | 2.6 | | | | | 2.6 |

| Elevation = 730.0 | | | Slope = 0.0027 | | | | |
|---|---|---|---|---|---|---|---|
| Sub-section ID | I | II | III | IV | V | VI | Totals/Average |
| Area (square feet) | | 25 | 6.6 | | | | 32 |
| Wetted Perimeter (feet) | | 15 | 22 | | | | |
| Hydraulic Radius (feet) | | 1.7 | 0.30 | | | | |
| $R^{2/3}$ | | 1.4 | 0.45 | | | | |
| n | 0.060 | 0.035 | 0.050 | | | | |
| Q = (cubic feet a second) | | 77 | 4.6 | | | | 82 |
| Sub-section Velocity (feet per second) | | 3.1 | 0.70 | | | | 2.6 |

**Figure 8-7 Tabular Stage-Discharge Calculations**

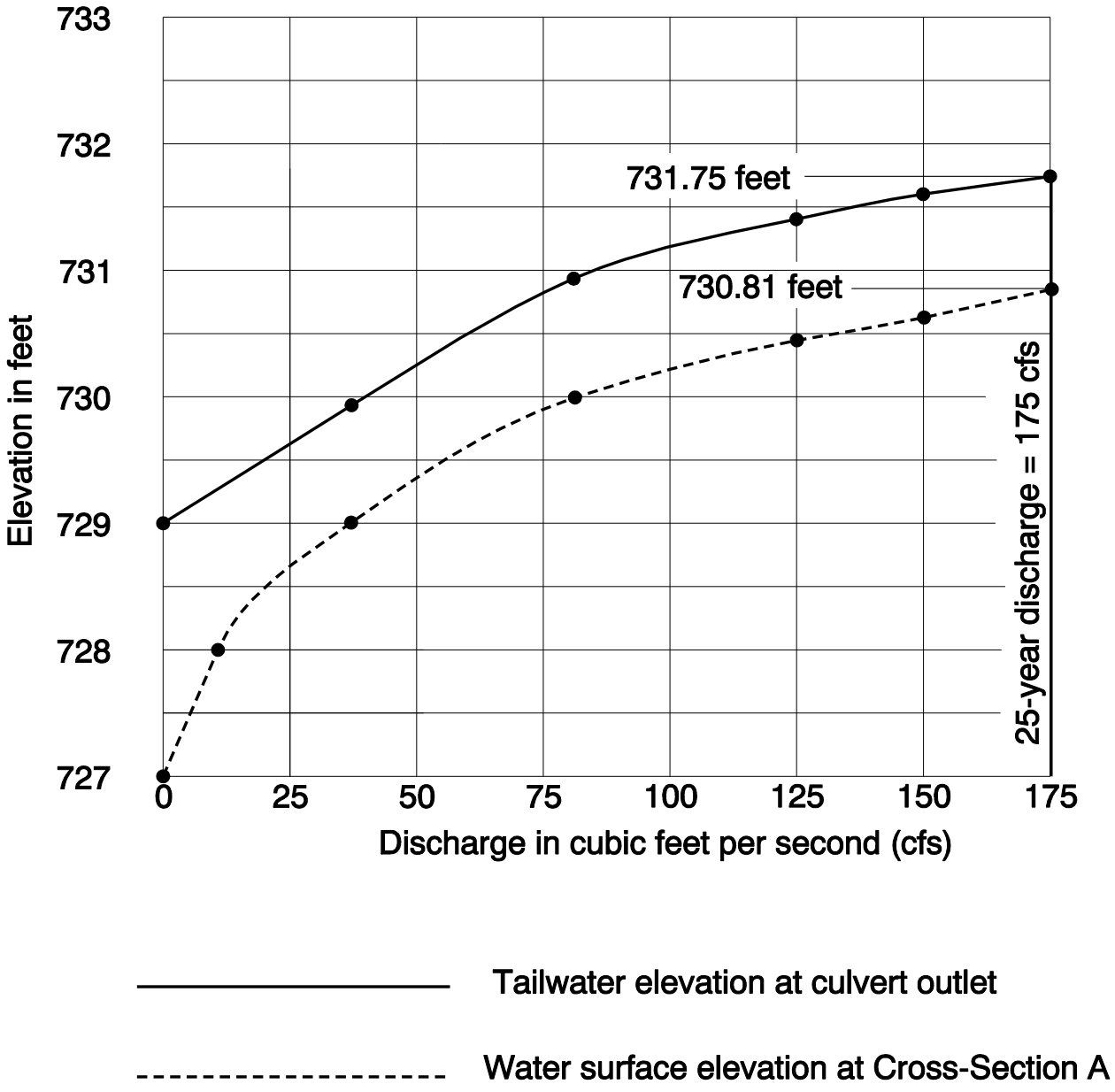



**Figure 8-8 Stage-Discharge Curves**

## 8.12 Profile Computation

Water surface profile computation requires a beginning value of water surface elevation, depth, or energy grade line slope at the initial or starting section. These beginning values are often called boundary conditions. The analysis proceeds section-by-section upstream in the case of subcritical flow, or downstream in supercritical flow. Boundary conditions are an approximation of the water surface elevation or slope at the initial section, and they are usually determined by the following methods.

- The water surface or channel bottom slope at the initial section is assumed to be the energy grade line slope at that location. This assumption can be used if there is uniform or nearly uniform flow at the initial section. Often the initial section is deliberately located in an area of nearly uniform flow so this assumption can be used.

- Critical depth is assumed to occur at the initial section. This assumption is usually made if there is a rapidly varied flow such as a hydraulic drop or jump and flow passes through critical depth at the initial section. Sometimes the initial section is located just upstream or downstream from a rapids, cataract, or waterfall so this assumption can be used.

- A range of reasonable water surface elevations or depths are assumed at the initial section and water surface profiles are computed upstream or downstream to the desired location using the same discharge. These profiles should converge toward the same elevation or depth at the cross-section where the stage-discharge relationship is desired, as shown in Figure 8-9a.

  *Note: Estimating the boundary conditions introduces unavoidable error into the hydraulic model, and this convergence indicates that this error does not affect the water surface elevations at the study site. If the profiles do not converge, as shown in Figure 8-9b, the error in estimating the boundary conditions does affect the results, and more cross-sections are needed in the model. The model can be extended downstream or upstream as shown in Figure 8-9c. This is the method of adding sections that most often produces the desired results. Satisfactory results can also be obtained in some instances by adding one or more intermediate cross-sections between the sections in the original model, as shown in Figure 8-9d.*

- Water surface elevations of a river, lake, ocean, or other body of water can be used as boundary conditions if they are located upstream or downstream. Elevation data are often available from published sources such as Flood Insurance Studies.

## 8.13 Curbs and Gutters

Gutter flow hydraulics are an essential part of storm drain system design. Gutter hydraulics can be determined by several methods, such as equations, nomographs, and computer programs. Equations and nomographs are discussed in this section.

Computer programs to calculate gutter flow are available from public and proprietary sources. The most common gutter shapes can be analyzed using the FHWA VISUAL URBAN program. This software is based on the FHWA Hydraulic Engineering Circular No. 22 "Urban Drainage Design Manual" (HEC-22). Irregular shaped gutters and the Valley Gutter can be analyzed using the U.S. Corps of Engineers HEC-RAS step backwater program.

Gutters should be designed to convey the 10-year event and limit the maximum flow widths to the values listed in **Chapter 13**, Storm Drainage. **The rational or unit hydrograph methods discussed in Chapter 7 are used to estimate runoff from the pavement and side drainage areas adjacent to the gutters.**

### 8.13.1 Gutter Types and Dimensions

A pavement gutter is defined as the section of pavement next to the curb which conveys water during a storm runoff event. Gutter cross-sections usually have a triangular shape with the curb forming the near-vertical leg of the triangle. The gutter may have a straight cross slope or a cross slope composed of two straight lines. Shallow swale gutters typically have V-shaped or circular sections and are often used in paved median areas on roadways with inverted crowns. Curbs and gutters are shown on Figure 8-10 and ODOT Standard Drawings RD700, RD701, and RD720. Dimensions for ODOT gutters are listed in Table 8-1.

The ODOT Curb and Gutter and Mountable Curb and Gutter provide a composite gutter cross-section. The composite section has a smooth concrete apron adjacent to the curb called a depressed gutter section. The apron edge near the curb is slightly depressed because the apron has a steeper cross slope than the adjacent pavement. These are the most hydraulically efficient curb and gutter sections. Using these curbs may result in the most economical design since fewer inlets are usually required to drain the pavement.

Another gutter type is the extruded curb. It has an asphalt concrete or Portland cement concrete curb and the pavement forms the gutter. The cross slope of the gutter is the cross-slope of the pavement. These gutters are not as hydraulically efficient as those with depressed sections. Extruded curbs are often used to keep highway drainage on the pavement and prevent it from eroding fill slopes.

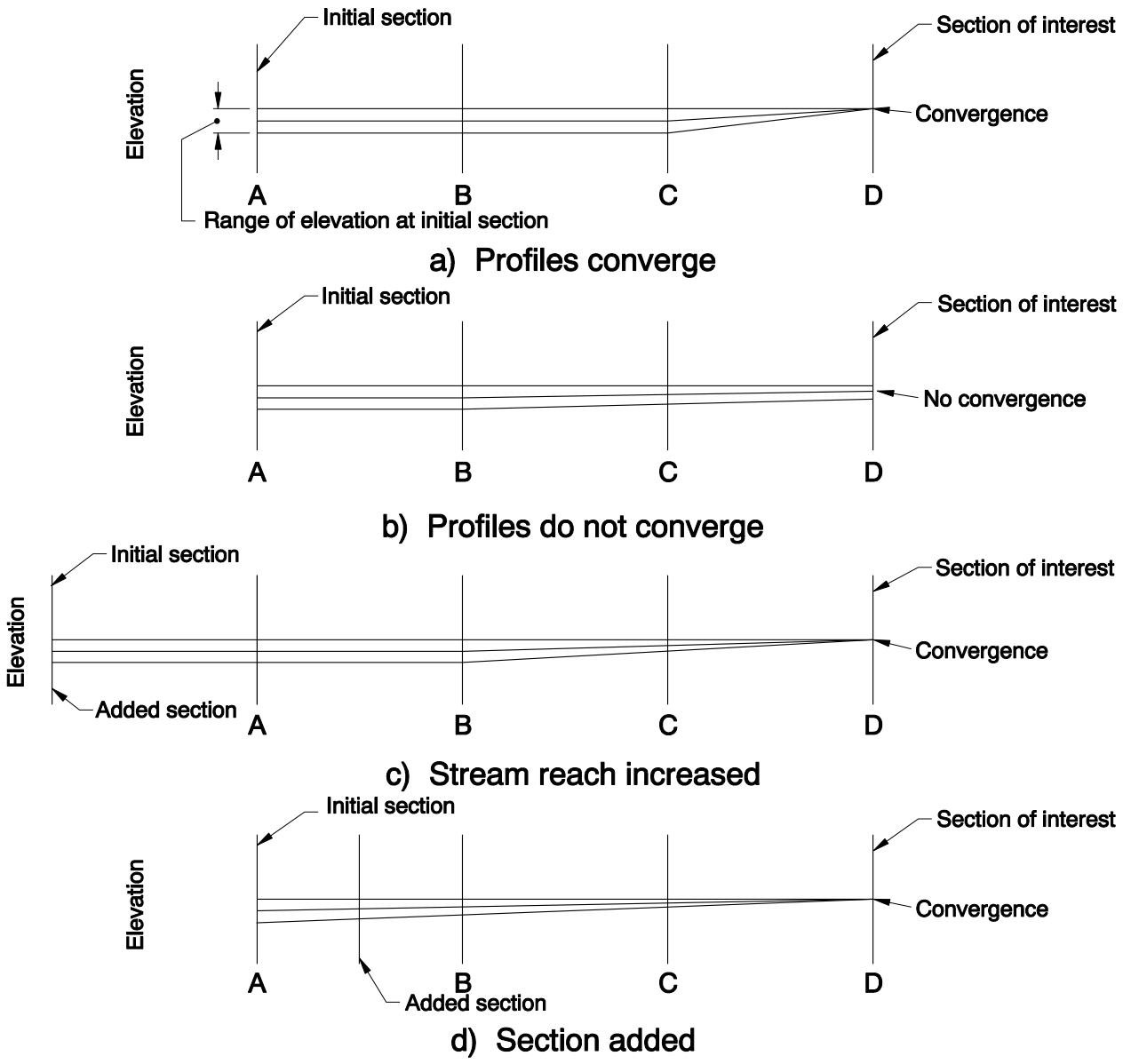



**Figure 8-9 Convergence**

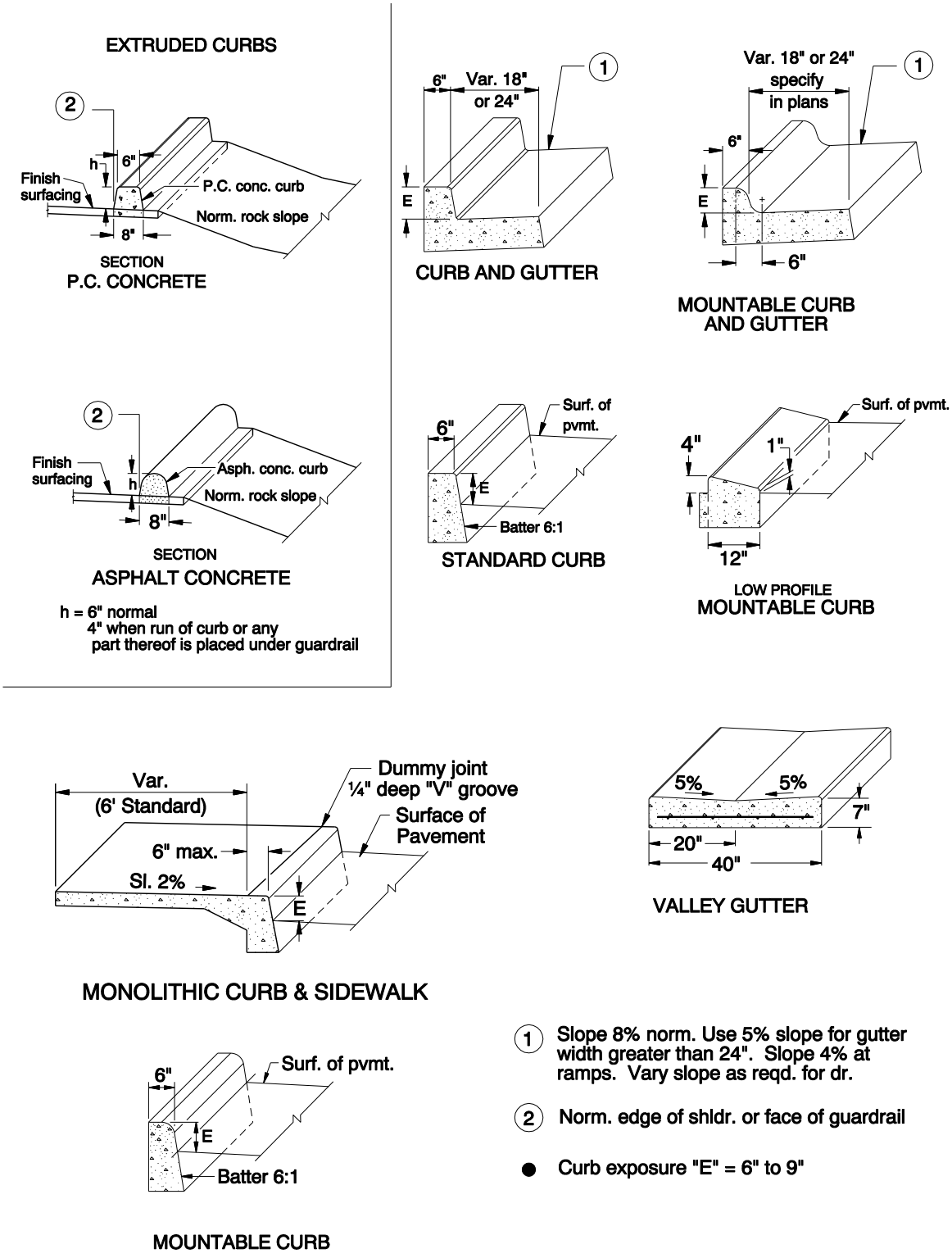



**Figure 8-10 Curb and Gutter Types and Dimensions**

**Table 8-1 ODOT Curbs and Gutters**

| Type | Configuration | | | Notes |
|---|---|---|---|---|
| | Curb Height (inches) | Depressed Gutter Width (inches) | Gutter Cross Slope (percent) | |
| Standard Curb | 6-9 | N/A | Same as Pavement | See Note 1. |
| Curb and Gutter | 6-9 | 18-24 | 8 | |
| | | more than 24 | 5 | |
| Mountable Curb | 6-9 | N/A | Same as Pavement | See Note 1. |
| Low Profile Mountable Curb | 3 | N/A | Same as Pavement | Recommended for rural highways or freeways. See Note 1. |
| Mountable Curb and Gutter | 6-9 | 18-24 | 8 | |
| | | more than 24 | 5 | |
| Extruded Curb | 4-6 | N/A | Same as Pavement | See Note 1. |
| Monolithic Curb and Sidewalk | 6-9 | N/A | Same as Pavement | See Note 1. |

*Note 1: Gutter grades more than or equal to 0.3 percent recommended. Do not use where gutter grades are less than 0.2 percent.*

### 8.13.2 Curb and Gutter Flow Equations

Channel capacities, average velocities, and flow widths for gutter types can be calculated using a modified form of the Manning equation. This modified equation is necessary to compute flow in a triangular channel because the hydraulic radius in the conventional equation does not adequately describe a gutter cross-section. Cross-sections of typical gutters are shown in Figure 8-11.

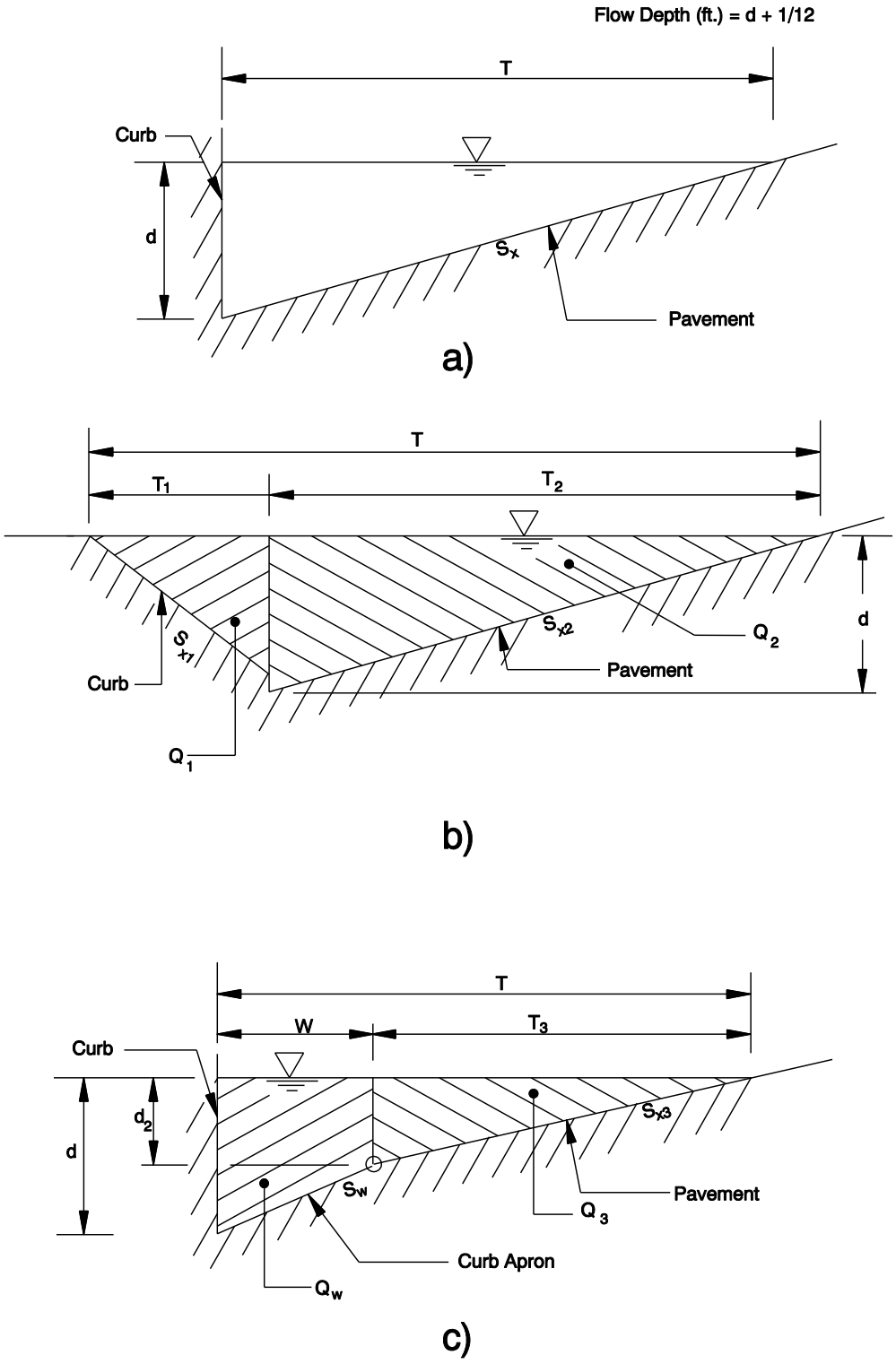



**Figure 8-11 Curb and Gutter Cross-Sections**

The terms in the gutter flow equations follow:

| | |
|---|---|
| $Q$ | Total gutter flow (cubic feet per second) |
| $Q_1$, $Q_2$, $Q_3$ | Partial gutter flow (cubic feet per second) |
| $Q_W$ | Depressed gutter flow (cubic feet per second) |
| n | Manning's "n" value (see Appendix A) |
| $S_X$, $S_{X1}$, $S_{X2}$, $S_{X3}$ | Gutter cross slope (foot per foot) |
| S | Longitudinal gutter slope (foot per foot) |
| T, $T_1$, $T_2$, $T_3$ | Flow width in gutter (feet) |
| V | Average gutter flow velocity (feet per second) |
| d | Flow depth at curb (feet) |
| $d_2$ | Flow depth at break in cross slope (feet) |
| $S_W$ | Depressed gutter cross slope (foot per foot) |
| W | Depressed gutter width (feet) |

These curbs and gutters have a straight cross slope ($S_X$) and a curb face that is near vertical, as shown in Figure 8-11a:

- Portland Cement Concrete Extruded Drainage Curb,
- Asphalt Concrete Extruded Drainage Curb,
- Monolithic Curb and Sidewalk,
- Standard Curb, and
- Mountable Curb (with 6:1 batter).

These formulae can be used:

$$Q = \left(\frac{0.56}{n}\right) S_X^{1.67}\, S^{0.5}\, T^{2.67} \qquad \text{(Equation 8-24)}$$

$$T = \left(\frac{nQ}{0.56\, S_X^{1.67}\, S^{0.5}}\right)^{0.375} \qquad \text{(Equation 8-25)}$$

$$V = \frac{Q}{A} = \frac{1.12}{n} S^{0.5}\, S_X^{0.67}\, T^{0.67} = \frac{2Q}{T^2 S_X} \qquad \text{(Equation 8-26)}$$

The Low Profile Mountable Curb has two gutter cross slopes ($S_{X1}$ and $S_{X2}$), as shown in Figure 8-11b. These formulae can be used:

$$Q = Q_1 + Q_2 = \left(\frac{0.56}{n}\right)\left(S^{0.5}\right)\left(S_{X1}^{1.67}\, T_1^{2.67} + S_{X2}^{1.67}\, T_2^{2.67}\right) \qquad \text{(Equation 8-27)}$$

$$T = T_1 + T_2 = \left[\frac{Q_1 n}{0.56\, S^{0.5}\, S_{X1}^{1.67}}\right]^{0.375} + \left[\frac{Q_2 n}{\left(0.56\, S^{0.5}\, S_{X2}\right)^{1.67}}\right]^{0.375} \quad \text{(Equation 8-28)}$$

$$V = \frac{Q}{A} = \frac{2Q}{\left(T_1^2\, S_{X1} + T_2^2\, S_{X2}\right)} \quad \text{(Equation 8-29)}$$

$$d = T_2 S_{X2} \quad \text{(Equation 8-30)}$$

The Curb and Gutter has a near vertical curb and two gutter cross slopes ($S_{X3}$ and $S_W$), as shown in Figures 8-11c. These formulae are derived to calculate discharges for the Curb and Gutter. They can be used to approximate discharges in the Mountable Curb and Gutter.

$$Q_3 = \left(\frac{0.56}{n}\right) S_{X3}^{1.67}\, S^{0.3}\, T_3^{2.67} = \left(\frac{0.56}{n\, S_{X3}}\right) S^{0.5}\, d_2^{2.67} \quad \text{(Equation 8-31)}$$

$$Q_w = \left(\frac{0.56}{n S_W}\right) S^{0.5} \left(d^{2.67} - d_2^{2.67}\right) \quad \text{(Equation 8-32)}$$

Where:

$$d = T_3 S_{X3} + W S_W \quad \text{(Equation 8-33)}$$

$$d_2 = T_3 S_{X3} \quad \text{(Equation 8-34)}$$

$$Q = Q_3 + Q_W = \left(\frac{0.56}{n}\right)\left(S^{0.5}\right)\left[\left(\frac{d_2^{2.67}}{S_{X3}}\right) + \frac{\left(d^{2.67} - d_2^{2.67}\right)}{S_W}\right] \quad \text{(Equation 8-35)}$$

$$V = \frac{Q}{A} = \frac{2Q}{\left(T_3^2\, S_{X3} + W^2\, S_W + 2W T_3 S_{X3}\right)} = \frac{2Q}{\frac{w\,(d + d_2) + d_2^2}{S_{X3}}} \quad \text{(Equation 8-36)}$$

### 8.13.3 Gutter Flow Examples Using Equations

The following examples illustrate the use of the gutter flow equations.

#### 8.13.3.1 Example 1: Determine Discharge and Velocity for Low Profile Mountable Curb

Calculate the discharge, average velocity, and flow depth in a gutter alongside an ODOT Low Profile Mountable Curb with a flow width of 6 feet, as shown in Figure 8-12.

Given:

S = longitudinal gutter slope = 0.01 foot per foot,
$S_{X1}$ = 3 inches / 12 inches = 0.25 inch per inch = 0.25 foot / foot (Std. Dwg. RD 700)
$S_{X2}$ = pavement cross slope 0.025 foot per foot,
T = $T_1 + T_2$ = 6 feet, and
n = 0.016 (Appendix A).

Need: Q = discharge in cubic feet per second,
V = average velocity in feet per second, and
d = maximum flow depth in feet.

**Step 1 -** Determine $T_1$ and $T_2$

$$S_{X1} T_1 = d - \frac{1}{12}$$

$$d = S_{X2} T_2$$

$$\therefore S_{X1} T_1 = S_{X2} T_2 - \frac{1}{12}$$

(1) $0.25T_1 = 0.025T_2 - 0.083$
(2) $0.25T_1 - 0.025T_2 = -0.083$

Divide equation (2) above by 0.025

(1) $10T_1 - T_2 = -3.3$
(2) $T_1 + T_2 = 6$ see "Given"

Add equation (1) + (2) as shown below

(1) $10T_1 - T_2 = -3.3$

(2) $T_1 + T_2 = 6$

$11T_1 = 2.7$

$$T_1 = \frac{2.7}{11} = 0.25 \text{ feet} \quad \leftarrow$$

Solve for $T_2$ as follows:

$T_2 = 6 - T_1$
$T_2 = 6 - 0.25 = 5.75$ feet ←

**Step 2 -** Determine d using Equation 8-30

$d = T_2 S_{X2} = (5.75)(0.025) = 0.14$ feet = 1.7 inches ← Answer

*Note: Flow is contained within 4-inch high curb and assumptions in Step 1 are valid. The Step 1 assumptions would not be valid if the flow is over the curb and onto the sidewalk. This should always be checked before proceeding to the next step.*

**Step 3 -** Determine Q using Equation 8-27 and V using Equation 8-29

$$Q = \left(\frac{0.56}{n}\right)\left(S^{0.5}\right)\left(S_{X1}^{1.67}\, T_1^{2.67} + S_{X2}^{1.67}\, T_2^{2.67}\right)$$

$$Q = \left(\frac{0.56}{0.016}\right)(0.01^{0.5})\,(0.25^{1.67}\, 0.25^{2.67} + 0.025^{1.67}\, 5.75^{2.67})$$

Q = 0.80 cubic feet per second ← Answer

$$V = \frac{2Q}{T_1^2\, S_{X1} + T_2^2\, S_{X2}}$$

$$V = \frac{2\,(0.80)}{\left[(0.25^2)\,(0.25) + (5.75^2)\,(0.025)\right]} = 1.9 \text{ feet per second} \leftarrow \text{Answer}$$

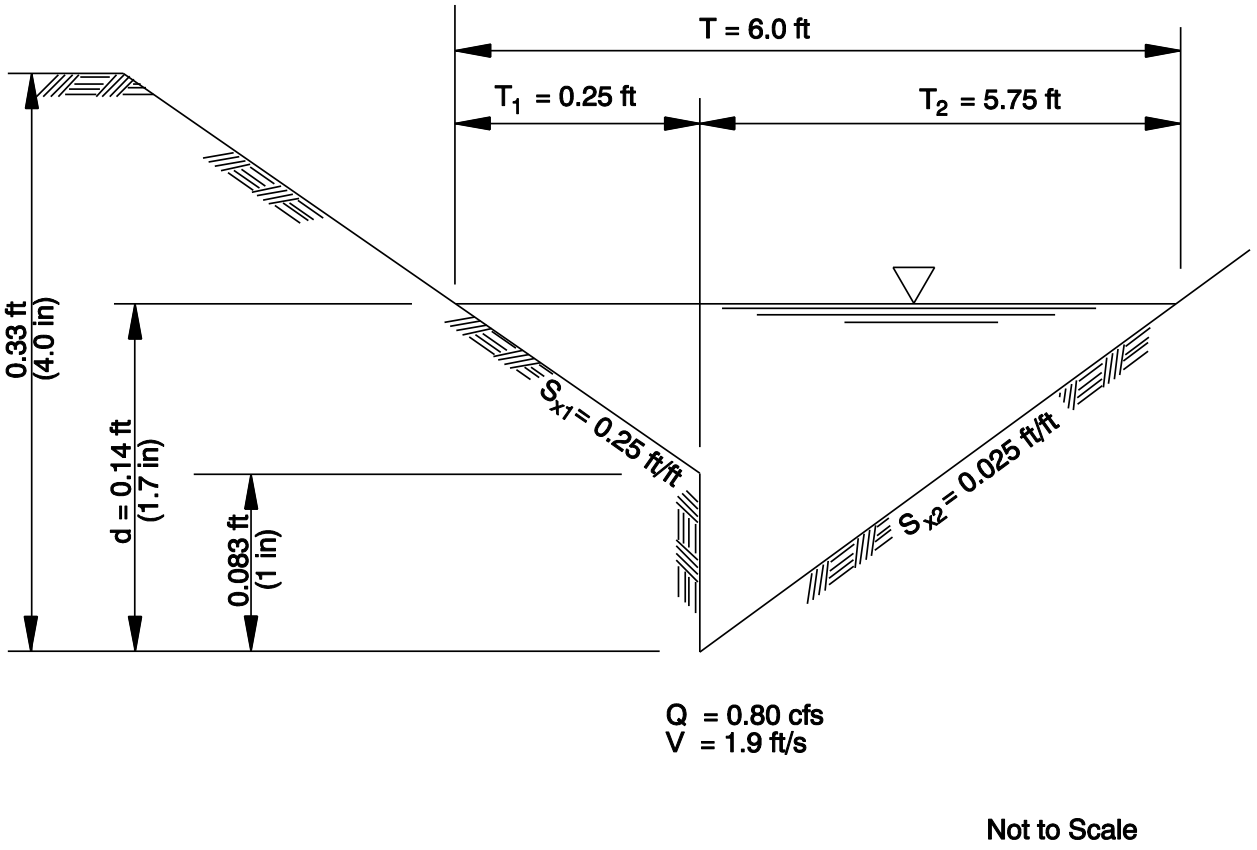



**Figure 8-12 Gutter Cross-Section for Example 1**

### 8.13.3.2 Example 2: Determine Flow Width and Depth for Curb and Gutter

Calculate the flow width and depth adjacent to an ODOT Curb and Gutter during a 1.0 cubic foot per second discharge, as shown in Figure 8-13. A 24-inch wide apron will be used. Verify the flow depth will not be higher than the 7-inch tall curb.

This is a trial and error solution. Steps 1 and 2 are repeated as necessary to calculate an answer with the desired accuracy.

Given:

| | |
|---|---|
| S | = longitudinal gutter slope = 0.001 foot per foot, |
| $S_W$ | = 0.080 foot per foot for the 24-inch apron (See Std. Dwg. RD 700), |
| $S_{X3}$ | = 0.02 foot per foot, |
| W | = 24 inches = 2 feet |
| Q | = 1.0 cubic foot per second, and |
| n | = 0.014 (Appendix A). |

curb height = 7 inches

Need:

T = flow width in gutter in feet, and
d = maximum flow depth in feet.

**Step 1 -** Determine flow depths, $d_2$ and d, using Equations 8-33 and 8-34

Assume $T_3$ = 6 feet

then

$d_2 = T_3 S_{X3} = (6)(0.02) = 0.12$ foot

$d = T_3 S_{X3} + W S_W = (6)(0.02) + (2.0)(0.080) = 0.28$ foot

**Step 2 -** Calculate flow (Q) for flow depths determined in Step 1 using Equation 8-35

$$Q = \left(\frac{0.56}{0.014}\right)\left(0.001^{0.5}\right)\left(\frac{0.12^{2.67}}{0.02}\right) + \left(\frac{0.28^{2.67} - 0.12^{2.67}}{0.080}\right)$$

$$Q = \left[(40)\,(0.032)\right](0.17) + \left(\frac{0.030}{0.080}\right)$$

Q = 0.70 cubic feet per second …. too low

assume $T_3$ = 7 feet

then $d_2 = (7)(0.02) = 0.14$ foot

d = (7)(0.02) + (2.0)(0.080) = 0.30 foot

Q = 0.90 cubic feet per second …..too low

assume $T_3$ = 7.5 feet

then $d_2 = (7.5)(0.02) = 0.15$ ft.

d = (7.5)(0.02) + (2.0)(0.080) = 0.31 foot

Q = 1.0 cubic foot per second ←

$T = T_3 + W = 7.5 + 2.0 = 9.5$ feet

d = 0.31 foot = 3.7 inches ← Answer

Since the curb exposure (7 inches) is greater than the flow depth at curb (3.7 inches), the gutter flow will not overtop the curb.

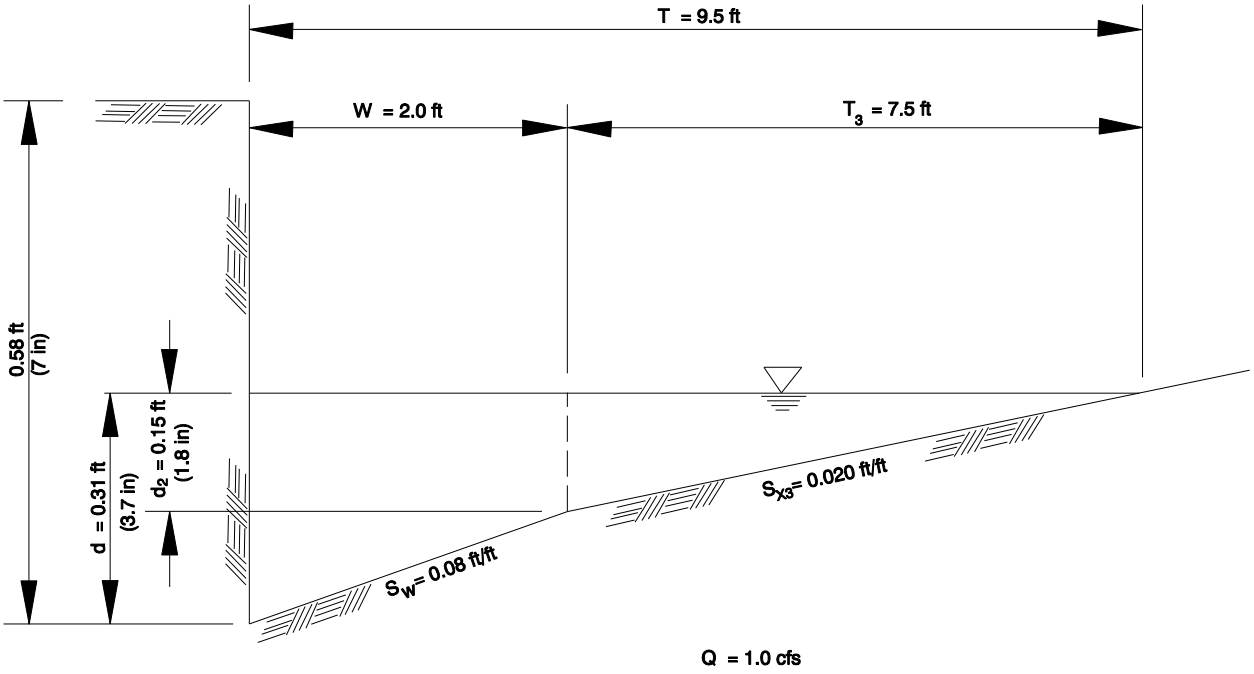



**Figure 8-13 Gutter Cross-Section for Example 2**

### 8.13.4 Curb and Gutter Flow Nomographs

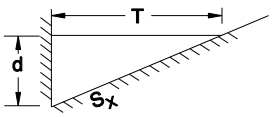
Curb and gutter flow properties can be estimated using nomographs. Flow properties in a gutter with a near vertical curb face and a single gutter cross slope can be analyzed using Figure 8-14 for discharge and depth, and Figure 8-15 for average velocity. These gutters are illustrated in Figure 8-11a and they are:

- Portland Cement Concrete Extruded Drainage Curb,
- Asphalt Concrete Extruded Drainage Curb,
- Monolithic Curb and Sidewalk,
- Standard Curb, and
- Mountable Curb (with 6:1 batter).

Discharge and flow depth can be estimated in a gutter with a two gutter cross slopes, such as the Low Profile Mountable Curb, using Figure 8-16. This gutter is shown in Figure 8-10b.

Discharges and flow depths in a gutter with a near vertical curb and two gutter cross slopes can be estimated using Figure 8-17 and 8-18. This gutter is shown in Figure 8-10c. The nomograph provides an accurate estimate of flow properties in the Curb and Gutter. It provides an approximation for the Mountable Curb and Gutter.

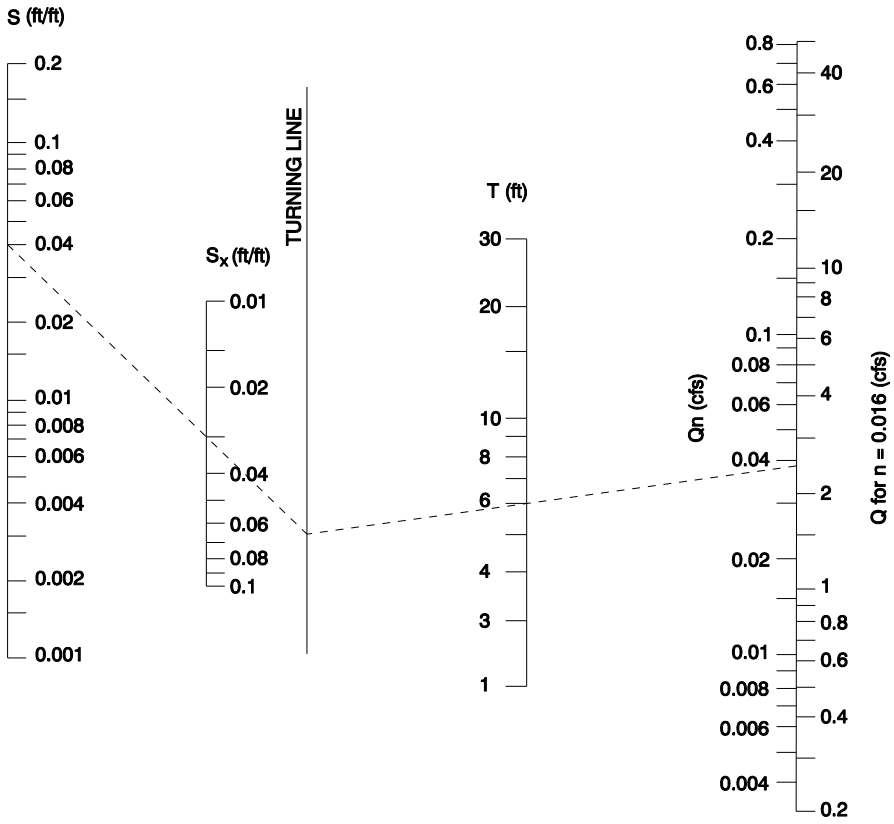
The procedures and example calculations are illustrated on the nomographs.




$$Q = \frac{0.56}{n} S_X^{1.67} S^{0.5} T^{2.67}$$

$$d = TS_X$$

Example: Given:
$n = 0.016$, $S_X = 0.03$ ft/ft
$S = 0.04$ ft/ft, $T = 6$ ft
Find:
$Q = 2.4$ cfs
$d = (6)(0.03) = 0.18$ ft




S = longitudinal gutter slope (ft/ft)

$S_X$ = gutter cross-slope (ft/ft)

T = flow width in gutter (ft)

Q = total gutter flow (cfs)

n = Manning's "n"

d = flow depth at curb (ft)

Example: Given:
$n = 0.012$, $S_X = 0.03$ ft/ft
$S = 0.04$ ft/ft, $T = 6$ ft
Find:
$Q_n = 0.038$ cfs
$Q = \frac{Q_n}{n} = \frac{0.038}{0.012} = 3.2$ cfs

**Figure 8-14 Discharge Nomograph for Gutter with Near Vertical Curb and Single Gutter Cross Slope**

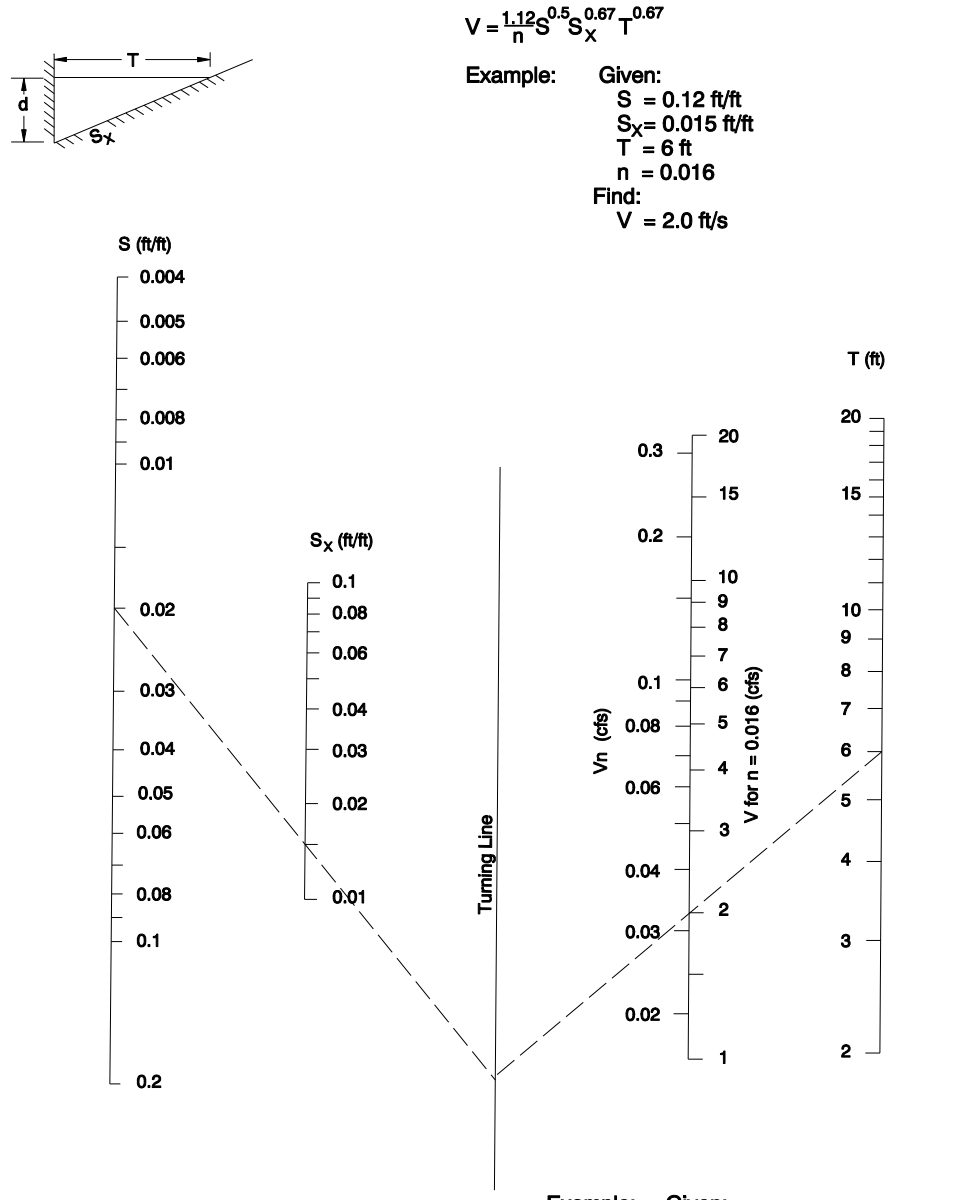
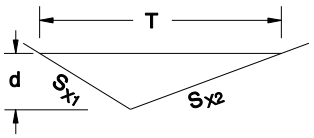



**Figure 8-15 Velocity Nomograph for Gutter with Near Vertical Curb and Single Gutter Cross Slope**

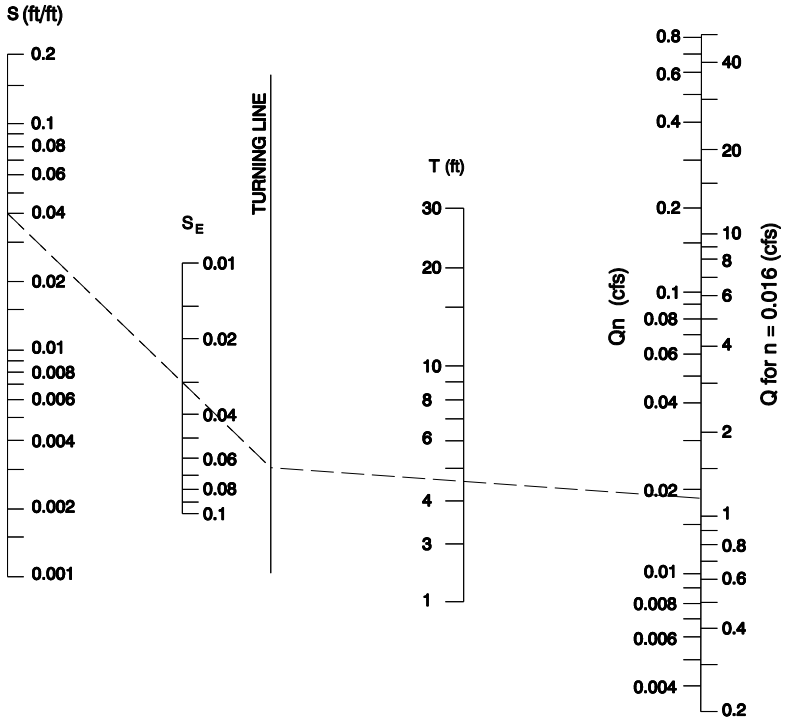



$$Q = \frac{0.56}{n} S^{1.67} S^{0.5} T^{2.67}$$

$$S_E = S_{X1} S_{X2} / (S_{X1} + S_{X2})$$

$$d = TS_E$$

Example: Given:
$n = 0.016$, $S_{X1} = 0.50$ ft/ft, $S_{X2} = 0.02$ ft/ft
$S = 0.04$ ft/ft, $T = 6$ ft
Find:
$S_E = (0.50)(0.02)/(0.50 + 0.02)$
$S_E = 0.019$
$Q = 1.2$ cfs
$d = (6)(0.019) = 0.11$ ft




S = longitudinal gutter slope (ft/ft)

$S_{X1}$, $S_{X2}$ = gutter cross-slope (ft/ft)

$S_E$ = equivalent gutter cross-slope (ft/ft)

T = flow width in gutter (ft)

Q = total gutter flow (cfs)

n = Manning's "n"

d = flow depth at curb (ft)

Example: Given:
$n = 0.012$, $S_{X1} = 0.50$ ft/ft,
$S_{X2} = 0.02$ ft/ft, $S = 0.04$ ft/ft
$T = 6$ ft
Need:
$S_E = (0.50)(0.02)/(0.50 + 0.02)$
$S_E = 0.019$ ft/ft
$Q_n = 0.019$ cfs
$Q = \frac{Q_n}{n} = \frac{0.019}{0.012} = 1.6$ cfs

**Figure 8-16 Discharge Nomograph for Gutter with Two Cross Slopes**

To determine discharge in gutter with composite cross slopes, find $Q_S$ using $T_S$ and $S_X$. Then, use Figure 8-18 to find $E_O$. The total discharge is $Q = Q_S/(1 - E_O)$, and $Q = Q - Q_S$

$d = WS_W + T_S S_X$

Example: Given:

$n = 0.016$, $S_W = 0.08$ ft/ft, $S_X = 0.02$ ft/ft

$T = 6$ ft, $W = 1.5$ ft, $S = 0.04$ ft/ft

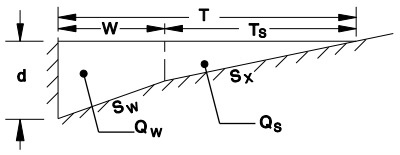
Find:

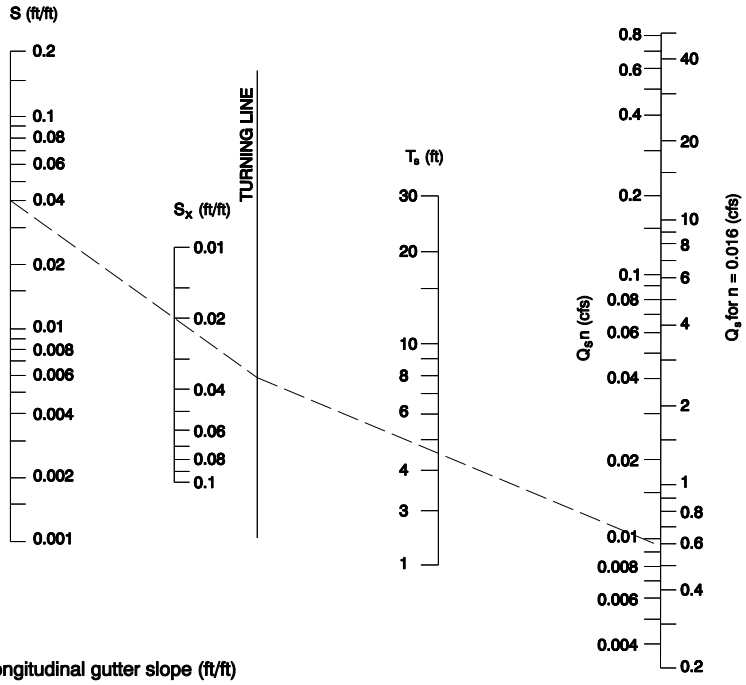
$T_S = T - W = 6 - 1.5 = 4.5$ ft

$Q_S = 0.59$ cfs

$E_O = 0.67$ (from Figure 8-18)

$Q = 0.59/(1 - 0.67) = 1.8$ cfs




S = longitudinal gutter slope (ft/ft)

$S_W$ = depressed gutter cross slope (ft/ft)

$S_X$ = gutter cross-slope (ft/ft)

T = flow width in gutter (ft)

$T_S$ = flow width outside of depressed gutter (ft)

W = depressed gutter flow width (ft)

Q = total gutter flow (cfs)

$Q_S$ = flow outside of depressed gutter (cfs)

$Q_W$ = depressed gutter flow (cfs)

$E_O$ = ratio of depressed gutter flow to total gutter flow

Example: Given:

$n = 0.012$, $S_W = 0.08$ ft/ft, $S_X = 0.02$ ft/ft

$T = 6$ ft, $W = 1.5$ ft, $S = 0.04$ ft/ft

Find:

$T_S = T - W = 6 - 1.5 = 4.5$ ft

$Q_S n = 0.0093$ cfs

$Q_S = \frac{Q_S n}{n} = \frac{0.0093}{0.012} = 0.77$ cfs

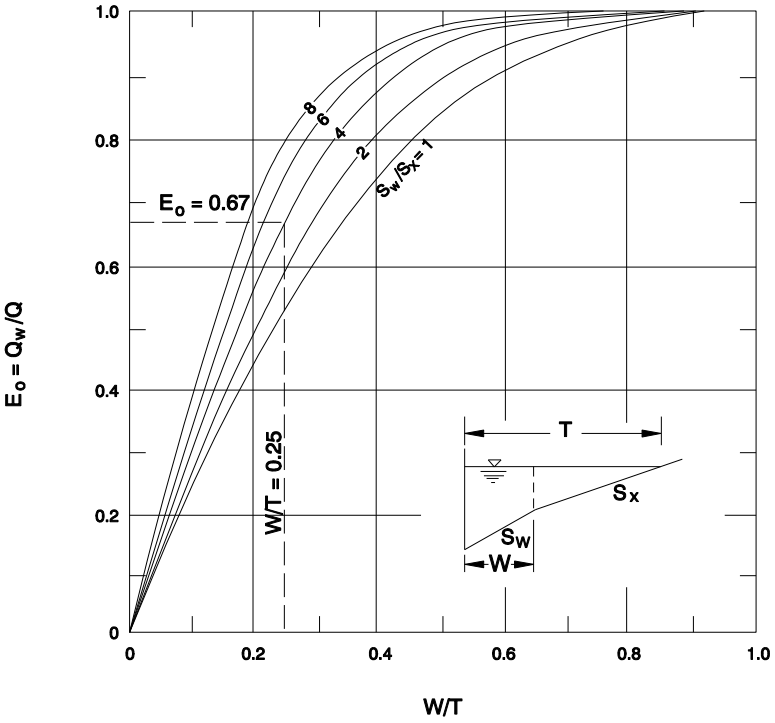
$E_O = 0.67$ (from Figure 8-18)

$Q = 0.77/(1 - 0.67) = 2.3$ cfs

**Figure 8-17 Discharge Nomograph for Gutter with Near Vertical Curb and Two Gutter Cross Slopes**

Example: Given:
$W = 1.5$ ft, $T = 6$ ft, $S_X = 0.02$ ft/ft, $S_W = 0.08$ ft/ft

Find:
$W/T = 1.5/6 = 0.25$ $S_W/S_X = 0.08/0.02 = 4$
$E_O = 0.67$




$E_O$ = ratio of depressed gutter flow to total gutter flow

$Q$ = total gutter flow (cfs)

$Q_W$ = depressed gutter flow (cfs)

$W$ = depressed gutter width (ft)

$T$ = flow width in gutter (ft)

$S_W$ = depressed gutter cross slope (ft/ft)

$S_X$ = gutter cross slope (ft/ft)

**Figure 8-18 Ratio of Depressed Gutter Flow to Total Gutter Flow**

## 8.14 Closed Conduits

Determining the hydraulic properties of closed conduits is essential to evaluate and design culverts, storm drains, and many other hydraulic structures. Many techniques can be used, such as equations, hydraulic element charts, nomographs, and computer programs. Equations, hydraulic element charts, and nomographs are discussed in this chapter.

A comprehensive set of nomographs for common conduit and channel shapes is in the Federal Highway Administration (FHWA) publication Hydraulic Design Series No. 3 "Design Charts for Open-Channel Flow." The publication is available from the following FHWA website http://www.fhwa.dot.gov/engineering/hydraulics/

Many computer programs to calculate closed conduit hydraulics are available from public and proprietary sources. The most common conduit shapes can be analyzed using the FHWA VISUAL URBAN program. This software is based on FHWA Hydraulic Engineering Circular No. 22 "Urban Drainage Design Manual" (HEC-22).

### 8.14.1 Equations for Full Flow in Closed Conduits

The hydraulic properties of a conduit flowing full, but not under surcharge or pressure flow, are useful tools in hydraulic design. The Manning's equation can be used to calculate the velocity and discharge in conduits flowing full, but not under pressure flow. The Manning's equation can also be used to estimate approximate discharges and velocities in conduits under slight surcharge such as storm drains conveying check discharges. In this case the pressure flow component of the total flow is considered insignificant and it is ignored. Usually the Manning's equation is adequate for these estimates. Accurate hydraulic analyses of critical applications or systems under significant pressure flow should use other methods, such as the Hazen-Williams formula.

Output and input variables of Manning's equations for full flow in conduits follow:

$V_{FULL}$ = velocity in full conduit in feet per second
$Q_{FULL}$ = discharge in full conduit in cubic feet per second
$D_{FULL}$ = diameter of circular conduit needed to convey full flow, in feet
$S_f$ = friction slope of full or slightly surcharged circular conduit, in feet per foot
n = Manning's roughness coefficient
R = hydraulic radius = A / P, in feet
D = diameter of circular pipe, in feet
A = cross-sectional area of full conduit in square feet
P = inside circumference of conduit in feet
S = slope of energy gradeline in foot per foot (S = conduit slope for steady uniform flow.)
V = velocity in conduit flowing full in cubic feet per second

Flow areas and hydraulic radii for many common closed conduits are listed in Appendix B. Use of these values will simplify calculations.

**Manning's Roughness Coefficients –** In Manning's equations the roughness of the wetted perimeter is represented by the Manning's roughness coefficient. Roughness values for most conduits are listed in Appendix A. Many of the listed values are for culverts and storm drains with a constant roughness coefficient throughout their wetted perimeter. An example is a typical new circular concrete pipe. The roughness throughout the wetted perimeter has the same value, and the Appendix lists a typical value of 0.013.

The appendix also lists Manning's coefficients for many conduits with varying roughness on their wetted perimeters. An example is a corrugated metal pipe with a paved invert. The roughness varies because the conduit has a smooth concrete bottom and relatively rough corrugated sides. The Appendix lists several roughness values for different flow depths ranging from 0.013 to 0.021.

The Appendix does not list roughness values for all conduits. An example is a box culvert with smooth reinforced concrete sides and a relatively rough channel bottom of cobbles and gravels. This conduit has varying roughness coefficients, and it can be analyzed using a composite Manning's roughness value, $n_c$. This method is also used in the analysis of natural stream channels, as discussed previously in this chapter. $n_c$ is calculated by dividing the wetted perimeter (P) into parts (N), each with an individual roughness coefficient ($n_i$) and use of Equation 8-20. The following formula is used:

$$n_c = \left[ \frac{\sum (n_i^{3/2} P_i)}{P} \right]^{2/3} \qquad \text{(Equation 8-37)}$$

$i = 1$

Where:

$n_c$ = composite Manning's roughness value for wetted perimeter
$P$ = wetted perimeter of entire conduit in feet
$P_i$ = wetted perimeter of the individual subdivision of the entire wetted perimeter
$n_i$ = Manning's coefficient of roughness for the individual subsection

*Example:* The composite roughness coefficient $n_c$ is needed for a box culvert with a 2.0-foot wide bottom. The roughness of the gravels and cobbles on the culvert bottom is similar to typical ODOT Class 25 loose riprap with a roughness coefficient of 0.070. The sides are formed concrete with a roughness coefficient of 0.013. The estimated flow depth is 0.5 foot.

$$n_c = \left[ \frac{\left(0.013^{3/2} \text{ x } 0.5\right) + \left(0.070^{3/2} \text{ x } 2.0\right) + \left(0.013^{3/2} \text{ x } 0.5\right)}{\left(0.5 + 2.0 + 0.5\right)} \right]^{2/3} = 0.55 \leftarrow$$

The composite Manning's roughness value of the culvert is 0.055 at a flow depth of 0.5 foot.

**Manning's Equation for Full Flow Velocity in Conduits -** The mean velocity, V, can be computed with Manning's equation for a conduit flowing full using the following formula:

$$V_{FULL} = \left(\frac{1.486}{n}\right) R^{2/3} S^{1/2} \qquad \text{(Equation 8-38)}$$

**Manning's Equation for Full Flow Discharge in Conduits -** The continuity equation can be combined with Manning's equation to obtain Manning's equation for discharge:

$$Q_{FULL} = \left(\frac{1.486}{n}\right) A\, R^{2/3} S^{1/2} \qquad \text{(Equation 8-39)}$$

**Manning's Equation to Determine n Value –** Manning's n

value can be calculated for a conduit under full flow, if either velocity or discharge is known, using one of the following versions of the Manning's equation:

$$n = \frac{1.486\,(R^{2/3} S^{1/2})}{V_{FULL}} \text{ if velocity is known} \qquad \text{(Equation 8-40)}$$

$$n = \frac{1.486\,(A\, R^{2/3} S^{1/2})}{Q_{FULL}} \text{ if discharge is known} \qquad \text{(Equation 8-41)}$$

**Manning's Equation for Full Flow in Circular Conduits –** A version of the Manning's equation can be used to calculate full flow discharge in circular conduits. Another version can be used to calculate the diameter needed to convey full flow. The equations follow.

$$Q_{Full} = \left(\frac{0.464}{n}\right) D^{8/3} S^{1/2} \qquad \text{(Equation 8-42)}$$

$$D_{Full} = \left(\frac{2.16Qn}{S^{1/2}}\right)^{3/8} \qquad \text{(Equation 8-43)}$$

**Manning's Equation to Determine Friction Slope for Circular Conduits –** Friction loss ($H_f$) is the energy required to overcome barrel roughness and is the most significant energy loss in a closed conduit system. It is particularly significant for surcharged or pressure flow. Friction loss is directly related to the velocity in the pipe; therefore, the higher the velocity, the greater the friction loss and vice versa. The friction loss can be calculated for circular conduits flowing full using Equations 8-44 and 8-45. These equations calculate the friction slope for conduits flowing full, and they estimate the friction slope for a conduit flowing with slight surcharge under pressure flow.

$$H_f = S_f L \qquad \text{(Equation 8-44)}$$

Where:

$H_f$ = friction loss in feet
$S_f$ = friction slope in feet per foot
$L$ = conduit length in feet

The friction slope can be calculated using a version of the Manning's equation:

$$S_f = \frac{V^2 n^2}{(2.21)(R^{4/3})} \qquad \text{(Equation 8-45)}$$

### 8.14.2 Nomographs for Full Flow in Closed Conduits

Nomographs can also be used to estimate full flow in conduits. Full flow hydraulic characteristics for circular concrete and corrugated metal pipes are shown in Figures 8-19 and 8-20, respectively. Examples of typical applications are shown on the figures. Nomographs for full flow in many common conduit shapes are in the FHWA publication listed at the beginning of this section.

Full flow nomographs can also be used to estimate friction loss in pipes under slight surcharge. This is often needed to evaluate a storm drain system during a check discharge, as shown in the following example.

#### 8.14.2.1 Example – Estimate Friction Loss in Pipe Under Slight Surcharge

A storm drain is designed to have the smallest possible conduits providing free surface flow during the 10-year event. The pipes are sized to be 75 to 95 percent full during this design storm. Local regulations require that the system be checked for the 50-year event. It is necessary to verify that water is contained within the system and does not flow out of manholes or inlets. One task in this analysis is to determine the friction loss in a section of 100-foot long 24-inch diameter corrugated metal pipe during a 15 cubic feet per second check discharge.

Given:

$Q_{50}$ = 20 cubic feet per second
$n$ = 0.024 (corrugated metal)

L = 100 feet

Need:

$H_f$ = friction loss in feet

**Step 1 -** The nomograph in Figure 8-20 is used to determine the friction slope. A line is drawn between the 15 cubic feet per second discharge on the DISCHARGE scale, through the 24-inch diameter on the PIPE DIAMETER scale, to the SLOPE scale. The friction slope is read from the SLOPE scale, and it is 0.015 foot per foot, as shown on Figure 8-20.

$S_f$ = 0.015 foot per foot

**Step 2 -** The friction loss is calculated using Equation 8-44, as follows:

$H_f = (0.015)(100) = 1.5$ feet ← Answer

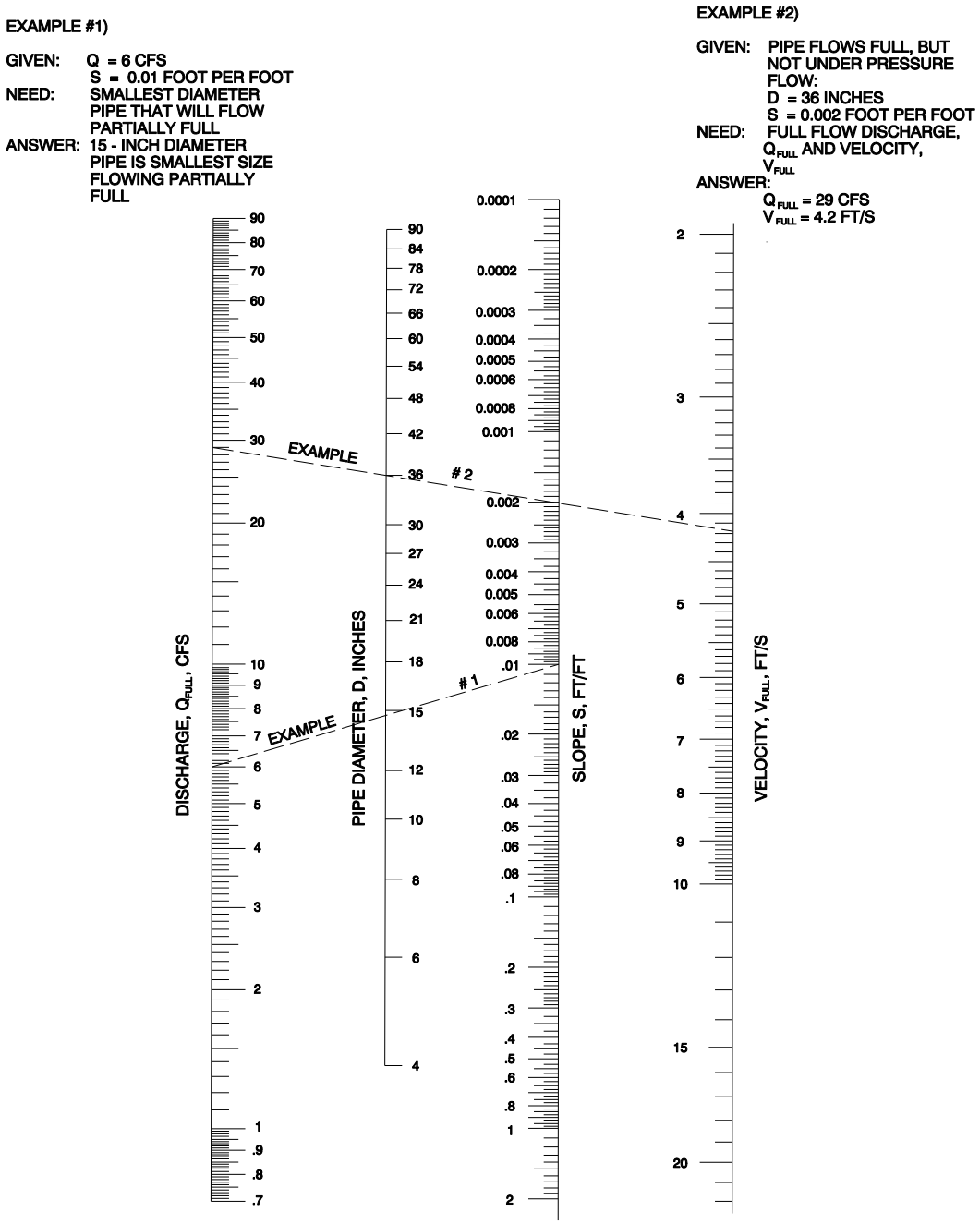



**Figure 8-19 Capacity and Velocity Nomograph for Circular Concrete Pipes Flowing Full**

EXAMPLE
GIVEN: Q = 15 CFS
D = 24 INCHES
NEED: ESTIMATE OF FRICTION
SLOPE IN FEET PER FOOT
ANSWER: FRICTION SLOPE = 0.015 FOOT
PER FOOT

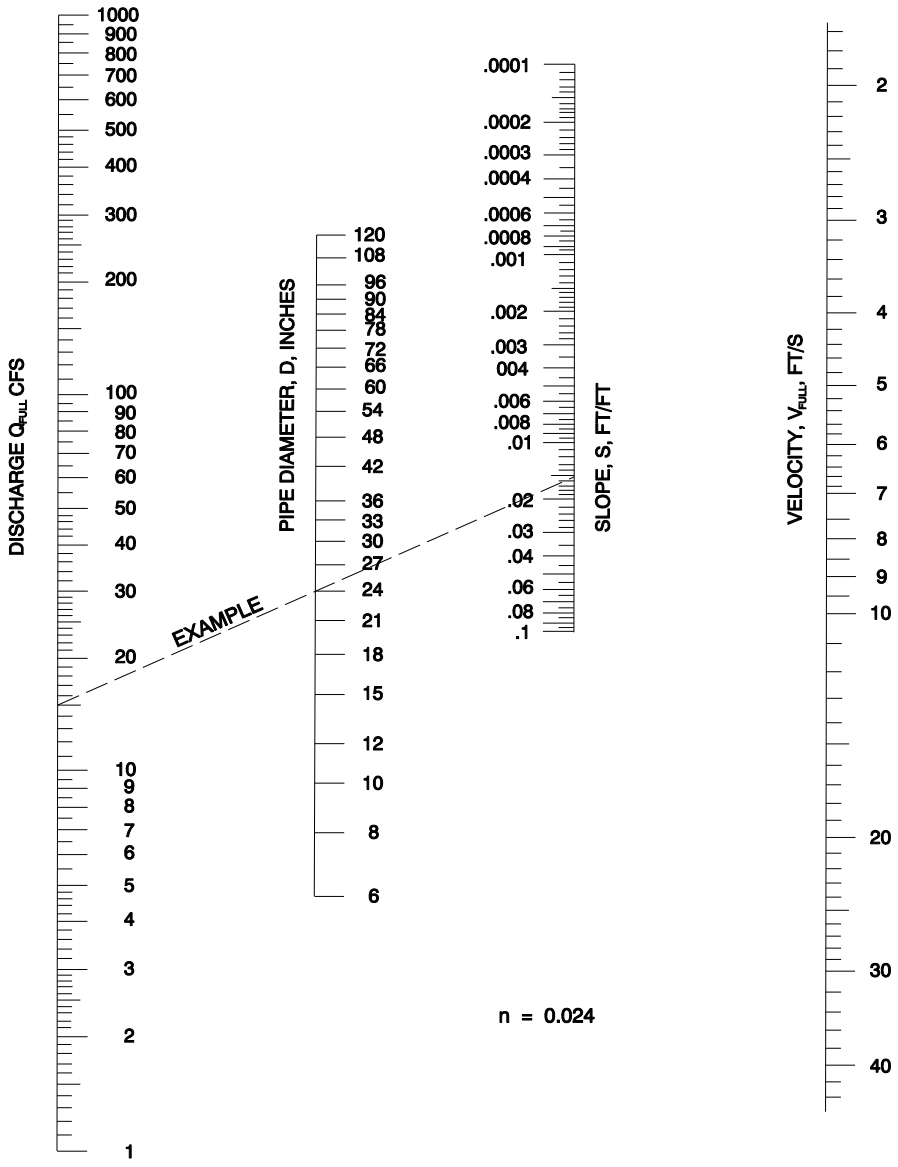



**Figure 8-20 Capacity and Velocity Nomograph for Circular Corrugated Metal Pipes Flowing Full**

### 8.14.3 Equations for Normal Flow in Partially Full Conduits

The hydraulic properties of normal flow are a useful hydraulic analysis tool. Normal flow properties of open-channel discharge in closed conduits can be determined by many methods. These procedures can also be used for open-channels with shapes similar to partially full conduits, such as flumes and lined prismatic canals. Normal flow occurs:

- in prismatic conduits such as pipes or boxes where the cross-section and hydraulic roughness does not vary significantly between upstream and downstream sections,
- during open-channel flow,
- where the depth of flow is the same in upstream and downstream sections,
- where the flow streamlines are fairly parallel to each other and to the long axis of the conduit, and
- where turbulence such as hydraulic jumps and drops do not occur.

Manning's equations for velocity (Equation 8-12) and discharge (Equation 8-15) can be used with the continuity equation (Equation 8-11) to analyze normal flow in conduits. This method can be used for open channels of any shape. It is most useful for odd or irregular shapes that can not be analyzed by other procedures. It involves an iterative solution where the depth of flow is assumed and the flow area, hydraulic radius, and discharge are calculated. The iterations are repeated using various flow depths until the calculated discharge matches the desired discharge. In the last step the discharge is divided by the flow area to determine the average velocity. An example of this procedure using Manning's equation follows this subsection.

Irregular shapes often have varying hydraulic roughness throughout their cross-section. An example would be a box culvert with a hydraulically rough fish ladder on one half of the bottom, a smooth concrete surface on the other half of the bottom, and brick sides. A composite Manning's n values based on Equation 8-37 would be used when analyzing this conduit.

Normal velocity in circular pipes can be approximated within $\pm$ 5 percent using the following version of the Manning equation. It is sufficiently accurate for many purposes.

$$V_n = 0.863 \frac{\left(S^{0.366}\right)\left(Q^{0.268}\right)}{\left(n^{0.732}\right)\left(D^{0.048}\right)} \qquad \text{(Equation 8-46)}$$

Where:

$V_n$ = normal velocity for partial flow in feet per second
S = channel slope in foot per foot
Q = flow rate in cubic feet per second
N = Manning's roughness coefficient (Appendix A)
D = pipe diameter in feet

### 8.14.3.1 Example – Manning's Equation to Determine Normal Flow Properties

Normal flow velocity needs to be determined for an elliptical pipe laid with its long axis horizontal.

Given:

14-inch rise by 23-inch span horizontal elliptical concrete pipe,
pipe slope is 0.005 foot per foot,
discharge is 1.0 cubic foot per second, and
Manning's n = 0.013 from Appendix A.

Need:

average flow velocity in pipe assuming normal flow

The procedure is to assume a flow depth and solve for the discharge. The process is repeated until the discharge is close to 1.0 cubic feet per second. The average velocity is determined by dividing the discharge by the flow area. Intermediate steps in these calculations require the hydraulic radius and flow area. These values are typically measured from a drawing of the conduit cross-section.

| Assumed Flow Depth, d (feet) | Flow Area, A (square feet) | Hydraulic Radius, R (feet) | Discharge, Q $\left(\frac{1.486}{n}\right) A R^{2/3} S^{1/2}$ (cubic feet per second) |
|---|---|---|---|
| 0.33 | 0.445 | 0.225 | 1.33 |
| 0.26 | 0.323 | 0.179 | 0.829 |
| 0.29 | 0.373 | 0.197 | 1.02 |
| 0.285 | 0.370 | 0.195 | 1.00 |

$$V = \frac{Q}{A} = \frac{1.00}{0.370} = 2.7 \text{ feet per second} \leftarrow \text{Answer}$$

### 8.14.4 Nomographs for Normal Flow in Partially Full Conduits

Nomographs can be used to determine the hydraulic properties of partially full conduits. Nomographs for many common conduit shapes are in the FHWA publication listed at the beginning of this section.

### 8.14.5 Hydraulic Elements Charts for Normal Flow in Partially Full Conduits

Normal flow properties can be determined by the hydraulic element charts in Appendix B for circular pipes, pipe-arches, semicircular arches, and reinforced concrete boxes. Although this method is easy to use, it has some limitations. First, the method can only be used for the conduit shapes represented by charts. The iterative solution shown previously is recommended for conduits not shown on the charts. Second, the hydraulic element method gives accurate results if the conduit has a constant roughness value throughout its entire wetted perimeter at full flow, such as a corrugated metal pipe. This method gives approximate results for conduits with varying roughness coefficients such as box culverts with gravel covered bottoms or corrugated metal pipes with paved inverts. The iterative solution method is recommended for accurate analysis of these conduits.

#### 8.14.5.1 Example – Hydraulic Element Chart to Determine Normal Flow Properties in a Circular Pipe

Hydraulic properties such as depth and velocity are needed for a 40 cubic foot per second discharge through a 48-inch diameter circular corrugated metal pipe on a 0.6 percent grade.

Given:

D = 48 inches = 4.0 feet = pipe diameter
S = 0.6 / 100 = 0.006 foot per foot = pipe slope
q = 40 cubic feet per second
n = 0.023 from Appendix A for a 48-inch diameter corrugated metal pipe

Need:

d = flow depth in feet
V = average flow velocity in feet, and
a = flow area in square feet

**Step 1 –** Determine one of the following proportional ratios:

q / $Q_{FULL}$ will be selected as the proportional ratio since q is known. $Q_{FULL}$ is calculated by Equation 8-42 as follows:

$$Q_{FULL} = \left(\frac{0.464}{0.023}\right)(4.0^{8/3})\,(0.006^{1/2}) = 63 \text{ cubic feet per second}$$

$\frac{d}{D_{FULL}}$ or

$\frac{a}{A_{FULL}}$

$$\frac{q}{Q_{FULL}} = \frac{40}{63} = 0.63$$

**Step 2 -** Locate the known proportional value from Step 1 on the appropriate axis of the hydraulic elements chart, as shown in Figure 8- 21. This would be a value of 0.63 on the horizontal axis for q / $Q_{FULL}$ ratio of 0.63. Extend a line upward from the known proportional value on the axis to the curve representing the known proportional value. This will be the q / $Q_{FULL}$ curve.

**Step 3 -** Extend a horizontal line from the intersection point of the line and curve determined in the previous step. This line will intercept one of the axes and several of the curves. In this example, the horizontal line intersects the d / $D_{FULL}$ axis at a ratio of 0.57. Draw vertical lines from the intersection points on the various curves to the axes that represent these curves. The vertical lines from the a / $A_{FULL}$, v / $V_{FULL}$, and r / $R_{FULL}$ curves intercept the horizontal axis at ratios of 0.58, 1.05, and 1.08, respectively.

**Step 4 -** Calculate desired values by multiplying the full flow values by the corresponding proportional values. As an example: v = ($V_{FULL}$)(v / $V_{FULL}$). In some cases the

$\frac{q}{Q_{FULL}}$

$\frac{v}{V_{FULL}}$

continuity equation (Equation 8-11) can also be used. In the example:

$A_{FULL} = \pi r^2$ = ( 3.142 ) (2 feet)$^2$ = 12.5 square feet

$$a = (A_{FULL})\left(\frac{a}{A_{FULL}}\right) = (12.5)(0.58) = 7.3 \text{ Square feet} \leftarrow \text{ Answer}$$

$$d = (D_{FULL})\left(\frac{d}{D_{FULL}}\right) = (2)(0.57) = 1.1 \text{ feet} \leftarrow \text{ Answer}$$

Using the continuity equation:

$$v = \frac{q}{a} = \frac{40}{7.3} = 5.5 \text{ feet per second} \leftarrow \text{ Answer}$$

### 8.14.6 Critical Depth in Conduits

Critical depth ($d_c$) is defined as the depth for which the specific energy (sum of the flow depth and velocity head) of a given discharge is at a minimum. A slight change in specific energy can result in a significant rise or fall in the water depth when flow is at or near critical depth. Because of this, critical depth is an unstable condition and it rarely occurs for any distance along a water surface profile.

Flow at critical depth, however, is a common occurrence. Flow always passes through critical depth during transitions between flow regimes, such as the transition between the subcritical flow regime, where normal depth is greater than critical depth, and the supercritical flow regime, where normal depth is less than critical depth. This typically occurs when flow passes through a contraction, either horizontal or vertical, such as a culvert inlet, outlet, storm sewer manhole, etc. A detailed discussion of the relationship between critical depth and specific energy is beyond the scope of this manual. The designer should refer to any open-channel flow reference text for further information.

Critical depth is a useful tool in hydraulic analysis because it can be used to determine the type of flow profile inside the conduit and at the outlet. Critical depth is dependent on the shape of the conduit and the discharge, and it can easily be shown on a chart. Critical depth charts for most common conduit shapes are in Appendix B.

Critical depth can be calculated for conduit shapes that are not included in the charts. The following method can be used to determine critical depth for open channels of any shape. It is based on the principle that critical depth occurs during critical flow. It involves an iterative solution where the depth of flow is assumed, the flow area is calculated, the top width of the flow is determined, and the discharge at critical flow calculated. The procedure is repeated until the critical flow based on the assumed depth equals the discharge of interest. The flow depth at critical flow is critical depth.

The following equation is used:

$$Q_C = \left( \frac{A^3 g}{T} \right)^{1/2}$$ (Equation 8-47)

Where:

$Q_C$ = discharge at critical depth in cubic feet per second
g = acceleration of gravity (32.2 feet per second squared)
A = flow area in square feet
T = top width of flow in feet

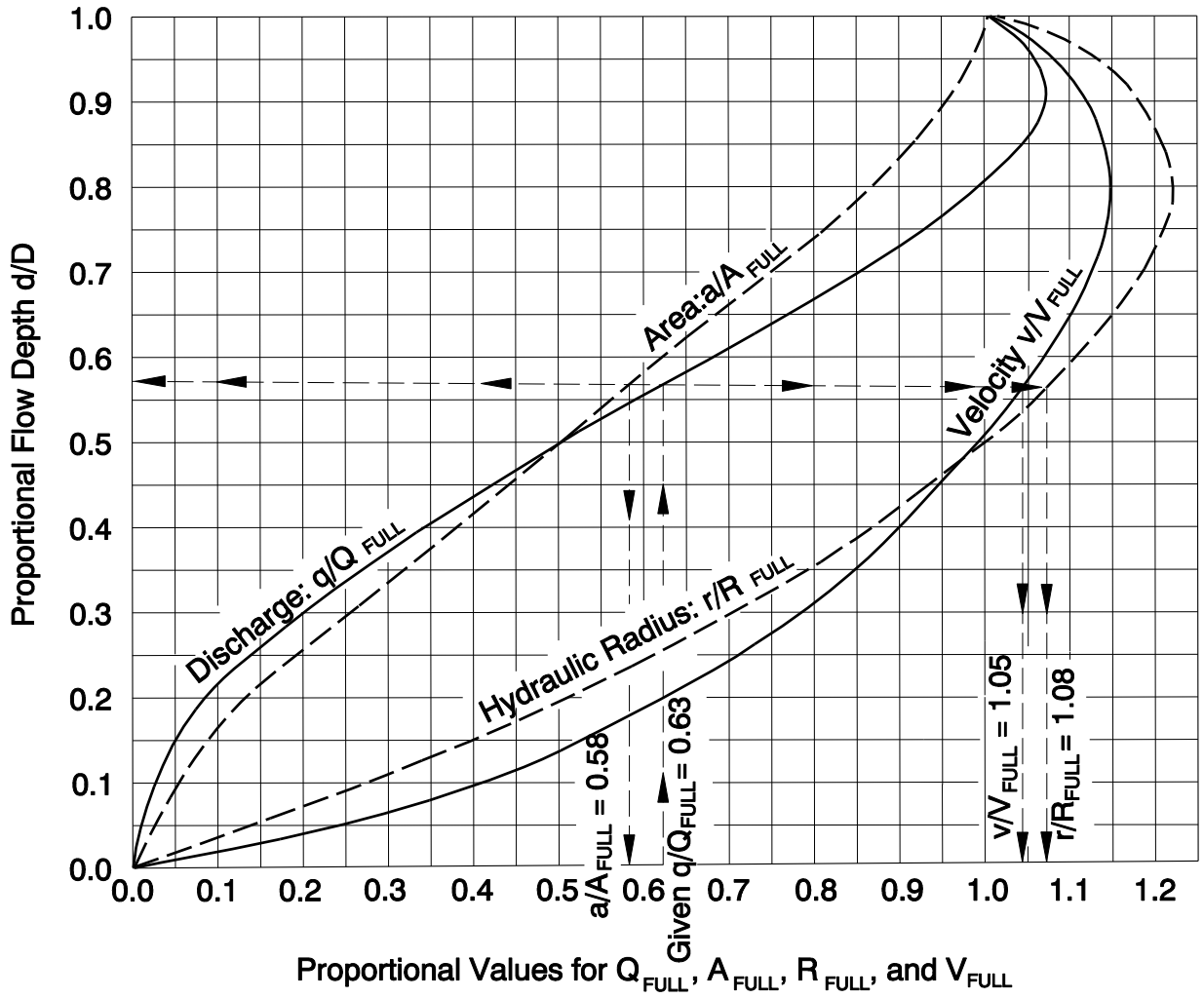



**Figure 8-21 Use of Hydraulic Elements Chart**

*Example:* Critical depth is needed for a trapezoidal shaped culvert during a 1200 cubic feet per second discharge.

Given:

trapezoidal channel has a 9-foot wide bottom width and 1V:2H side slopes, and
$Q = Q_C$ = 1200 cubic feet per second.

Need:

$D_C$ = critical depth in feet

Assume a depth and solve for $Q_C$. Repeat process until $Q_C$ is close to 1200 cubic feet per second. Areas and top widths are typically measured from a drawing of the conduit cross-section. A programmable calculator is strongly recommended for the calculations.

| Assumed d (feet) | A (square feet) | T (feet) | $\left(\frac{A^3 g}{T}\right)^{1/2}$ (cubic feet a second) |
|---|---|---|---|
| 3 | 45 | 21 | 370 |
| 6 | 126 | 33 | 1400 |
| 5.2 | 101 | 30 | 1050 |
| 5.56 | 112 | 31.2 | 1200 |

The critical depth for the given channel and discharge is approximately 5.56 feet ← Answer

Critical depth can be determined for a circular channel using Appendix B, or it can be approximated as follows:

$$D_C = 0.420\left(\frac{Q^{1/2}}{D^{1/4}}\right) \qquad \text{(Equation 8-48)}$$

Where:

$D_C$ = critical depth in a circular section in feet
$Q$ = flow in cubic feet per second
$D$ = diameter in feet

*Example:* Find critical depths for a 3-foot diameter pipe with flows of 18 and 180 cubic feet per second.

Given:

D = 3 feet
$Q_1 = Q_{C1}$ = 18 cubic feet per second and $Q_2 = Q_{C2}$ = 180 cubic feet per second

Need:

$D_{C1}$ and $D_{C2}$ in feet

Using Equation 8-48:

$$D_{C1} = 0.420\left(\frac{18^{1/2}}{3^{1/4}}\right) = 1.3 \text{ feet for a flow of } 18 \text{ cubic feet per second}$$

$$D_{C2} = 0.420\left(\frac{180^{1/2}}{3^{1/4}}\right) = 4.3 \text{ feet for a flow of } 180 \text{ cubic feet per second}$$

The critical depth for a discharge of 18 cubic feet per second is 1.3 feet ← Answer

The critical depth for a discharge of 180 cubic feet per second is 4.3 feet. The maximum critical depth that can occur is 3 feet because this is the diameter of the pipe. Greater critical depths cannot occur because the conduit will not be conveying open-channel flow. As a result, the critical depth of 4.3 feet is called a "calculated critical depth." Although the calculated critical depth cannot occur, it is useful because it indicates the conduit is flowing full with a surcharge.

Critical depth, and consequently critical flow, do not occur during an 180 cubic foot per second discharge in a 3-foot diameter pipe ← Answer

Critical depth can be calculated for a rectangular channel, such as a box culvert, using Appendix B, or it can be approximated as follows:

$$D_C = \left(\frac{0.176\,Q}{S}\right)^{2/3} \qquad \text{(Equation 8-49)}$$

Where:

$D_C$ = critical depth in feet
Q = flow in cubic feet per second
S = span of box section in feet

*Example:* Calculate critical depth in a box culvert with a 15-foot span by 10-foot rise during a discharge of 140 cubic feet per second.

Given:

S = 15 feet
Q = 140 cubic feet per second

Need:

$D_C$ in feet

Using Equation 8-49:

$$D_C = \left[\frac{(0.176)(140)}{15}\right]^{2/3} = 1.4 \text{ feet} \leftarrow \text{ Answer}$$

Critical depth can be calculated for a triangular channel, such as a flume, using the following approximation:

$$D_C = 0.757\left(\frac{Q}{Z_1 + Z_2}\right)^{2/5} \qquad \text{(Equation 8-50)}$$

Where:

$D_C$ = critical depth in feet
Q = flow in cubic feet per second
$Z_1$ = slope of channel side in feet per feet
$Z_2$ = slope of opposite channel side in feet per feet

*Example:* Find the critical depth in a triangular shaped channel with 1V:1.75H side slopes during a 880 cubic foot per second discharge

Given:

$Z_1 = Z_2 = 1.75$
Q = 880 cubic feet per second

Need:

$D_C$ in feet

Using Equation 8-50:

$$D_C = 0.757\left(\frac{880}{1.75 + 1.75}\right)^{2/5} = 6.9 \text{ feet} \leftarrow \text{ Answer}$$

Critical depths can be determined for pipe-arches and semicircular arches using charts in Appendix B.

## 8.15 Artificial Channels

Roadside or median ditches or channels, ditches leading from highway drainage systems to outfalls, canals, or other ditches conveying flow through the highway right-of-way are the artificial channels most often encountered in highway design. These artificial channels typically have trapezoidal, rectangular, or triangular cross-sections, and they are in bare earth or lined with grass, riprap, or other protective linings. Design discharge recurrence intervals for these smaller artificial channels are summarized in **Chapter 3** and Section 8.3.2. Design procedures are presented in this section for:

- channels with a prismatic cross-section conveying uniform or nearly uniform flow,
- 50 cubic foot per second or less discharge, and
- 10 percent or flatter channel bottom grade.

Procedures in the latest version of FHWA Hydraulic Engineering Circular Number 11, "Design of Riprap Revetment" can be used to design channels with flows greater than 50 cubic feet per second. Methods in FHWA Hydraulic Engineering Circular Number 15, "Design of Roadside Channels with Flexible Linings" can be used to design channels with grades greater than 10 percent. Both publications are available from the FHWA website: http://www.fhwa.dot.gov/bridge/elibrary.htm

### 8.15.1 Artificial Channel Hydraulics

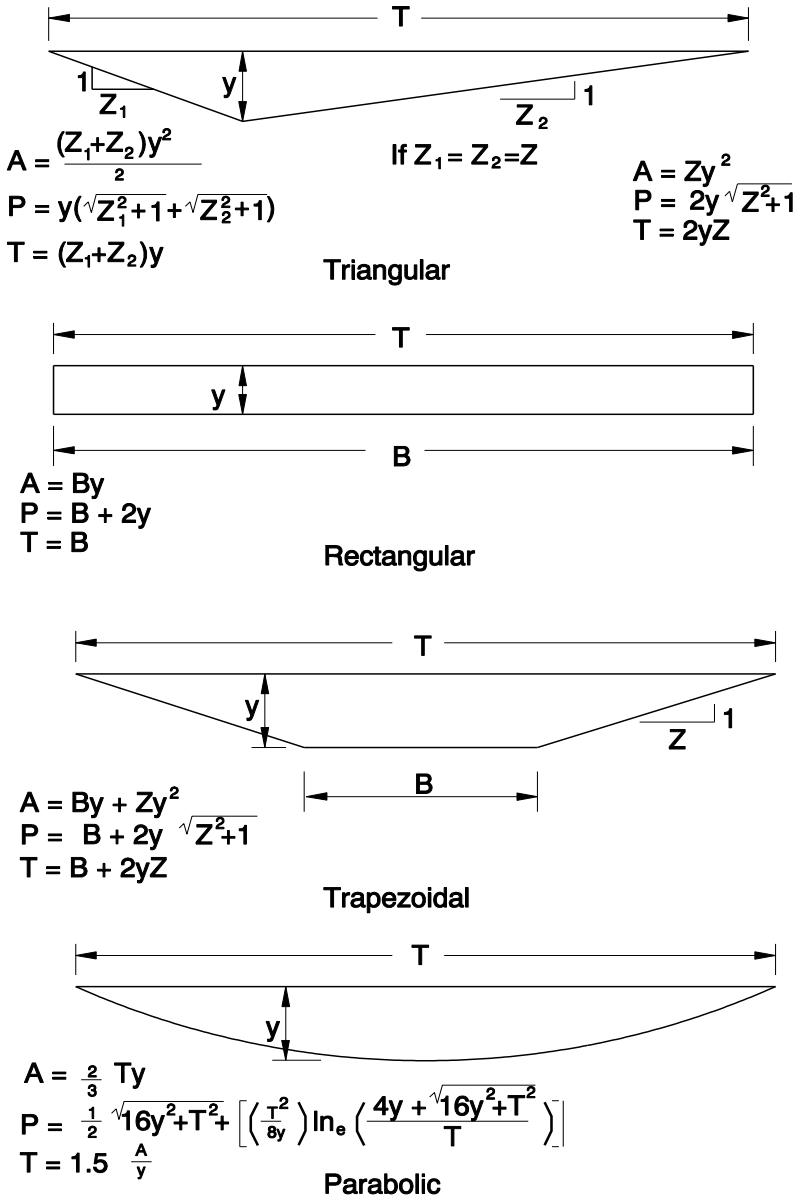
Artificial channels are composed of prismatic shapes where uniform or nearly uniform flow occurs. They also have transition sections where the cross-sections, profiles, surface roughness, or other properties change, and non-uniform or turbulent flow occurs.

Channel sections with non-uniform flow can be analyzed using many methods. Regardless of the procedure, transition and other energy losses need to be considered. The FHWA WSPRO module in HYDRAIN or the US Corps of Engineers HEC-RAS programs are often used.

Hydraulic characteristics of prismatic channel sections with uniform or nearly uniform flow are typically calculated by single section analysis using versions of Manning's equation (Equations 8-12 and 8-15). These procedures are discussed in this section. Single section analysis can also be done using the HYCHL module in the FHWA HYDRAIN software package, or the FHWA VISUAL URBAN program.

Versions of Manning's equation relate the average flow velocity or discharge to channel properties, such as cross-sectional flow area (A), wetted perimeter length (P), energy gradeline slope (S), and the hydraulic roughness (n). Cross-sections of triangular, rectangular, trapezoidal, and parabolic channels are shown in Figure 8-22 with formulae to calculate A, P, and the top width of the flow (T). It is usually assumed S is equal to the slope of the channel bottom if uniform or nearly uniform flow. Artificial channel Manning's n values are listed in Appendix A.

Flow depths, discharges, and other hydraulic properties of rectangular or trapezoidal channels can be estimated using the nomograph in Figure 8-23. Hydraulic properties of triangular channels can be analyzed using the procedures for gutters in Section 8.13. Nomographs for many channel shapes are in the FHWA HDS #3 publication mentioned at the beginning of Section 8.14.




A = Area of flow in square feet
P = Wetted perimeter in feet
T = Top width in feet
B = Bottom width in feet
y = Depth of flow in feet

**Figure 8-22 Channel Geometries**

### 8.15.2 Grassed Channel Linings

Grassed channel Manning's n values are related to the retardance class of the lining, the length of the hydraulic radius, and the channel slope, as shown in Figures 8-24 through 8-28. The retardance classification of various grass linings are summarized in Table 8-2. Retardance is related to the type of grass in the lining and whether it is cut or uncut. In general, cut grass has more retardance than uncut grass of the same height.

Information about the height of the grass before and after mowing can be provided by maintenance personnel, and a description of the grass type can be provided by the designer. The height of uncut grass can be estimated by measuring the height of similar types of uncut grasses in the project vicinity. Typically, the height of the grass during the season of peak runoff is used in the channel design.

**Table 8-2 Retardance Classes of Grass Channel Linings**
(Adapted from FHWA HEC-15)

| Retardance Class | Lining |
|---|---|
| A | Uncut grass, excellent stand, average of 30 to 36 inches tall. |
| B | Uncut grass, good stand, average of 11 to 24 inches tall. |
| C | Uncut grass, fair stand, average of 10 to 48 inches tall. |
| C | Uncut grass, good stand, average of 6 to 18 inches tall. |
| C | Cut grass, good stand, average of 6 inches tall. |
| D | Uncut grass, good to excellent stand, average of 3.6 to 6 inches tall. |
| D | Cut grass, good to very good stand, 2.4 inches tall. |
| E | Cut grass, good stand, average of 1.5 inches tall. |

*Note: Retardance classes are for green and generally uniform cover with flows of 50 cubic feet per second or less. Shorter heights within each retardance class apply to denser grass stands or ground covers.*

Example 1:

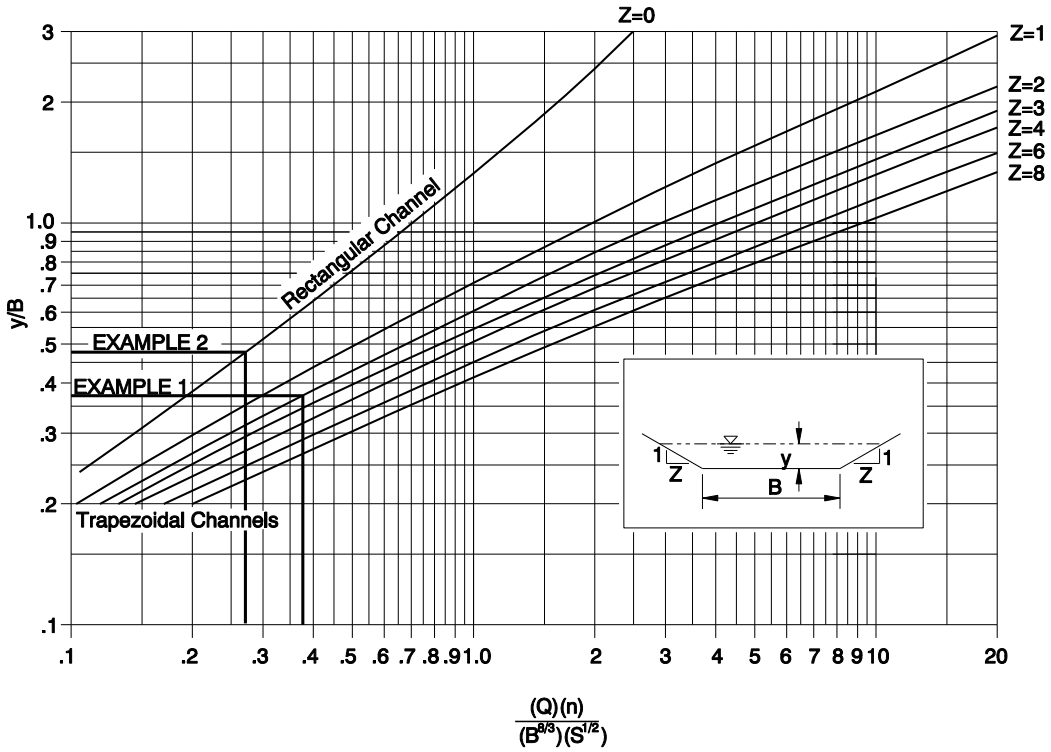
Given: y = 1.1 ft
n = 0.032
B = 3 ft
S = 0.05 ft/ft
Z = 2
Need: Q

$\frac{y}{B} = \frac{1.1}{3} = 0.37$

From Figure: $\frac{(Q)(n)}{(B^{8/3})(S^{1/2})} = 0.375$

$Q = 0.375 \ \frac{(B^{8/3})(S^{1/2})}{(n)} \quad 0.375 \ \frac{(3^{8/3})(0.05^{1/2})}{(0.032)}$

Q = 49 cfs ←




Example 2:

Given: Q = 120 cfs
n = 0.015
B = 6 ft
S = 0.003 ft/ft
Z = 0
Need: y

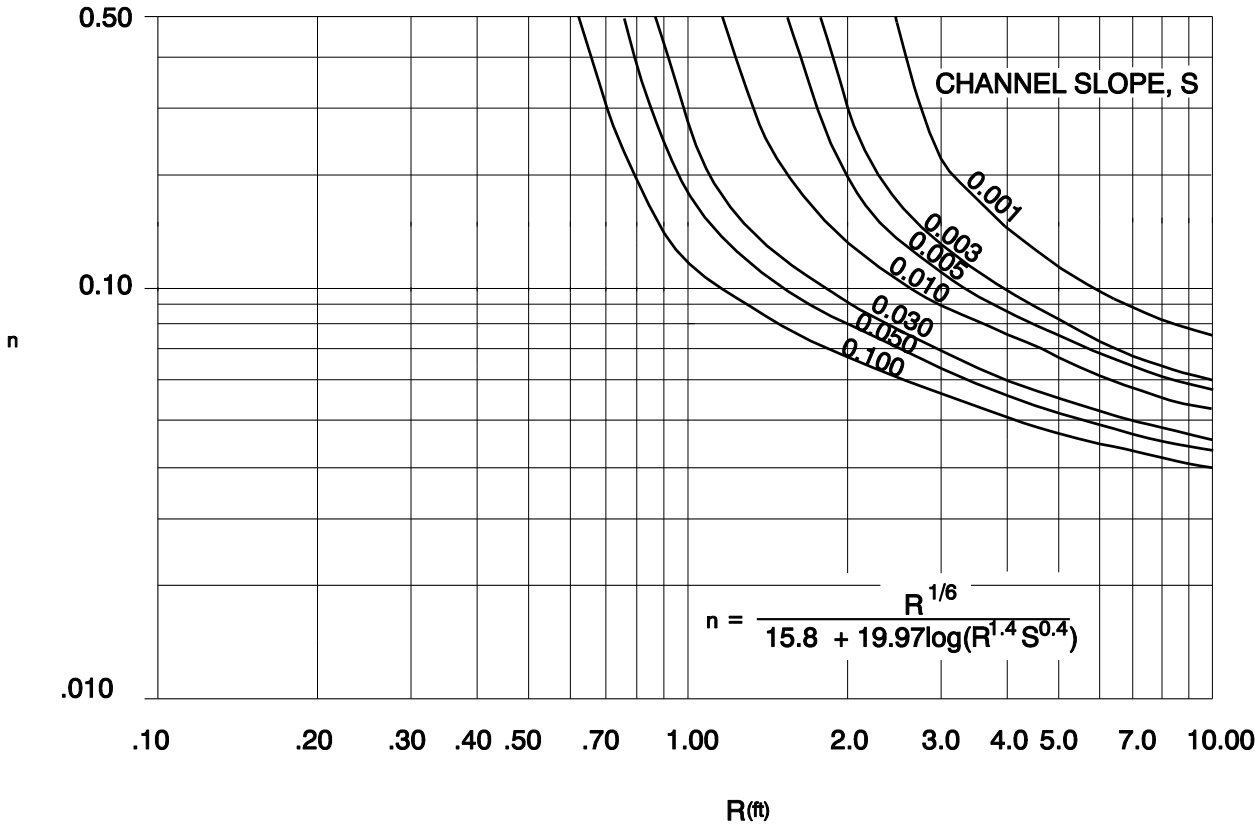
$\frac{(Q)(n)}{(B^{8/3})(S^{1/2})} = \frac{(120)(0.015)}{(6^{8/3})(0.003^{1/2})} = .28$

From Figure, y/B = 0.48

y = (y/B)(B) = (0.48)(6) = 2.9 ft ←

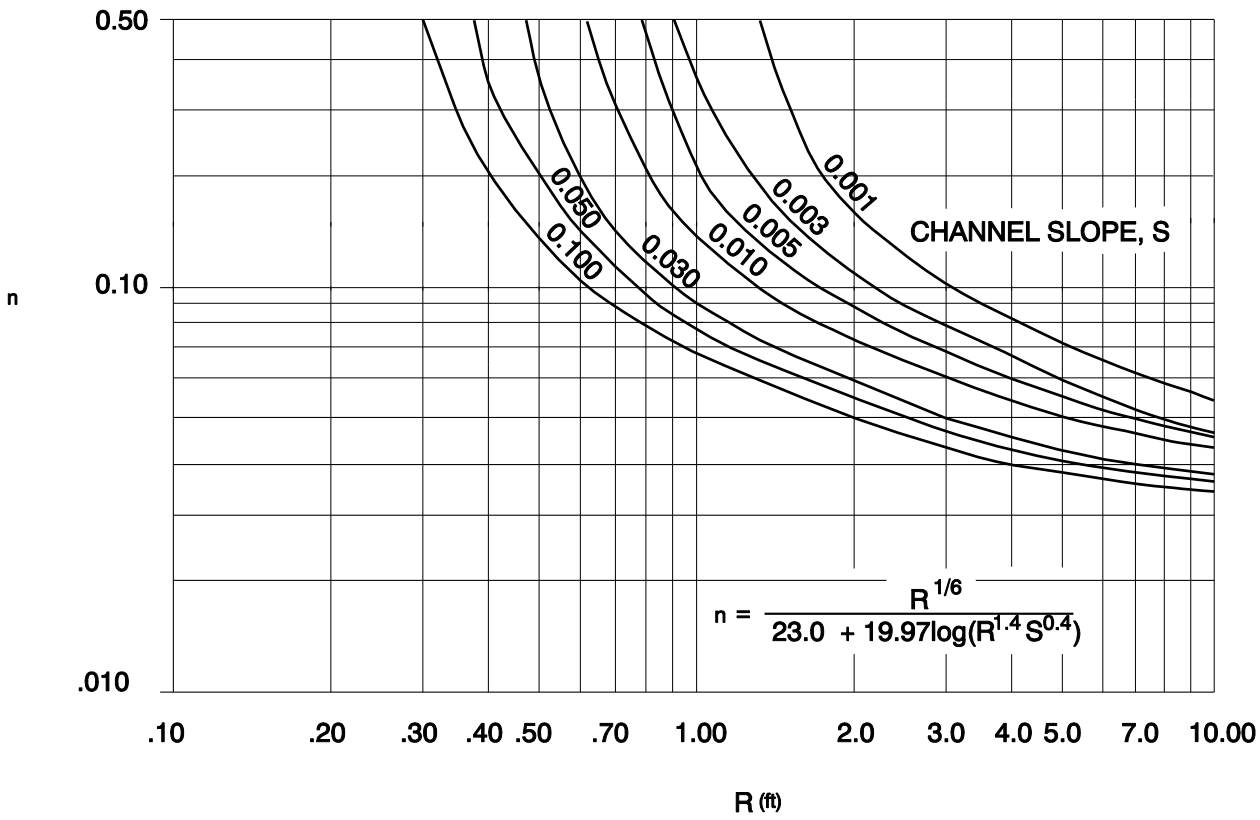
Q = discharge in cubic feet per second
n = Manning's roughness coefficient
B = channel bottom width in feet
y = flow depth in feet
S = channel slope in feet per foot

**Figure 8-23 Rectangular and Trapezoidal Channel Capacity Nomograph**




n = Manning's roughness coefficient
R = hydraulic radius in feet
S = channel slope in feet per foot

**Figure 8-24 Manning's n Versus Hydraulic Radius, R, for Class A Vegetation**




n = Manning's roughness coefficient
R = hydraulic radius in feet
S = channel slope in feet per foot

**Figure 8-25 Manning's n Versus Hydraulic Radius, R, for Class B Vegetation**

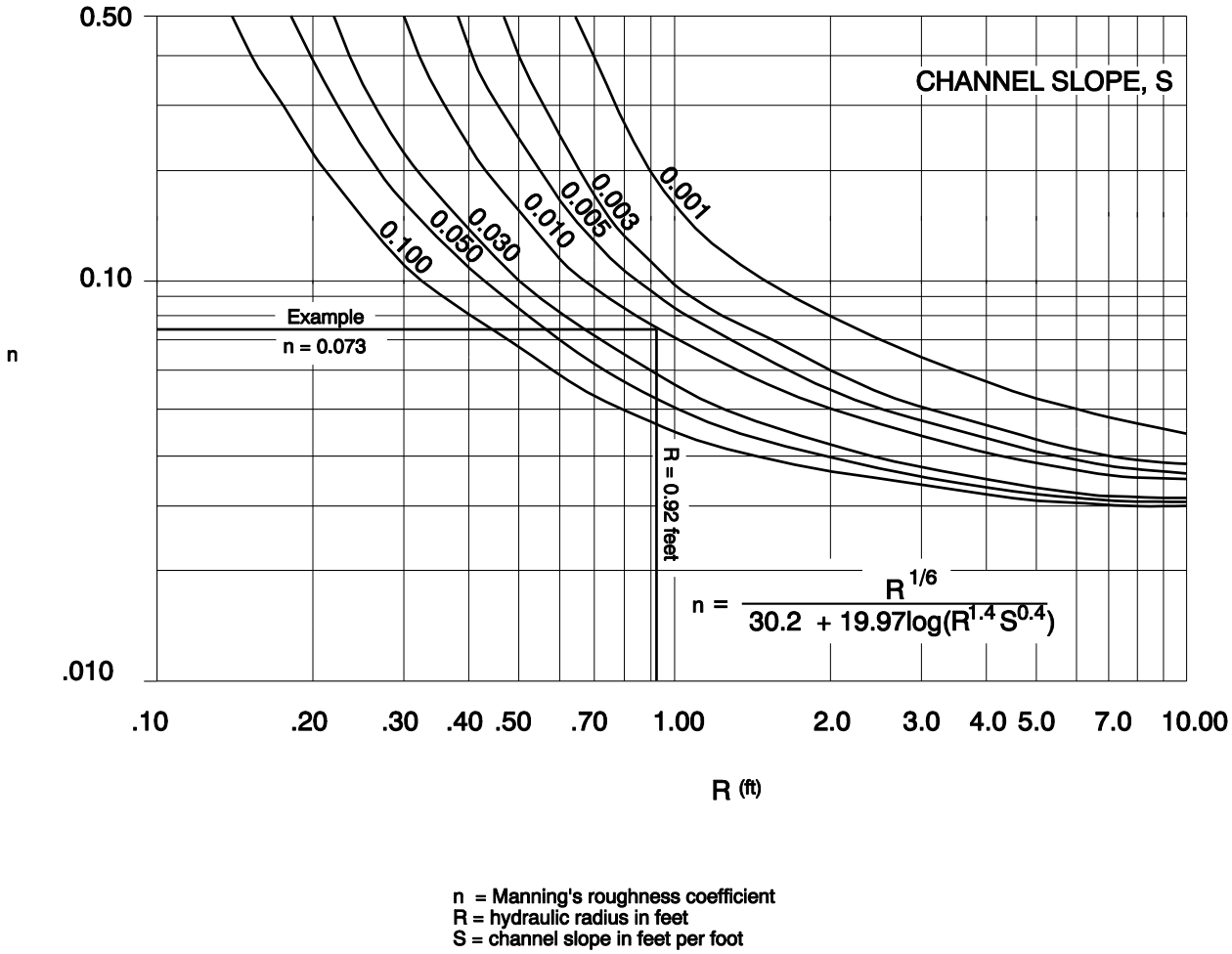
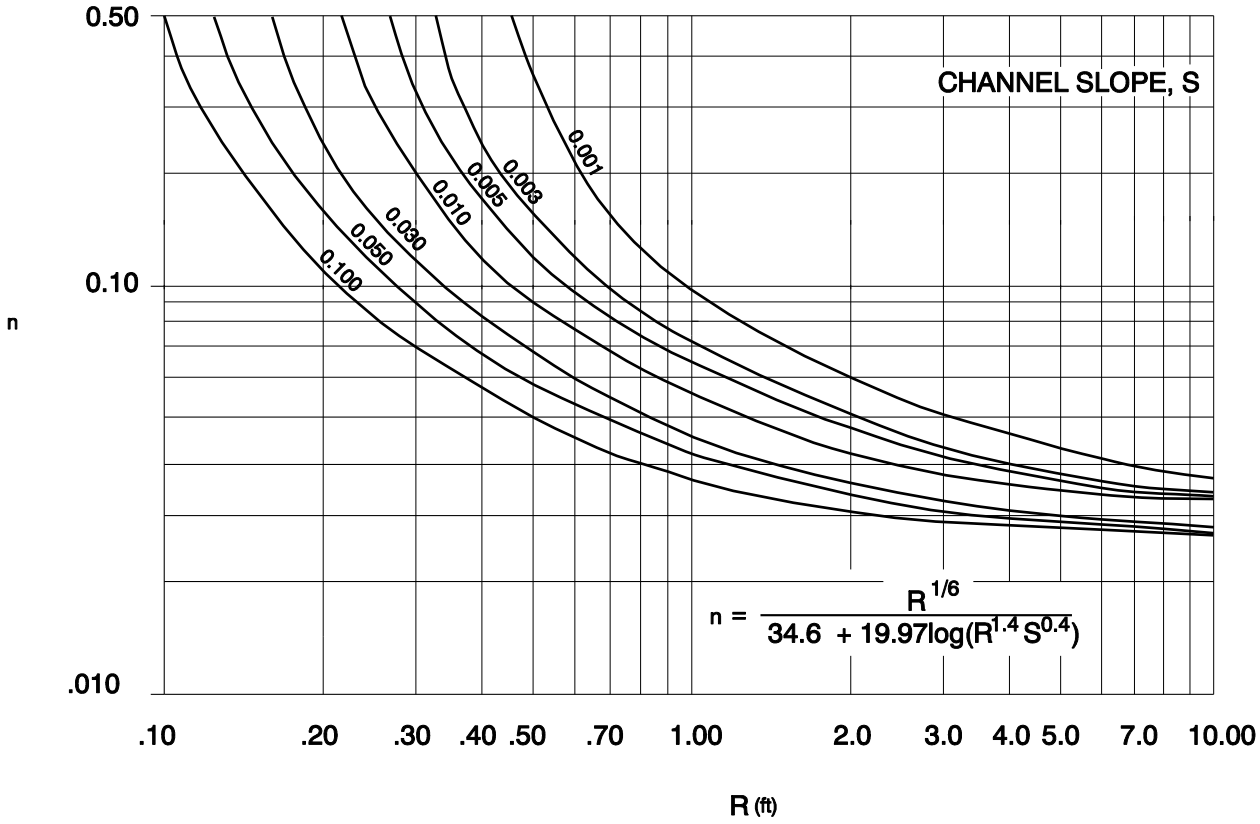
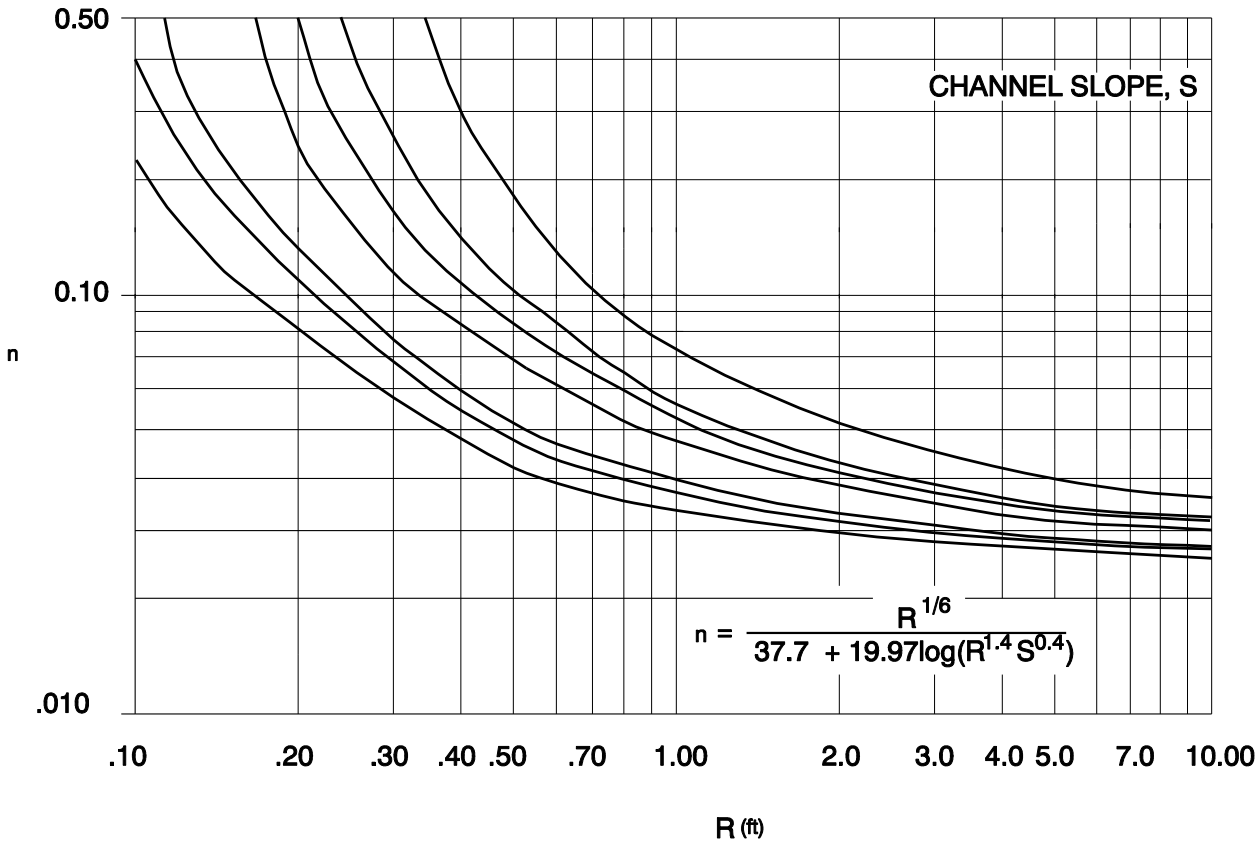



**Figure 8-26 Manning's n Versus Hydraulic Radius, R, for Class C Vegetation**




n = Manning's roughness coefficient
R = hydraulic radius in feet
S = channel slope in feet per foot

**Figure 8-27 Manning's n Versus Hydraulic Radius, R, for Class D Vegetation**




n = Manning's roughness coefficient
R = hydraulic radius in feet
S = channel slope in feet per foot

**Figure 8-28 Manning's n Versus Hydraulic Radius, R, for Class E Vegetation**

### 8.15.2.1 Example Problem - Artificial Channel Hydraulics

The hydraulic characteristics of a trapezoidal channel with two different linings are determined in this example. The linings are:

- ODOT Class 50 loose riprap, and
- a good stand of mowed grass, 6 inches tall after mowing and 10 inches tall before mowing.

Given:

S = channel slope = 0.01 foot per foot
B = channel bottom width = 2.6 feet
Z = 3 for a 1V : 3H side slope
y = maximum allowable flow depth = 1.6 feet
n = 0.070 (see Appendix A Table 3)

Find:

Q = channel capacity
V = average flow velocity

Solution 1: Riprap Lining

**Step 1 -** Calculate and determine input variables for Manning's and continuity equations (Equations 8-12 and 8-11). Geometric formulae for trapezoidal channels are shown in Figure 8-22. Input to Manning's equation is:

$$A = By + Zy^2 = (2.6)(1.6) + (3)(1.6^2) = 12 \text{ square feet}$$

$$P = B + 2y(Z^2 + 1)^{1/2} = 2.6 + (2)(1.6)[(3^2 + 1)^{1/2}] = 13 \text{ feet}$$

$$R = \frac{A}{P} = \frac{12}{13} = 0.92 \text{ feet}$$

**Step 2 -** Calculate average velocity based on Manning's equation.

$$V = \left(\frac{1.486}{n}\right) R^{2/3} S^{1/2} = \left(\frac{1.486}{0.07}\right) (0.92)^{2/3} (0.01)^{1/2} = 2.0 \text{ feet per second} \leftarrow \text{Answer}$$

**Step 3 -** Discharge is calculated using continuity equation as follows:

$$Q = (V)(A) = (2.0)(12) = 24 \text{ cubic feet per second} \leftarrow \text{Answer}$$

Solution 2: Grass Lining

**Step 1 -** Determine channel roughness (n). Channel roughness is related to retardance of the grass lining. Maintenance forces say the grass will not be mowed during the wet season so hydraulic properties will be based on the "before mowing" grass height of 10 inches. Based on Table 8-2, this grass is in Retardance Class C. S = 0.01 foot per foot, R = 0.92 foot, and A = 12 square feet (see Solution 1). As shown in Figure 8-26, n = 0.073.

**Step 2 -** The nomograph in Figure 8-23 will be used to determine discharge. An example similar to this problem is shown on the Figure.

$$\frac{y}{B} = \frac{1.6}{2.6} = 0.61$$

From Figure 8 - 23, for $\frac{y}{B} = 0.61$ and $Z = 3$:

$$1.35 = \frac{Qn}{B^{8/3}\, S^{1/2}}$$

$$\therefore \quad Q = \frac{(1.35)\left(B^{8/3}\right)\left(S^{1/2}\right)}{n}$$

$$Q = \frac{(1.35)\left(2.6^{8/3}\right)\left(0.01^{1/2}\right)}{0.073} = 24 \text{ cubic feet per second} \leftarrow \text{Answer}$$

**Step 3 -** The continuity equation will be used to determine velocity.

$$V = \frac{Q}{A} = \frac{24}{12} = 2.0 \text{ feet per second} \leftarrow \text{Answer}$$

### 8.15.3 Flow in Bends

The water surface elevation at the outside of a channel bend is higher than it is on the inside of the bend. This superelevation should be considered in channel design to assure there is adequate freeboard. The difference in water surface elevations between the outside and inside of a bend can be calculated by the following formula:

$$\Delta y = \frac{\left(V^2\right)(T)}{(g)(R_C)} \qquad \text{(Equation 8-51)}$$

Where:

- $\Delta y$ = difference in water surface elevation between the inner and outer banks of the channel in the bend in feet
- $V$ = average velocity in feet per second
- $T$ = surface width of the channel in feet

g = gravitational acceleration (32.2 feet per second squared)
$R_c$ = radius to the centerline of the channel in feet

This equation is valid for subcritical flow. The water surface elevations at the outside and inside of the bend are $\Delta y/2$ higher and lower, respectively, than the water surface at centerline.

Changes in water surface elevation around bends during supercritical flow are difficult to predict by equations. Often it is necessary to construct a model of the design, route water through it, and measure the differences in water surface elevations. The modeling results are used to predict the actual flow characteristics.

### 8.15.4 Shear Stresses on Channel Linings

Hydrodynamic forces created by water flowing in a channel cause shear stresses on the channel sides and bottom. The lining will be displaced and erosion may occur if the hydrodynamic forces create shear stresses great enough to initiate movement of the lining material. Consequently, to assure the channel will be stable, the maximum shear stress caused by the design flow must be less than the maximum shear stress the lining can resist without particle movement. Maximum shear stress in a straight trapezoidal channel occurs near the center of the channel bottom, as follows:

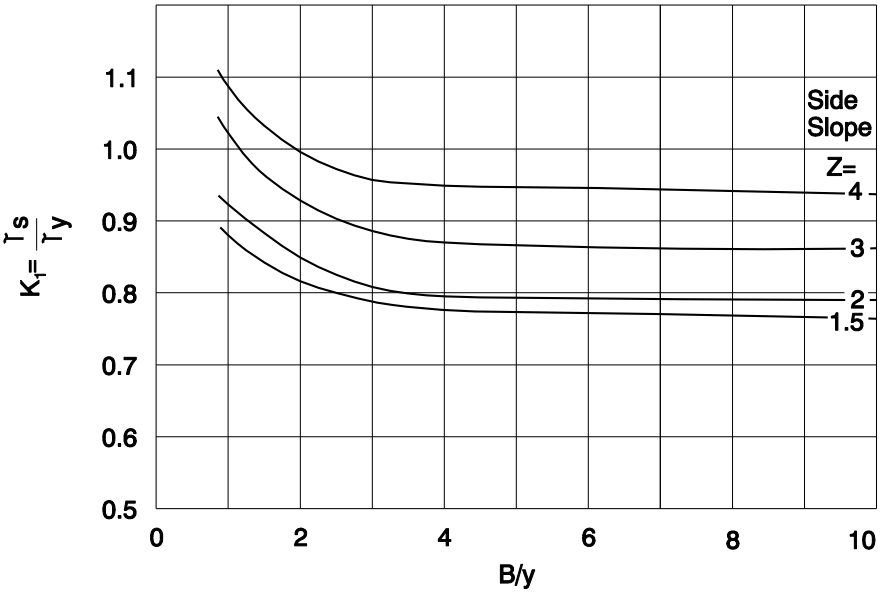
$$\tau_y = \gamma y S \qquad \text{(Equation 8-52)}$$

Where:
$\tau_y$ = maximum shear stress on channel bottom in pounds per square foot
$\gamma$ = unit weight of water (62.4 pounds per cubic foot)
y = maximum flow depth in feet
S = average bed or energy gradeline slope in feet per foot

Shear stresses on the channel sides are used to design channels with different materials on the sides and bottom. These stresses can be calculated using the shear stress ratio from Figure 8-29 and the following equation:

$$\tau_s = K_1 \tau_y \qquad \text{(Equation 8-53)}$$

Where:
$\tau_s$ = maximum shear stress on channel side in pounds per square foot
$K_1$ = ratio of maximum stress on channel side to maximum stress on channel bottom, as determined from Figure 8-29
$\tau_y$ = maximum shear stress on channel bottom, as determined from Equation 8-52




$\tau_s$ = maximum shear stress on channel sides in pounds per square foot

$\tau_y$ = maximum shear stress on channel bottom in pounds per square foot

$K_1$ = factor

B = channel bottom width in feet

y = depth of flow in feet

Z = channel side slope as shown

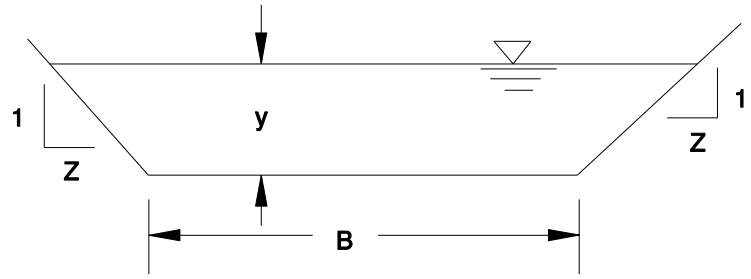



**Figure 8-29 Ratio of Channel Side Shear Stress to Bottom Shear Stress, $K_1$**

Flow in a channel with a bend creates a higher shear stress on the channel lining than the same flow in a straight channel. Areas of higher shear stress are shown in Figure 8-30. The increased shear stress persists downstream from the bend a distance $L_p$ as shown in the figure, and $L_p$ can be calculated as follows:

$$L_p = \frac{0.604\, R^{7/6}}{n_b} \qquad \text{(Equation 8-54)}$$

Where:

$L_p$ = length of increased shear stress downstream from the point of tangency (P.T) in feet
R = hydraulic radius in feet
$n_b$ = Manning's roughness of the channel bend

The initial upstream point on the bend where shear stresses start to increase is difficult to estimate. Often it is assumed the point of tangency is the upstream limit of the area affected by increased scour. This is the shown on Figure 8-30.

The maximum shear stress due to the bend is related to the ratio of the radius of the bend to the bottom width of the channel, as shown in Figure 8-31. The maximum shear stress is calculated as follows:

$$\tau_b = K_b\, \tau_y \qquad \text{(Equation 8-55)}$$

Where:

$\tau_b$ = maximum shear stress due to bend in pounds per square foot
$K_b$ = function of $R_c$ and B as shown in Figure 8-31
$\tau_y$ = maximum shear stress on channel bottom in pounds per square foot as calculated by Equation 8-52
$R_c$ = radius of bend to the centerline of the channel in feet
B = bottom width of channel in feet

*Note: In almost all situations, the greatest shear stresses are on the channel bottom rather than the channel sides, and $\tau_y$ more than $\tau_s$. In some instances the greatest shear stresses are on the channel sides and , $\tau_s$ more than $\tau_y$. In these cases, $\tau_s$ should be used in Equation 8-55 instead of $\tau_y$.*

### 8.15.4.1 Example Problem - Shear Stress in Straight and Bent Channels

Lining shear stresses need to be calculated for a prismatic trapezoidal channel with a straight section and a 90 degree bend having a centerline radius of 14.8 feet. The lining is a good stand of grass 6 inches (mowed) to 10 inches (unmowed) tall. This is the grass lined channel for the example problem in Subsection 8.15.2.1.

Given:

$\gamma$ = unit weight of water (62.4 pounds per cubic foot)
y = maximum flow depth = 2.6 feet
S = channel slope = 0.01 foot per foot
$R_C$ = radius of bend to the centerline of the channel = 14.8 feet
B = channel bottom width = 2.6 feet

Need:

$\tau_y$ = maximum shear stress on channel bottom in pounds per square foot
$\tau_b$ = maximum shear stress due to bend in pounds per square foot

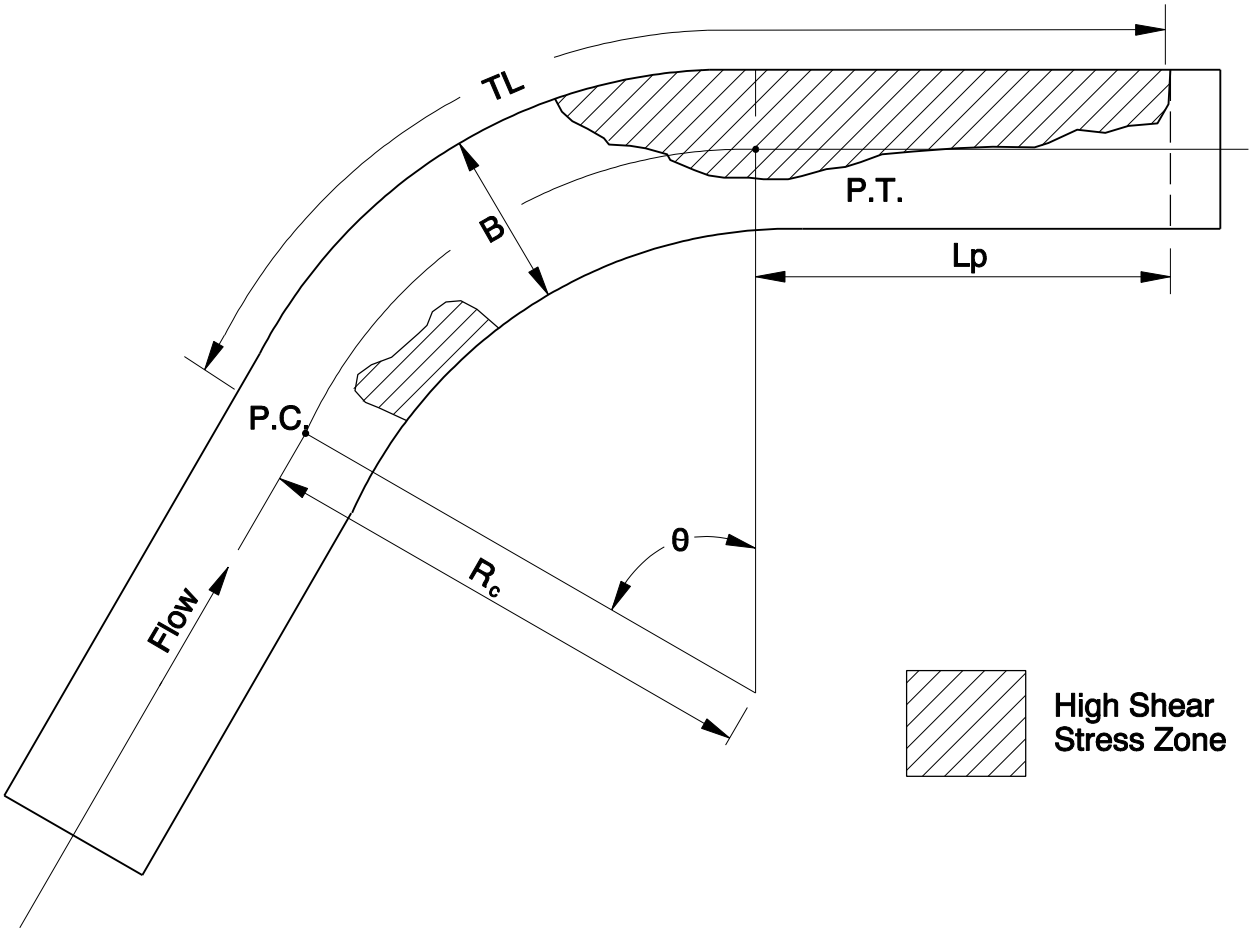
**Step 1 -** Calculate maximum shear stress on channel bottom in straight section using Equation 8-52

$\tau_y = \gamma yS = (62.4)(2.6)(0.01) = 1.6$ pounds per square foot ← Answer

**Step 2 -** Compute maximum shear stress in the bend using Figures 8-30 and 8-31 with Equation 8-55.

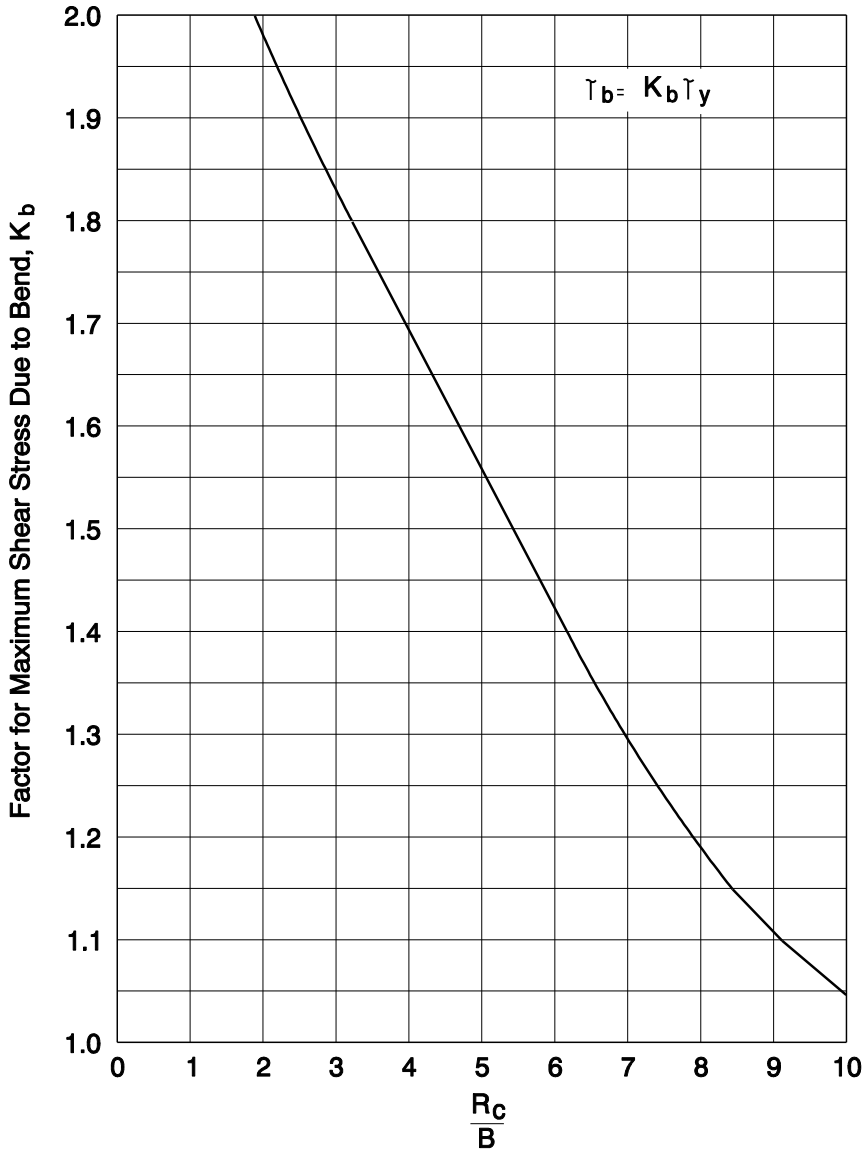
$$\frac{R_C}{B} = \frac{14.8}{2.6} = 5.7 \quad \text{From Figure } 8\text{-}31,\ K_b = 1.45$$

$\tau_b = K_b\ \tau_y = (1.45)(1.6) = 2.3$ pounds per square foot ← Answer




TL = Total channel length subject to increased shear
Lp = Length of increased shear stress downstream from the point of tangency in feet
P.C. = Point of curvature
P.T. = Point of tangency
Rc = Radius of bend in feet
B = Bottom width of channel in feet
θ = Bend angle

**Figure 8-30 Shear Stress Distribution in Channel Bend**




$\tau_b$ = maximum shear stress due to channel bend in pounds per square foot

$\tau_y$ = maximum shear stress on bottom of straight channel in pounds per square foot

$K_b$ = factor ($\tau b/\tau y$) to account for increased shear in bend

$R_c$ = radius of curvature of channel centerline in feet

B = channel bottom width in feet

**Figure 8-31 $K_b$ Factor for Maximum Shear Stress Due to Channel Bend**

### 8.15.5 Permissible Shear Stresses on Channel Linings

Sometimes a channel does not have a lining and the flow contacts bare soil. This is acceptable if some erosion can be tolerated and the flow velocities are low. The permissible shear stresses for non-cohesive soils are related to the mean diameter of the soil particles, as shown in Figure 8-32. The permissible shear stresses of cohesive soils are related to the Plasticity Index (P.I.), as shown in Figure 8-33. Soil particle size and plasticity information is available from soil tests. These properties can be estimated for native soils using the Engineering Index Property tables in the applicable National Resources Conservation Service (NRCS) Soil Survey. These surveys are available for most counties and populated areas. They are published by the NRCS and available in many libraries. They are also accessible from the NRCS website: http://www.or.nrcs.usda.gov/pnw_soil/index.html

Temporary channel linings are typically designed to withstand shear during the 2-year event, as discussed in Section 8.3.2. They are intended to provide scour and erosion protection until vegetation is established on the channel sides and bottom. Many of these temporary linings, such as straw with net, are truly temporary because they decompose quickly as the underlaying vegetation becomes established. Other linings, such as some types of plastic and fiberglass roving, are semi-permanent. They do not decompose quickly, and they continue to add strength to the channel lining for many years after the vegetation is established.

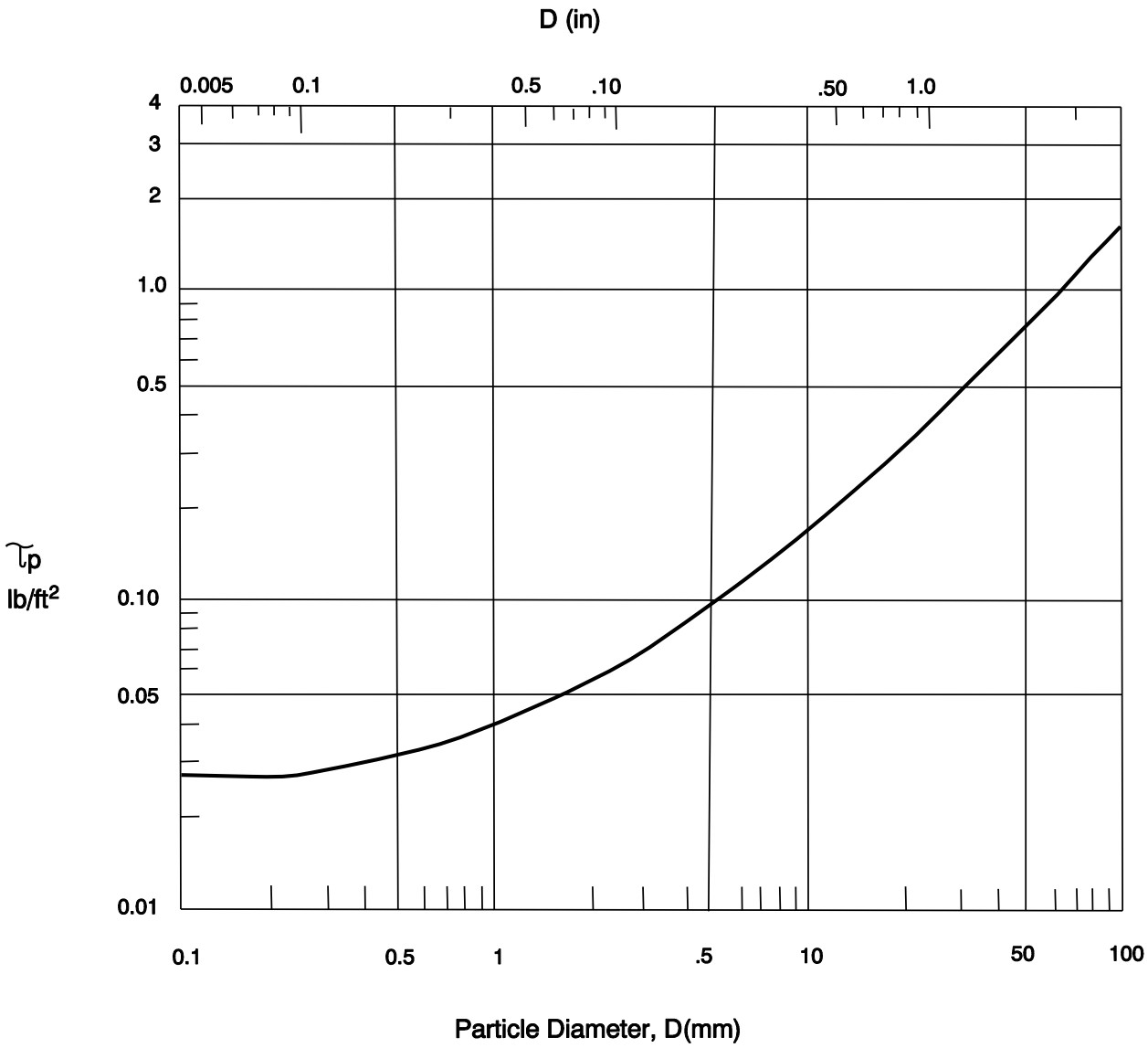



**Figure 8-32 Permissible Shear Stress for Non-Cohesive Soils**

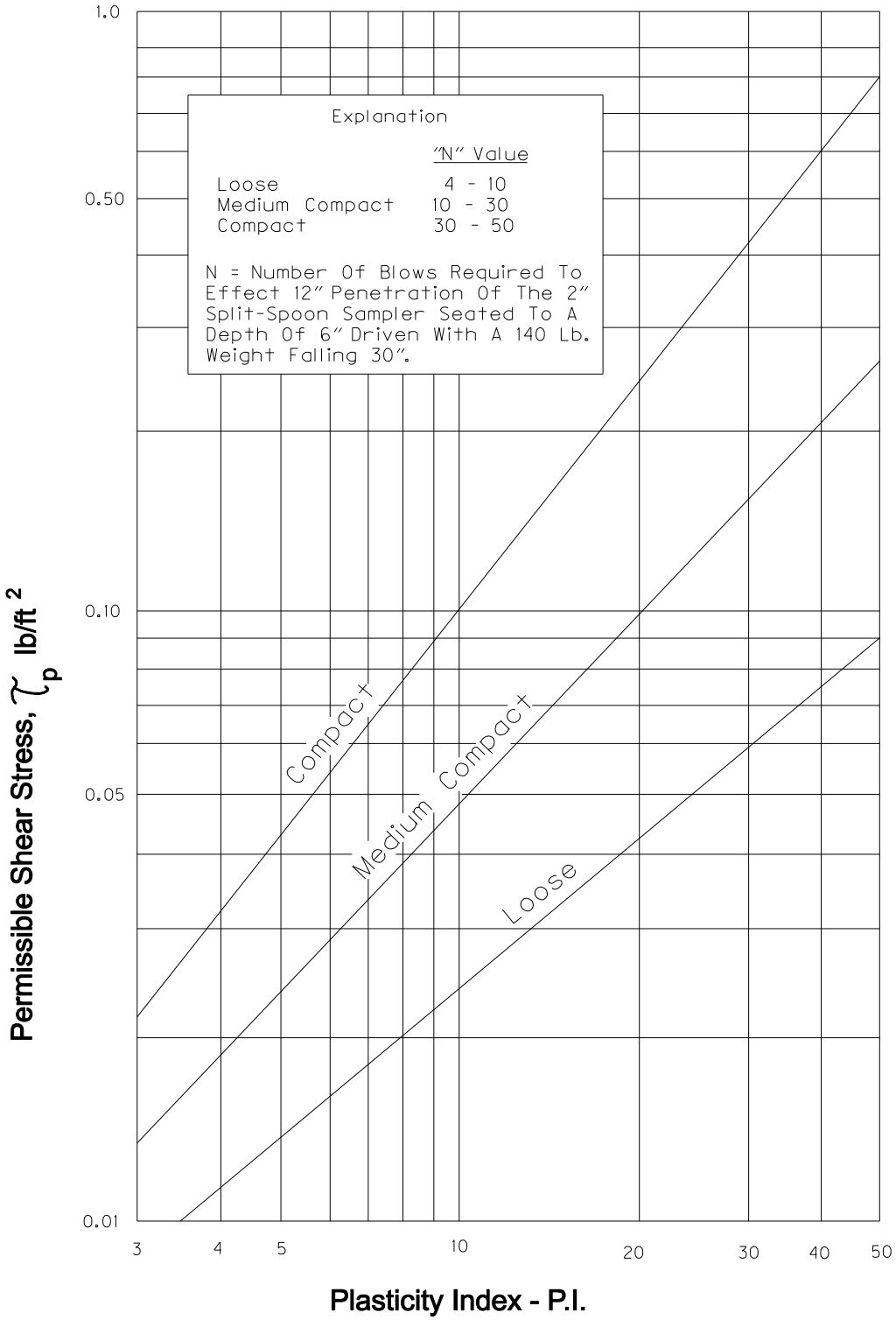



**Figure 8-33 Permissible Shear Stress for Cohesive Soils**

A more durable temporary lining may be desirable in some circumstances to provide erosion

protection if the vegetal lining fails or does not become fully established. Examples are a location where a scour resistant vegetation layer may not be fully established for some time, or a critical installation where a lining failure could be expensive, endanger public safety, or harm adjacent property.

Permanent channels are typically lined with vegetation or rock riprap. A lining is selected which will withstand the design event, and different lining materials have varying abilities to resist hydraulic shear stresses. The maximum permissible unit shear stresses for many temporary and permanent linings are summarized in Table 8-3.

The shear stresses a grass lining can withstand are related to the retardance class of the grasses. Taller grasses that provide more retardance also withstand greater shear stresses. The retardance classes of cut and uncut grasses of various heights are listed in Table 8-2.

Riprap channel linings should use angular rock produced to ODOT standards. Channel banks as steep as 1-1/2H: 1 V are allowed if the underlying soils will be stable, unless other design standards require flatter side slopes. The riprap or large stone size needed to provide stability on the channel bottom is calculated as follows:

$$(D_{50})_{bottom} = \frac{\tau_y}{4.0} \qquad \text{(Equation 8-56)}$$

Where:

$(D_{50})_{bottom}$ = mean effective diameter of riprap on channel bottom in feet

$\tau_y$ = maximum shear stress on channel bottom using Equation 8-52 (for channel bends, use maximum shear stress on channel bottom in a bend, $\tau_b$, using Equation 8-55)

The mean effective diameters of ODOT loose riprap are:

- Class 50 = 0.59 feet
- Class 100 = 0.71 feet
- Class 200 = 1.01 feet
- Class 700 = 1.45 feet
- Class 2000 = 2.06 feet

The maximum allowable shear stresses for ODOT riprap on the channel bottom are listed in Table 8-3.

Side slope stability must also be considered for gravel or riprap linings with side slopes steeper than 3 Horizontal : 1 Vertical. This analysis is done by comparing the shear force ratio of the channel sides and bottom to the tractive force ratio of the channel sides and bottom.

**Table 8-3 Permissible Shear Stresses for Linings**

| Lining Category | Lining Type | Underlying Soil | Permissible Shear Stress in Pounds per Square Foot |
|---|---|---|---|
| Bare Soil | | Non-Cohesive | See Figure 8-33 |
| | | Cohesive | See Figure 8-34 |
| Temporary* | Woven Paper Net | | 0.15 |
| | Jute Net | | 0.45 |
| | Fiberglass Roving: | | |
| | Single | | 0.60 |
| | Double | | 0.85 |
| | Straw with Net | | 1.45 |
| | Curled Wood Mat | | 1.55 |
| | Synthetic Mat | | 2.00 |
| Vegetal | Class A Retardance | | 3.7 |
| | Class B Retardance | | 2.1 |
| | Class C Retardance | | 1.0 |
| | Class D Retardance | | 0.60 |
| | Class E Retardance | | 0.35 |
| Riprap | ODOT Class 50 | | 2.36 |
| | ODOT Class 100 | | 2.84 |
| | ODOT Class 200** | | 4.04 |
| | ODOT Class 700** | | 5.80 |
| | ODOT Class 2000** | | 8.24 |
| Other | 6-Inch Thick Gabions | *** | 35 |
| | 4-Inch Thick Geoweb | *** | 10 |
| | Soil Cement (8% cement) | *** | more than 45 |
| | Dycel w/o Grass | *** | more than 7.0 |
| | Petraflex w/o Grass | *** | more than 32 |
| | Armorflex w/o Grass | *** | 12-20 |
| | Erikamat w/3-in Asphalt | *** | 13-16 |
| | Erikamat w/1-in Asphalt | *** | less than 5 |
| | Armorflex Class 30 with Longitudinal and Lateral Cables, No Grass | *** | more than 34 |
| | Dycell 100, Longitudinal Cables, Cells Filled with Mortar | *** | less than 12 |
| | Concrete Construction Blocks w/Granular Filter Underlayer | *** | more than 20 |
| | Wedge-shaped Blocks w/Drainage Slot | *** | more than 25 |

*Some “temporary” linings become permanent when buried.
**Riprap backing required.
***Shear stress values are for linings on silty clay to silty sand (SC-SM) with AASHTO classification A-4(0).

The minimum riprap size needed to provide stability on channel sides is calculated as follows:

$$(D_{50})_{sides} = \frac{(K_1)(D_{50})_{bottom}}{K_2}$$ (Equation 8-57)

Where:

$(D_{50})_{sides}$ = minimum mean effective riprap diameter on channel sides in feet
$K_1$ = ratio of maximum shear stress on channel side to maximum shear stress channel bottom, as determined from Figure 8-29
$(D_{50})_{bottom}$ = minimum mean effective riprap diameter on channel bottom, in feet, from Equation 8-56
$K_2$ = ratio of tractive force on channel side to tractive force on channel bottom, as shown in Figure 8-34

### 8.15.6 Channel Lining Selection

Selection of an artificial channel lining involves many factors, such as hydraulic capacity, ease of maintenance, aesthetics, and most importantly, the ability of the lining to resist erosion. This subsection describes a method to select a lining to resist erosion. The channel lining should have a permissible shear stress ($\tau_p$) which is higher than the shear stresses in the channel to prevent erosion. As applicable, the lining needs to resist shear stresses at the following locations:

- maximum shear stress on channel bottom ($\tau_y$),
- maximum shear stress on channel side($\tau_s$), and
- maximum shear stress due to bends in the channel ($\tau_b$).

#### 8.15.6.1 Example Problem - Lining Selection

An ODOT Class 50 loose riprap lining is proposed for the prismatic channel with a bend. It must be verified whether or not this lining is adequate.

Given:

S = channel slope = 0.012 feet per foot
B = channel bottom width = 3 feet
Z = 2 for a 1V: 2H side slope
Q = design discharge = 18 cubic feet per second
$R_C$ = channel radius of curvature = 25 feet
n = 0.070 (Table 3 of Appendix A)
$\gamma$ = unit weight of water = 62.4 pounds per cubic foot

Find:

Minimum mean effective diameter ($D_{50}$) needed to resist displacement. $D_{50}$(minimum) must be less than 0.59 feet. This is the mean effective diameter for ODOT Class 50 loose riprap.

**Step 1 -** Determine maximum shear stress on channel bottom in straight section.

Using Figure 8-23:

$\therefore y = 0.47B = (0.47)(3) = 1.4$ feet

From Figure with $Z = 2 : \frac{y}{B} = 0.47$

From Equation 8-52:

$$\frac{(Q)(n)}{(B^{8/3})(S^{1/2})} = \frac{(18)(0.070)}{(3^{8/3})(0.012^{1/2})} = 0.61$$

$\tau_y = \gamma yS = (62.4)(1.4)(0.012) = 1.0$ pounds per square foot ←

**Step 2 -** Determine maximum shear stress on channel side in straight section.

Using Figure 8 - 29 with $Z = 2$, and $\frac{B}{y} = \frac{3}{1.4} = 2.1$:

$K_1 = 0.88$

Using Equation 8-53:

$\tau_S = K_1\tau_y = (0.88)(1.0) = 0.88$ pounds per square foot ←

**Step 3 -** Determine maximum shear stress in bend. The channel bottom shear stress is greater than the channel side shear stress. The bottom shear stress will be used in the bend shear stress calculation. Using Figures 8-30 and 8-31 with Equation 8-55:

$\frac{R_C}{B} = \frac{25}{3} = 8.3$ feet From Figure 8 - 31, $K_b = 1.6$

$\tau_b = K_b\tau_y = (1.6)(1.0) = 1.6$ pounds per square foot ←

**Step 4 -** Determine riprap size on channel bottom.

Using Equation 8-56 with $\tau_y = 1.6$ pounds per square foot (see Step 3):

$$(D_{50})_{bottom} = \frac{\tau_y}{4.0} = \frac{1.6}{4.0} = 0.40 \text{ feet} \leftarrow$$

**Step 5 -** Determine riprap size on channel sides.

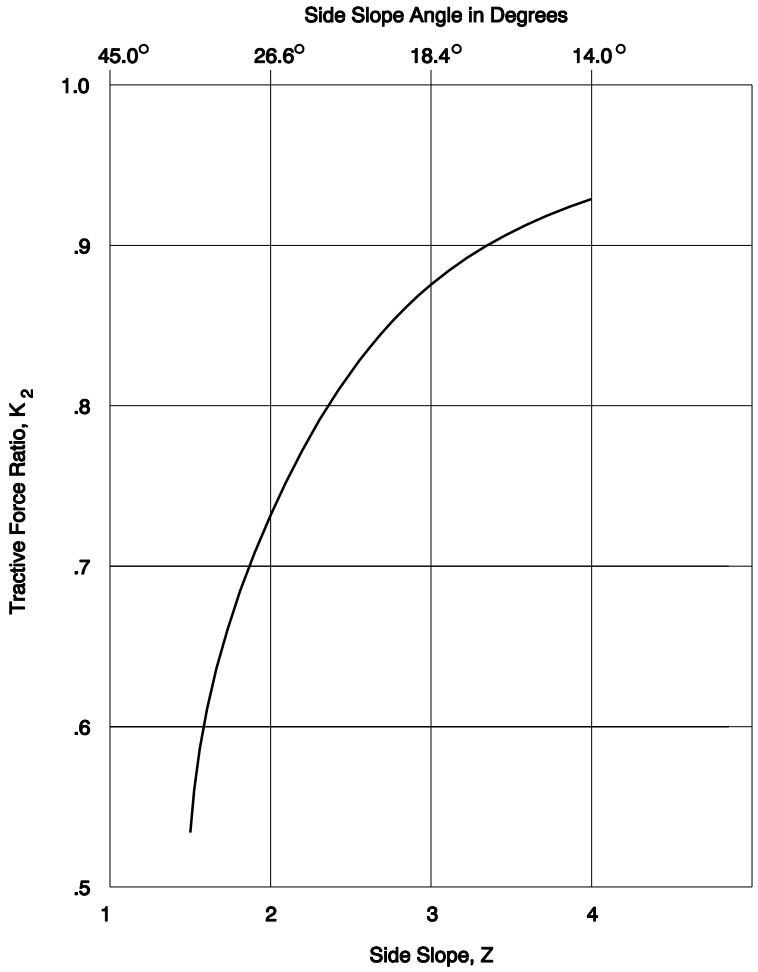
$$\frac{B}{y} = \frac{3}{1.4} = 2.1 \text{ and } Z = 2. \text{ Using Figure } 8\text{-}29: \; K_1 = 0.85$$

Using Figure 8-34 with Z = 2: $K_2 = 0.73$

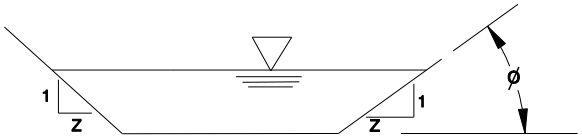
Using Equation 8-57 with the channel bottom shear stress in the bend:

$$(D_{50})_{sides} = \frac{(K_1)\left[(D_{50})_{bottom}\right]}{K_2} = \frac{(0.85)(0.40)}{0.73} = 0.47 \text{ feet} \leftarrow \text{Answer}$$

The mean effective diameter needed to resist displacement is 0.47 feet. This is less than the 0.59 foot mean effective diameter of ODOT Class 50 loose riprap. Therefore, Class 50 riprap is verified to be adequate.




NOTE:
1) Chart valid for riprap that meets ODOT Specifications (angle of repose = 41°)
2) $K_2$ for Z = 1-1/2 is 0.53




$$K_2 = \sqrt{1 - \frac{\sin^2 \varnothing}{\sin^2 \Theta}}$$

$K_2$ = ratio of tractive force on channel side to tractive force on channel bottom
$\varnothing$ = angle of side slope as shown in degrees
$\Theta$ = angle of repose in degrees (See Note 1)

**Figure 8-34 Tractive Force Ratios for Channel Side Slopes**

## 8.16 Changes in Channel Cross-Section and Slope

Changes in cross-section between upstream and downstream channels, where the slope does not vary, are often called transition sections. Changes in channel slope, or gradient, between upstream and downstream channels are often called slope or grade breaks. In some cases a channel can have a transition section and a slope break where both the cross-section geometry and slope change.

The hydraulic characteristics of flow, such as velocity and depth, are related to the slope and cross-section of the channel. As a result, there is always a change in flow characteristics at transition sections and slope breaks. Sudden changes in flow depth and velocity may occur, and these changes may be accompanied by turbulence. The channel design may need to consider the effects of turbulence in these areas. Extra freeboard may be needed to keep the water in the channel, and additional erosion protection may be needed to prevent scour.

Flow transitions are detected by determining the flow regimes in the upstream and downstream channel reaches, as follows:

**Step 1 -** Determine normal depths in the upstream and downstream channels using single-section analyses.

**Step 2 -** Calculate the Froude numbers for the channels to determine the flow regimes, either supercritical or subcritical.

**Step 3 -** Compare flow regimes. A hydraulic jump or drop will occur, and significant turbulence may result if the upstream and downstream Froude numbers are in different regimes. Lesser amounts of turbulence will occur if the upstream and downstream Froude numbers differ and are in the same regime.

Hydraulic jumps and drops can be avoided in some cases by designing the upstream channels and downstream channels to have Froude numbers as similar as possible and both within the same regime. Significant turbulence and resulting contraction or expansion head losses may be prevented in many cases by avoiding sudden transitions from one channel configuration to another. Limited experimental data suggests that a contraction with 30 degrees or less taper (measured between tapered face and stream centerline) is significantly more efficient than an abrupt (90 degree) transition. The same experimental data shows that an expansion with 2:1, 3:1 or 4:1 tapers are much more efficient than an abrupt transition, and the 4:1 taper is significantly more efficient than the 2:1 taper. (A 4:1 taper, for example, would occur when both sides of a 20 foot wide channel contract into a 10 foot wide channel within a 20 foot long transition section.)

Water surface profiles through simple transitions where the cross-section geometry changes and the channel slope is constant can be approximated by hand calculations using the energy

equations. This method is best suited for transitions where expansion and contraction losses predominate and boundary friction losses are minor. These calculations are described in detail in the FHWA Hydraulic Design Circular No 14 “Hydraulic Design of Energy Dissipators for Culverts and Channels.” This publication is available from the FHWA website http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=13
This publication also discusses other aspects of transition section design, such as minimizing standing waves and scour and erosion protection.

Water surface profiles through long and gradual transitions must consider boundary friction losses. These sections, as well as more complex situations where there are channel slope breaks, are typically analyzed using a step-backwater computer program. Several programs are discussed previously in this chapter. Transition sections and slope breaks are discussed in many texts on hydraulics such as Ven Te Chow's "Open-Channel Hydraulics."

## 8.17 Channel Changes

Channel changes are sometimes required to reduce flood hazards to highways or to relocate channels displaced by the highway. A channel change should be avoided when possible without adding unreasonable costs to the project. Alternate solutions which minimize the disruption of the natural channel should be given thorough consideration.

Step-backwater modeling using computer programs such as the FHWA’s WSPRO or the U.S. Corps of Engineers HEC-RAS is recommended for channel changes. Both the existing and proposed channels should be modeled. These analyses provide detailed information about velocities, water surface elevations, and energy, and they are a convenient means to compare the hydraulic aspects of channel modifications. The hydraulic modeling should include all proposed biological enhancements, such as fish rocks, woody debris, and vegetation.

Channel changes may have impacts which could damage fish habitat, reduce the quality of the stream side environment, and increase flow velocities as a result of reductions in channel roughness and an increase in channel slope. Disrupting channel stability may cause erosion or deposition problems after construction. These problems may occur for considerable distances upstream and downstream from the channel change. Other impacts from channel changes include the removal of vegetation from stream banks leaving raw cut-slopes that require riprap protection or concrete linings. These impacts usually reduce the aesthetics of a natural channel and may increase water temperature and flow velocities.

The following guidelines should be used when designing a channel change:

1. Design the channel to be stable under design flow conditions. A riprap blanket will usually provide sufficient protection. Generally riprap protection will only be needed along the

outside bends of curves and along any highway embankments which form a channel bank. A sinuous channel typically requires more protection to be stable than does a straight channel.

2. Contact the appropriate regulatory agencies such as the Oregon Department of Fish and Wildlife, National Marine Fisheries Service, Corps of Engineers, etc, early in the design. These agencies may recommend the appropriate enhancement features necessary to mitigate the undesirable effects of the channel change. Typical enhancement features are replanting streamside vegetation, placing fish rocks, and including vegetation and woody debris in the bank stabilization.

3. Design a channel with curvature, and when feasible, attempt to match the meander pattern, stage-discharge relationship, slope, and cross-section of the existing channel. Field surveys are required to determine the cross-section of the existing channel and the slope of the natural stream. This survey data is also used to calculate the stage-discharge relationship for the existing channel.

4. The channel change should maintain the energy conditions of the existing channel during the design event. The energy gradeline (EGL) and hydraulic gradeline (HGL) of the existing and realigned channels should be plotted and compared. The HGL and EGL of the original and realigned channels should match as closely as possible. The HGL shows the potential energy of the stream above a known datum plane and usually can be considered coincident with the water surface elevation at any given section. The EGL shows the total available energy of a stream above a known datum plane and is the sum of the potential energy and the kinetic energy.

   As an example, if a channel change were designed for a small creek having a 500 cubic foot per second design flow, numerous trapezoidal sections could be found which convey this discharge. The proper size channel would maintain about the same energy relationships as the existing channel under design conditions.

### 8.17.1 Channel Change Example

This example illustrates a channel change designed by hand calculations. It is intended to illustrate the procedure. This channel change would normally be designed using a step-backwater hydraulic analysis. The modified channel would normally include extensive biological components. These are omitted from the example for clarity.

A highway project requires replacing a 225-foot long reach of DeLores Creek with a 230-foot long channel change and also requires an 8-foot extension to each end of the existing 7-foot span by 4-foot rise reinforced concrete box culvert (RCBC). The proposed channel change and RCBC lengthening are shown on Figure 8-35. The existing creek bottom (thalweg) and a water surface

profile 1000 feet upstream and 1000 feet downstream are shown on Figure 8-36. The water surface profile represents the 2 February 1989 flood. The largest known flood at the project site occurred during 22 December 1964. The December 1964 highwater elevation of 2947.47 feet (NGVD) was determined from information provided by Mr. John Oldtimer, telephone number (541) 468-1234, a 50-year resident of the area. The channel Manning's n value is estimated to be 0.04.

The channel change will be designed to convey the December 1964 flood and maintain the energy conditions of the existing channel. The peak discharge and average velocity of the 1964 flood will be estimated. The following procedure is used to design the channel change.

**Step 1 -** Draw a straight line through the channel bottom points at stations "D" 11 and "D" 26 and identify the elevations of these two points on this line as shown on Figure 8-37. This line should be approximately parallel to the water surface profile also shown on Figure 8-37. (It is recommended that the water surface profile be used to determine the slope for highly irregular channel bottom profiles.) The estimated mean slope is shown below:

$$\text{Mean Slope} = \frac{(2949.9 - 2941.5)}{(26 - 11)(100)} = 0.0056 \text{ feet per foot}$$

**Step 2 -** Draw a straight line through the December 1964 highwater mark which is parallel to the mean slope as shown on Figure 8-37.

**Step 3 -** Locate and identify on Figure 8-37 the downstream and upstream controls which will be used to determine the mean channel cross-section.

**Step 4 -** Sketch the mean cross-section as shown on Figure 8-37. The mean cross-section represents a cross-section at the control sections. This is also the recommended cross-section for the channel change. Energy conditions of the existing channel will be maintained since the channel will provide the same cross-section and slope as the existing channel.

**Step 5 -** Plot the channel change section on the plan view, as shown on Figures 8-35. Plot the typical channel change cross-section, as shown on Figure 8-38.

The estimated 1964 peak discharge and average velocity are calculated by Manning's equation for discharge and the continuity equation as shown below. These equations are presented in Section 8.6.

$$Q = \frac{1.486}{n}(A)(R^{2/3})(S^{1/2})$$

$$A = \left(\frac{7 + 7 + 3.4 + 3.4}{2}\right)(1.7) = 17.68 \text{ Square feet}$$

$$P = 7 + 2(1.7^2 + 3.4^2)^{1/2} = 14.60 \text{ feet}$$

$$R = \frac{A}{P} = \frac{17.68}{14.60} = 1.21 \text{ feet}$$

$$Q = \left(\frac{1.486}{0.04}\right)(17.68)(1.21^{2/3})(0.0056^{1/2}) = 56 \text{ cubic feet per second}$$

$$V = \frac{Q}{A} = \frac{56}{17.68} = 3.2 \text{ feet per second}$$

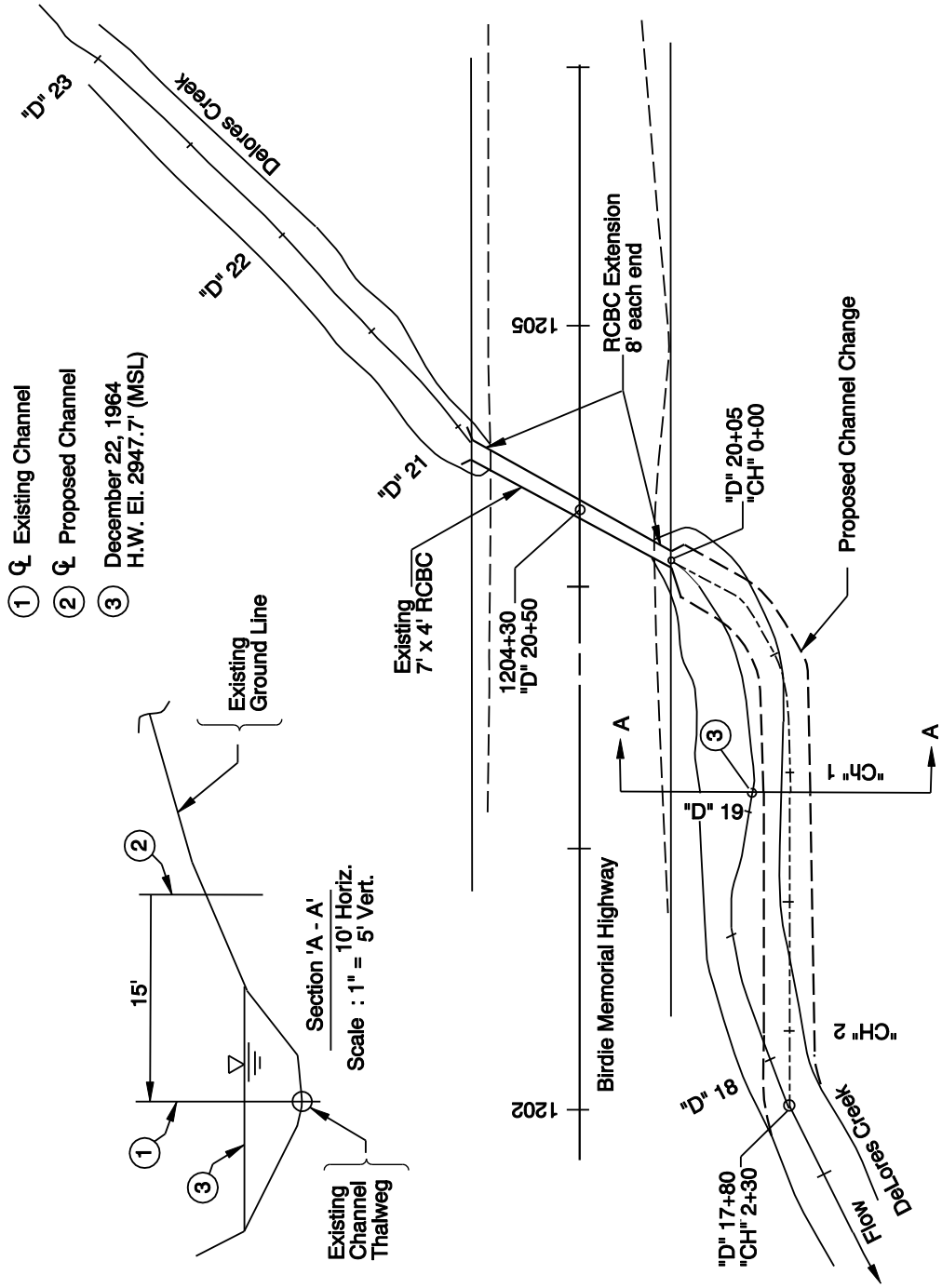



**Figure 8-35 Plan View**

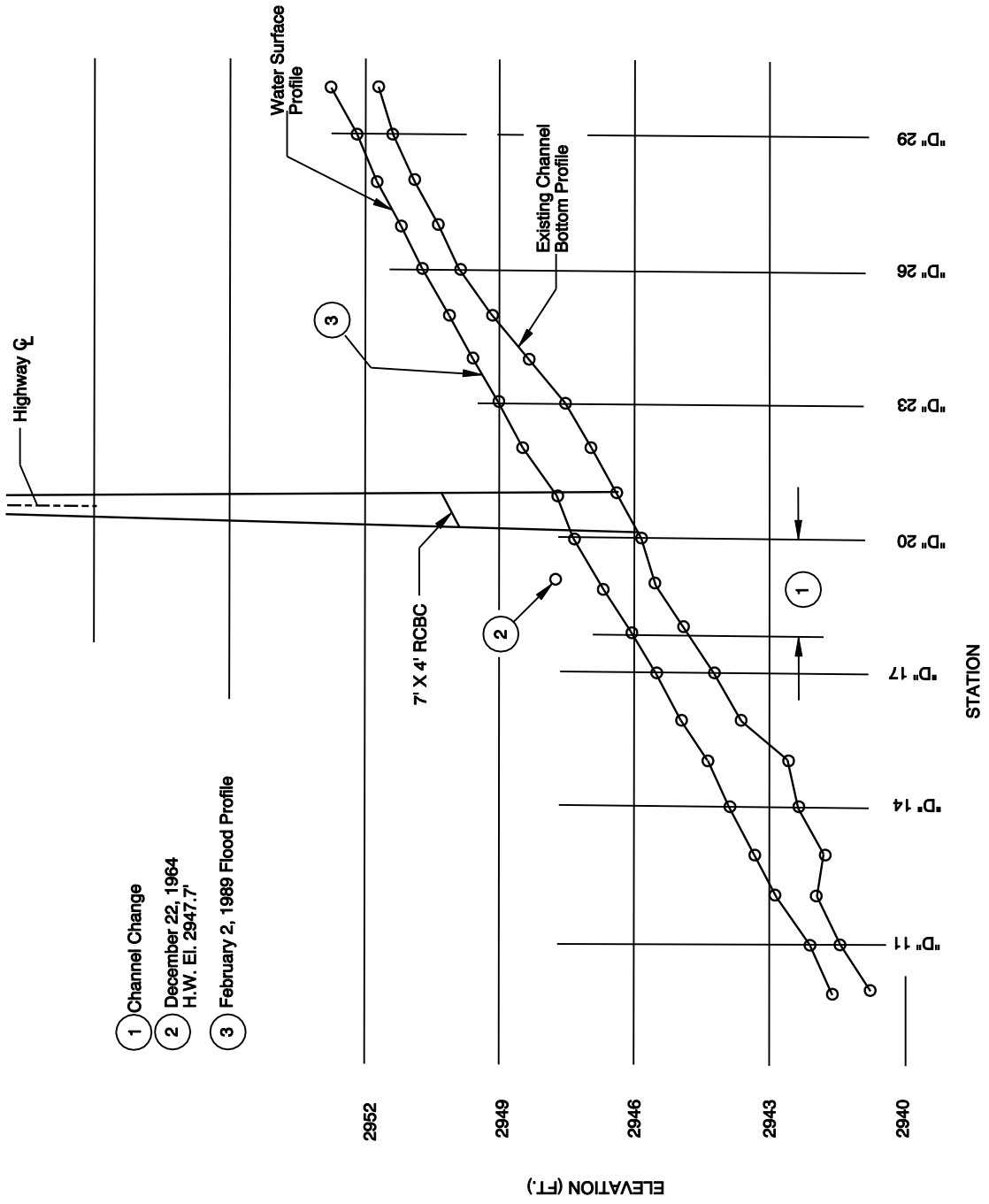



**Figure 8-36 Existing Channel Profile**

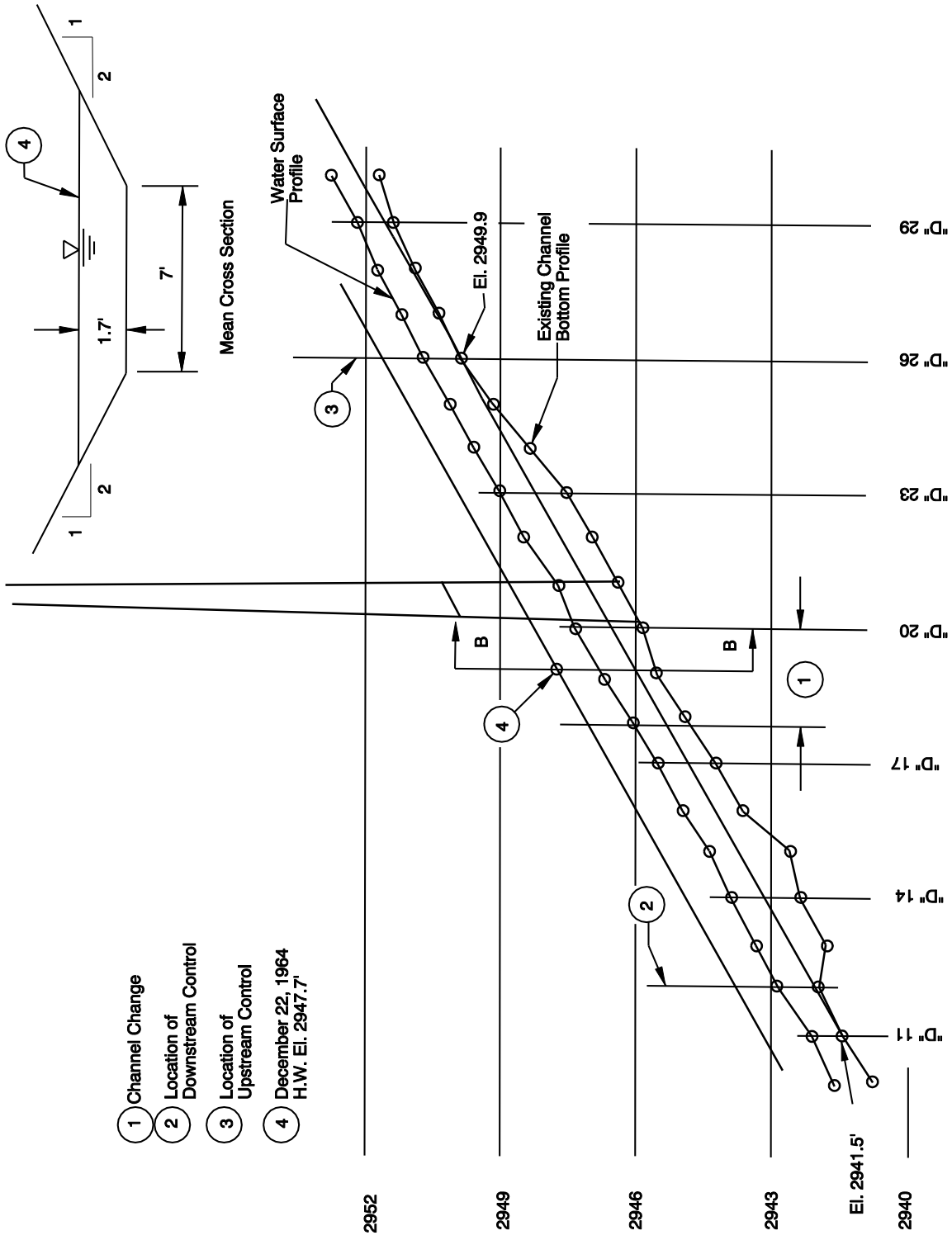



**Figure 8-37 Proposed Channel Profile**

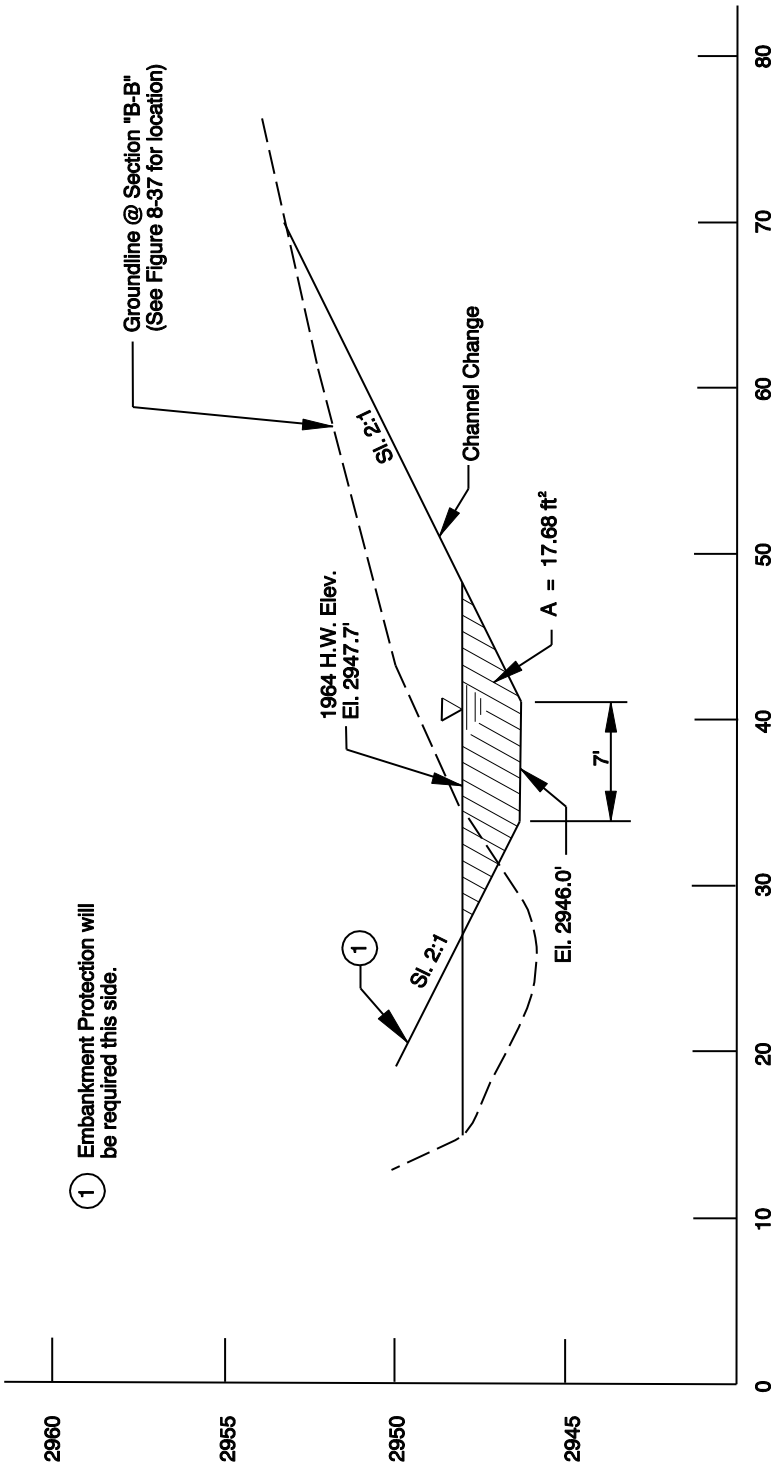



**Figure 8-38 Channel Change Cross-Section**

## 8.18 Design Procedures

The design procedure for all types of channels has common elements as well as substantial differences. This section will outline a process for assessing a natural stream channel and a more specific design procedure for roadside channels.

### 8.18.1 Natural Stream Channel Procedure

A stream channel analysis is often part of a culvert or bridge hydraulic design. In general, the objective is to convey the water along or under the highway in an environmentally acceptable manner that will not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The detail of the study should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining flood plain. The following step-by-step procedure outlines many of the steps in a typical modification of a natural channel or larger artificial channel.

**Step 1 -** Assemble available data (see **Chapter 6**) and design criteria (see **Chapter 3** and Subsection 8.3.1).

**Step 2 -** Determine the level of assessment. Information may be required to:

- show compliance with floodway and floodplain land use regulations,
- apply for permits,
- prepare a Hydraulic Report if a significant structure such as a bridge or culvert is involved,
- assure regulatory agencies that fish passage is provided, and
- provide needed data for designs. Generally, structures with the most potential for causing damage are studied in greater detail.

**Step 3 -** Determine type of hydraulic analysis. Examples are:

- Qualitative assessment. These are generally used for low-risk designs and preliminary estimates.
- Single-section analysis. Sometimes used for structures over prismatic channels such as irrigation canals where contraction scour does not need to be calculated.
- Step-backwater analysis. Used in most structural analyses.

**Step 4 -** Obtain additional survey information such as streambed profiles and cross-sections, elevations of flood-prone property, etc (see **Chapter 6**).

**Step 5 -** Determine hydrology (see **Chapter 7**).

**Step 6 -** Perform hydraulic analysis.

- Select representative cross-section (single-section method) or cross-sections (step-backwater method).
- Select appropriate n values (both methods) and reach lengths (step-backwater method).
- Calculate energy slope (single-section method) or starting water surface slope or elevations (step-backwater method).
- Compute stage-discharge relationship (single-section method) or profiles (step-backwater method).
- Calibrate model if historical flows and corresponding high water elevations are known.

**Step 7-** Perform stability analysis if needed. This is beyond the scope of this manual. A useful reference is the latest FHWA Hydraulic Engineering Circular Number 20 "Stream Stability at Highway Structures."

**Step 8 -** Design channel change and/or bank protection.

- Calculate and compare hydraulic characteristics of existing and modified channels.
- Design bank protection. (see **Chapter 15**)

**Step 9 -** Prepare report and file with background information (see **Chapter 4**).

### 8.18.2 Artificial Channel Procedure

An artificial channel is an open channel which typically conveys surface runoff through or from the highway right-of-way. A generalized procedure for the design of roadside and median channels follows. A discussion of design procedures including equations and examples is included in Section 8.15. Although many projects may require different tasks, many of the following design steps will be applicable.

**Step 1 -** Assemble available data (see **Chapter 6**) and design discharge recurrence intervals (see **Chapter 3** and Subsection 8.4.2).

**Step 2 -** Establish a preliminary drainage plan.

- Prepare plans and profiles of the existing and proposed channels. Include any constraints on design such as highway and road locations, right-of-way boundaries, culverts, utilities, etc. An example of a plan and profile is shown in Figure 8-39.
- Determine and plot on the plan the locations of natural basin divides and channel outfalls. Include all areas contributing runoff to the drainage system, both on and off of the highway right-of-way.

**Step 3 -** Obtain or establish typical ditch cross-section.

- Provide adequate depth in roadside ditches to drain the subbase and prevent freeze-thaw damage.
- Select channel side slopes considering design policy, safety, maintenance, economics, soil, aesthetics, and access.

**Step 4 -** Determine initial channel grades. Plot initial grades on the plan and profile. Slopes of roadside ditches in cuts are usually controlled by highway grades. Use the following guidelines when establishing initial grades.

- Provide minimum grade of 0.3% to minimize ponding and sediment accumulation, if possible.
- Identify and avoid features which may influence or restrict grade such as utilities.

**Step 5 -** Locate outfall and perform a preliminary hydrologic estimate to determine flow to outfall. (Detailed design of the outfall occurs at a later stage in the design process.)

**Step 6 -** Identify any adverse impacts on downstream properties from changes in peak flow or runoff volume. Compliance with Oregon drainage law (see **Chapter 2**) and the hydraulic capacity of the downstream receiving channel should be considered. In addition, any flow diversions should consider the water rights of affected parties. Mitigation measures may be needed to reduce impacts to an acceptable level. Mitigation measures may include:

- increasing capacity and adequately protecting the banks of the downstream channel,
- installing an energy dissipator or other control structure to reduce excessive outlet velocities,
- providing flow detention,
- diverting flow to another outfall where increased flow does not cause problems, or
- obtaining drainage easements.

**Step 7 -** Identify need for water quality treatment (see **Chapter 14**).

**Step 8 -** Size ditch to convey flow. Typically, the flow in a ditch is related to the time of concentration ($T_c$) of the runoff reaching the ditch. The $T_c$, however, is not known unless the flow time can be estimated in upstream ditches, and flow time is related to the size and lining of the upstream ditch. As a result, ditches are usually designed in segments, one after the other, proceeding downstream. Segments typically include ditches which convey approximately the same flow, and a new segment starts where there is a significant change of flow.

Typically $T_c$ and the resultant design flow are calculated for a ditch segment and the ditch is sized and lining selected. Then $T_c$ and the resultant flow are calculated for the adjacent

downstream ditch section, and those ditches are designed. The procedure is repeated segment-by-segment until the outfall is reached. Steps in the sizing of ditches follow.

- Compute the design discharge at the downstream end of the channel segment (see **Chapter 7**).
- Estimate liner type and roughness of channel during period of peak runoff. Calculate depth of flow using preliminary estimates of ditch cross-section and Manning's equations or nomograph. One or more of these changes may be needed if flow is deeper than maximum allowable flow depth including freeboard:

  - increase bottom width,
  - make channel side slopes flatter,
  - make channel slope steeper,
  - provide smoother channel lining, and/or
  - install drop inlets and a parallel storm drain pipe beneath the channel to supplement channel capacity.

**Step 9 -** Check adequacy of lining. Calculate maximum shear stress on lining including effects of bends in the channel.

- Compare maximum to permissible shear stress for all linings except riprap. Reduce maximum shear stress by one or more of the following if maximum shear stress is greater than allowable:

  - enlarging channel, or
  - changing the channel slope,
  - flattening the bank side slopes, or
  - increasing the radius of curvature if the channel has bends.

  Increase the allowable shear stress by changing the lining material if reducing the shear stress is impractical.

- Calculate $D_{50}$ needed to resist displacement for riprap linings. Compare $D_{50}$ needed to $D_{50}$ provided. Reduce the $D_{50}$ needed by the measures previously listed. Increase the $D_{50}$ of the liner if these measures are not practical.

**Step 10 -** Verify adequacy of ditch size. Verify that the depth of flow in the ditch is within allowable limits if liner roughness or ditch cross-section is changed. Adjust ditch parameters listed in Step 9 above if depth is excessive. Verify that the liner is adequate for revised ditch, if needed.

**Step 11 -** Design a temporary ditch lining, if needed. Consider whether or not a stand of grass with complete coverage can be quickly established before high flows occur. Liner

materials which quickly decompose may be adequate if a stand of grass with full coverage will grow before high flows occur. Conversely, a more permanent liner may be needed if high flows occur before the grass is established.

**Step 12 -** Check channel transitions where there are changes in cross-section, slope, discharge, or roughness. Transitions may require special design considerations, such as:

- gradual transitions to minimize turbulence, and
- adequate erosion protection and channel capacity, including freeboard. A hydraulic analysis using energy equations may be needed to determine flow depths and velocities.

**Step 13 -** Design outfall improvements, detention, and water quality treatment facilities, as needed. The ditches and liners designed in Steps 8 and 9 should be used to determine the $T_c$ and the resultant design flows.

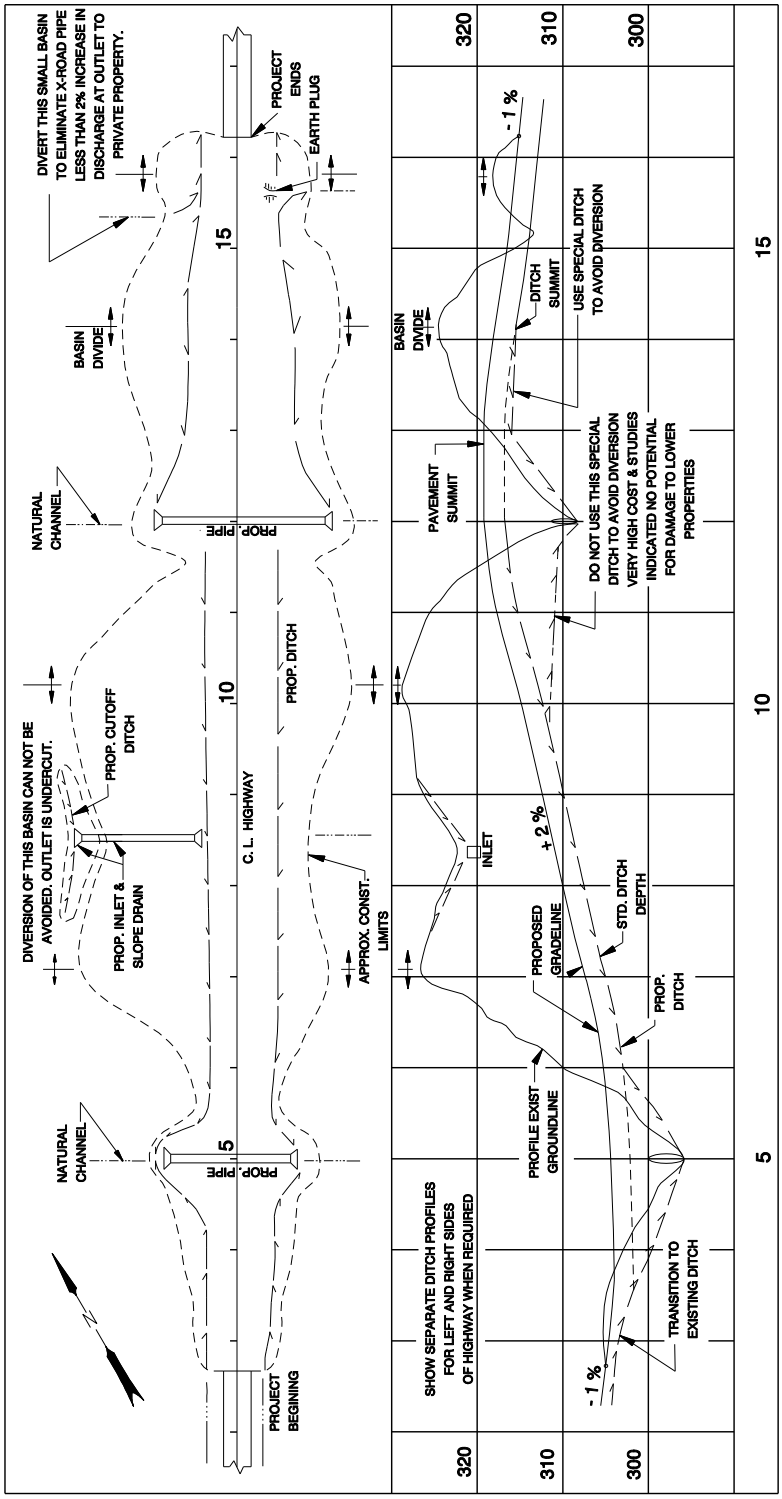



**Figure 8-39 Roadside Drainage Plan and Profile**

# APPENDIX A - HYDRAULIC ROUGHNESS (MANNING'S n) VALUES OF CONDUITS AND CHANNELS

This appendix lists Manning's roughness (n) values for various conduits and channels, as follows:

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Sources:
- Chow, Ven Te, "Open-Channel Hydraulics," 1959
- FHWA, "Design of Urban Highway Drainage, The State of the Art," 1979
- FHWA, "Hydraulic Design Series No. 3, Design Charts for Open-Channel Flow," 1961
- FHWA, "Hydraulic Engineering Circular No. 15, Design of Roadside Channels with Flexible Linings," 1988
- FHWA, "Hydraulic Engineering Circular No. 22, Urban Drainage Design Manual," 1996
- ODOT, "Memo to Designers, Helical Corrugated Pipe," 1992

TABLE 1: CONDUITS

| Conduit | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| A. Concrete or asbestos-cement pipe | 0.011 | 0.013 | 0.015 |
| B. Metal pipe or pipe-arch with annular corrugations | | | |
| 1. 2-2/3-inch x ½-inch corrugations | | | |
| a. Plain or fully coated | . . . . . | 0.024 | . . . . . |
| b. Paved invert (range represents 25 and 50 percent of circumference paved, with larger n value representing 25 percent paved) | | | |
| 1. Full flow depth | 0.018 | . . . . . | 0.021 |
| 2. Flow 80 percent of depth | 0.016 | . . . . . | 0.021 |
| 3. Flow 60 percent of depth | 0.013 | . . . . . | 0.019 |
| 2. 3-inch x 1-inch corrugations | . . . . . | 0.027 | . . . . . |
| 3. 6-inch x 2-inch corrugations | . . . . . | 0.032 | . . . . . |
| C. Smooth walled helical spiral rib pipe | 0.012 | . . . . . | 0.013 |
| D. Corrugated metal subdrain | 0.017 | 0.019 | 0.021 |
| E. Plastic pipe | | | |
| 1. Smooth | 0.011 | . . . . . | 0.015 |
| 2. Corrugated | . . . . . | 0.024 | . . . . . |
| F. Metal pipe or pipe arch with helically wound corrugations | | | |
| 1. Smaller pipes | | | |
| 12 inch | . . . . . | 0.013 | . . . . . |
| 15 inch | . . . . . | 0.014 | . . . . . |
| 18 *inch* | . . . . . | 0.015 | . . . . . |

| Conduit | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| 21 inch | . . . . . | 0.016 | . . . . . |
| 24 inch | . . . . . | 0.017 | . . . . . |
| 27 inch | . . . . . | 0.018 | . . . . . |
| 30 inch | . . . . . | 0.019 | . . . . . |
| 33 inch | . . . . . | 0.020 | . . . . . |
| 36 inch | . . . . . | 0.021 | . . . . . |
| 42 inch | . . . . . | 0.022 | . . . . . |
| 48 inch | . . . . . | 0.023 | . . . . . |
| 2. Diameters larger than 48 inches with 2-2/3-inch x 1/2-inchcorrugations | . . . . . | 0.024 | . . . . . |
| 3. Diameters larger than 48 inches with 3-inch x 1-inch corrugations | . . . . . | 0.027 | . . . . . |
| G. Wrought iron | | | |
| 1. Black | 0.012 | 0.014 | 0.015 |
| 2. Galvanized | 0.013 | 0.016 | 0.017 |
| H. Cast iron | | | |
| 1. Coated | 0.010 | 0.013 | 0.014 |
| 2. Uncoated | 0.011 | 0.014 | 0.016 |
| I. Steel pipe, welded | 0.010 | 0.012 | 0.014 |
| J. Brick | | | |
| 1. Glazed | 0.011 | 0.013 | 0.015 |
| 2. Lined with cement mortar | 0.012 | 0.015 | 0.017 |
| K. Common clay drainage tile | 0.011 | 0.013 | 0.017 |
| L. Vitrified clay sewer | 0.011 | 0.014 | 0.017 |

| Conduit | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| M. Sanitary sewer, coated with sewage slime, with bends and connections | 0.012 | 0.013 | 0.016 |
| N. Monolithic concrete | | | |
| 1. Wood forms, rough | 0.015 | 0.017 | 0.020 |
| 2. Wood forms, smooth | 0.012 | 0.014 | 0.016 |
| 3. Steel forms | 0.012 | 0.013 | 0.014 |
| O. Rubble masonry, cemented | 0.018 | 0.025 | 0.030 |
| P. Laminated treated wood | 0.015 | 0.017 | 0.020 |

**TABLE 2: GUTTERS AND PAVEMENTS**

| Channel | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| A. Concrete gutter, troweled finish * | . . . . . | 0.012 | . . . . . |
| B. Asphalt pavement | | | |
| 1. Smooth texture | . . . . . | 0.013 | . . . . . |
| 2. Rough texture | . . . . . | 0.016 | . . . . . |
| 3. ODOT Standard Curb, Low Profile Mountable Curb, Monolithic Curb and Sidewalk, and Mountable Curb | . . . . . | 0.016** | . . . . . |
| C. Concrete gutter with asphalt pavement | | | |
| 1. Smooth | . . . . . | 0.013 | . . . . . |
| 2. Rough | . . . . . | 0.015 | . . . . . |
| 3. ODOT Curb and Gutter, Mountable Curb and Gutter, and Valley Gutter | . . . . . | 0.014 | . . . . . |
| D. Concrete pavement | | | |
| 1. Float finish | . . . . . | 0.014 | . . . . . |
| 2. Broom finish | . . . . . | 0.016 | . . . . . |
| 3. ODOT Standard Curb, Low Profile Mountable Curb, Monolithic Curb and Sidewalk, and Mountable Curb | . . . . . | 0.016** | . . . . . |
| E. Brick | . . . . . | 0.016 | . . . . . |
| F. For gutters listed above with small slope, where sediment may accumulate, increase above values of n by | . . . . . | 0.002 | . . . . . |

* Flow contained within gutter.

* * The most common value used for gutters with Asphalt bottoms on ODOT project is 0.016

## TABLE 3: SMALLER ARTIFICIAL CHANNELS

Values are for artificial channels with flows of 50 cubic feet per second or less. Roughness values vary with depth, as follows:

- Minimum values are for flow depths greater than 2 feet.
- Normal values are for depths between 6 inches and 2 feet.
- Maximum values are for flow depths less than 6 inches.

| Channel | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| A. Rigid Linings | | | |
| 1. Concrete | 0.013 | 0.013 | 0.015 |
| 2. Grouted riprap | 0.028 | 0.030 | 0.040 |
| 3. Stone masonry | 0.030 | 0.032 | 0.042 |
| 4. Soil cement | 0.020 | 0.022 | 0.025 |
| 5. Asphalt | 0.016 | 0.016 | 0.018 |
| B. Unlined | | | |
| 1. Bare soil | 0.020 | 0.020 | 0.023 |
| 2. Rock cut | 0.025 | 0.035 | 0.045 |
| C. Temporary | | | |
| 1. Woven paper net | 0.015 | 0.015 | 0.016 |
| 2. Jute net | 0.019 | 0.022 | 0.028 |
| 3. Fiberglass roving | 0.019 | 0.021 | 0.028 |
| 4. Straw with net | 0.025 | 0.033 | 0.065 |
| 5. Curled wood mat | 0.028 | 0.035 | 0.066 |

| Channel | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| 6. Synthetic mat | 0.021 | 0.025 | 0.036 |
| D. Grass (See pages 8-12 through 8-16 of this chapter.) | | | |
| E. Riprap | | | |
| 1. ODOT Class 50 | 0.036 | 0.070 | 0.106 |
| 2. ODOT Class 100 | 0.039 | 0.075 | . . . . . |

*Note: Roughness values vary with depth. See previous page.*

**TABLE 4: LINED ARTIFICIAL CHANNELS**

| Channel | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| A. Concrete, with surfaces as indicated: | | | |
| 1. Formed, no finish | 0.014 | 0.017 | 0.020 |
| 2. Trowel finish | 0.011 | 0.013 | 0.015 |
| 3. Float finish | 0.013 | 0.015 | 0.016 |
| 4. Finished, with gravel on bottom | 0.015 | 0.017 | 0.020 |
| 5. Gunite, good section | 0.016 | 0.019 | 0.023 |
| 6. Gunite, wavy section | 0.018 | 0.022 | 0.025 |
| 7. On good excavated rock | 0.017 | 0.020 | . . . . . |
| 8. On irregular excavated rock | 0.022 | 0.027 | . . . . . |
| B. Concrete, bottom float finished, sides as indicated: | | | |
| 1. Dressed stone in mortar | 0.015 | 0.017 | 0.020 |
| 2. Random stone in mortar | 0.017 | 0.020 | 0.024 |
| 3. Cement rubble masonry | 0.020 | 0.025 | 0.030 |
| 4. Cement rubble masonry, plastered | 0.016 | 0.020 | 0.024 |
| 5. Dry rubble or riprap | 0.020 | 0.030 | 0.035 |
| C. Gravel bottom, sides as indicated: | | | |
| 1. Formed concrete | 0.017 | 0.020 | 0.025 |
| 2. Random stone in mortar | 0.020 | 0.023 | 0.026 |
| 3. Dry rubble or riprap | 0.023 | 0.033 | 0.036 |
| D. Glazed brick | 0.011 | 0.013 | 0.015 |
| E. Brick in cement mortar | 0.012 | 0.015 | 0.018 |

| Channel | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| F. Asphalt | | | |
| 1. Smooth | . . . . . | 0.013 | . . . . . |
| 2. Rough | . . . . . | 0.016 | . . . . . |
| G. Wood | | | |
| 1. Planed, untreated | 0.010 | 0.012 | 0.014 |
| 2. Planed, creosoted | 0.011 | 0.012 | 0.015 |
| 3. Unplaned | 0.011 | 0.013 | 0.015 |
| 4. Plank with battens | 0.012 | 0.015 | 0.018 |
| H. Cemented masonry rubble | 0.017 | 0.025 | 0.030 |
| I. Dry masonry rubble | 0.023 | 0.032 | 0.035 |

*Note: Values are for straight alignment.*

TABLE 5: EXCAVATED ARTIFICIAL CHANNELS

| Channel | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| A. Earth, straight and uniform | | | |
| 1. Clean, recently completed | 0.016 | 0.018 | 0.020 |
| 2. Clean, after weathering | 0.018 | 0.022 | 0.025 |
| 3. Gravel, uniform section, clean | 0.022 | 0.025 | 0.030 |
| 4. With short grass, few weeds | 0.022 | 0.027 | 0.033 |
| B. Earth, winding and sluggish | | | |
| 1. No vegetation | 0.023 | 0.025 | 0.030 |
| 2. Grass, some weeds | 0.025 | 0.030 | 0.033 |
| 3. Dense weeds or aquatic plants in deep channels | 0.030 | 0.035 | 0.040 |
| 4. Earth bottom and rubble sides | 0.028 | 0.030 | 0.035 |
| 5. Stony bottom and weedy banks | 0.025 | 0.035 | 0.040 |
| 6. Cobble bottom and clean sides | 0.030 | 0.040 | 0.050 |
| C. Dragline-excavated or dredged | | | |
| 1. No vegetation | 0.025 | 0.028 | 0.033 |
| 2. Light brush on banks | 0.035 | 0.050 | 0.060 |
| D. Rock cuts | | | |
| 1. Smooth and uniform | 0.025 | 0.035 | 0.040 |
| 2. Jagged and irregular | 0.035 | 0.040 | 0.050 |
| E. Channels not maintained, weeds and brush uncut | | | |
| 1. Dense weeds, high as flow depth | 0.050 | 0.080 | 0.120 |
| 2. Clean bottom, brush on sides | 0.040 | 0.050 | 0.080 |

| Channel | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| 3. Clean bottom, brush on sides, highest stage of flow | 0.045 | 0.070 | 0.110 |
| 4. Dense brush, high stage | 0.080 | 0.100 | 0.140 |

*Note: Values are for excavated or dredged channels with natural linings.*

**TABLE 6: HIGHWAY CHANNELS AND SWALES WITH MAINTAINED VEGETATION**

| Channel | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| Range of roughness values represents flow velocities from 2 to 6 feet per second with the higher roughness values representing the 2 feet per second flow velocity. | | | |
| A. Depth of flow up to 0.7 feet | | | |
| 1. Grass | | | |
| a. Mowed to 2 inches | 0.045 | . . . . . | 0.07 |
| b. Length 2 inches to 6 inches | 0.05 | . . . . . | 0.09 |
| 2. Grass, good stand | | | |
| a. Length about 12 inches | 0.09 | . . . . . | 0.18 |
| b. Length about 24 inches | 0.15 | . . . . . | 0.30 |
| 3. Grass, fair stand | | | |
| a. Length about 12 inches | 0.08 | . . . . . | 0.14 |
| b. Length about 24 inches | 0.13 | . . . . . | 0.25 |
| B. Depth of flow 0.7 feet to 1.5 feet | | | |
| 1. Grass | | | |
| a. Mowed to 2 inches | 0.035 | . . . . . | 0.05 |
| b. Length 2 inches to 6 inches | 0.04 | . . . . . | 0.06 |
| 2. Grass, good stand | | | |
| a. Length about 12 inches | 0.07 | . . . . . | 0.12 |
| b. Length about 24 inches | 0.10 | . . . . . | 0.20 |
| 3. Grass, fair stand | | | |
| a. Length about 12 inches | 0.06 | . . . . . | 0.10 |
| b. Length about 24 inches | 0.09 | . . . . . | 0.17 |

**TABLE 7: NATURAL CHANNELS AND FLOODPLAINS**

| Channel | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| A. Minor streams (top width at flood stage less than 100 feet) | | | |
| 1. Streams on plain | | | |
| a. Clean, straight, full stage, no rifts or deep pools | 0.025 | 0.030 | 0.033 |
| b. Same as above, but more stones and weeds | 0.030 | 0.035 | 0.040 |
| c. Clean, winding, some pools and shoals | 0.033 | 0.040 | 0.045 |
| d. Same as above, but some weeds and stones | 0.035 | 0.045 | 0.050 |
| e. Same as above, lower stages, irregular slopes and sections with more ineffective flow area | 0.040 | 0.048 | 0.055 |
| f. Same as d, but more stones | 0.045 | 0.050 | 0.060 |
| g. Sluggish reaches, weedy, deep pools | 0.050 | 0.070 | 0.080 |
| h. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush | 0.075 | 0.100 | 0.150 |
| 2. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages | | | |
| a. Bottom: gravels, cobbles, and few boulders | 0.030 | 0.040 | 0.050 |
| b. Bottom: cobbles with large boulders | 0.040 | 0.050 | 0.070 |
| B. Floodplains | | | |
| 1. Pasture, no brush | | | |
| a. Short grass | 0.025 | 0.030 | 0.035 |
| b. High grass | 0.030 | 0.035 | 0.050 |
| 2. Cultivated areas | | | |
| a. No crop | 0.020 | 0.030 | 0.040 |
| b. Mature row crops | 0.025 | 0.035 | 0.045 |

| Channel | HYDRAULIC ROUGHNESS (MANNING'S n) VALUES | | |
|---|---|---|---|
| | **Minimum** | **Normal** | **Maximum** |
| c. Mature field crops | 0.030 | 0.040 | 0.050 |
| 3. Brush | | | |
| a. Scattered brush, heavy weeds | 0.035 | 0.050 | 0.070 |
| b. Light brush and trees, in winter | 0.035 | 0.050 | 0.060 |
| c. Light brush and trees, in summer | 0.040 | 0.060 | 0.080 |
| d. Medium to dense brush, in winter | 0.045 | 0.070 | 0.110 |
| e. Medium to dense brush, in summer | 0.070 | 0.100 | 0.160 |
| 4. Trees | | | |
| a. Dense willows, summer, straight | 0.110 | 0.150 | 0.200 |
| b. Cleared land with tree stumps, no sprouts | 0.030 | 0.040 | 0.050 |
| c. Same as above, but with heavy growth of sprouts | 0.050 | 0.060 | 0.080 |
| d. Heavy stand of timber, a few down trees, little undergrowth,flood stage below branches | 0.080 | 0.100 | 0.120 |
| e. Same as above, but with flood stage reaching branches | 0.100 | 0.120 | 0.160 |
| C. Major streams (top width at flood stage more than 100 feet). The n values are less than those of minor streams with similar description because banks offer less effective resistance. | | | |
| 1. Regular section with no boulders or brush | 0.025 | . . . . . | 0.060 |
| 2. Irregular and rough section | 0.035 | . . . . . | 0.100 |

# APPENDIX B - HYDRAULIC PROPERTIES OF CONDUITS

This appendix lists hydraulic properties for various conduits, as follows:

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TABLE -1
HYDRAULIC PROPERTIES OF
CIRCULAR PIPE

(Size, Waterway Area, Hydraulic Radius)

| Diameter (inches) | Waterway Area (square feet) | Hydraulic Radius (feet) | Diameter (inches) | Waterway Area (square feet) | Hydraulic Radius (feet) |
|---|---|---|---|---|---|
| 12 | 0.79 | 0.25 | 156 | 132.7 | 3.25 |
| 15 | 1.23 | 0.31 | 135 | 143.1 | 3.38 |
| 18 | 1.77 | 0.38 | 168 | 153.9 | 3.50 |
| 21 | 2.41 | 0.44 | 174 | 165.1 | 3.63 |
| 24 | 3.14 | 0.50 | 180 | 176.7 | 3.75 |
| 30 | 4.91 | 0.63 | 186 | 188.7 | 3.88 |
| 36 | 7.07 | 0.75 | 192 | 201.1 | 4.00 |
| 42 | 9.62 | 0.88 | 198 | 213.8 | 4.13 |
| 48 | 12.57 | 1.00 | 204 | 227.0 | 4.25 |
| 54 | 15.90 | 1.13 | 210 | 240.5 | 4.38 |
| 60 | 19.64 | 1.25 | 216 | 254.5 | 4.50 |
| 66 | 23.76 | 1.38 | 222 | 268.8 | 4.63 |
| 72 | 28.27 | 1.50 | 228 | 283.5 | 4.75 |
| 78 | 33.18 | 1.63 | 234 | 298.6 | 4.88 |
| 84 | 38.49 | 1.75 | 240 | 314.2 | 5.00 |
| 90 | 44.18 | 1.88 | 246 | 330.1 | 5.13 |
| 96 | 50.27 | 2.00 | 252 | 346.4 | 5.25 |
| 108 | 63.62 | 2.25 | 258 | 363.1 | 5.38 |
| 114 | 70.88 | 2.38 | 264 | 380.1 | 5.50 |
| 120 | 78.54 | 2.50 | 270 | 397.6 | 5.63 |
| 126 | 86.59 | 2.63 | 276 | 415.5 | 5.75 |
| 132 | 95.03 | 2.75 | 282 | 433.7 | 5.88 |
| 138 | 103.87 | 2.88 | 288 | 452.4 | 6.00 |
| 144 | 113.10 | 3.00 | 294 | 471.4 | 6.13 |
| 150 | 122.70 | 3.13 | 300 | 490.9 | 6.25 |

# TABLE -2
# HYDRAULIC PROPERTIES OF
# CORRUGATED STEEL PIPE-ARCHES

(Size, Waterway Area, Hydraulic Radius, Equivalent Diameter)

| 2 2/3 x 1/2 inch Corrugations | | | | 3 x 1 inch Corrugations | | | |
|---|---|---|---|---|---|---|---|
| Span x Rise (inches) x (inches) | Waterway Area (square feet) | Hydraulic Radius (feet) | Equivalent Diameter (inches) | Span x Rise (inches) x (inches) | Waterway Area (square feet) | Hydraulic Radius (feet) | Equivalent Diameter (inches) |
| 17 x 13 | 1.1 | .28 | 15 | 60 x 46 | 15.6 | 1.10 | 54 |
| 21 x 15 | 1.6 | .34 | 18 | 66 x 51 | 19.3 | 1.23 | 60 |
| 24 x 18 | 2.2 | .40 | 21 | 73 x 55 | 23.2 | 1.34 | 66 |
| 28 x 20 | 2.9 | .46 | 24 | 81 x 59 | 27.4 | 1.45 | 72 |
| 35 x 24 | 4.5 | .57 | 30 | 87 x 63 | 32.1 | 1.57 | 78 |
| 42 x 29 | 6.5 | .69 | 36 | 95 x 67 | 37.0 | 1.68 | 84 |
| 49 x 33 | 8.9 | .81 | 42 | 103 x 71 | 42.4 | 1.80 | 90 |
| 57 x 38 | 11.6 | .92 | 48 | 112 x 75 | 48.0 | 1.91 | 96 |
| 64 x 43 | 14.7 | 1.04 | 54 | 117 x 79 | 54.2 | 2.03 | 102 |
| 71 x 47 | 18.1 | 1.15 | 60 | 128 x 83 | 60.5 | 2.14 | 108 |
| 77 x 52 | 21.9 | 1.27 | 66 | 137 x 87 | 67.4 | 2.26 | 114 |
| 83 x 57 | 26.0 | 1.38 | 72 | 142 x 91 | 74.5 | 2.37 | 120 |

# TABLE -3
# HYDRAULIC PROPERTIES OF
# STRUCTURAL PLATE STEEL PIPE-ARCHES

(Size, Waterway Area, Hydraulic Radius, Equivalent Diameter)

| 6 x 2 inch Corrugations 18-inch Corner Radius | | | | 6 x 2 inch Corrugations 31-inch Corner Radius | | | |
|---|---|---|---|---|---|---|---|
| Span x Rise (feet - inches) x (feet -inches) | Waterway Area (square feet) | Hydraulic Radius (feet) | Equivalent Diameter (inches) | Span x Rise (feet - inches) x (feet -inches) | Waterway Area (square feet) | Hydraulic Radius (feet) | Equivalent Diameter (inches) |
| 6-1 x 4-7 | 22 | 1.29 | 66 | 13-3 x 9-4 | 97 | 2.68 | 138 |
| 6-4 x 4-9 | 24 | 1.35 | 69 | 13-6 x 9-6 | 102 | 2.74 | 141 |
| 6-9 x 4-11 | 26 | 1.39 | 72 | 14-0 x 9-8 | 105 | 2.78 | 144 |
| 7-0 x 5-1 | 28 | 1.45 | 75 | 14-2 x 9-10 | 109 | 2.83 | 147 |
| 7-3 x 5-3 | 30 | 1.51 | 78 | 14-5 x 10-0 | 114 | 2.90 | 150 |
| 7-8 x 5-5 | 33 | 1.55 | 81 | 14-11 x 10-2 | 118 | 2.94 | 153 |
| 7-11 x 5-7 | 35 | 1.61 | 84 | 15-4 x 10-4 | 123 | 2.98 | 156 |
| 8-2 x 5-9 | 38 | 1.67 | 87 | 15-7 x 10-6 | 127 | 3.04 | 159 |
| 8-7 x 5-11 | 40 | 1.71 | 90 | 15-10 x 10-8 | 132 | 3.10 | 162 |
| 8-10 x 6-1 | 43 | 1.77 | 93 | 16-3 x 10-10 | 137 | 3.14 | 165 |
| 9-4 x 6-3 | 45 | 1.81 | 96 | 16-6 x 11-0 | 142 | 3.20 | 168 |
| 9-6 x 6-5 | 48 | 1.87 | 99 | 17-0 x 11-2 | 146 | 3.24 | 171 |
| 9-9 x 6-7 | 51 | 1.93 | 102 | 17-2 x 11-4 | 151 | 3.30 | 174 |
| 10-3 x 6-9 | 54 | 1.97 | 105 | 17-5 x 11-6 | 157 | 3.36 | 177 |
| 10-8 x 6-11 | 57 | 2.01 | 109 | 17-11 x 11-8 | 161 | 3.40 | 180 |
| 10-11 x 7-1 | 60 | 2.07 | 111 | 18-1 x 11-10 | 167 | 3.45 | 183 |
| 11-5 x 7-3 | 63 | 2.11 | 114 | 18-7 x 12-0 | 172 | 3.50 | 186 |
| 11-7 x 7-5 | 66 | 2.17 | 117 | 18-9 x 12-2 | 177 | 3.56 | 189 |
| 11-10 x 7-7 | 70 | 2.23 | 120 | 19-3 x 12-4 | 182 | 3.59 | 192 |
| 12-4 x 7-9 | 73 | 2.26 | 123 | 19-6 x 12-6 | 188 | 3.65 | 195 |
| 12-6 x 7-11 | 77 | 2.32 | 126 | 19-8 x 12-8 | 194 | 3.71 | 198 |
| 12-8 x 8-1 | 81 | 2.38 | 129 | 19-11 x 12-10 | 200 | 3.77 | 201 |
| 12-10 x 8-4 | 85 | 2.44 | 132 | 20-5 x 13-0 | 205 | 3.81 | 204 |
| 13-5 x 8-5 | 88 | 2.48 | 135 | 20-7 x 13-2 | 211 | 3.87 | 207 |
| 13-11 x 8-7 | 91 | 2.52 | 138 | | | | |
| 14-1 x 8-9 | 95 | 2.57 | 141 | | | | |
| 14-3 x 8-11 | 100 | 2.63 | 144 | | | | |
| 14-10 x 9-1 | 103 | 2.67 | 147 | | | | |
| 15-4 x 9-3 | 107 | 2.71 | 150 | | | | |
| 15-6 x 9-5 | 111 | 2.77 | 153 | | | | |
| 15-8 x 9-7 | 116 | 2.83 | 156 | | | | |
| 15-10 x 9-10 | 121 | 2.89 | 159 | | | | |
| 16-5 x 9-11 | 125 | 2.92 | 162 | | | | |
| 16-7 x 10-1 | 130 | 2.98 | 165 | | | | |

# TABLE -4
# HYDRAULIC PROPERTIES OF
# STRUCTURAL PLATE STEEL ARCHES

(Size, Waterway Area, Hydraulic Radius, Rise/Span, Radius)
6 x 2 inch Corrugations

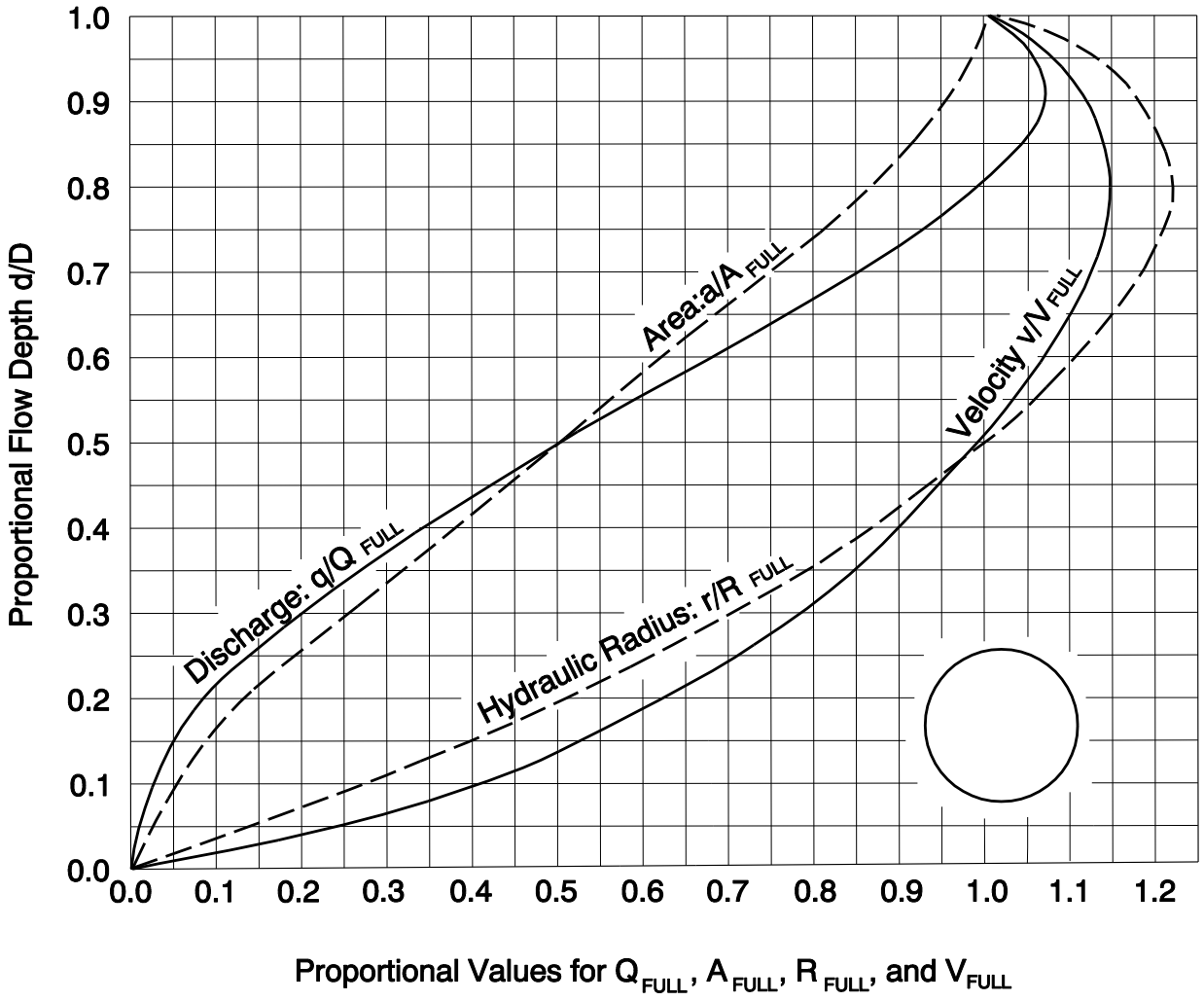
| Span (feet) | Rise (feet-inches) | Waterway Area (square feet) | Hydraulic Radius (feet) | Rise/ Span | Radius (inches) |
|---|---|---|---|---|---|
| 6.0 | 1-9½ | 7½ | .58 | 0.30 | 41 |
| | 2-3½ | 10 | .72 | 0.38 | 37½ |
| | 3-2 | 15 | .97 | 0.53 | 36 |
| 7.0 | 2-4 | 12 | .77 | 0.34 | 45 |
| | 2-10 | 15 | .91 | 0.40 | 43 |
| | 3-8 | 20 | 1.11 | 0.52 | 42 |
| 8.0 | 2-11 | 17 | .93 | 0.37 | 51 |
| | 3-4 | 20 | 1.05 | 0.42 | 48½ |
| | 4-2 | 26 | 1.26 | 0.52 | 48 |
| 9.0 | 2-11 | 18½ | .93 | 0.32 | 59 |
| | 3-10½ | 26½ | 1.23 | 0.43 | 55 |
| | 4-8½ | 33 | 1.43 | 0.52 | 54 |
| 10.0 | 3-5½ | 25 | 1.11 | 0.35 | 64 |
| | 4-5 | 34 | 1.41 | 0.44 | 60½ |
| | 5-3 | 41 | 1.60 | 0.52 | 60 |
| 11.0 | 3-6 | 27½ | 1.22 | 0.32 | 73 |
| | 4-5½ | 37 | 1.43 | 0.41 | 67½ |
| | 5-9 | 50 | 1.77 | 0.52 | 66 |
| 12.0 | 4-0½ | 35 | 1.30 | 0.34 | 77½ |
| | 5-0 | 45 | 1.58 | 0.42 | 73 |
| | 6-3 | 59 | 1.91 | 0.52 | 72 |
| 13.0 | 4-1 | 38 | 1.32 | 0.32 | 86½ |
| | 5-1 | 49 | 1.62 | 0.39 | 80½ |
| | 6-9 | 70 | 2.10 | 0.52 | 78 |
| 14.0 | 4-7½ | 47 | 1.50 | 0.33 | 91 |
| | 5-7 | 58 | 1.77 | 0.40 | 86 |
| | 7-3 | 80 | 2.22 | 0.52 | 84 |
| 15.0 | 4-7½ | 50 | 1.51 | 0.31 | 101 |
| | 5-8 | 62 | 1.79 | 0.38 | 93 |
| | 6-7 | 75 | 2.07 | 0.44 | 91 |
| | 7-9 | 92 | 2.39 | 0.52 | 90 |

# TABLE -4 CONTD.
# HYDRAULIC PROPERTIES OF
# STRUCTURAL PLATE STEEL ARCHES

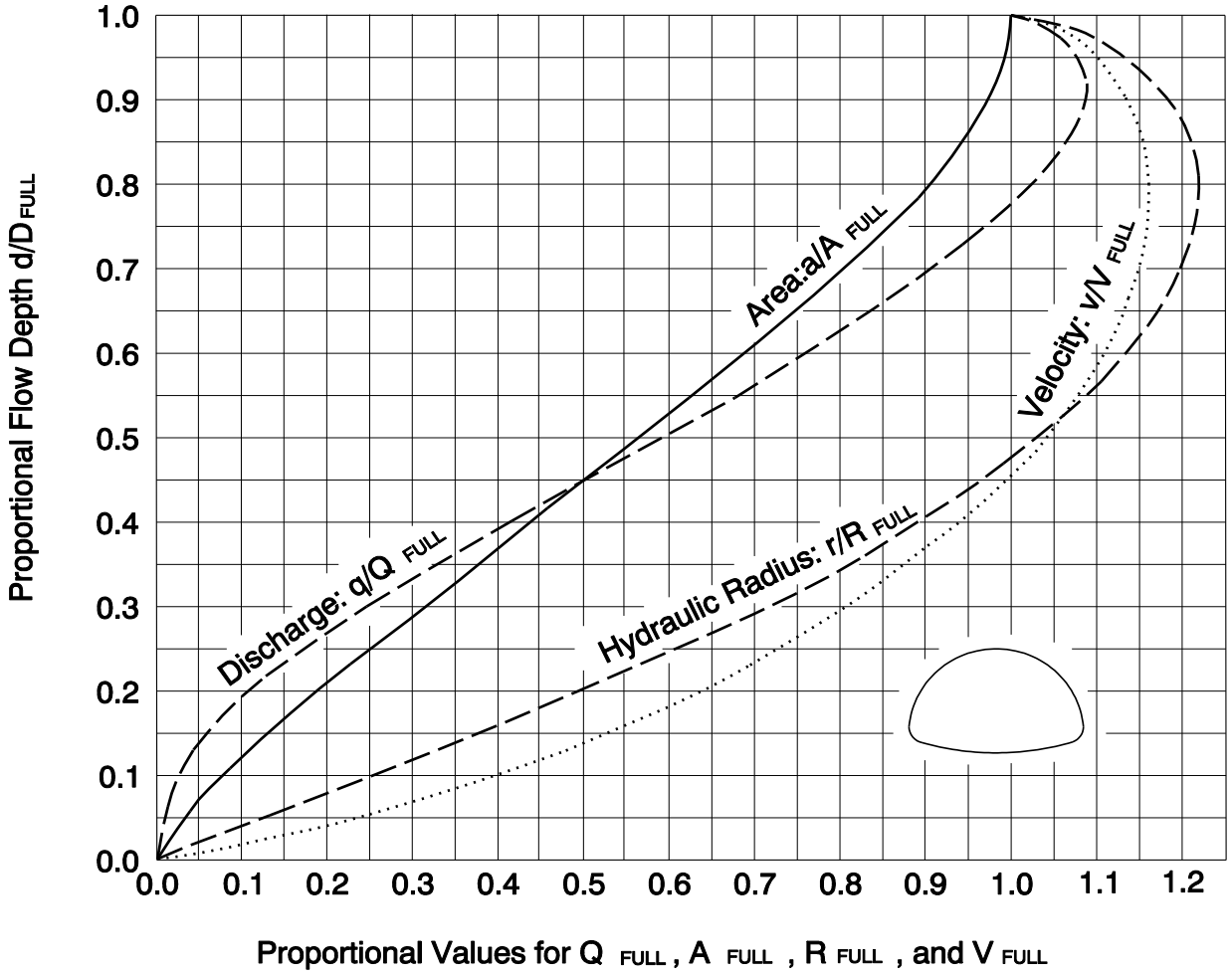
(Size, Waterway Area, Hydraulic Radius, Rise/Span, Radius)
6 x 2 inch Corrugations (cont)

| Span (feet) | Rise (feet-inches) | Waterway Area (square feet) | Hydraulic Radius (feet) | Rise/ Span | Radius (inches) |
|---|---|---|---|---|---|
| 16.0 | 5-2 | 60 | 1.68 | 0.32 | 105 |
| | 7-1 | 86 | 2.22 | 0.45 | 97 |
| | 8-3 | 105 | 2.55 | 0.52 | 96 |
| 17.0 | 5-2½ | 63 | 1.68 | 0.31 | 115 |
| | 7-2 | 92 | 2.18 | 0.42 | 103 |
| | 8-10 | 119 | 2.72 | 0.52 | 102 |
| 18.0 | 5-9 | 75 | 1.88 | 0.32 | 119 |
| | 7-8 | 104 | 2.41 | 0.43 | 109 |
| | 8-11 | 126 | 2.77 | 0.50 | 108 |
| 19.0 | 6-4 | 87 | 2.05 | 0.33 | 123 |
| | 8-2 | 118 | 2.58 | 0.43 | 115 |
| | 9-5½ | 140 | 2.91 | 0.50 | 114 |
| 20.0 | 6-4 | 91 | 2.05 | 0.32 | 133 |
| | 8-3½ | 124 | 2.61 | 0.42 | 122 |
| | 10-0 | 157 | 3.10 | 0.50 | 120 |
| 21.0 | 6-11 | 104 | 2.22 | 0.33 | 137 |
| | 8-10 | 140 | 2.80 | 0.42 | 128 |
| | 10-6 | 172 | 3.23 | 0.50 | 126 |
| 22.0 | 6-11 | 109 | 2.24 | 0.31 | 146 |
| | 8-11 | 146 | 2.82 | 0.40 | 135 |
| | 11-0 | 190 | 3.41 | 0.50 | 132 |
| 23.0 | 8-0 | 134 | 2.58 | 0.35 | 147 |
| | 9-10 | 171 | 3.10 | 0.43 | 140 |
| | 11-6 | 208 | 3.57 | 0.50 | 138 |
| 24.0 | 8-6 | 150 | 2.75 | 0.35 | 152 |
| | 10-4 | 188 | 3.26 | 0.43 | 146 |
| | 12-0 | 226 | 3.71 | 0.50 | 144 |
| 25.0 | 8-6½ | 155 | 2.75 | 0.34 | 160 |
| | 10-10½ | 207 | 3.43 | 0.43 | 152 |
| | 12-6 | 247 | 3.89 | 0.50 | 150 |

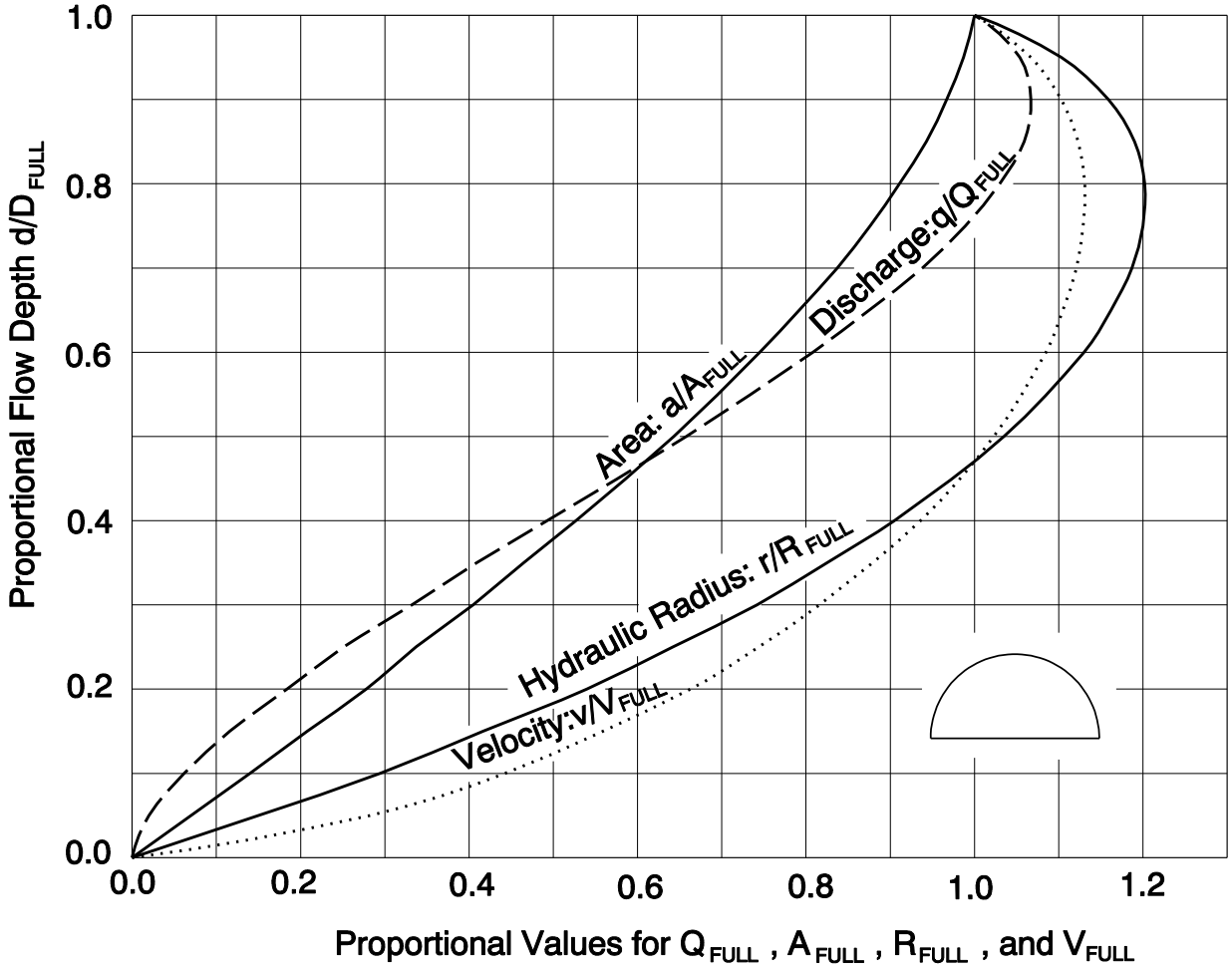
# CHART 1
# HYDRAULIC ELEMENTS OF
# CIRCULAR PIPE



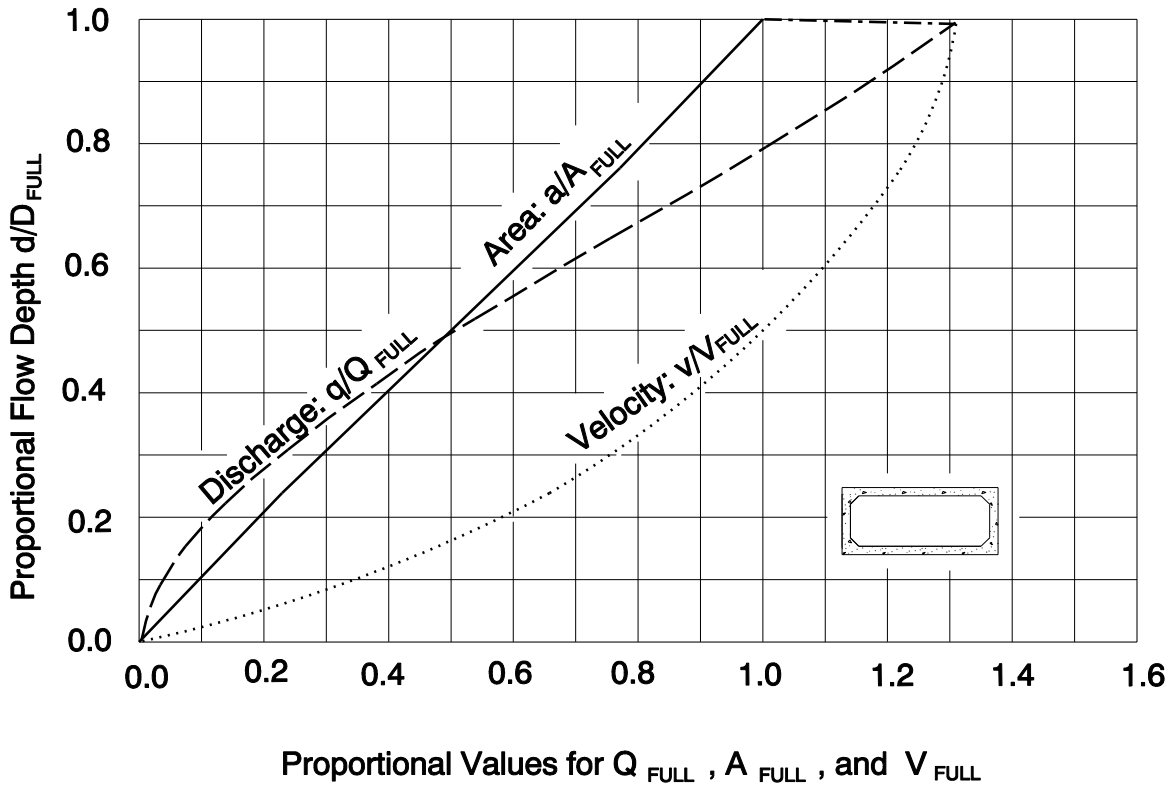
# CHART 2
# HYDRAULIC ELEMENTS OF
# PIPE-ARCHES



# CHART 3
# HYDRAULIC ELEMENTS OF
# SEMICIRCULAR ARCHES

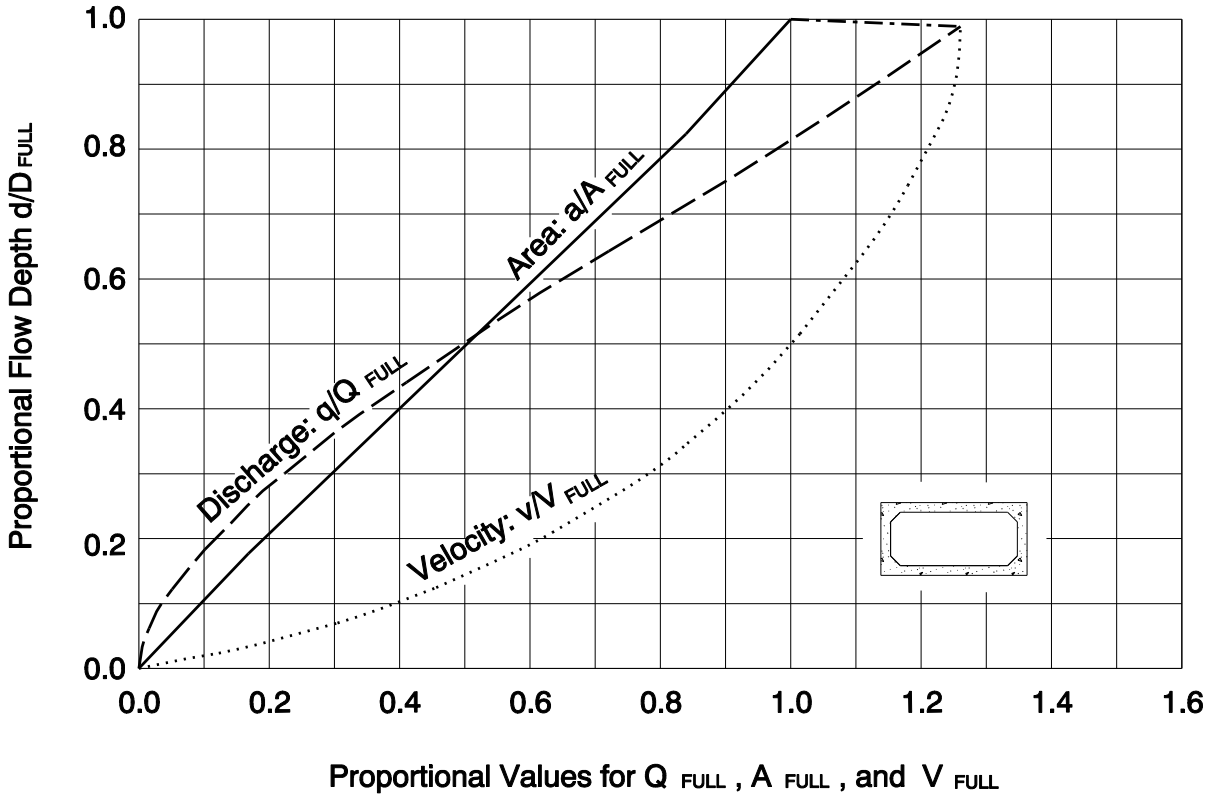


# CHART 4
# HYDRAULIC ELEMENTS OF RCBC 0.33 ≤ RISE / SPAN ≤ 0.36



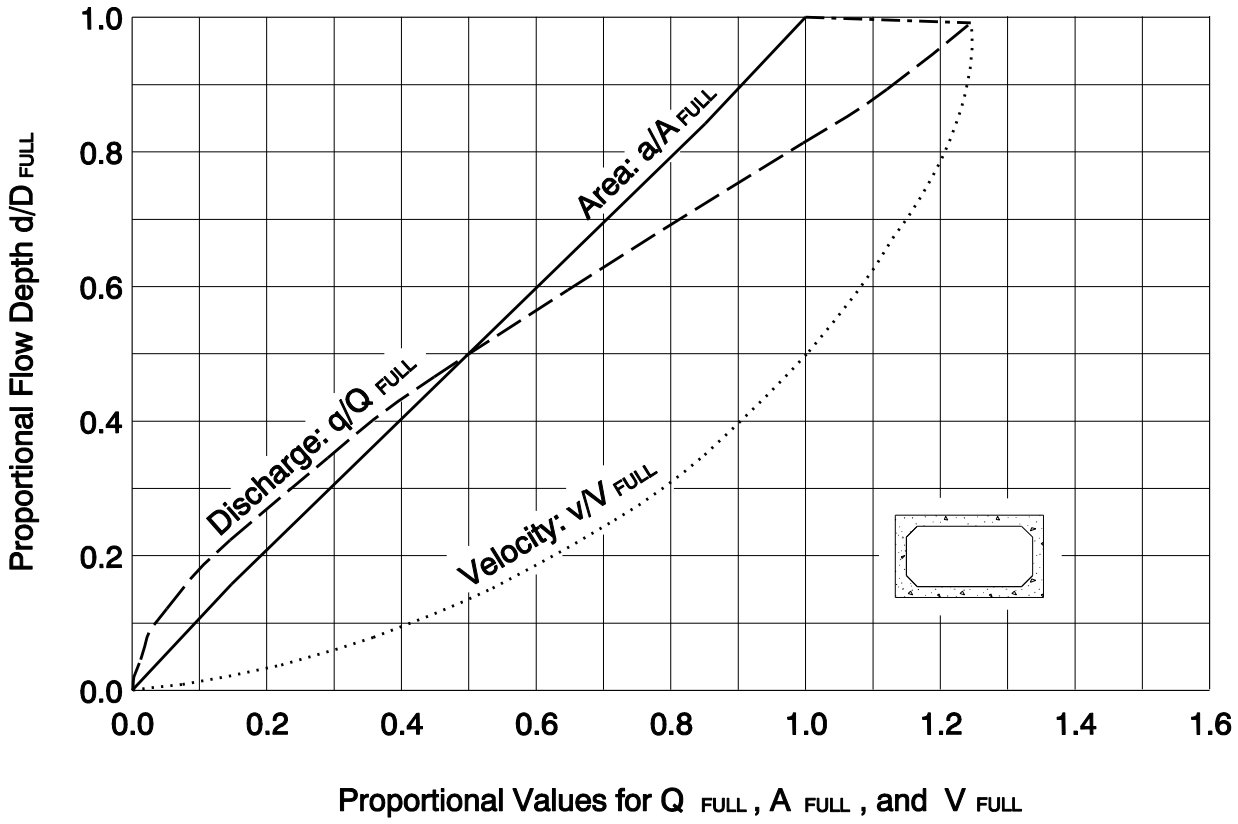

Applicable for 11-foot x 4-foot
and 12-foot x 4-foot sections

# CHART 5
# HYDRAULIC ELEMENTS OF RCBC RISE / SPAN = 0.50



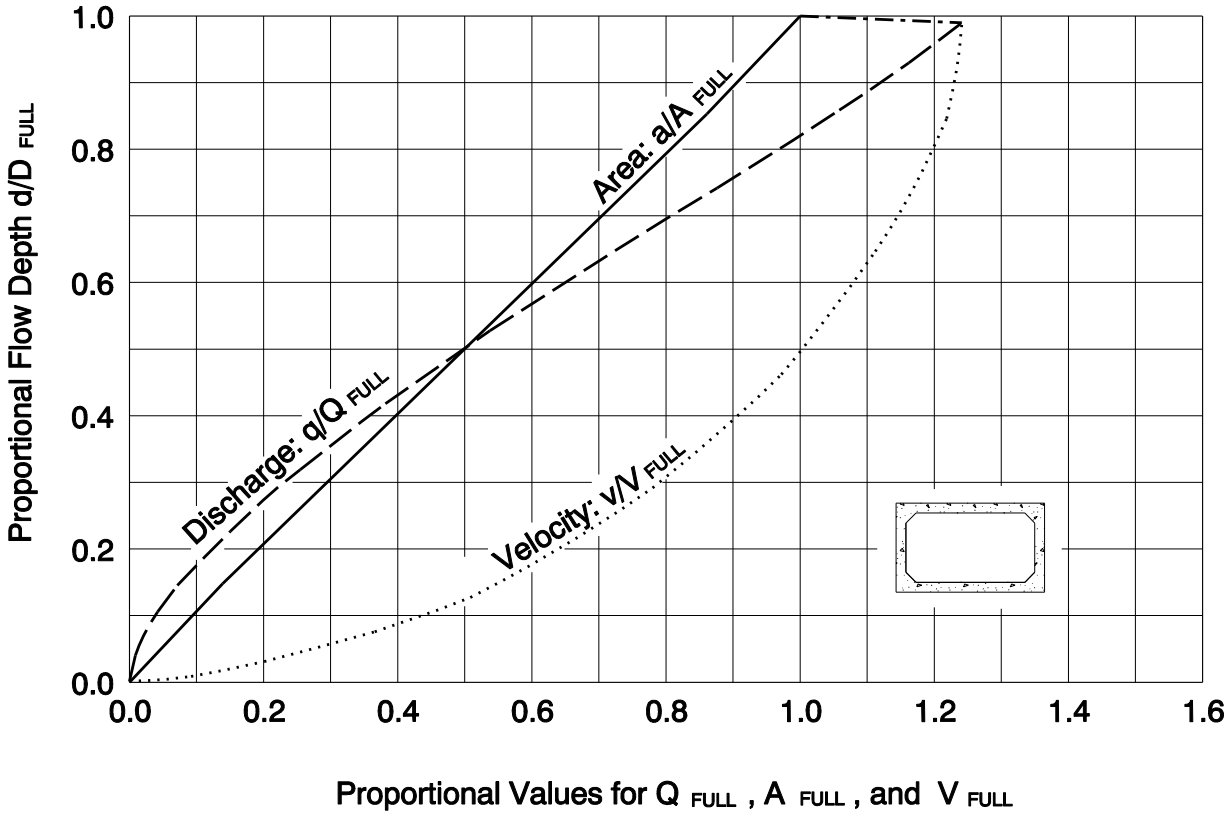

Applicable for 4-foot x 2-foot, 6-foot x 3-foot,
8-foot x 4-foot, 10-foot x 5-foot, 12-foot x 6-foot sections

# CHART 6
## HYDRAULIC ELEMENTS OF RCBC 0.55 ≤ RISE / SPAN ≤ 0.57



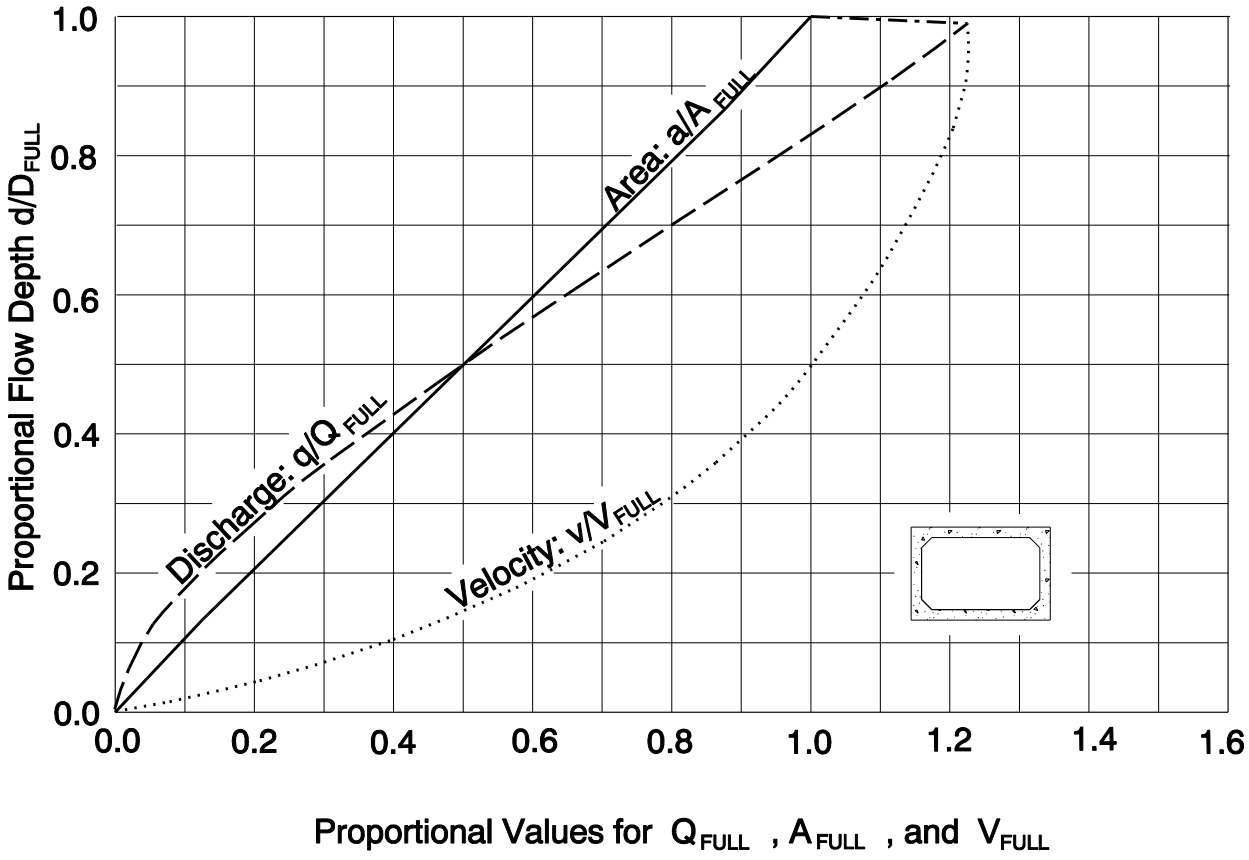

Applicable for 7-foot x 4-foot,
9-foot x 5-foot, and 11-foot x 6-foot sections

# CHART 7
# HYDRAULIC ELEMENTS OF RCBC 0.60 ≤ RISE / SPAN ≤ 0.63



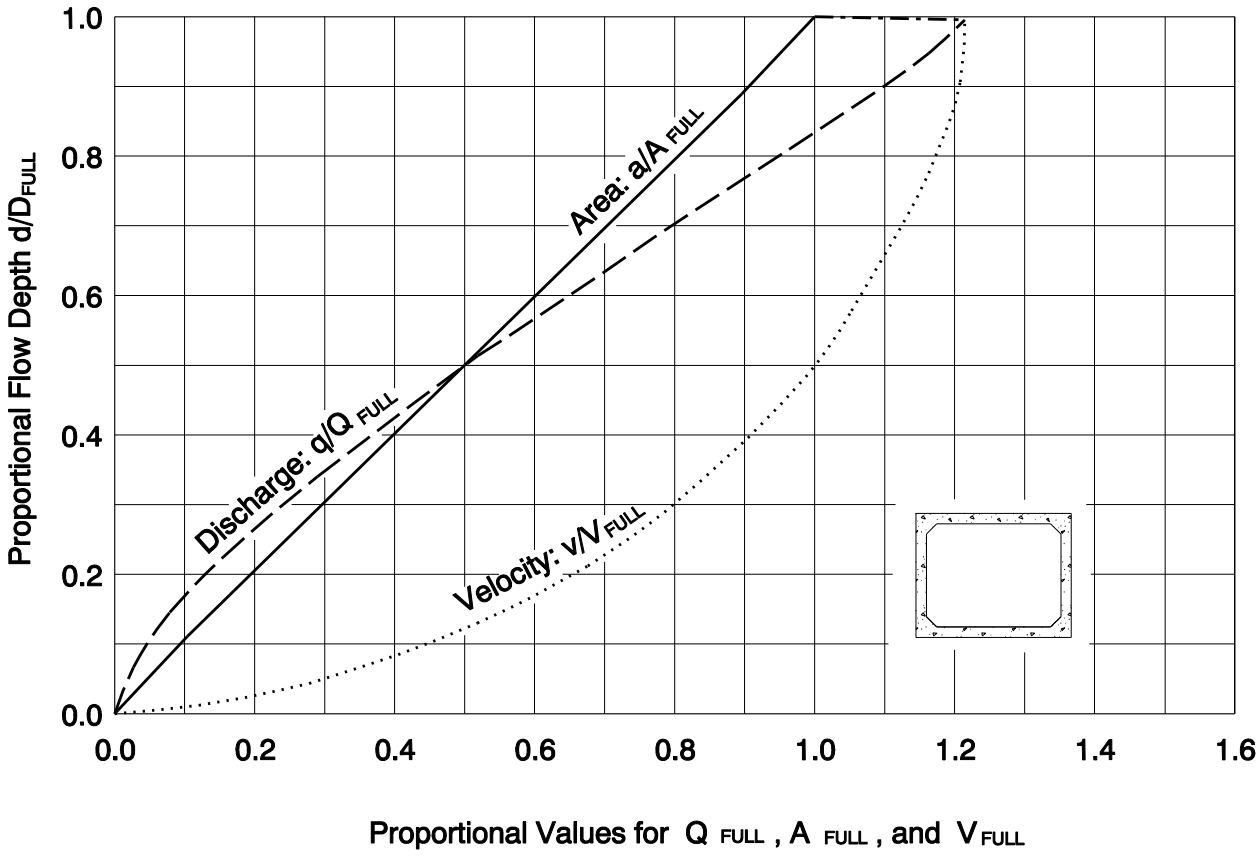

Applicable for 5-foot x 3-foot, 8-foot x 5-foot,
and 10-foot x 6-foot sections

# CHART 8
# HYDRAULIC ELEMENTS OF RCBC RISE / SPAN = 0.67



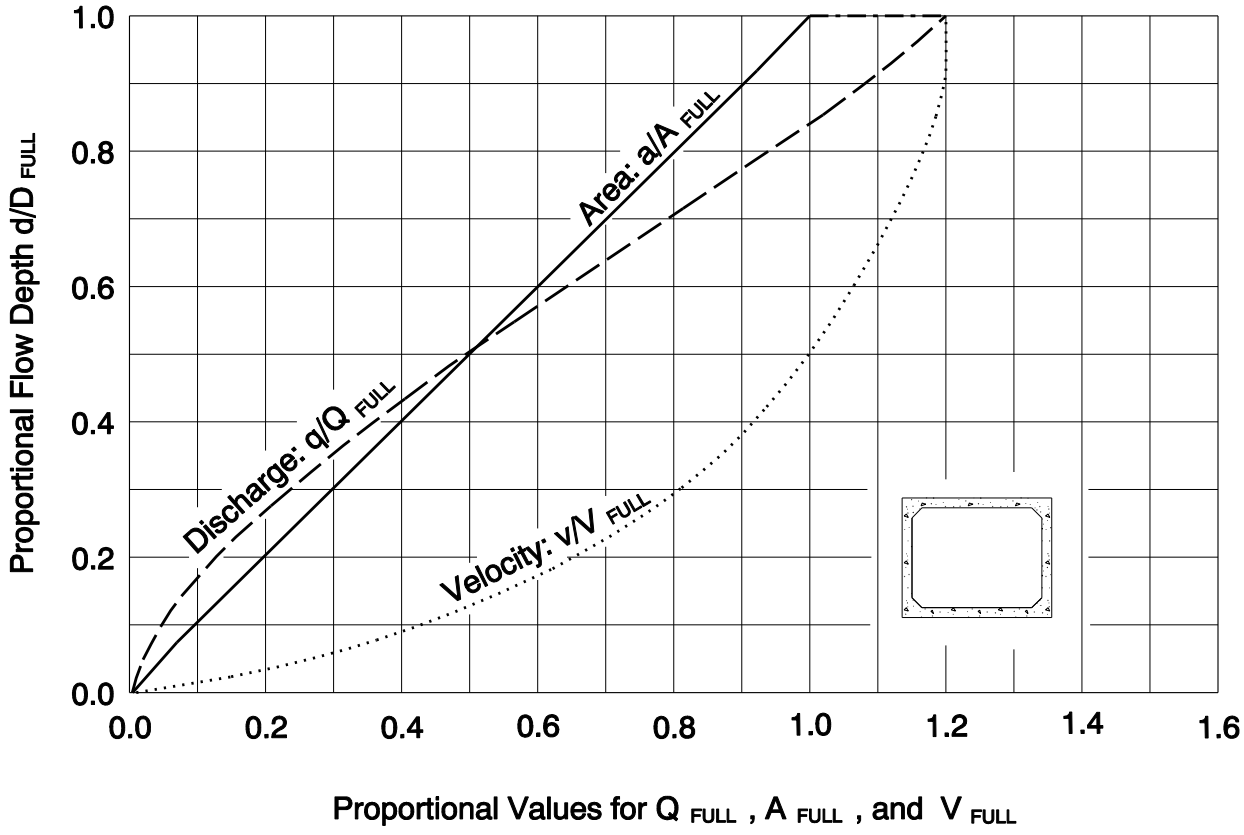

Applicable for 3-foot x 2-foot, 6-foot x 4-foot,
9-foot x 6-foot, and 12-foot x 8-foot sections

# CHART 9
# HYDRAULIC ELEMENTS OF RCBC 0.70 ≤ RISE / SPAN ≤ 0.75



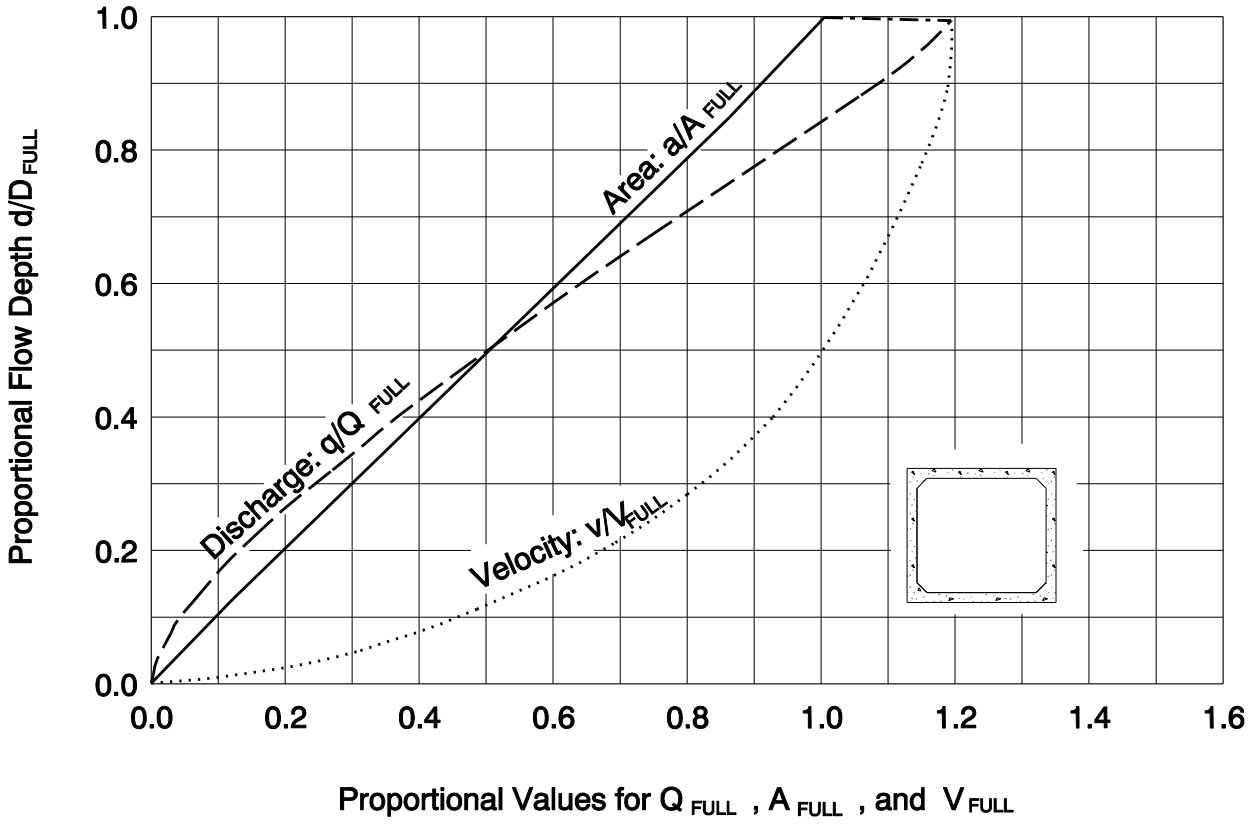

Applicable for 4-foot x 3-foot, 7-foot x 5-foot, 8-foot x 6-foot, 10-foot x 7-foot, and 11-foot x 8-foot sections

# CHART 10
## HYDRAULIC ELEMENTS OF RCBC 0.77 ≤ RISE / SPAN ≤ 0.83



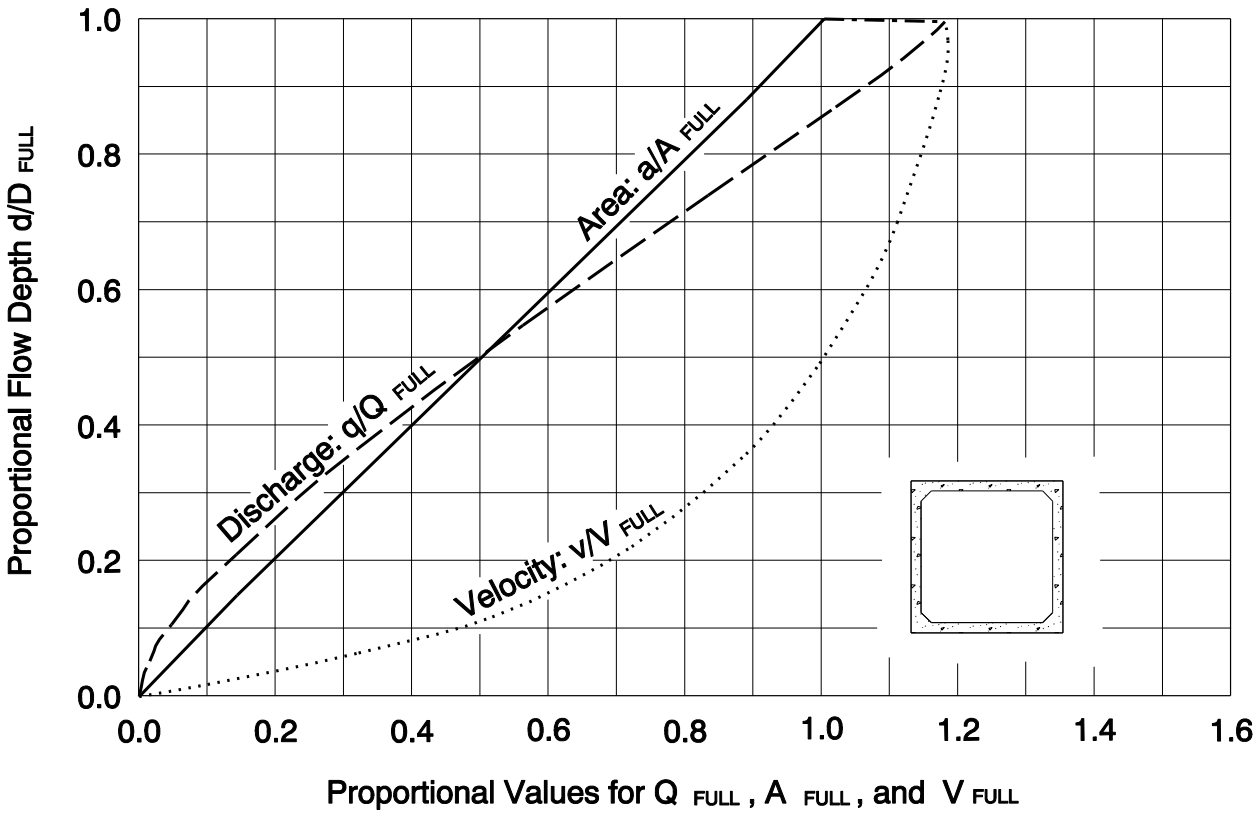

Applicable for 5-foot x 4-foot, 6-foot x 5-foot, 9-foot x 7-foot, 10-foot x 8-foot, 11-foot x 10-foot, and 12-foot x 10-foot sections

# CHART 11
# HYDRAULIC ELEMENTS OF RCBC 0.85 ≤ RISE / SPAN ≤ 0.91

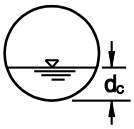



Applicable for 7-foot x 6-foot, 8-foot x 7-foot,
9-foot x 8-foot, and 10-foot x 9-foot sections

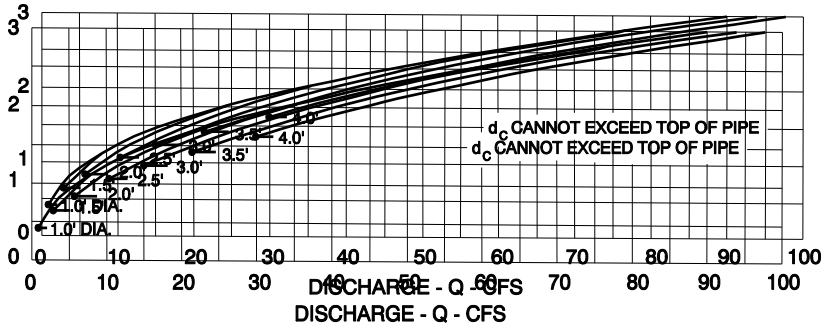
# CHART 12
# HYDRAULIC ELEMENTS OF RCBC RISE / SPAN = 1.0

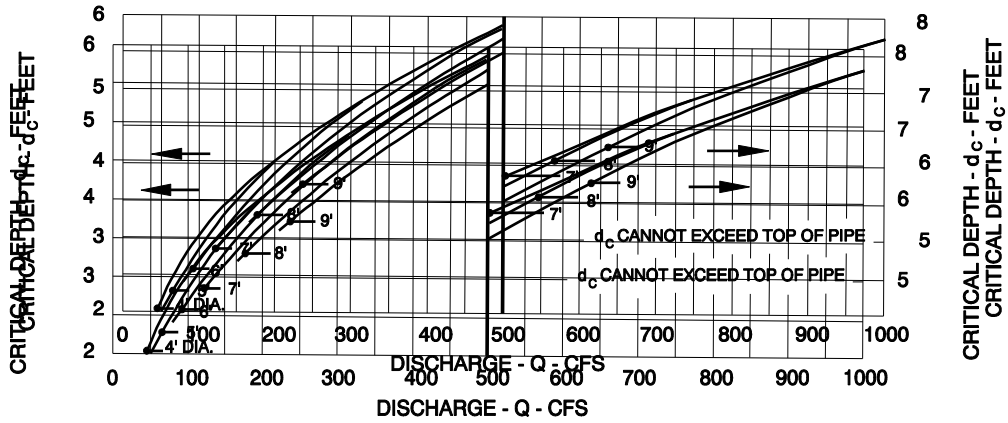


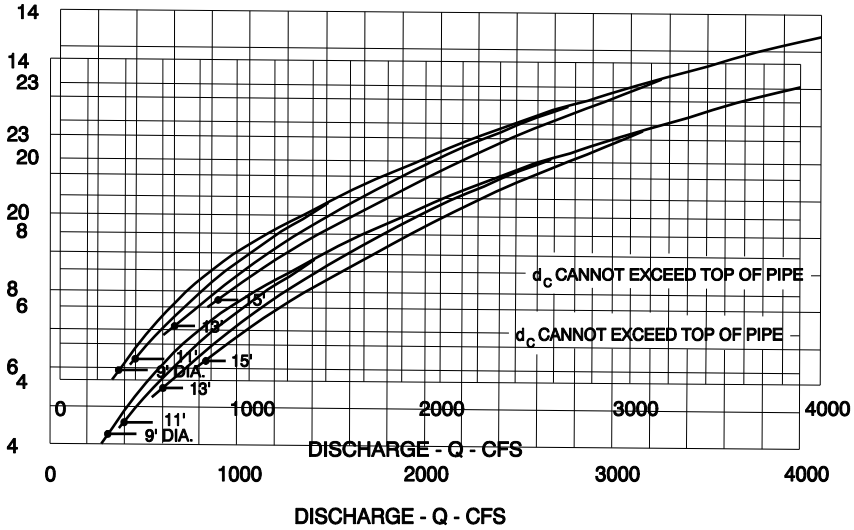

Applicable for 3-foot x 3-foot, 4-foot x 4-foot, 5-foot x 5-foot, 6-foot x 6-foot, 7-foot x 7-foot, 8-foot x 8-foot, 9-foot x 9-foot 10-foot x 10-foot, 11-foot x 11-foot, and 12-foot x 12-foot sections



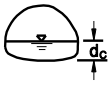

# CHART 13
# CRITICAL DEPTH IN CIRCULAR PIPES

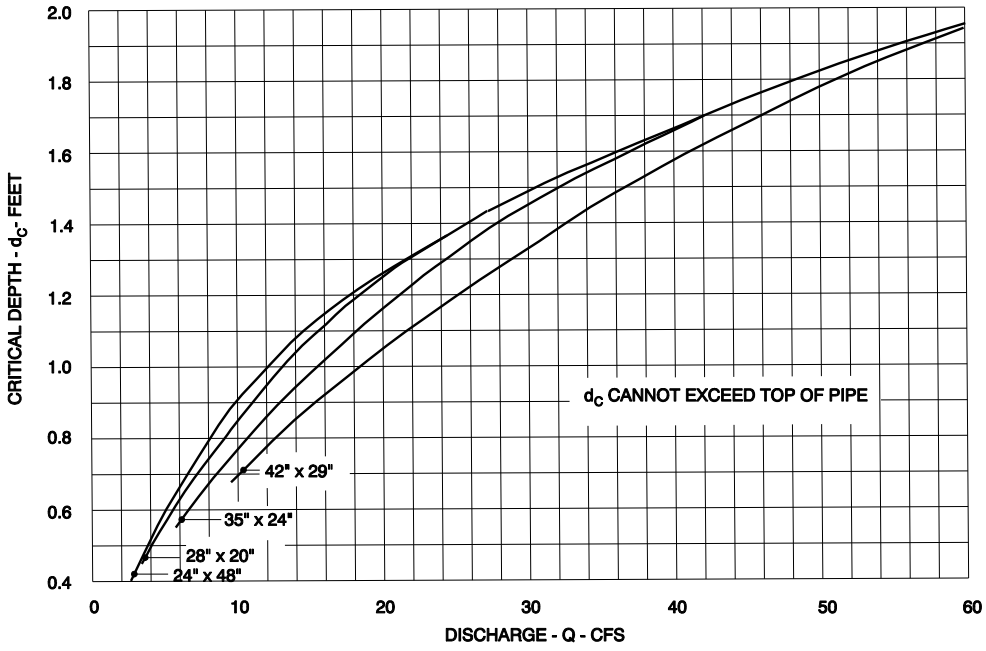


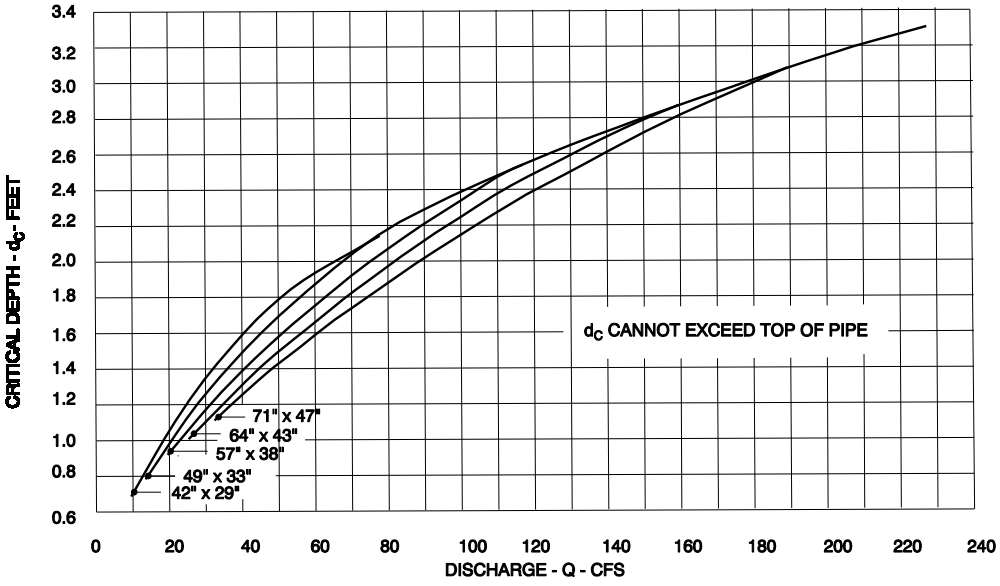



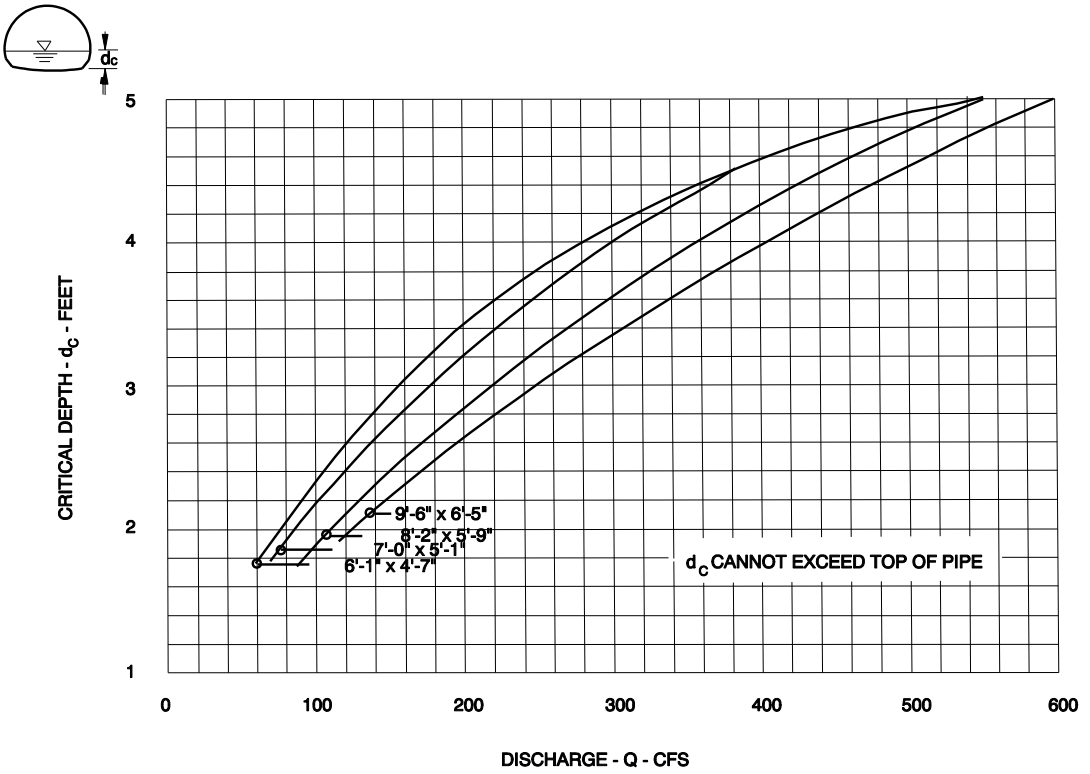
# CHART 14
# CRITICAL DEPTH IN CORRUGATED
# PIPE ARCHES

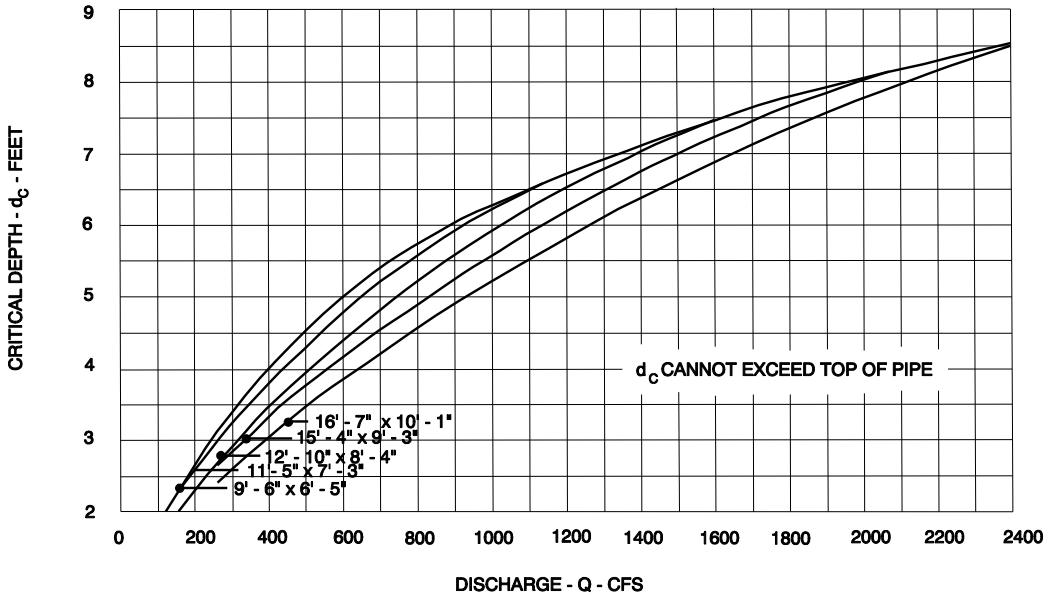


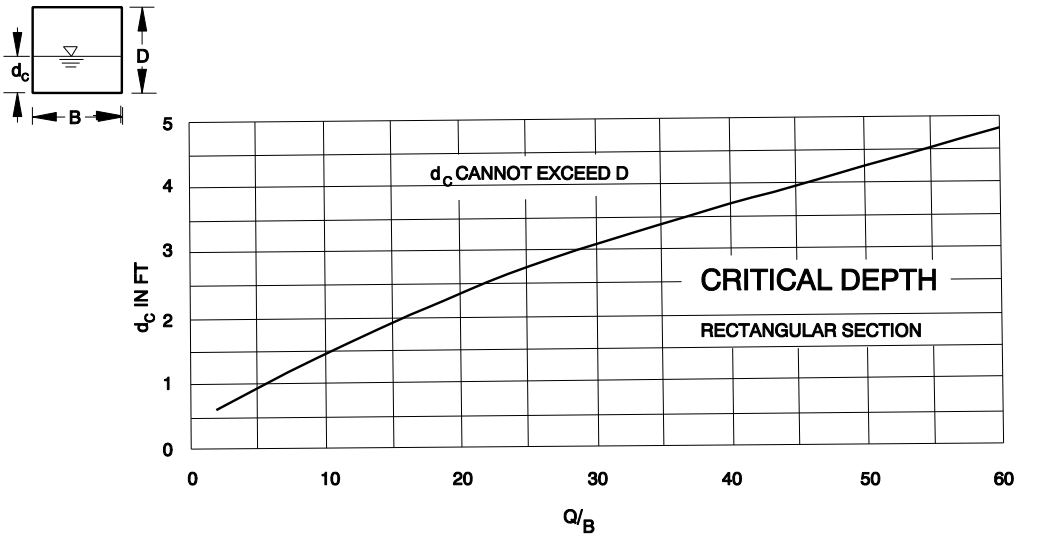



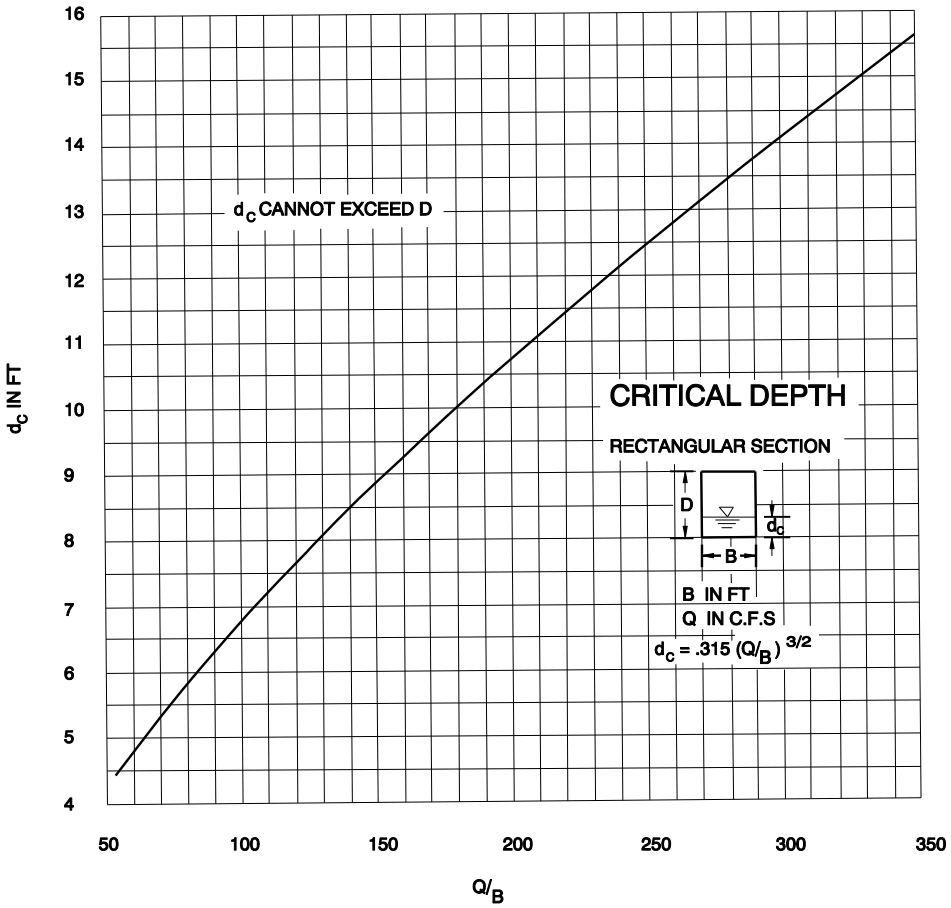
# CHART 15
# CRITICAL DEPTH IN STRUCTURAL PLATE PIPE ARCHES



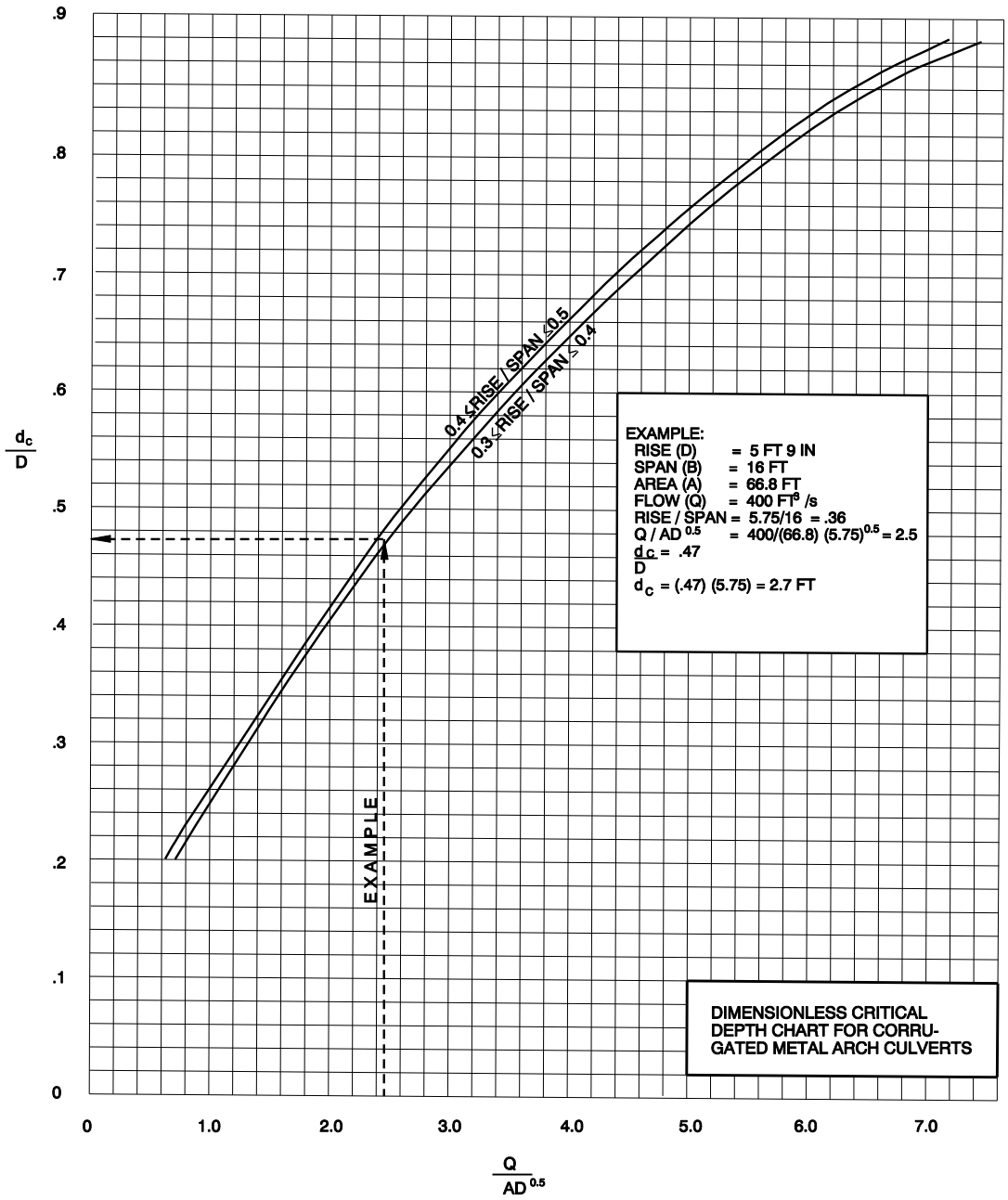



# CHART 16
# CRITICAL DEPTH IN BOXES



# CHART 17
# CRITICAL DEPTH IN ARCHES



Chapter 9

# CULVERTS

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## 9.1 Introduction

This chapter provides information for the planning and design of highway culverts. The methodology is intended for those with an understanding of basic hydrologic and hydraulic methods and some experience in the design of hydraulic structures.

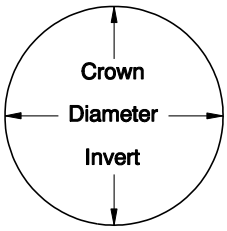
A culvert is a conduit that conveys stream flow or storm runoff through a roadway embankment or past some other flow obstruction. Culverts are constructed from a variety of materials and are available with many different shapes, sizes, and end treatments, as shown in Figures 9-1 and 9-2. Culvert selection factors include roadway profiles, channel characteristics, fish passage requirements, hydraulic performance, resistance to flotation, construction and maintenance costs, and estimated service life.

Culverts may be cost-effective replacements for bridges at some sites. Culverts often do not provide the clearance for navigation, debris, ice, and fish passage as compared to bridges. In addition, scour downstream from a culvert outlet is often deeper and more extensive than scour downstream from a bridge. These factors should be considered before a culvert is recommended at the site of an existing bridge.

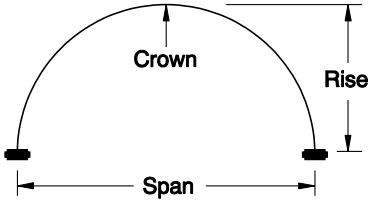
## 9.2 Policy and Practice

General policies of the Federal Highway Administration (FHWA) and ODOT pertaining to hydraulic design are discussed in **Chapter 3**. ODOT practice specific to culvert design are as follows.

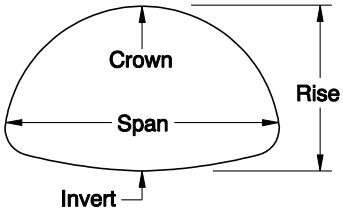
- Coordination with other Federal, state, and local agencies concerned with water resources planning will have a high priority in culvert design.
- Safety of the general public is an important consideration in culvert selection.
- Culvert design shall consider the frequency and type of maintenance and make allowance for maintenance equipment and personnel access.
- Sediment deposition and scour shall be considered during culvert design and be minimized when possible.
- Environmental impacts of culverts such as fish passage and disturbance of fish habitat, wetlands, and riparian areas must be considered and minimized as needed.
- Culvert hydraulic performance during the design event shall consider the class of the roadway, the consequences of traffic interruption, the flood hazard risks, economics, and local site conditions.



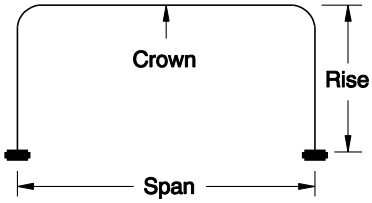

Circular Culvert



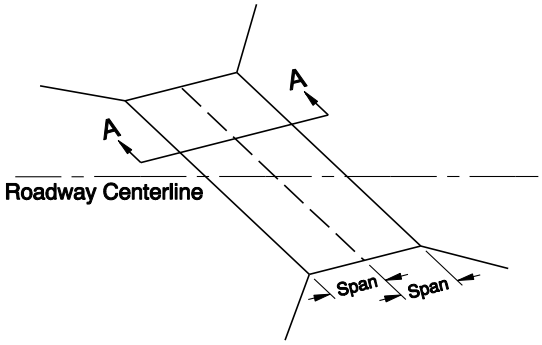

Bottomless Arch Culvert



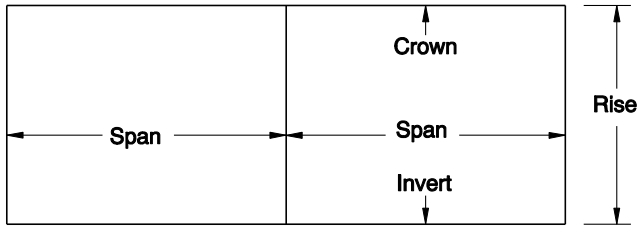

Pipe-Arch Culvert




Bottomless Box or 3-sided Culvert



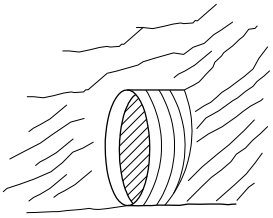

Double Box Culvert
Plan
N.T.S.



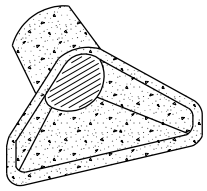

Double Box Culvert
Section A-A

Note: All spans shown are hydraulic spans.
For a definition of Bridge span refer to
the Bridge Design and Drafting Manual.

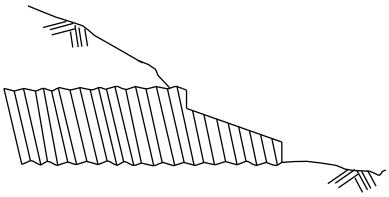
**Figure 9-1 Common Culvert Shapes**



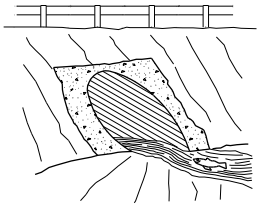
Projecting Barrel



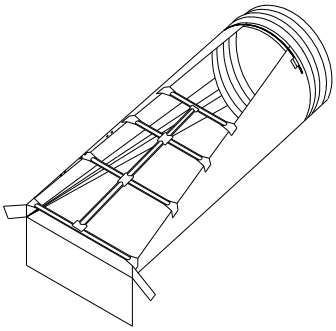
Precast Concrete End Section



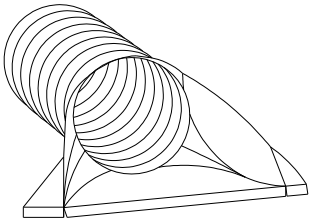
Sloped Mitered End Without Slope Paving



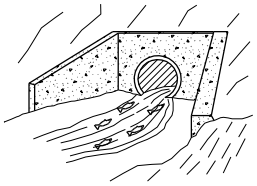
Sloped Mitered End with Slope Paving



Safety End Section
with Safety Bars



Prefabricated Sheet
Metal End Section



Cast-in-Place Concrete
Headwall and Wingwalls

**Figure 9-2 Common End Treatments**

## 9.3 Large, Medium, and Small Culverts

Culverts are classified as large, medium, or small. The type and amount of data collected for the design, the complexity of the hydraulic study, and the level of documentation depends on the culvert classification. Regardless of culvert size, they are designed by a professional engineer registered in Oregon or others under the direct supervision of the engineer. Final plans and other documents require an engineer's seal. This is discussed in **Chapter 3**.

Large culverts are:

- circular culverts with diameters more than or equal to 72 inches,
- arch-pipes with equivalent diameters more than or equal to 72 inches,
- box culverts with spans more than or equal to 72 inches,
- multiple barrel culverts with cumulative span widths more than or equal to 72 inches,
- open-bottom culverts of all sizes,
- culverts with complex hydraulic structures such as cast-in-place energy dissipators; drop, side tapered, or slope-tapered inlets, etc. and
- culverts providing fish passage regardless of size.

Medium culverts are:

- circular culverts with diameters more than or equal to 48 inches and less than 72 inches,
- arch-pipes with equivalent diameters more than or equal to 48 inches and less than 72 inches, and
- box culverts with spans more than or equal to 48 inches less than 72 inches.

Small culverts are:

- circular culverts with diameters less than 48 inches,
- arch-pipes with diameters less than 48 inches, and
- box culverts with spans less than 48 inches.

## 9.4 Sources of Information

The type, source, and complexity of data for a culvert design will vary depending on the location and type of culvert. Large culverts typically require data similar to a bridge location survey, such as a vicinity map, stream or drainage profile and cross-sections, geologic data, etc. Small and medium culverts usually require less data and most of the information can be provided by

the roadway location survey. Often information and experience from other agencies can be used to supplement the location survey. This is especially important when fish passage is a concern, as the requirements and guidelines of fish management agencies may dictate the type, size and location of the culvert. Data requirements for typical culvert designs are listed in **Chapter 6**.

## 9.5 Culvert Design Documentation

The documentation for large culverts is similar to bridges and large channel studies. Smaller and medium culverts require less documentation. Report and documentation guidelines are in **Chapter 4**.

## 9.6 Definitions

Definitions for many terms used in culvert design are in the glossary at the end of this manual.

## 9.7 Culvert Vertical Alignment

The vertical alignment of a culvert with respect to the stream is important to culvert hydraulic performance, stream stability, construction and maintenance costs, and the safety and integrity of the highway. Proper alignment is also of particular importance to promote fish passage, and to prevent outlet scour or excessive sediment buildup in the culvert barrels.

A culvert placed too high in relation to the streambed may become a barrier to fish passage if the channel bottom degrades, or lowers, during the life of the facility. Conversely, a culvert placed too low in relation to the channel bottom may lose hydraulic performance if the channel aggrades, or rises. In addition, a culvert placed at a slope different than the natural channel slope may have problems related to both sediment deposition and bed material scour, and this affects both fish passage and hydraulic performance.

### 9.7.1 Use of Streambed Profiles in Design

Accurate assessment of streambed profiles is essential to determine potential changes in channel bottom elevations. It is important to:

- determine the profile of the existing streambed, called the "existing profile,"
- estimate the profile the streambed will have immediately after the structure is constructed, called the "design profile," and
- estimate the profile of the streambed throughout the life of the structure, called the "long-term profile."

#### 9.7.1.1 Existing Profile

The existing profile is the streambed profile before construction. An example of an existing profile is shown in Figure 9-3a. A small culvert is shown in the figure. This culvert is too small to convey the stream discharges and water pools upstream from the inlet. The pooled water cannot transport bed material conveyed by the stream, and as a result, there is a mound of gravel upstream from the inlet. Excessive flow velocities in the narrow barrel have created a scour hole downstream from the outlet. The existing profile also shows two breaks in the channel grade. The upstream break is due to a rock ledge. The downstream break is caused by a log across the channel bottom.

The existing profile is determined as follows.

**Step 1 -** Obtain streambed and water surface elevations for some distance upstream and downstream from the culvert. Obtain the elevations and descriptions of control points such as dams, weirs, existing culverts, etc. This information is available from the site survey as discussed in **Chapter 6**.

**Step 2 -** Plot the profiles of the streambed and the water surface, either electronically or on paper. Note the location and elevations of all control points.

#### 9.7.1.2 Design Profile

The channel profile will almost always be altered to some degree when the new culvert is built. Possible causes of these changes are: removing a culvert and installing another in a different location, eliminating or construction of a control feature such as a weir or tide gate, smoothing out natural irregularities in the channel bottom, roughening the channel with fish rocks, wood, or other structures, or a channel change. This revised profile is the design profile. The design profile is often the basis of the profile shown on project plans. The following steps are used to determine this profile.

**Step 1 -** Obtain a copy of the existing profile. Locate the proposed structure on the profile. Note the limits of agency right-of-way or easements on the profile.

**Step 2 -** Estimate and plot the channel profile through the proposed structure. The design profile grade will match the existing stream grade on some installations. Often when a fish-passage structure is built, irregularities in the channel bottom or barriers to fish passage caused by the existing structure are removed during construction, and these changes should be shown. Show control structures such as weirs on this profile if they will be needed. Check to see that all channel re-grading and control structures can be built within agency right-of-way or easements. If not, additional right-of-way or drainage easements may be needed.

An example of construction related changes are shown in Figure 9-3b. In the figure, a longer and larger embedded culvert will replace the smaller existing culvert. It is expected that the sediment mound will be removed and the scour hole will be filled during the construction of the new culvert. As a result, the design profile is expected to have a constant grade upstream and downstream from the culvert. The rock ledge and the log across the channel are shown on the design profile because they will not be affected by construction.

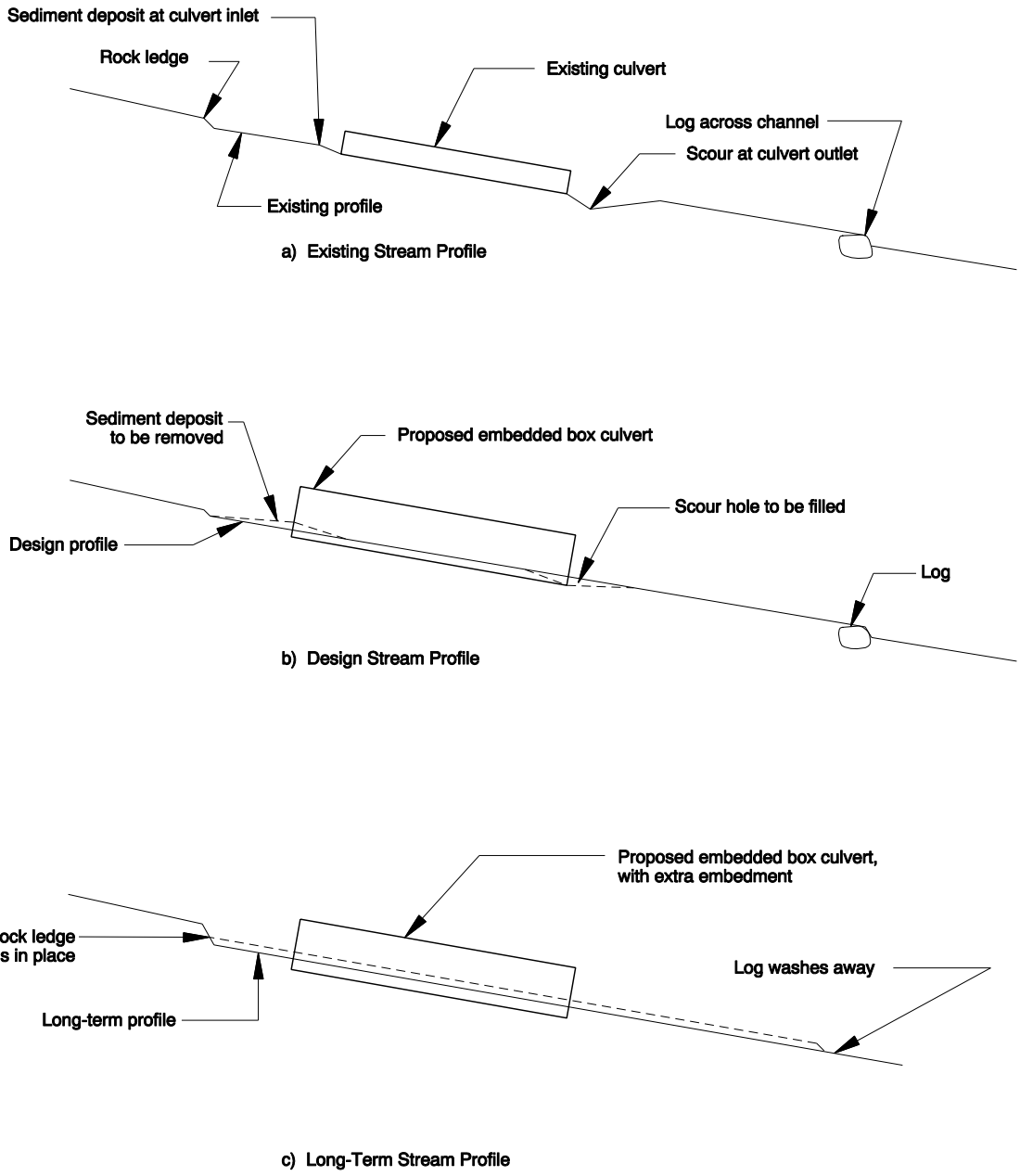



**Figure 9-3 Stream Profiles**

### 9.7.1.3 Long-Term Profile

Stream profiles often change with time. Changes in streambed profile are often called aggradation or degradation. Aggradation and degradation are the raising or lowering of the channel bottom elevation, respectively. An alteration to sediment continuity is the primary cause of these changes.

Sediment continuity is a function of sediment supply and channel transport capacity. Sediment supply is provided by the upstream drainage. It is materials carried to the site by the stream, and the greatest amount of transport occurs during floods. Sediment transport capacity is the ability of the stream to move sediment along the streambed. Streams where the sediment supply exceeds the stream transport capacity tend to aggrade. Conversely, streams degrade when the transport capacity exceeds the sediment supply. A detailed discussion of sediment continuity with a list of factors causing changes is in FHWA Hydraulic Engineering Circular No. 20 "Stream Stability at Highway Structures."

These long-term changes in channel profile can reduce the ability of the culvert to pass fish, provide adequate hydraulic performance, and be structurally sound. Channel degradation can cause embedded, non-embedded, and baffled culverts to become barriers to fish passage. This occurs when the downstream end of the culvert becomes higher than the channel bottom, and fish cannot swim upstream through the barrel. This is often called a "perched" condition. Degradation can undermine open-bottom culvert foundations. Channel aggradation can reduce the flow capacity of all culvert types.

Many factors can cause long-term changes in streambed profile. These causes are most often detected during a site visit. Typical items to investigate are in the following list.

1. Will a dam or other obstruction be built downstream that will decrease the flow velocity through the culvert? If the velocity is decreased, the ability of the stream to carry sediment will also be reduced, and the channel bed could aggrade in or near the culvert. This often occurs at the mouths of streams that discharge into newly constructed reservoirs.

2. Will a dam or other obstruction be removed upstream? The removal could be done by man, such as the demolition of a dam, or by nature, such as the rotting and decomposition of a log across the channel. If the dam or other obstruction has retained bed material, its removal or failure can increase the amount of bed material passing though the site and the channel could aggrade. Conversely, removal of a dam that provides flood control could also increase the discharge and result in degradation.

3. Will changes in land use increase runoff rates? Activities such as converting a forest into a pasture, or converting a forest or pasture to a housing or commercial development almost always increases the discharge. This frequently causes degradation.

4. Will a dam or other obstruction be removed downstream? If the obstruction has retained bed material, the retained material can be carried downstream and the channel may degrade. In addition, if removing the obstruction increases flow velocities through the culvert site, the ability of the stream to carry sediment will also be increased, and the channel can degrade.

5. Will a dam or other obstruction be built upstream from the culvert site? If the obstruction will interrupt the transport of bed material, the channel could degrade.

6. Will changes in land use change their sediment contribution? Improved erosion control in riparian corridors can reduce the amount of sediment washing into the streambed. Converting wildlands to housing or commercial developments can also reduce the sediment availability. This may result in channel degradation.

7. Will the stream be channelized upstream from the culvert? Channelization, especially straightening the channel, can increase flow velocities through the site and cause the streambed to degrade. Conversely, restoring a straightened channel to a meandering waterway can decrease flow velocities and cause aggradation.

8. Is the streambed degrading due to natural erosion? Many streambeds are degrading due to long-term erosion of the stream bottom that is caused by lack of sufficient sediment inflow into the channel (transport capacity > sediment supply). These streams are most often in “V” shaped channels on steep gradients and they probably have headcuts.

9. Are there headcuts downstream that may proceed upstream through the crossing site? A headcut is an abrupt change in channel elevation as shown in Figure 9-4a. Headcuts typically proceed upstream until they are stopped by a ledge of non-erodible rock or a structure such as a dam, a weir, or a culvert if the streambed is erodible. This migration is shown in Figure 9-4b. This type of erosion usually indicates that the streambed profile is readjusting after a change in channel slope, discharge, or sediment load characteristics. In channels with flowing water, headcutting is often shown by the presence of 12 to 18-inch high waterfalls or rapidly moving water in an otherwise placid river or stream. In dry channels, a relatively abrupt drop in the bed of an erodible channel is evidence of a headcut. Headcuts may range in depth from 1 to 9 feet or more. Niagara Falls is an example of a large headcut that is slowly progressing upstream in the Niagara River between Lake Erie and Lake Ontario.

The long-term profile is an estimate of the stream bottom elevations after long-term changes. A long-term profile is shown in Figure 9-3c. This profile shows changes that are expected in the channel profile. It is assumed the rock ledge will last for the design life of the culvert and no changes in profile are expected upstream from the ledge. If the upstream obstruction would be temporary, such as a log, the effects of removing the obstruction should be considered. One effect may be the aggrading of the channel bed when the gravel retained by the obstruction passes through the culvert.

The other long-term change in channel profile is downstream from the culvert. It is assumed the log is a temporary obstruction and it will rot or wash away at some time during the design life of the culvert. If the obstruction was more substantial, such as a rock ledge, it would be considered permanent and the effects of its removal would be ignored. An estimate of the stream profile after the log rots away is shown in Figure 9-3c. It is anticipated that the sediment retained by the log will be carried downstream and the resulting degradation will remove most of the cover over the culvert invert. As a result, a deeper box culvert is chosen and the invert will be buried an extra amount. This way, there will be adequate cover over the invert of the box throughout the design life of the culvert.

The long-term profile is typically determined using the following steps.

**Step 1 -**Obtain a copy of the design stream profile including the proposed structure. Investigate possible long-term changes to the profile caused by the removal or addition of control structures, such as weirs, dams, logs, etc. The preceding list of factors can be used. Plot estimated profile changes on the design profile.

**Step 2 -**Estimate the changes to the profile that might occur if known headcuts proceed upstream through the culvert site. Plot these changes on the profile.

**Step 3 -** Estimate long-term changes, if any, to the entire stream profile. These changes could be caused by aggradation or degradation throughout the reach. These changes are often hard to estimate with any accuracy. Plot these changes on the profile. HEC-20 is recommended as a reference.

*Note: Long-term channel changes can be difficult to quantify with any certainty. This is especially true of aggradation. Culverts are sometimes designed with extra clearance over the streambed and extra embedment depth so they will function under a variety of sediment continuity conditions.*

**Step 4 -** Correlate the design data to the existing field conditions if the design is for a replacement culvert. Take into consideration any aggradation, degradation, outlet scour, outlet perching, debris problems, etc. Change as needed the type, size, or location of the proposed structure to assure that it will continue to function after the changes occur in the long-term profile.

### 9.7.2 Culvert Invert Slope Versus Streambed Slope

The culvert invert slope should match the stream bed slope. Placing the culvert on a flatter or steeper gradient than the natural streambed can cause sediment deposition in the barrel. It can also cause scour that removes sediment from the barrel. This is especially critical for fish passage culverts designed to have a sediment covered invert. Typical areas of deposition and scour are shown in Figure 9-5. Placing a culvert at a slope other than the natural streambed slope should be avoided. The following steps are recommended if it cannot be avoided.

**Step 1 -** Estimate the streambed profile through the culvert after it attains its expected long-term profile, including any deposition or scour.

**Step 2 -** Check to assure the culvert will have adequate hydraulic capacity with the profile determined in the previous step. The most common constriction point is the upper end of the barrel when culverts are installed at flatter slopes than the streambeds, as shown in Figure 9-5a. The constriction point can be at the lower end when the barrel is installed at a steeper slope than the streambed, as shown in Figure9-5b. Sometimes a taller or wider barrel will be needed.

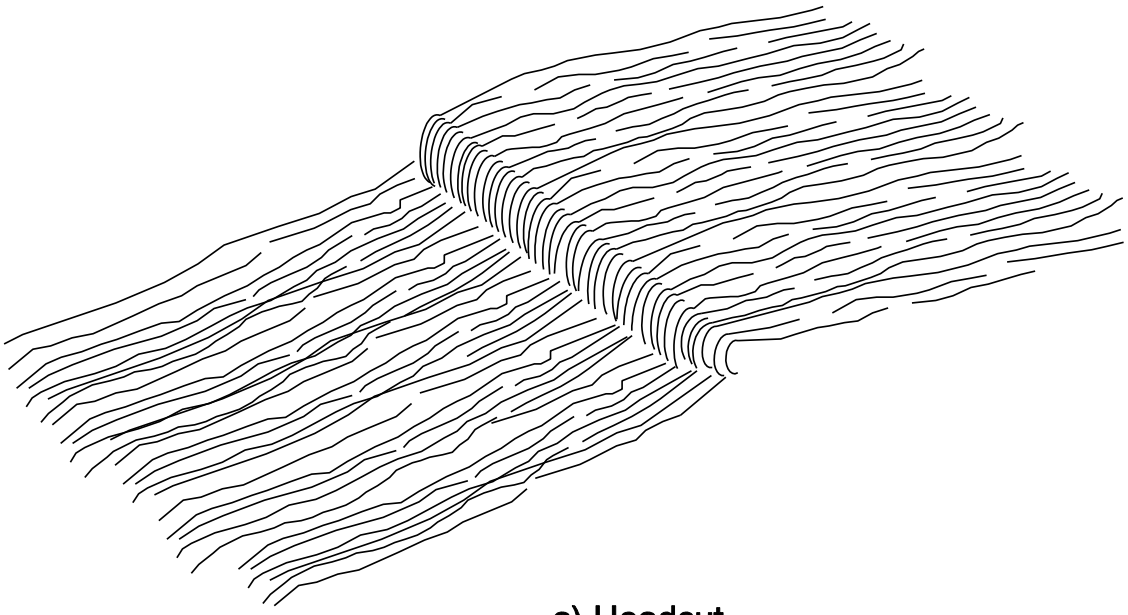
**Step 3 -** If the culvert is installed on a steeper slope than the stream, check to assure the upstream end of the culvert will retain a cover of bed material. Scour of bed material covering the upstream end is a common problem with culverts at slopes steeper than the natural channel. Scour of bed material throughout the barrel can be a problem with longer culverts, as shown in Figure 9-5c. It may be necessary to provide bed retention weirs within the culvert barrel. Consider the effects of these weirs on fish passage.

**Step 4 -** Periodic cleaning may be needed at some installations if a large enough barrel cannot be provided. This cleaning should be described in the Hydraulics Report. Approval from ODOT maintenance should be obtained before a culvert is installed that will require periodic cleaning.

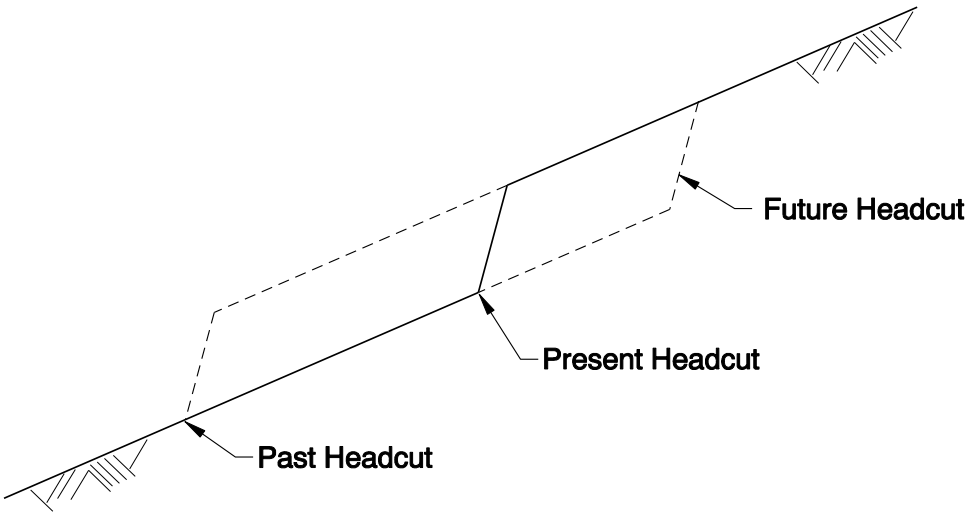
**Step 5 -** Provide access for maintenance forces if cleaning will be needed. This may include easements, a turnout for vacuum cleaning trucks, access roads, etc.

**Step 6 -** Verify the environmental documents describing the project impacts mention the need for periodic cleaning. Permits are often needed for cleaning.

**Step 7 -** Consider an overflow culvert if the subject culvert is susceptible to plugging due to deposition.



a) Headcut




b) Headcut Progressing Upstream

**Figure 9-4 Changes in Channel Profile Due to Headcuts**

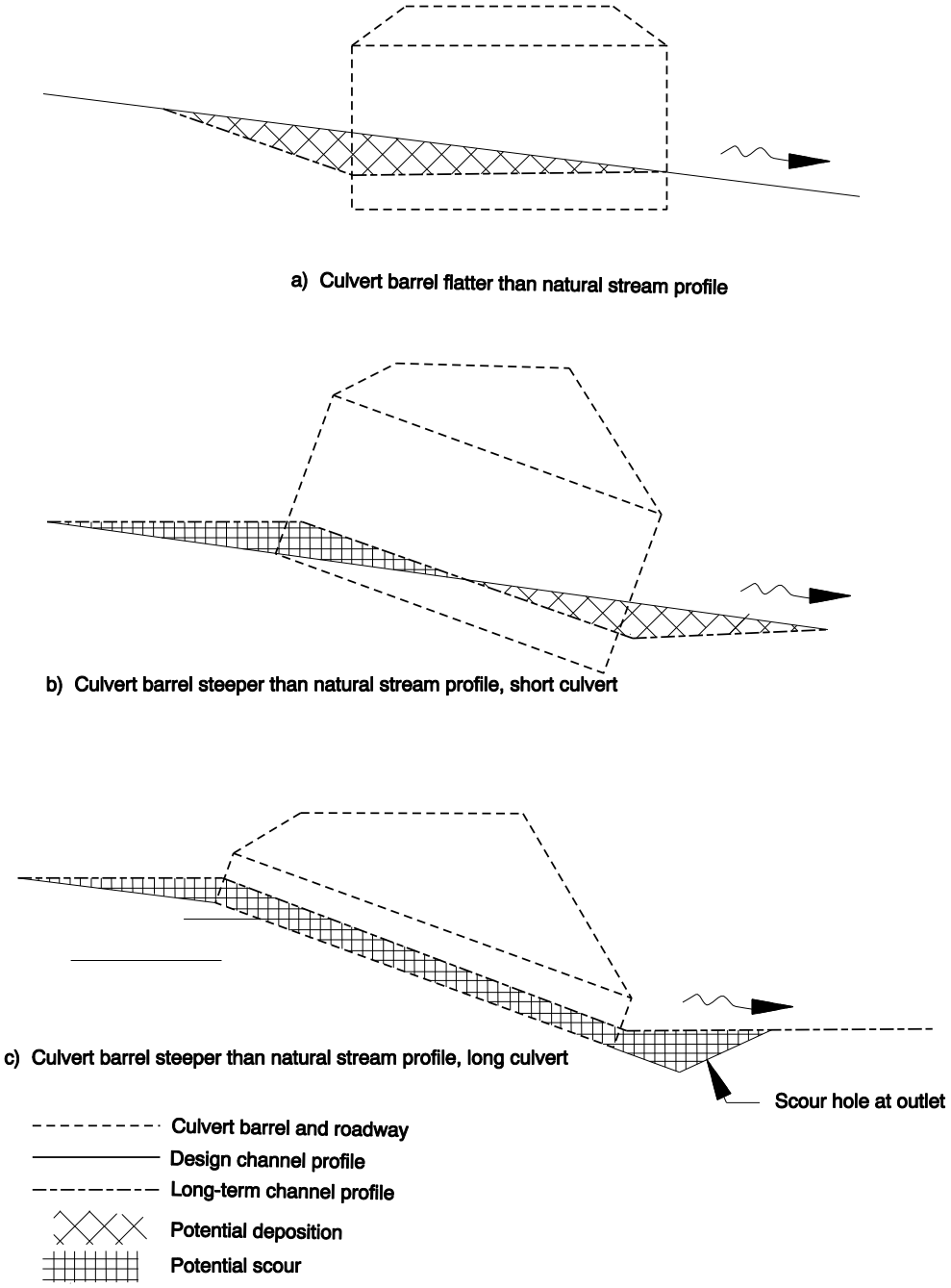



**Figure 9-5 Culvert Vertical Alignment**

## 9.8 Culvert Horizontal Alignment

The culvert horizontal alignment should match the natural stream alignment, as close as practical. This is shown in Figure 9-6a. This is often possible when installing an original culvert at a new crossing or when removing the existing culvert and replacing it with another at exactly the same location.

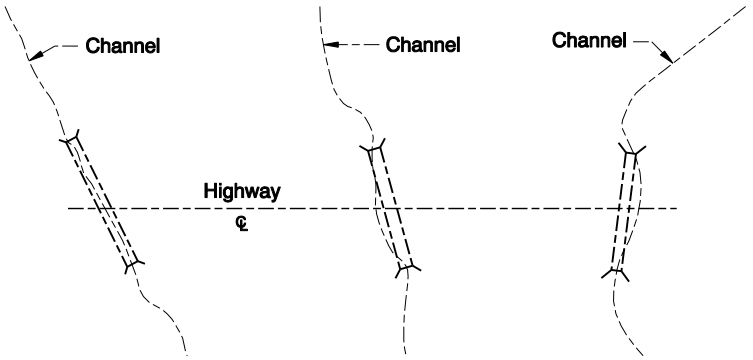
Replacement culverts frequently cannot be installed at exactly the same location as the existing pipes and channel changes are needed. In many cases, the existing barrel must convey flow while the new culvert is being built and it must remain in place during the construction of the new culvert. Short sections of modified channel are needed to route the stream to the new culvert. In other instances, the existing culvert location is not ideal for the relocated or widened road. Stream channel changes are needed to connect the stream to the relocated culvert. These culvert relocations and channel realignments must comply with applicable fish passage and other environmental requirements.

Severe or abrupt changes in channel alignment such as bends should be avoided immediately upstream or downstream of the culvert ends because they create head losses and reduce the culvert hydraulic efficiency, as shown in Figure 9-6. In addition, and of greater concern, a sharp bend in the channel immediately upstream and downstream from the culvert ends promote bank erosion on the outside of the bend and sediment deposition on the inside of the bend. This scour and erosion can adversely affect the culvert structural integrity and hydraulic performance.

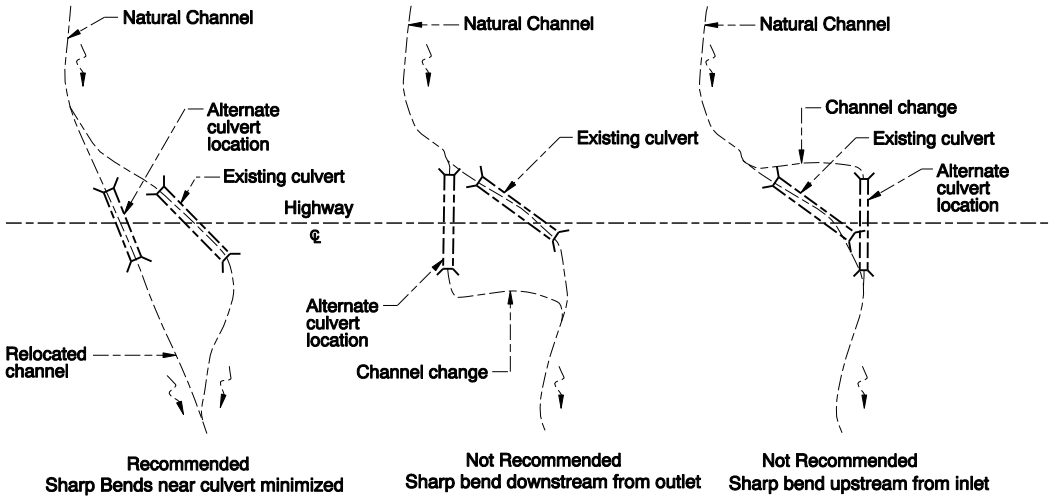
### 9.8.1 Alignment, Bent Culverts

Straight culverts matching the channel alignment are preferred. Straight culverts cannot be used in all situations, and matching the alignment of a sinuous channel may require a culvert with one or more bends. Design of a culvert with a bend should consider the following.

1. Debris passage. If long debris such as branches or trees are expected to pass through the culvert, these objects must pass through the bent section. Passing long debris may require extra culvert width or long bend radii. Access to the angle points or bends should be provided by a manhole or other means if debris can lodge at these locations. Use of a larger culvert that would not need a bend or a bridge may be the best option at debris prone sites.




a) Culvert located in natural channel




b) Culvert with Channel Change

**Figure 9-6 Culvert Location**

2. Hydraulic efficiency. A bend may reduce the hydraulic efficiency of a culvert, and the more sudden and sharper the bend, the greater the loss in efficiency. Bend hydraulic losses do not affect culverts under inlet control. They may significantly reduce the hydraulic performance of culverts under outlet control. Smooth large radius bends are preferred if bends are needed. They have a less adverse effect on hydraulic performance.

3. Scour. A bend increases flow velocities inside the barrel at the outside of the bend and downstream from the bend. The increased scour caused by these high velocities should be considered in the scour depth calculations and the footing designs of open-bottom arch culverts and 3-sided boxes. The locations of the higher shear stresses in bends are discussed in **Chapter 8**. The design of open-bottom culverts and 3-sided boxes is discussed within this chapter.

### 9.8.2 Alignment, Multiple Barrel Culverts and Multiple Culverts

Single barrel culverts are preferred where they can be used. Multiple barrel culverts or multiple culverts may be used where limited vertical clearance between the roadway and the streambed or other reasons preclude use of a single barrel culvert. A double or triple barrel reinforced concrete box (RCBC) is an example of a multiple barrel culvert. Two or more separate corrugated metal pipes at a single site are an example of a multiple culvert. Alignment of either culvert type should consider the following.

1. Barrel spacing. The separate barrels of multiple culverts should be spaced apart in accordance with ODOT Standard Drawing RD300. This will ensure sufficient space is available to compact the backfill. The barrels can be spaced closer if the area around the bottom half of the barrel is backfilled with concrete.

2. Channel characteristics. Installation of multiple barrel culverts or multiple culverts in channel bends or irregular channels may result in sediment deposition in one of the barrels. These problems can be minimized by relocating the culverts so they are in a straight reach of channel with a regular cross-section. Sometimes, with multiple barrel RCBCs, a sill is constructed across the apron to direct low flows into one barrel. A similar method is used with multiple culverts. One barrel is placed lower than the rest to handle the low flows and also to minimize potential sediment problems.

## 9.9 Alignment, Conflicts with Utilities

Culvert alignment, both vertical and horizontal, is often affected by utilities because many culverts are located in the vicinity of existing or future utilities, such as power transmission towers, sewers, fiber optic cables, etc. ODOT often has given the utility an easement to locate on ODOT right-of-way, and part of the easement stipulates that the utility must be relocated if it conflicts with the ODOT facilities. ODOT can request that the utility be relocated in these cases. In other instances, ODOT may have to locate its facilities to keep them away from the utility or pay for the utility to be moved. This most often occurs where the utility is on private property. ODOT region utilities personnel are responsible for utility location and negotiations with the utility owners.

The utility cannot always be relocated so it is far from the culvert. Instead, it will be near or adjacent to the structure. There are recommended minimum clearances between the utility and the culvert, in some instances. The utility company can often provide information about these clearances. The clearances can be reduced, or sometimes eliminated, if casings or other protection measures are installed around the utility.

Utility corridors should also be considered if they are present. This could be an unoccupied corridor reserved for future utilities or space for additional utilities near existing utilities. An example would be a community that has overhead phone and electrical lines and intends to relocate the lines underground. Corridors may be reserved for the relocation.

Precise utility location can be expensive because it requires detailed surveys. In some instances, excavation is needed to determine the exact utility location. As a result, preliminary culvert design is often done using limited utility data, a comprehensive utility location survey is made, and the detailed utility data is used in the final design. In all cases, the final culvert design should not be made until affected utilities are located. Unanticipated utility conflicts are almost always time consuming and expensive when they are resolved during construction.

## 9.10 Alignment, Construction Considerations

Often the anticipated construction method affects the culvert alignment. There should be at least one possible method of constructing the culvert, and the construction procedure should be considered when the culvert is designed. The roadway designer is a useful source of information about construction methods and constraints. As an example, the need to maintain the flow of traffic through the project may require special construction considerations such as trenchless excavation or stage construction.

Environmental concerns such as fish passage may also influence construction methods, construction sequence, and as a result, culvert alignment. The ODOT region environmental representative should be contacted about any requirements. Typical methods of constructing culverts are summarized below.

**Trenchless Installation –** Installing culverts in open trenches across the roadway is often impractical where the excavations are deep and traffic cannot be interrupted or detoured away from the trench location. There are several methods of installing culverts under highways without excavating trenches. These trenchless construction methods include auger boring, slurry boring, pipe jacking, pipe bursting, microtunneling, horizontal drilling, and pipe ramming. These methods are discussed in detail in **Chapter 16**.

The new pipe can be installed in exactly the same location as the existing culvert using pipe bursting and jacking. With the pipe bursting method, the existing pipe is destroyed and the new pipe is pushed or pulled through the old pipe using a pipe bursting head. With pipe jacking, a larger culvert is pushed into the embankment as a sleeve over a smaller culvert and the smaller pipe is removed in sections from the inside of the larger pipe.

The replacement culvert cannot be installed on the same alignment as the new pipe using the other trenchless methods. The existing pipe will interfere with the new pipe installation. An exception is pipe jacking. It can be used on new as well as existing alignments.

Regardless of the chosen method, the new pipe alignment must consider that heavy equipment will be needed at one end of the culvert to install the pipe, and this equipment may require a large flat dewatered pad at an elevation equal to or slightly lower than the streambed. In addition, if the culvert is to be installed on a flowing stream, the water must be kept away from the trenchless excavation operation. Usually the stream continues to flow through the existing culvert during construction and it is diverted to the new culvert after it is built.

**Trench Excavation with Detour or Road Closure -** Another method of building a culvert is to close the road to traffic and simply dig up the old culvert and install the new culvert. This method provides the most latitude for horizontal and vertical culvert alignment changes.

Two construction methods are commonly used if the culvert is located on a flowing stream. One procedure is to dig a trench alongside the existing culvert, place a bypass pipe in the trench, divert flow to the bypass pipe, remove and replace the existing culvert, divert flow back to the new culvert, remove the bypass pipe, and rebuild the road embankment. With this procedure the old and new culverts can be in the same location. The other procedure is to excavate and build the new culvert, divert streamflow from the old culvert to the new culvert, remove the old culvert (or fill the old barrel with concrete or grout and leave it in place), and rebuild the road. Using this method the old and new culverts have different alignments.

**Trench Excavation with Stage Construction –** With this method the highway remains open to traffic and the new culvert is built in the same or different location than the old culvert. This procedure is illustrated in Figure 9-7 and it requires careful planning. To summarize the procedure, a long bypass pipe is placed inside the existing culvert and flow is diverted into the bypass pipe. Then a temporary fill and roadway is built alongside the existing road and over one end of the bypass pipe. Traffic is diverted to the temporary road, and a portion or the entire existing culvert is removed and replaced. Last, traffic is rerouted over the new section of culvert, the temporary roadway is removed, the remainder of the old culvert is removed and replaced, and the bypass pipe is removed.

There must be sufficient clearance inside the new culvert for both the bypass pipe and construction activity if the stage-construction method is used. For example, if a structural plate pipe-arch is to be built, there must be enough room in the new culvert for both the bypass pipe and the workers bolting the culvert together.

## 9.11 Fill Heights

An important factor in culvert sizing and vertical alignment is the distance between the surface of the subgrade or the pavement and the culvert crown. This is typically called “cover thickness” or “fill height.” Fill height limitations often influence the choice of the culvert barrel material or the maximum diameter or rise of the culvert.

At a minimum, there should be enough fill over the culvert to dissipate the wheel loads from construction activities or traffic. Minimum cover is measured from the subgrade surface to the pipe crown. This distance is usually at its minimum at the roadway shoulder over the upstream end of the pipe but should be checked over the length of the culvert.

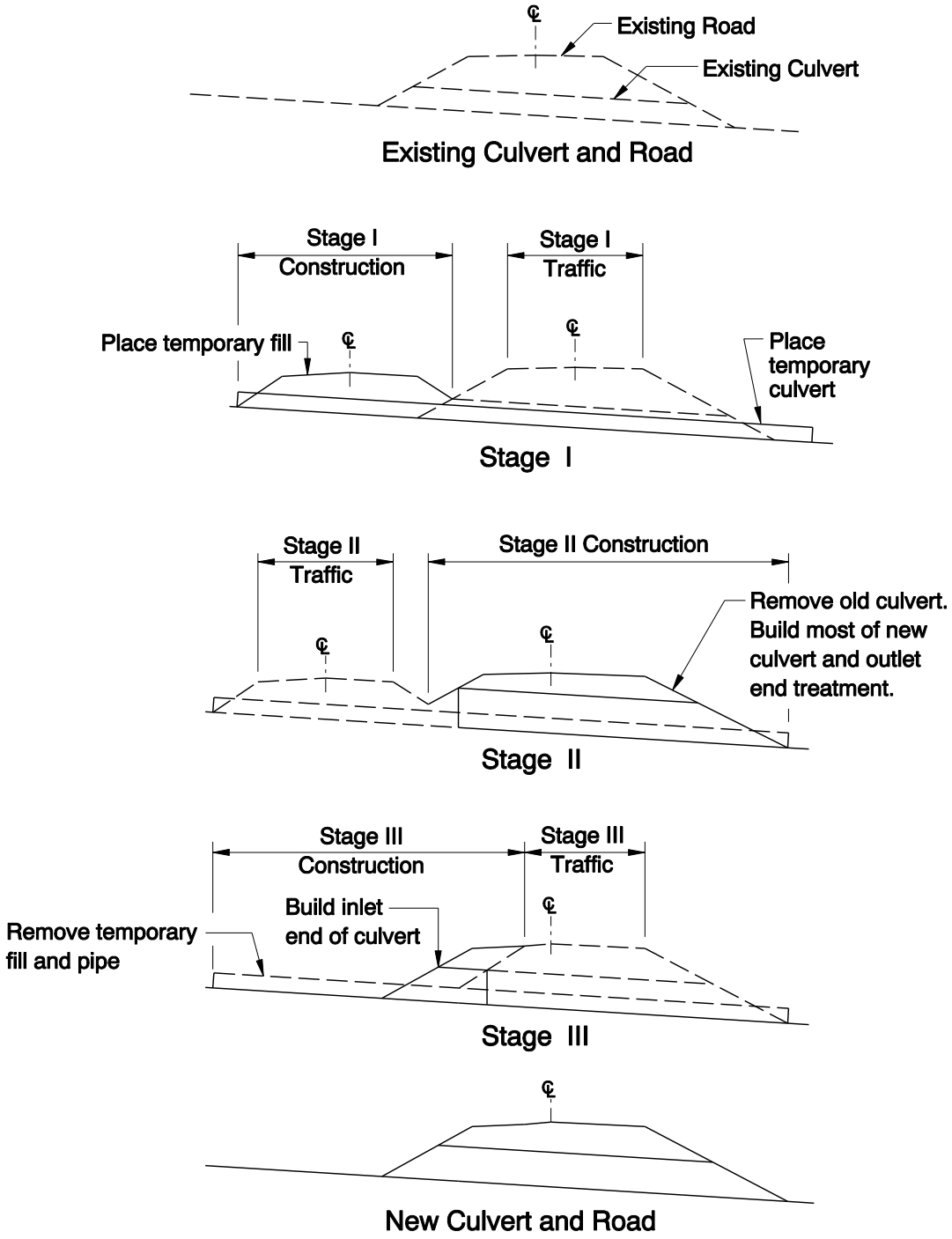



**Figure 9-7 Stage Construction**

At a maximum, the weight of the fill over the culvert should not be great enough to damage or distort the culvert. Maximum cover is measured from the finished road surface to the pipe crown. It is usually greatest at the roadway shoulder over the downstream end of the pipe but should be checked over the entire length of the culvert.

Fill height tables for metal pipes, concrete pipes, corrugated polyethylene pipes, and cast-in-place concrete box culverts are in the ODOT Standard Drawings. Fill heights for precast box culverts are listed in the American Association of State Highway and Transportation Officials (AASHTO) "Standard Specifications for Transportation Materials." Fill heights for polyvinyl chloride plastic pipes are in **Chapter 5**. Fill heights for the various bottomless arch and 3-sided box culverts are in the trade literature. Methods to protect pipes having minimal cover, and procedures to protect pipes from damage due to high fills, are discussed in **Chapter 5**.

Guardrail support is not a factor in calculating cover heights in most circumstances. There are several rail types that do not have posts projecting into the fill directly over the culvert barrel. Post penetration should be considered in fill height calculations if posts cannot be avoided. Guardrail posts can project 3-1/2 to 4-1/2 feet into the roadway. Exact post and guardrail dimensions are in the ODOT Standard Drawings.

## 9.12 Minimum Culvert Size

The minimum culvert size is the larger of the following:

- the minimum size allowed by ODOT design criteria, including fish passage and environmental criteria,
- the minimum size that produces an allowable headwater elevation (Section 9.17),
- the minimum size that allows access for maintenance , if needed,
- the minimum size that will allow passage of stream bed material, and
- for bottomless culverts, the minimum size that results in acceptable scour depths.

**ODOT Design Criteria -** ODOT design criteria specify minimum culvert sizes at many locations, as described in the following list.

- The minimum diameter for median culverts shall be 18 inches unless they are connected to a grated inlet or catch basin, in which case, the minimum diameter shall be 15 inches.
- Culvert pipe under roadway approaches shall have a minimum diameter of 12 inches. These roadway approaches are driveways, minor access roads, etc. Roadway culvert criteria apply to culverts under roadways that intersect the main highway.
- The minimum diameter for roadway cross-culverts shall be 18 inches. The minimum diameter for installations longer than 150 feet or under more than 15 feet of fill shall be 24 inches. Consideration should be given to using a 36-inch minimum diameter on interstate

routes.

**Clearance for Maintenance –** Culverts with buried inverts on waterways where fish passage is required should provide clearance for channel maintenance such as removing obstructions or replacing lost bed material. Culverts with fish baffles or weirs should provide clearance for inspection, cleaning, and repair. Open-bottom arches or 3-sided boxes should provide clearance for cleaning, footing inspection, footing repair, and renewal of revetment protecting the footings, as discussed in Subsection 9.22.3. ODOT maintenance should be contacted for preferred horizontal and vertical clearances. Maintenance usually requests an 8-foot by 8-foot opening.

**Passage of Streambed Material –** Almost all ODOT culverts intended to pass fish are designed to retain a layer of bed material over their invert. They are also aligned and sized to minimize scour at the outlet and bed material deposition at the inlet. This is done by providing adequate hydraulic capacity for sediment continuity. Sediment continuity is a balance between the bed load the stream transports into the culvert during floods versus the amount of bed material carried out of the culvert during the same flood. The concept of sediment continuity is discussed in the FHWA publications Hydraulic Design Series No. 6 "River Engineering for Highway Encroachments" (HDS#6) and Hydraulic Engineering Circular No. 20 "Stream Stability at Highway Structures" (HEC#20). Culvert design using the principles in HDS#5 and HEC#20 is beyond the scope of this chapter. At present, these guidelines can also be used to determine the minimum culvert span needed to pass streambed material.

**Case 1 –** Span the waterway, or channel width, that conveys the Ordinary High Water (OHW). OHW is defined in **Chapter 6**. In almost all instances the majority of sediment is transported within this waterway. This minimum span length is best suited for culverts meeting all of the following:

- culverts on straight stream reaches,
- culverts well aligned to the flow,
- culverts that do not go under pressure flow during the design flood,
- sites where most of the discharge is within the channel during floods,
- sites where the stream is stable in both vertical and horizontal alignment,
- sites where the hydrology can be determined with a fair degree of certainty, and
- sites without a history of problems related to excessive bed load or debris transport.

**Case 2 –** Span 125 percent of the width of the waterway that conveys the Ordinary High Water plus two feet. This is a minimum waterway width criteria used successfully by other agencies. It is a larger waterway than Case 1, and it is best suited for culverts or sites having any one of the following characteristics:

- culverts on curved stream reaches,
- culverts that cannot be well aligned to the flow,
- sites where there is considerable overbank flow during floods,

- sites where the stream may not be stable in either vertical or horizontal alignment,
- sites where there is greater than normal uncertainty in determining the hydrology, and
- sites with a history of problems related to excessive bed load or debris transport.

These are general guidelines. Regulatory agency guidelines may supercede these criteria in some circumstances.

## 9.13 Barrel Materials

Culvert barrels are made from a variety of materials. Barrel material selection is based on hydraulic and structural considerations, corrosion and abrasion resistance, the design life of the facility and the service life of the culvert, local government preference, and the need for experimental installations. Guidelines for barrel material selection follow.

1. Materials must comply with requirements in the ODOT "Alternate Materials Policy for Pipe Materials," as discussed in **Chapter 5**.

2. The barrel material must have an adequate design life for the specific site conditions. Special considerations are needed for pipes installed in corrosive conditions or culverts carrying abrasive sediments. Culvert design lives are listed in **Chapter 5**.

3. Culverts designed for fish passage must comply with applicable requirements.

4. Concrete pipe or box sections should not be used without pipe anchors on slopes steeper than 4 Horizontal: 1 Vertical because differential settlement may open the joints. Cast-in-place reinforced concrete structures are acceptable on these steep slopes.

Culvert barrels should be standard sizes whenever possible. Many circular, pipe-arch, and semi-circular arch sizes are listed in Chapter 8 Appendix B and the fill height tables in the ODOT Standard Drawings. Many cast-in-place reinforced concrete box sizes are listed in the ODOT Standard Drawings. Precast concrete box culvert sizes are listed in Chapter 8 Appendix B and the American Association of State Highway and Transportation Officials (AASHTO) "Standard Specifications for Transportation Materials." Sizes of smooth wall polyvinyl chloride pipe are listed in **Chapter 5**. Sizes of the various non-semicircular open-bottom arches and 3-sided boxes are listed in the trade literature.

## 9.14 End Treatment

The choice of end treatment depends on many interrelated and sometimes conflicting considerations. The designer must evaluate safety, aesthetics, the ability to pass debris, hydraulic efficiency, scouring, and economics. In addition, the end treatment should be consistent with ODOT design criteria unless the exception is approved by the ODOT Geo-Environmental Section's Engineering and Asset Management Unit. This subsection describes many commonly used end treatments, their advantages and disadvantages, and ODOT design practice.

**Projecting Ends** - A projecting end is a treatment where the culvert protrudes out of the embankment, as shown in Figure 9-2. This end treatment is seldom used on permanent installations, especially with large culverts. Its most common use is on temporary detour pipes and bypass pipes for temporary water management. Projecting ends have these advantages.

- They are the most simple and economical end treatment.
- The inlet edge of a projecting metal culvert has a ring structure with more bending resistance than a non-reinforced sloped end.

Projecting ends have these disadvantages.

- They produce more inlet and outlet head losses than other end treatments because they provide no flow transition into or out of the culvert. An exception is the socket or groove end of a concrete pipe.
- From an aesthetic standpoint, projecting ends may not be desirable in areas exposed to public view.
- They have a poor ability to pass debris. Material carried by the stream, especially woody debris, can snag on the inlet edge.
- Metal pipes with projecting ends are especially vulnerable to uplift from buoyant forces.
- Although the end of a projecting culvert offers some bending resistance, it is vulnerable to damage and collapse if flows carry debris and the culvert is operating under a high headwater. Reinforced sloped ends or headwalls with wingwalls offer greater bending resistance.
- Projecting ends do not have cutoff walls to protect the culvert end or embankment from being undermined by scour. The segments near the ends of concrete pipes with projecting ends are especially vulnerable to separation at the joints if there is embankment scour.

ODOT practice for projecting ends is as follows.

- Projecting ends cannot be used for circular culverts with diameters of 72 inches or greater or the equivalent pipe arch size, or semi-circular bottomless arches with spans of 72 inches or greater.
- Projecting ends are not allowed for culvert ends in unshielded areas within the clear zone,

approach road culverts in cut ditch sections, or on shallow flat sloped embankments five feet or less in height.
- Projecting ends are not allowed on outward helical spiral rib pipe.
- Projecting ends are not suitable for many locations as discussed in this subsection.

**Sloped Ends -** Sloped ends with the barrel ends cut to match the fill slopes are the standard end treatment for circular, pipe-arch, and bottomless metal arch culverts. These ends are often called mitered ends, as shown in Figure 9-2. Details are shown on Standard Drawings RD318 for concrete pipes, and RD316 for metal and plastic pipes.

Slope paving is a rectangular shaped section of reinforced concrete fabricated around the sloped end. It reinforces the end against bending, provides scour protection, and makes it easier to mow the embankment around the culvert end. ODOT Standard Drawing RD 320 shows slope paving on concrete and metal pipes.

Reinforced concrete collars are a type of slope paving that fits around the upper half of the pipe in a ring shape, rather than the rectangular shape associated with slope paving. The collar around the lower half of the pipe resembles slope paving, and like slope paving, a cutoff wall is included. This end treatment provides most of the benefits of slope paving with a savings in material and fabrication costs. Concrete collars are rarely cost-effective for smaller pipe sizes. When collars are considered during design, it is most often for circular culverts of 66-inch diameter or larger or the equivalent pipe arch size.

Advantages of sloped ends are as follows.

- They are simpler and less expensive than headwalls with wingwalls.
- In the case of metal culverts, they are more hydraulically efficient than projecting ends.
- They are more aesthetically pleasing than projecting ends.
- They are less of a hazard to traffic than projecting ends. For this reason the safety end section version of the sloped end is used in many locations. Safety bars can be fitted to the safety end sections to increase the chances that vehicles will pass over the inlets without damage or injury to the driver. This end treatment is shown in Figure 9-2 and details are shown on Standard Drawings RD324 for concrete pipe and RD322 for metal pipe. Safety end sections with paved end slopes are not shown on the standard drawings. If a paved end slope is needed for an inlet with safety bars it resembles the paved end slope shown on Standard Drawing RD320.

Disadvantages of sloped ends are:

- The inlet edges of sloped ends on metal culverts are especially susceptible to bending and uplift caused by high flows, collisions with debris, or contact with maintenance equipment. This is especially true for ends with flatter slopes and non-reinforced skewed ends on open-

bottom arches or 3-sided boxes.

- Sloped ends of both metal and concrete culverts can be undermined by scour. To prevent this damage, sloped ends are often protected by cutoff walls on both ends, as shown on Standard Drawing RD320. In addition, if concrete pipe is used, the sections of pipe near the ends are linked with tie bars as shown on ODOT Standard Drawing RD318.
- Sloped inlets on vegetated slopes are often hard to locate and mow around. To prevent these problems slope paving is required in some applications, as discussed in the remainder of this subsection.

ODOT practice for sloped ends within the clear zone that are not shielded from traffic is as follows.

1. Culverts parallel to the roadway centerline under driveways, road approaches, etc.

    a. Pipe diameters less than or equal to 24 inches and posted speed more than or equal to 45 miles per hour, provide 1V: 6H sloped end or safety end section. No safety bars are required.

    b. Pipe diameters less than or equal to 24 inches and posted speed less than 45 miles per hour, provide 1V: 4H sloped end or safety end section. No safety bars required.

    c. Pipe diameters more than 24 inches and posted speed more than or equal to 45 miles per hour, provide 1V: 6H safety end section with safety bars.

    d. Pipe diameters more than 24 inches and posted speed less than 45 miles per hour, provide 1V: 4H safety end section with safety bars.

    e. Multiple pipes with diameters more than or equal to 15 inches, provide safety end sections with safety bars, using above criteria.

2. Cross-culverts:

    a. Pipe diameters less than or equal to 36 inches, provide either a sloped end to match embankment slope, or a 1V: 6H or 1V: 4H safety end section. No safety bars are required. The embankment slope should be warped and shaped to match the safety end section

    b. Pipe diameters more than 36 inches, provide either a sloped end with safety bars to match the embankment slope, or a 1V: 6H or 1V: 4H safety end section with safety bars.

    c. Multiple pipes with diameters more than or equal to 36 inches, provide either a sloped end, with safety bars to match embankment slope, or a 1V: 6H or 1V: 4H

safety end section with safety bars.

*Note: Provide safety bars at the inlet if there are safety bars at the outlet, regardless of whether or not the inlet is in the clear zone.*

Additional ODOT design practice for sloped ends is as follows.

- Sloped ends are the standard end treatment for circular, pipe-arch, and metal arch culverts with diameters of 72 inches or greater, or the equivalent pipe-arch or arch size. Both ends will be sloped if this treatment is used.
- Reinforced concrete slope paving or collars are required on sloped ends of culverts with diameters equal to or greater than 72 inches. A cutoff wall with a depth of at least 36 inches is required on both ends.
- Slope paving on culverts with diameters less than 72 inches should include a cutoff wall on both ends with a minimum depth of ½ the barrel diameter or 1 foot, whichever is greater.
- Sloped ends with slope paving or safety end sections with slope paving are required on slopes that are 1V: 3H or flatter that will be mowed, or on the main roadways and interchange ramps of interstate highways. Slope paving will be used on both ends of the culvert if it is used. Contact the appropriate District Manager to determine if the slope will be mowed.
- Sloped ends or safety end sections are required on all approach road culverts in cut ditch sections or shallow flat sloped embankments, five feet or less in height.
- Sloped ends on plastic pipes are allowed only in locations where slope paving is required.
- Skew cut sloped ends are not permitted for arch-type pipes. The roadway slopes will be contour-graded to conform with the pipe as shown in detail "Alternate Skew Plan," Standard Drawing RD320, for arch-type pipes on a skewed alignment with sloped ends.

**Headwalls with Wingwalls -** Headwalls with wingwalls are the standard end treatment for box culverts and they are occasionally used on circular, pipe-arch, and bottomless culverts. Headwalls and wingwalls for circular pipes and box culverts are shown on Figure 9-2 and ODOT Standard Drawing BR800, respectively. The upstream headwall has a beveled edge to improve hydraulic efficiency. Beveled edges can also be incorporated into the headwall to improve the efficiency of circular pipes, arch-pipes, bottomless arches, and other shapes. This end treatment has these advantages.

- It is one of the most hydraulically efficient end treatments and the cutoff walls protect the culvert ends from undermining due to scour.
- The weight of this end treatment holds the culvert down and it can prevent buoyant failure.
- They are very efficient at passing debris.

A disadvantage of this end treatment is its relatively high complexity and cost as compared to other end treatments.

ODOT design practice for headwalls with wingwalls is as follows.

- Headwalls with wingwalls are an optional end treatment for culverts. Both ends will have headwalls with wingwalls if this option is used. Culverts with diameters equal to or larger than 72 inches should have a cutoff wall with a minimum depth of 36 inches. Culverts with diameters less than 72 inches should have a cutoff wall extending downward the greater distance of ½ the barrel diameter or 1 foot.
- Headwalls with wingwalls are not to be used for culvert ends in unshielded areas within the clear zone, approach road culverts in cut ditch sections, or on shallow flat sloped embankments five feet or less in height.

**Concrete Boxes with Grates -** Concrete boxes with grates are often used on small culverts draining medians, swales, gutters, depressions in paved surfaces, or roadside ditches. Details of these boxes are shown on Standard Drawings RD364, RD366, RD368, RD370, RD374, and RD378. Applicable safety standards apply to these ends within the clear zone.

**Prefabricated End Sections -** Prefabricated end sections such as precast end sections for concrete pipes and flared sheet metal ends for metal pipes are shown in Figure 9-2. In many cases, these prefabricated end sections look better and are more hydraulically efficient than projecting or sloped ends. Prefabricated metal end sections have cutoff walls to resist undermining due to scour. Prefabricated concrete end sections, however, do not have cutoff walls and they are vulnerable to scour unless a cutoff wall is constructed under the edges of their aprons. A cutoff wall should extend downward to the greater of ½ the pipe diameter or 1 foot. Although a cutoff wall is the best protection against scour, riprap around the end is also an option.

There are limitations and conditions on prefabricated end section use. These ends, with the exception of the safety end sections, are not approved by ODOT for use within the clear zone in locations unshielded from traffic. They may be used in areas shielded from traffic, only. In addition, ODOT does not have standard plans for the pipe connection details and installation of prefabricated end sections. As a result, plan details are prepared for each installation. Prior approval should be obtained from the Region Technical Center staff before these end sections are included in project plans.

**Depressed and Tapered Inlets -** A variety of depressed and tapered inlets are used to increase the capacity of a culvert flowing under inlet control. Three types of these inlets, the depressed, the side-tapered, and the slope-tapered inlets are shown in Figure 9-8. Outlet end treatments on culverts with these inlets are usually headwalls with wingwalls. The hydraulic design of these inlets is beyond the scope of this manual and it is covered in the FHWA's HDS #5 "Hydraulic Design of Highway Culverts." These inlets have these advantages.

- The depressed inlet is most often used to reduce the barrel size or lower the headwater elevations to an acceptable level. This can be done because of this inlet has greater hydraulic efficiency. In comparison to a conventional inlet, for a given headwater elevation the depressed inlet provides greater headwater depth above the control section at the

upstream end of the barrel (inlet control only). This increased headwater depth produces additional pressure to push the flow through the barrel.

- The depressed inlet provides a drop in elevation between the upstream edge of the inlet apron and the culvert invert. This may be useful if a lower barrel is needed to provide clearance for utilities or pavement. In addition, this inlet can be used to reduce the slope of a culvert and lower outlet velocities.
- The slope-tapered inlet provides the same benefits as the depressed inlet. It provides much greater hydraulic efficiency due to the gradual taper of the inlet throat section.
- The side-tapered inlets do not provide a drop but they have increased hydraulic efficiency when compared to conventional inlets. They may be cost effective in certain situations because they can reduce barrel size. They may also be added to existing barrels to increase their flow capacity.

These inlets have these disadvantages.

- They have a relatively high cost compared to other end treatments. They are specially designed for each application and they require forming reinforced concrete into shapes.
- These inlets are rarely adequate for fish passage.
- These inlets may be more susceptible to clogging than conventional inlets in heavy debris areas.

ODOT practice for depressed or tapered inlets follows.

- Depressed or tapered inlets are an optional end treatment for culverts. A reinforced concrete collar, slope paving, or a headwall with wingwalls should be used at the outlet. Culverts with diameters equal to or larger than 72 inches should have a cutoff wall with a minimum depth of 36 inches. Culverts with barrels less than 72 inches diameter should have a cutoff wall that extends downward the greater distance of ½ the barrel diameter or 1 foot.
- Depressed or tapered inlets are not to be used for culvert ends in unshielded areas within the clear zone, approach road culverts in cut ditch sections, or on shallow flat sloped embankments five feet or less in height.
- Depressed or tapered inlets are not to be used for culverts in debris prone areas or where fish passage is required.

## 9.15 Culvert Design Flows

An essential step in culvert design is to determine the flood flow versus recurrence interval relationship. Design and check flood recurrence intervals are listed in **Chapter 3**, the contents of the hydrology section of a Hydraulics Report are discussed in **Chapter 4**, and hydrologic methods are presented in **Chapter 7**.

The discharges used in culvert design are different for large, medium, and small culverts. The culvert size classification is defined at the beginning of this chapter. Discharges used in large culvert and some medium culvert designs are:

- the 6-month flow for selection of invert protection (see **Chapter 5**),
- the Design Event (typically the 25 or 50-year storm event, and in some instances, the 100-year flood,
- the 100-year Base Flood, and
- the check flood. The check flood is one of the two flows listed below that results in the highest headwater elevations and outlet flow velocities,
  - the Overtopping Flood, or
  - the 500-year Flood.

Discharges used in small culvert and some medium culvert designs are:

- the 6-month flow for selection of invert protection (see **Chapter 5**), or
- the Design Event (typically the 10-year storm event for road approach culverts, or the 25-year or 50-year storm events for cross-culverts), and
- the Check Flood. The check flood is the more frequent of:
  - the Overtopping Flood, or
  - the 100-year base flood.

*Note: The invert protection design is based on sediment and bed material conveyed by the 6-month discharge. This is adequate for most installations. Greater floods may transport larger material at some sites. Designing the invert to resist damage from these greater and less frequent floods is recommended for costly or critical installations.*

Culverts are also used for temporary installations such as detours around construction sites, temporary water management, and other applications. Guidance about the discharges to use in their design is provided in **Chapter 3**.

## 9.16 Peak vs. Attenuated Flows

Discharge increases to a maximum and then recedes when runoff from a storm event passes a point on a stream. The maximum discharge rate is called the peak flow. A culvert is sized to pass this peak flow from one side of the roadway embankment to the other without producing an excessive headwater elevation using the design method presented in this manual.

Often there is considerable volume in the floodplain upstream from the culvert, and a significant amount of water can be stored in this area without producing an excessive headwater elevation. This storage can be considered in the culvert analysis. When upstream storage is considered, it will

often reduce the peak flow the culvert must pass. Consequently, a smaller culvert can be used without producing excess headwater. This lower rate of flow is often called the attenuated flow.

Use of peak flows in the culvert design greatly simplifies the hydraulic analysis and it produces conservative results. Most culverts should be designed using this method, including fish passage culverts. In some cases, however, the added complexity of analyzing the attenuated flow may be justified. An example would be an area where there is considerable volume available upstream for storage, fish passage is not an issue, and a reduction in barrel size would significantly reduce the cost of the culvert.

Attenuated flow analyses can be done by hand. Computer programs are available and they greatly reduce the amount of hand calculations. The handiest programs are those which perform all four of the major calculations, such as generating the hydrograph, determining the storage vs. elevation relationship, routing the flows, and performing the culvert hydraulic analysis. The FHWA's HY-8 program in the "HYDRAIN" software package has these capabilities. Guidance on using storage in hydraulic design is presented in **Chapter 12**.

The ODOT culvert design practice is based on experience with culverts sized by the peak flow method, and designing a culvert using attenuated flows should be done with care. The maximum headwater depths upstream from the culvert should not exceed the criteria in the following section. The same method must be used in both analyses in cases when comparing an existing to a proposed condition. An erroneous comparison results if storage is considered in one analysis and not in the other.

## 9.17 Maximum Allowable Headwater

---

ODOT policy is to design the culvert so it does not have a design event headwater elevation ($EL_{hd}$) that is greater than the maximum headwater elevation the site can tolerate. The headwater elevation $EL_{hd}$ is shown in Figure 9-9. The maximum headwater elevation the site can tolerate is called the maximum allowable headwater elevation. Typical items to consider when determining the maximum allowable headwater elevation are listed below.

1. Damage to upstream property. In general, unless a flood easement is obtained, flood elevations should not be increased on adjacent property. In most cases, the maximum allowable headwater elevation should not be higher than the floor elevations of upstream buildings or an increase above existing headwater elevations.

2. Requirements of local development ordinances and the National Flood Insurance Program (NFIP). These ordinances often have restrictions on increases in flood elevations. It is recommended that the designer contact the Region Technical Center hydraulics staff for guidance if a culvert is to be placed or replaced in a floodway or floodplain identified in a NFIP Flood Insurance Study.

3. Diversion of flow. The maximum allowable headwater elevation should not be higher than the elevation where flow diverts around the culvert. This could be a diversion where the flow rejoins the stream of origin downstream from the culvert. It could also be a diversion where flow is diverted to drainage. An exception can be made if the effects of the bypass flow are considered in the design.

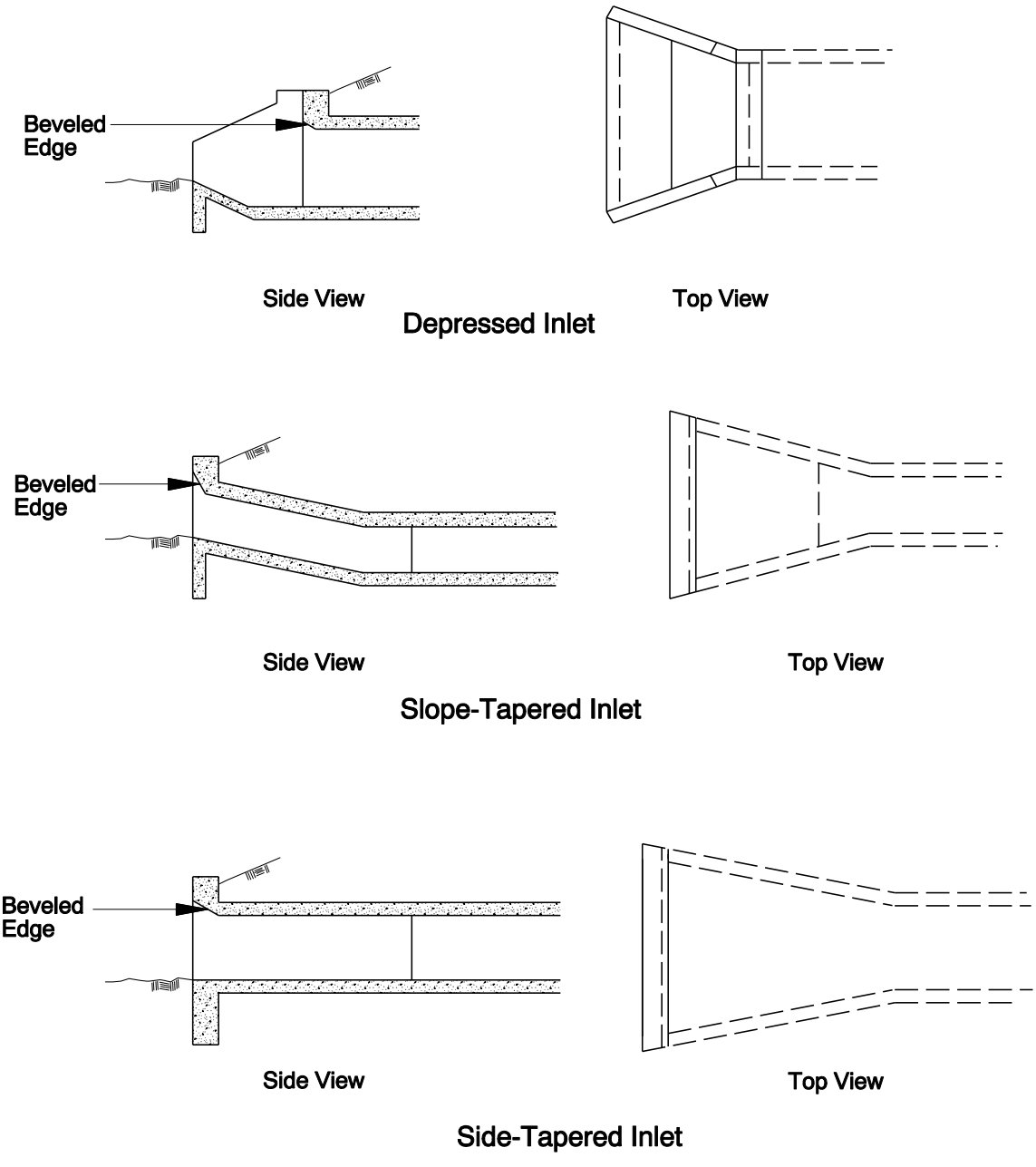



**Figure 9-8 Depressed and Tapered Inlets**

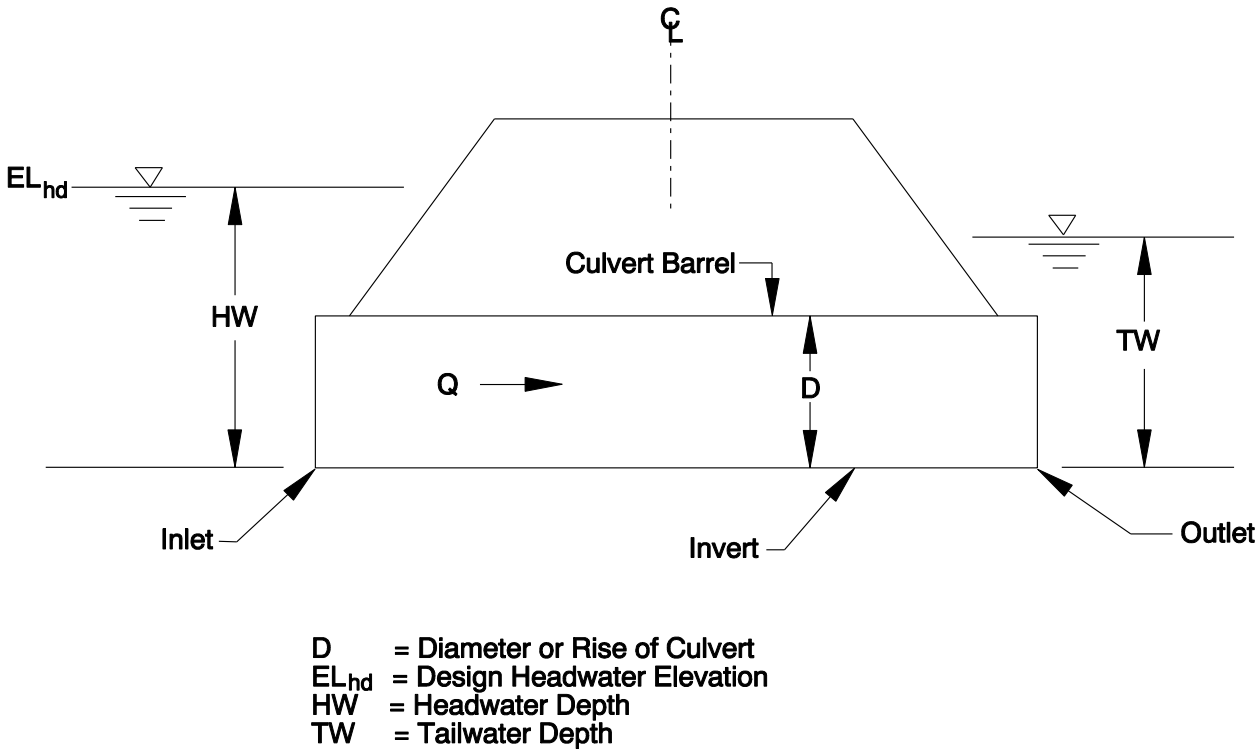



**Figure 9-9 Headwater and Tailwater Diagram**

4. Traffic interruption. This depends on the importance of the highway and the viability of alternate flood free routes. The maximum allowable headwater elevation should not be high enough to interrupt the flow of traffic, during the design event listed in **Chapter 3**.

5. Strength of the highway embankment. Water pooled at the culvert inlet can cause considerable hydrostatic pressure on the upstream side of the highway embankment and the embankment may collapse if the pressure becomes excessive. As a result, an excessively high headwater depth upstream from an embankment may create a hazard to the public, and should be avoided.

6. Hazard to human life. Headwater depths in locations such as urban areas may be a hazard to local residents as well as the traveling public.

7. Damage to the culvert. The hydraulic pressure on the edge of the inlet end of the culvert can be quite large if the culvert is operating under a high headwater. These pressures can collapse the end of the culvert. The thin edges of metal pipes with projecting ends, mitered ends without slope paving or flared sheet metal end sections especially susceptible to this damage.

*Note: Circular culverts, box culverts, and pipe-arch culverts should be designed such that the ratio of the headwater (HW) to the diameter or rise (D) during the design flow event is less than or equal to 1.25 (HW/D less than or equal to 1.25). Design discharge HW/D ratios greater than 1.25 are permitted, provided that the impounding embankment is determined to be stable based on the results of an appropriate geotechnical stability analysis and the existing site conditions dictate or warrant a larger ratio. An example of this may be an area with a high roadway fill, minimal debris in the discharge, and no impacted upstream property owners. Generally, the maximum HW/D ratio during the check flood should not exceed 3 to 5.*

## 9.18 Culvert Hydraulic Design

An analysis of culvert hydraulics is an essential tool to design culverts, and it can be done by several methods. One method is to use the procedures presented in this chapter. These methods provide sufficient accuracy for most applications. Much of the information in this chapter was originally presented in the FHWA's "Hydraulic Design Series No. 5 – Hydraulic Design of Highway Culverts" (HDS 5). HDS 5 contains detailed information on culvert hydraulics and culvert design for special applications. It is available from the FHWA website: http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=7

Computer programs such as the HY-8 or WSPRO modules in the FHWA's HYDRAIN computer program, or the U.S. Corps of Engineers HEC-RAS program can also be used to analyze culvert flow. Most of these programs can evaluate special situations, such as barrels with irregular shapes or groups of culverts with non-identical barrels. The HEC-RAS and WSPRO programs can also model a culvert and the stream reaches upstream and downstream from the culvert. The HEC-RAS program is particularly useful because, unlike the other programs, it can model a mixed water surface profile when there are both supercritical and subcritical flows in the culvert or the channel.

A fish passage culvert design almost always requires detailed information about the hydraulic characteristics of the channel as well as the culvert. The primary use of this information is to verify the proposed culvert has hydraulic characteristics suitable for passage, and to analyze sediment transport and retention in both the stream and culvert. HEC-RAS or WSPRO modeling has the ability to provide the needed data, and they are strongly recommended for fish passage culvert modeling.

An exact theoretical analysis of culvert flow is extremely complex because the flow is usually nonuniform with regions of both gradually varying and rapidly varying flow. An exact analysis involves backwater and drawdown calculations, energy and momentum balance, and application of the results of hydraulic model studies. In addition, the flow types change in a given culvert as the flow rate and tailwater elevations change. In order to simplify culvert analysis the various

flow types are classified and analyzed on the basis of a control section.

A control section is a location where there is a unique relationship between the flow rate and the upstream water surface elevation. Many different flow conditions exist over time, but at a given time the flow is either governed by the inlet geometry (**inlet control**); or by a combination of the culvert inlet configuration, the characteristics of the barrel, and the tailwater (**outlet control**). Control may oscillate from inlet to outlet; however, the concept of "minimum performance" applies. That is, while the culvert may operate more efficiently at times (more flow for a given headwater level), it will never operate at a lower level of performance than calculated. The governing control is determined by calculating the headwater depth for both inlet control and outlet control. The higher headwater elevation indicates the type of control. This method of determining the type of control is accurate except for a few cases where the headwater depth is approximately the same for both types of control.

A general description of the characteristics of inlet and outlet control flow is as follows: A culvert flowing in inlet control has shallow, high velocity flow categorized as "supercritical". For supercritical flow, the control section is at the upstream end of the barrel (the inlet). Conversely, a culvert flowing in outlet control will have relatively deep, lower velocity flow termed "subcritical" flow. For subcritical flow the control is at the downstream end of the culvert (the outlet). The tailwater depth is either critical depth at the culvert outlet or the downstream channel depth, whichever is higher. In a given culvert, the type of flow is dependent on all of the factors listed in Table 9-1.

### 9.18.1 Headwater

Energy is required to force flow through a culvert. This energy takes the form of an increased water surface elevation on the upstream side of the culvert. The depth of the upstream surface measured from the invert at the culvert entrance is generally referred to as **headwater depth**.

**TABLE 9-1 FACTORS INFLUENCING CULVERT PERFORMANCE**

| Factor | Inlet Control | Outlet Control |
|---|---|---|
| Headwater Depth | X | X |
| Inlet Area | X | X |
| Inlet Edge Configuration | X | X |
| Inlet Shape | X | X |
| Barrel Roughness | | X |
| Barrel Area | | X |
| Barrel Shape | | X |
| Barrel Length | | X |
| Barrel Slope | | X |
| Tailwater Depth | | X |

### 9.18.2 Tailwater

**Tailwater** is defined as the depth of water downstream of the culvert measured from the outlet invert. It is an important factor in determining culvert capacity under outlet control conditions. Tailwater may be caused by an obstruction in the downstream channel or by the hydraulic resistance of the channel. In either case, backwater calculations from the downstream control point are required to precisely define tailwater. When appropriate, normal depth approximations may be used instead of backwater calculations. This is shown in the single-section analysis example in **Chapter 8**.

### 9.18.3 Outlet Velocity

Culvert outlet velocities should be calculated to determine the need for erosion protection at the culvert outlet. Since culverts usually have outlet velocities which are higher than the natural stream velocities, riprap protection or an energy dissipator may be required to prevent downstream erosion. Outlet velocity calculation procedures are discussed and illustrated in **Chapter 11**.

### 9.18.4 Performance Curves

A performance curve is a plot of headwater depth or headwater elevation versus discharge. The resulting graphical depiction of culvert operation is useful in evaluating the hydraulic capacity of a culvert for various headwaters. Among its uses, the performance curve displays the

consequences of higher flow rates at the site and the benefits of inlet improvements. The determination of a performance curve should be routinely done as part of the hydraulic analysis.

In developing a culvert performance curve, both inlet and outlet control curves must be calculated. This is necessary to determine the dominant control for a given discharge as control may shift from the inlet to the outlet, or vice-versa over a range of discharges. The overall performance curve reflects the controlling portions of the individual inlet control and outlet control curves. Figure 9-10 illustrates a typical culvert performance curve. At the design headwater, the culvert operates under inlet control. With inlet improvement the culvert performance can be increased to take better advantage of the culvert barrel capacity.

### 9.18.5 Culverts Flowing in Inlet Control

Since the control is at the upstream end in inlet control, only the headwater and the inlet configuration affect the culvert performance (Table 9-1). The **headwater depth** is measured from the invert of the inlet control section to the surface of the upstream pool. The **inlet area** is the cross-sectional area of the face of the culvert. Generally, the inlet face area is the same as the barrel area, except for some types of improved inlets. The **inlet edge configuration** describes the entrance type. Some typical inlet edge configurations are thin edge projecting, mitered, square edges in a headwall, and beveled edge. The **inlet shape** is usually the same as the shape of the culvert barrel; however, it may be enlarged as in the case of a tapered inlet. Typical shapes are rectangular, circular, and elliptical. Whenever the inlet face is a different size or shape than the culvert barrel, the possibility of an additional control section within the barrel exists.

An additional factor which influences inlet control performance is the barrel slope. The effect is small, however, and it can be ignored or a small slope correction factor can be inserted in the inlet control equations.

The inlet edge configuration is a major factor in inlet control performance, and it can be modified to improve performance. As the inlet edge condition improves, the flow contraction at the inlet decreases. The reduced flow contraction increases inlet performance and allows more flow through the barrel for the same headwater.

A method of increasing inlet performance is the use of beveled edges at the entrance of the culvert. Beveled edges reduce the contraction of the flow by effectively enlarging the face of the culvert. Although any beveling will improve the hydraulic efficiency, design charts are available for two bevel angles, 45 degrees and 33.7 degrees.

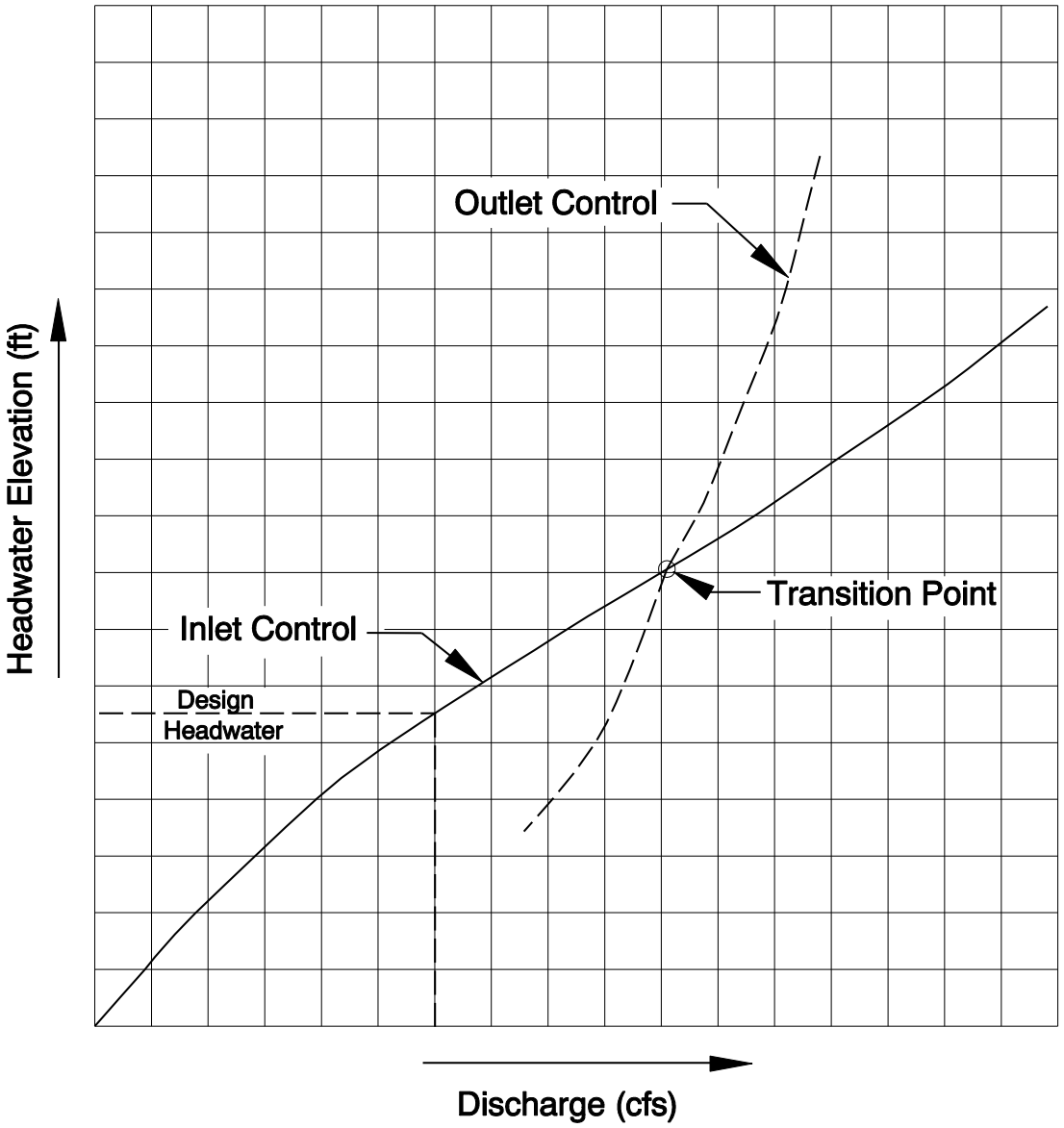
#### 9.18.5.1 Inlet Control Examples

Several different examples of inlet control are shown in Figure 9-11. The flow type depends on the degree of submergence of the inlet and outlet ends of the culvert. In all of these examples,

the control section is at the inlet end. Depending on the tailwater, a hydraulic jump may occur downstream from the inlet.

Figure 9-11a depicts a condition where neither the inlet nor the outlet ends are submerged. The flow passes through critical depth just downstream of the culvert entrance and the flow in the barrel is supercritical. The barrel flows partially full over its length and the flow approaches or has reached normal depth at the outlet end.

Figure 9-11b shows that outlet submergence does not assure outlet control. In this case, the flow just downstream of the inlet is supercritical and a hydraulic jump forms in the culvert barrel.




Note: Control shifts from inlet control to outlet control at the transition point.

**Figure 9-10 Culvert Performance Curve**

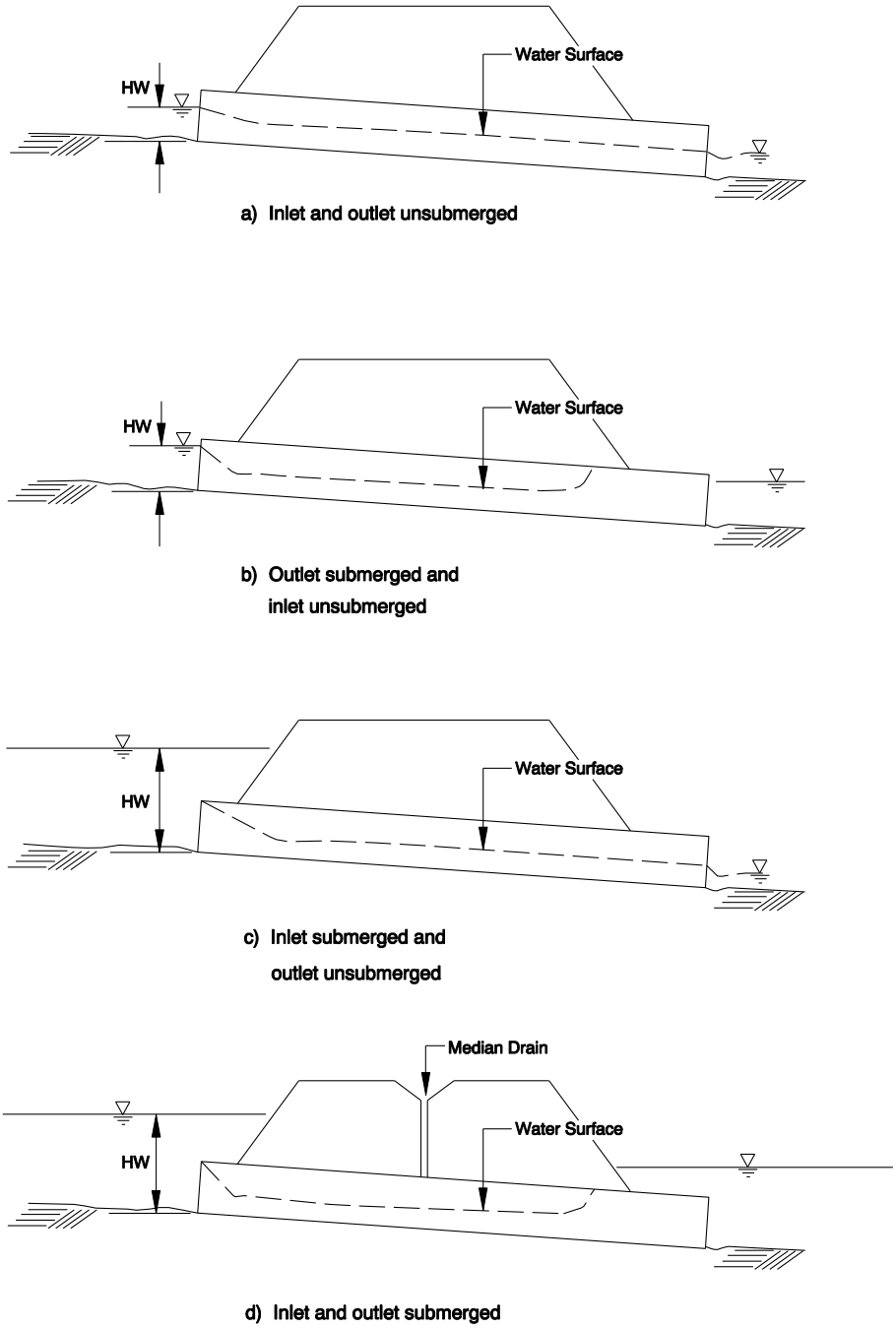



**Figure 9-11 Inlet Control Types**

Figure 9-11c is a more typical design situation. The inlet end is submerged and the outlet end flows freely. Again, the flow is supercritical and the barrel flows partially full over its length. Critical depth is located just downstream of the culvert entrance, and the flow is approaching or has reached normal depth at the downstream end of the culvert.

Figure 9-11d is an unusual condition illustrating the fact that even submergence of both the inlet and the outlet ends of the culvert does not assure full flow. In this case, a hydraulic jump will form in the barrel. The median inlet provides ventilation of the culvert barrel. If the barrel were not ventilated, sub-atmospheric pressures could develop which might create an unstable condition during which the barrel would alternate between full flow and partial flow.

### 9.18.5.2 Inlet Control Design Equations

The equations used to develop the inlet control nomographs are based on two basic inlet headwater conditions. If the inlet is not submerged, it performs as a weir. If the inlet is submerged, it performs as an orifice. Equations for both conditions are in this subsection.

Between the unsubmerged and the submerged conditions a transition zone exists where the equation results do not match. The zone is defined empirically by drawing a curve between and tangent to the curves defined by the unsubmerged and submerged equations. In most cases, the transition zone is short and the curve is easily constructed.

**FHWA Inlet Control Equations –** The Federal Highway Administration has developed inlet control equations for many culvert shapes and inlet configurations. These equations are discussed in Hydraulic Engineering Series No. 5 “Hydraulic Design of Highway Culverts.”

Equations 9-1a, 9-1b, and 9-1c are unsubmerged and submerged inlet control design equations, respectively. Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at critical depth, adjusted with two correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but form (2) is easier to apply.

Either form of unsubmerged inlet control equation will produce adequate results. The constants for the equations are given in Table 9-2. Table 9-2 provides the unsubmerged and submerged equation coefficients for each shape, material, and edge configuration. For the unsubmerged equations, the form of the equation is also noted.

Unsubmerged inlet control design equations:

$$\text{Form (1)} \quad \frac{HW_i}{D} = \frac{H_c}{D} + K\left(\frac{Q}{AD^{0.5}}\right)^M - 0.5S \quad \text{(see Notes 1 and 2)} \qquad \text{(Equation 9-1a)}$$

$$\text{Form (2)} \quad \frac{HW_i}{D} = K\left(\frac{Q}{AD^{0.5}}\right)^M \quad \text{(see Note 1)} \qquad \text{(Equation 9-1b)}$$

Submerged inlet control design equation:

$$\frac{HW_i}{D} = c\left(\frac{Q}{AD^{0.5}}\right)^2 + Y - 0.5S \quad \text{(see notes 2 and 3)} \qquad \text{(Equation 9-1c)}$$

Where:

| | | |
|---|---|---|
| $HW_i$ | = | headwater depth above inlet control section invert in feet, |
| D | = | interior height of culvert barrel in feet, |
| $H_c$ | = | specific head at critical depth ($D_c + V_c^2/2g$) in feet, |
| Q | = | discharge in cubic feet per second, |
| A | = | full cross sectional area of culvert barrel in square feet, |
| K,M,c,Y | = | constants from Table 9-2, and |
| S | = | culvert barrel slope in feet per foot. |

*Notes:* *(1) Equations, (unsubmerged) apply up to about $Q/AD^{0.5} = 3.5$*
*(2) For mitered inlets use +0.7S instead of -0.5S as the slope correction factor.*
*(3) Equation (submerged) applies above about $Q/AD^{0.5} = 4.0$*

The equations may be used to develop design curves for any conduit shape or size. Careful examination of the equation constants for a given form of equation reveals that there is very little difference between the constants for a given inlet configuration. Therefore, given the necessary conduit geometry for a new shape from the manufacturer, a similar shape is chosen from Table 9-2, and the constants are used to develop new design curves. Note that coefficients for rectangular shapes should not be used for nonrectangular shapes and vice-versa. A constant slope value of 2 percent is usually selected for the development of design curves. This is because the slope effect is small and the resultant headwater is conservatively high for sites with slopes exceeding 2 percent (except for mitered inlets).

**K-TRAN Inlet Control Equations –** Inlet control equations for several prefabricated end section types were derived from laboratory model test data by the Kansas Department of Transportation (K-TRAN). Several prefabricated end sections available from proprietary sources were tested. Study results for tapered metal and concrete end sections are published in K-TRAN Report No. KU-94-4 "Development of Hydraulic Design Charts for Type I and Type III Metal and Concrete End Sections for Pipe Culverts." The study results for the non-tapered metal mitered end resembling the ODOT Safety End Section are published in K-TRAN Report 93-5 "Development of Hydraulic

Design Charts for Type IV End Section for Pipe Culverts." The equation variables are the same as those described previously for the FHWA equations, with one addition, as follows:

g = gravitational acceleration, 32.2 feet per second squared

The tapered and mitered prefabricated concrete end section is shown on Figure 9-12 with the dimensions of the largest and smallest end sections tested by K-TRANS. The inlet control equations are as follows:

$$\frac{HW_i}{D} = 1.53\left(\frac{Q}{gD^5}\right)^{0.55} \quad \text{when } 0 \le \left(\frac{Q}{gD^5}\right) \le 0.42 \qquad \text{(Equation 9-2a)}$$

$$\frac{HW_i}{D} = 2.13\left(\frac{Q}{gD^5}\right) + 0.055 \quad \text{when } 0.42 < \left(\frac{Q}{gD^5}\right) \le 0.68 \qquad \text{(Equation 9-2b)}$$

$$\frac{HW_i}{D} = 1.367 - 1.50\left(\frac{Q}{gD^5}\right) + 2.50\left(\frac{Q}{gD^5}\right)^2 \quad \text{when } 0.68 < \left(\frac{Q}{gD^5}\right) \le 1.30 \qquad \text{(Equation 9-2c)}$$

*Note:* $\le$ *(Less than or equal to)*
$\ge$ *(Greater than or equal to)*

The tapered and mitered prefabricated metal end section is shown on Figure 9-13 with the dimensions of the largest and smallest end sections tested. The inlet control equations follow:

$$\frac{HW_i}{D} = 1.60\left(\frac{Q}{gD^5}\right)^{0.60} \quad \text{when } 0 \le \left(\frac{Q}{gD^5}\right) \le 0.41 \qquad \text{(Equation 9-3a)}$$

$$\frac{HW_i}{D} = 2.23\left(\frac{Q}{gD^5}\right) + 0.023 \quad \text{when } 0.41 < \left(\frac{Q}{gD^5}\right) \le 0.62 \qquad \text{(Equation 9-3b)}$$

$$\frac{HW_i}{D} = 1.289 - 1.61\left(\frac{Q}{gD^5}\right) + 2.90\left(\frac{Q}{gD^5}\right)^2 \quad \text{when } 0.62 < \left(\frac{Q}{gD^5}\right) \le 1.20 \qquad \text{(Equation 9-3c)}$$

Ten mitered prefabricated metal safety end sections of different sizes, safety bar configurations, and miter slopes were tested. The miter slopes were 1V: 4H and 1V: 6H. The safety end sections shown on ODOT Standard Drawings RD322 and RD324 are typical of the end sections tested. The study developed the following inlet control equations for safety end sections with safety bars.

$$\frac{HW_i}{D} = 1.69\left(\frac{Q}{gD^5}\right)^{0.60} \text{ when } \left(\frac{Q}{gD^5}\right) \leq 0.42 \qquad \text{(Equation 9-4a)}$$

$$\frac{HW_i}{D} = 1.11 - 1.93\left(\frac{Q}{gD^5}\right) + 4\left(\frac{Q}{gD^5}\right)^2 \text{ when } \left(\frac{Q}{gD^5}\right) > 0.42 \qquad \text{(Equation 9-4b)}$$

The preceding inlet control equations were developed with unobstructed inlet sections. ODOT design practice is to use the hydraulic properties of unobstructed sections in design. The safety end sections were also tested by K-TRANS with varying amounts of debris obstruction. The hydraulic performance equations and conclusions are in K-TRANS Report KU-93-5.

## TABLE 9-2 CONSTANTS FOR INLET CONTROL DESIGN EQUATIONS

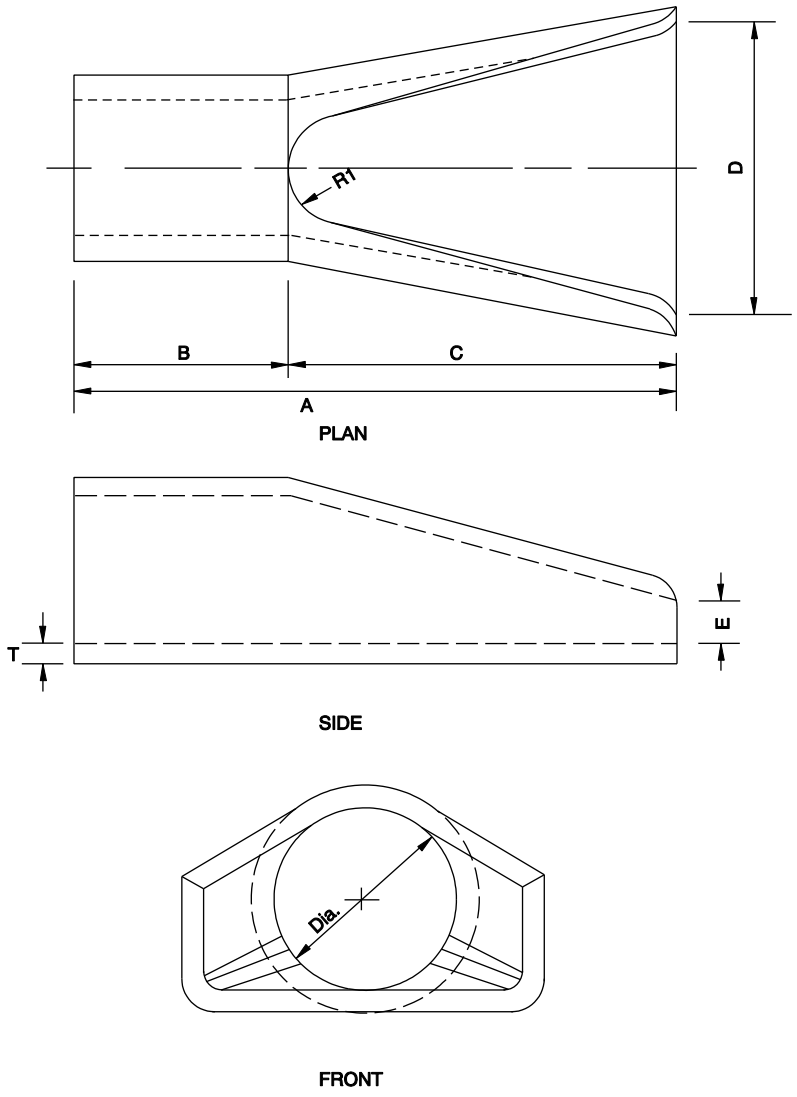
| | | | UNSUBMERGED | | | SUBMERGED | |
|---|---|---|---|---|---|---|---|
| **SHAPE & MATERIAL** | **NOMOGRAPH (Appendix A)** | **INLET EDGE DESCRIPTION** | **EQUATION FORM** | **K** | **M** | **c** | **Y** |
| Circular Concrete | Chart 1 | Square edge w/headwall | 1 | 0.0098 | 2.0 | 0.0398 | 0.67 |
| | | Groove end w/ headwall | | .0078 | 2.0 | .0292 | .74 |
| | | Groove end projecting | | .0045 | 2.0 | .0317 | .69 |
| Circular CMP | Chart 3 | Headwall | 1 | .0078 | 2.0 | .0379 | .69 |
| | | Mitered to slope | | .0210 | 1.33 | .0463 | .75 |
| | | Projecting | | .0340 | 1.50 | .0553 | .54 |
| Circular | Chart 6 | Beveled ring, 45º bevel | 1 | .0018 | 2.50 | .0300 | .74 |
| | | Beveled ring, 33.7º bevel | | .0018 | 2.50 | .0243 | .83 |
| Rectangular Box | Chart 10 | 30º to 75º wingwall flare | 1 | .026 | 1.0 | .0385 | .81 |
| | | 90º and 15º wingwall flare | | .061 | 0.75 | .0400 | .80 |
| | | 0º wingwall flare | | .061 | 0.75 | .0423 | .82 |
| Rectangular Box | Chart 11 | 45º wingwall flare d=.043D | 2 | .510 | .667 | .0309 | .80 |
| | | 18º to 33.7º wingwall flare d=.083D | | .486 | .667 | .0249 | .83 |
| Rectangular Box | | 90º headwall w/3/4" chamfer | 2 | .515 | .667 | .0375 | .79 |
| | | 90º headwall w/45º bevel | | .495 | .667 | .0314 | .82 |
| | | 90º headwall w/33.7º bevel | | .486 | .667 | .0252 | .865 |
| Rectangular Box | | ¾" chamfers; 45º skewed headwall | 2 | .522 | .667 | .0402 | .73 |
| | | ¾" chamfers; 30º skewed headwall | | .533 | .667 | .0425 | .705 |
| | | ¾" chamfers; 15º skewed headwall | | .545 | .667 | .04505 | .68 |
| | | 45º bevels; 10º – 45º skewed headwall | | .498 | .667 | .0327 | .75 |

## TABLE 9-2 CONSTANTS FOR INLET CONTROL DESIGN EQUATIONS (cont)

| | | | UNSUBMERGED | | | SUBMERGED | |
|---|---|---|---|---|---|---|---|
| **SHAPE & MATERIAL** | **NOMOGRAPH (Appendix A)** | **INLET EDGE DESCRIPTION** | **EQUATION FORM** | **K** | **M** | **c** | **Y** |
| Rectangular<br>Box<br>3/4" Chamfers | | 45º non-offset wingwall flare<br>18.4º non-offset wingwall flare<br>18.4º non-offset wingwall flare<br>30º skewed barrel | 2 | .497<br>.493<br>.495 | .667<br>.667<br>.667 | .0339<br>.0361<br>.0386 | .803<br>.806<br>.71 |
| Rectangular<br>Box<br>Top bevels | | 45º wingwall flares – offset<br>33.7º wingwall flares – offset<br>18.4º wingwall flares - offset | 2 | .497<br>.495<br>.493 | .667<br>.667<br>.667 | .0302<br>.0252<br>.0227 | .835<br>.881<br>.887 |
| C M Boxes | | 90º headwall<br>Thick wall projecting<br>Thin wall projecting | 1 | .0083<br>.0145<br>.0340 | 2.0<br>1.75<br>1.5 | .0379<br>.0419<br>.0496 | .69<br>.64<br>.57 |
| Horizontal<br>Ellipse<br>Concrete | | Square edge with headwall<br>Groove end with headwall<br>Groove end projecting | 1 | .0100<br>.0018<br>.0045 | 2.0<br>2.5<br>2.0 | .0398<br>.0292<br>.0317 | .67<br>.74<br>.69 |
| Vertical<br>Ellipse<br>Concrete | | Square edge with headwall<br>Groove end with headwall<br>Groove end projecting | 1 | .0100<br>.0018<br>.0095 | 2.0<br>2.5<br>2.0 | .0398<br>.0292<br>.0317 | .67<br>.74<br>.69 |
| Pipe Arch<br>18” Corner<br>Radius CM | Chart 13 | 90º headwall<br>Mitered to slope<br>Projecting | 1 | .0083<br>.0300<br>.0340 | 2.0<br>1.0<br>1.5 | .0496<br>.0463<br>.0496 | .57<br>.75<br>.53 |

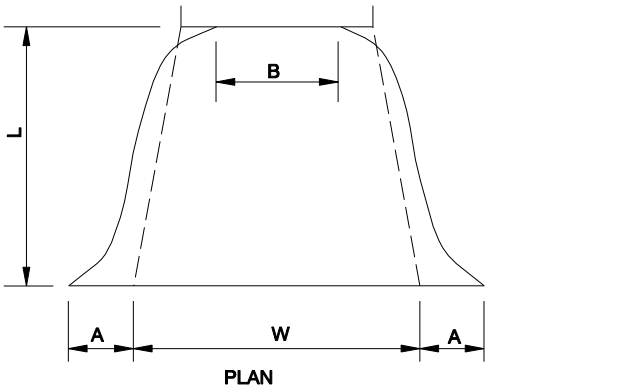
## TABLE 9-2 CONSTANTS FOR INLET CONTROL DESIGN EQUATIONS (cont)

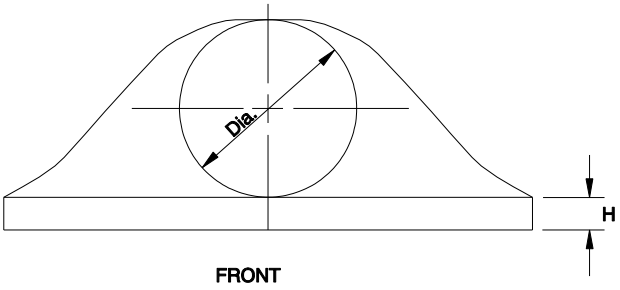
| | | | UNSUBMERGED | | | SUBMERGED | |
|---|---|---|---|---|---|---|---|
| **SHAPE & MATERIAL** | **NOMOGRAPH (Appendix A)** | **INLET EDGE DESCRIPTION** | **EQUATION FORM** | **K** | **M** | **c** | **Y** |
| Pipe Arch<br>18" Corner<br>Radius CM | Chart 14 | Projecting<br>No bevel<br>33.7º Bevel | 1 | .0296<br>.0087<br>.0030 | 1.5<br>2.0<br>2.0 | .0487<br>.0361<br>.0264 | .55<br>.66<br>.75 |
| Pipe Arch<br>31" Corner<br>Radius CM | Chart 15 | Projecting<br>No bevel<br>33.7º Bevel | 1 | .0296<br>.0087<br>.0030 | 1.5<br>2.0<br>2.0 | .0487<br>.0361<br>.0264 | .55<br>.66<br>.75 |
| Arch CM | Chart 18 –<br>Chart 20 | 90º headwall<br>Mitered to slope<br>Thin wall projecting | 1 | .0083<br>.0300<br>.0340 | 2.0<br>2.0<br>1.5 | .0379<br>.0463<br>.0496 | .69<br>.75<br>.57 |
| Circular | | Smooth tapered inlet throat<br>Rough tapered inlet throat | 2 | .534<br>.519 | .555<br>.64 | .0196<br>.0289 | .89<br>.90 |
| Elliptical<br>Inlet Face | | Tapered inlet-beveled edge<br>Tapered inlet-square edge<br>Tapered inlet-thin edge projecting | 2 | .536<br>.5035<br>.547 | .622<br>.719<br>.80 | .0368<br>.0478<br>.0598 | .83<br>.80<br>.75 |
| Rectangular | | Tapered inlet throat | 2 | .475 | .667 | .0179 | .97 |
| Rectangular<br>Concrete | | Side tapered-less favorable edge<br>Side tapered-more favorable edge | 2 | .56<br>.56 | .667<br>.667 | .0466<br>.0378 | .85.<br>.87 |
| Rectangular<br>Concrete | | Side tapered-less favorable edge<br>Side tapered-more favorable edge | 2 | .50<br>.50 | .667<br>.667 | .0466<br>.0378 | .65<br>.71 |

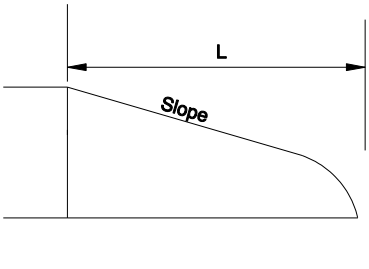



| Pipe Dia. (in.) | A (in.) | B (in.) | C (in.) | D (in.) | E (in.) | R1 (in.) | T (in.) |
|---|---|---|---|---|---|---|---|
| 24 | 73.5 | 30 | 43.5 | 48 | 9.5 | 14 | 3 |
| 60 | 99.0 | 39 | 60.0 | 96 | 35 | 24 | 6 |

**Figure 9-12 Prefabricated Concrete End Section Tested by K-TRANS**




| Pipe Dia. (in.) | A (in.) | B (in.) | H (in.) | L (in.) | W (in.) | SLOPE |
|---|---|---|---|---|---|---|
| 24 | 10 | 13 | 6 | 41 | 48 | 2.5:1 |
| 60 | 18 | 33 | 12 | 87 | 114 | 2:1 |

**Figure 9-13 Prefabricated Metal End Section Tested by K-TRANS**

### 9.18.5.3 Inlet Control Nomographs

Inlet control nomographs for many culverts are included in Appendix A. These nomographs are sufficiently accurate for most design purposes. They are not, however, as accurate as the equations. In formulating inlet and outlet control design nomographs in Appendix A, a certain degree of error is introduced into the design process. This error is due to the fact that the nomograph construction involves graphical fitting techniques resulting in scales which do not exactly match the equations.

Most of the nomographs are from FHWA publications. They have precisions of ±10 percent of the equation values in terms of headwater (inlet control) or head loss (outlet control). The inlet control nomographs for the prefabricated concrete and metal end sections, and the safety end section were developed by ODOT based on the K-TRANS equations listed previously in this chapter. Their precision is estimated to be ± 10 percent. The aluminum structural plate conduit nomographs were provided by Kaiser Aluminum, and their precision is estimated to be ±5 percent.

Instructions for Use

1. To determine **headwater (HW)**; given Q, and size and type of culvert.

    a. Connect with a straightedge the given culvert diameter or height, D and the discharge, Q, or Q/B for box culverts; mark intersection of straightedge on HW/D scale marked (1).

    b. If HW/D scale marked (1) represents entrance type used, read HW/D on scale (1). If another of the three entrance types listed on the nomograph is used, extend the point of intersection in (a) horizontally to scale (2) or (3) and read HW/D.

    c. Compute HW by multiplying HW/D by D.

2. To determine **Discharge (Q)** per barrel; given HW, and size and type of culvert.

    a. Compute HW/D for given conditions.

    b. Locate HW/D on scale for appropriate entrance type. If scale (2) and (3) is used, extend HW/D point horizontally to scale (1).

    c. Connect point on HW/D scale (1) as found in (b) above and the size of culvert on the left scale. Read Q or Q/B on the discharge scale.
    d. If Q/B is read in (c) multiply by B (span of box culvert) to find Q.

3. To determine **culvert size**; given Q, allowable HW and type of culvert.

a. Using a trial size, compute HW/D.

b. Locate HW/D on scale for appropriate entrance type. If scale (2) or (3) is used, extend HW/D point horizontally to scale (1).

c. Connect point on HW/D scale (1) as found in (b) above to given discharge and read diameter, height, or size of culvert required for HW/D value.

d. If D is not that originally assumed, repeat procedure with a new D.

## 9.19 Culverts Flowing in Outlet Control

All factors influencing culvert performance in inlet control also influence culverts in outlet control. In addition, the barrel characteristics (roughness, area, shape, length, and slope) and the tailwater elevation affect culvert performance in outlet control (Table 9-1).

The **barrel roughness** is a function of the material used to fabricate the barrel. Typical materials include concrete and corrugated metal. The roughness is represented by a hydraulic resistance coefficient such as the Manning "n" value. Typical Manning "n" values for culverts are presented in **Chapter 8**, Appendix A.

The **barrel area** and **barrel shape** are self-explanatory.

The **barrel length** is the length of the full barrel section from the entrance to the exit of the culvert. Because the design height of the barrel and the slope influence the actual length, an approximation of barrel length is usually necessary to begin the design process.

The **barrel slope** is the actual slope of the culvert barrel. The barrel slope is often the same as the natural stream slope.

The **tailwater elevation** is based on the downstream water surface elevation. Backwater calculations from a downstream control, a normal depth approximation, or field observations are used to define the tailwater elevation.

### 9.19.1 Outlet Control Examples

Various outlet control flow conditions are illustrated in Figure 9-14. The control section is at the outlet end of the culvert or further downstream in all cases. The flow in the barrel is subcritical for the partial flow situations.

Condition 9-14a represents the classic full flow condition, with both inlet and outlet submerged. The barrel is in pressure flow throughout its length. This condition is often assumed in calculations, but seldom actually exists.

Condition 9-14b depicts the outlet submerged with the inlet unsubmerged. For this case, the headwater depth is shallow so that the inlet crown is exposed as the flow contracts into the culvert.

Condition 9-14c shows the entrance submerged to such a degree that the culvert flows full throughout its entire length while the exit is unsubmerged. This is a rare condition. It requires an extremely high headwater to maintain full barrel flow with no tailwater. The outlet velocities are usually high under this condition. Critical depth must equal the barrel height for this flow condition to occur.

Condition 9-14d is more typical. The culvert entrance is submerged by the headwater and the outlet end flows freely with a low tailwater. For this condition, the barrel flows partially full over at least part of its length (subcritical flow) and the flow passes through critical depth just upstream of the outlet.

Condition 9-14e is also typical, with neither the inlet nor the outlet end of the culvert submerged. The barrel flows partially full over its entire length and the flow profile is subcritical.

### 9.19.2 Outlet Control Hydraulics

This section provides a method to accurately determine headwater depth for culverts flowing full as depicted in Figures 9-14a, 9-14b, and 9-14c. The procedure gives an approximate solution for a free water surface condition throughout the barrel length as shown in Figure 9-14e, or full barrel flow over part of its length as shown in Figure 9-14d. An exact solution to these flow conditions requires calculating the water surface profile through the culvert barrel.

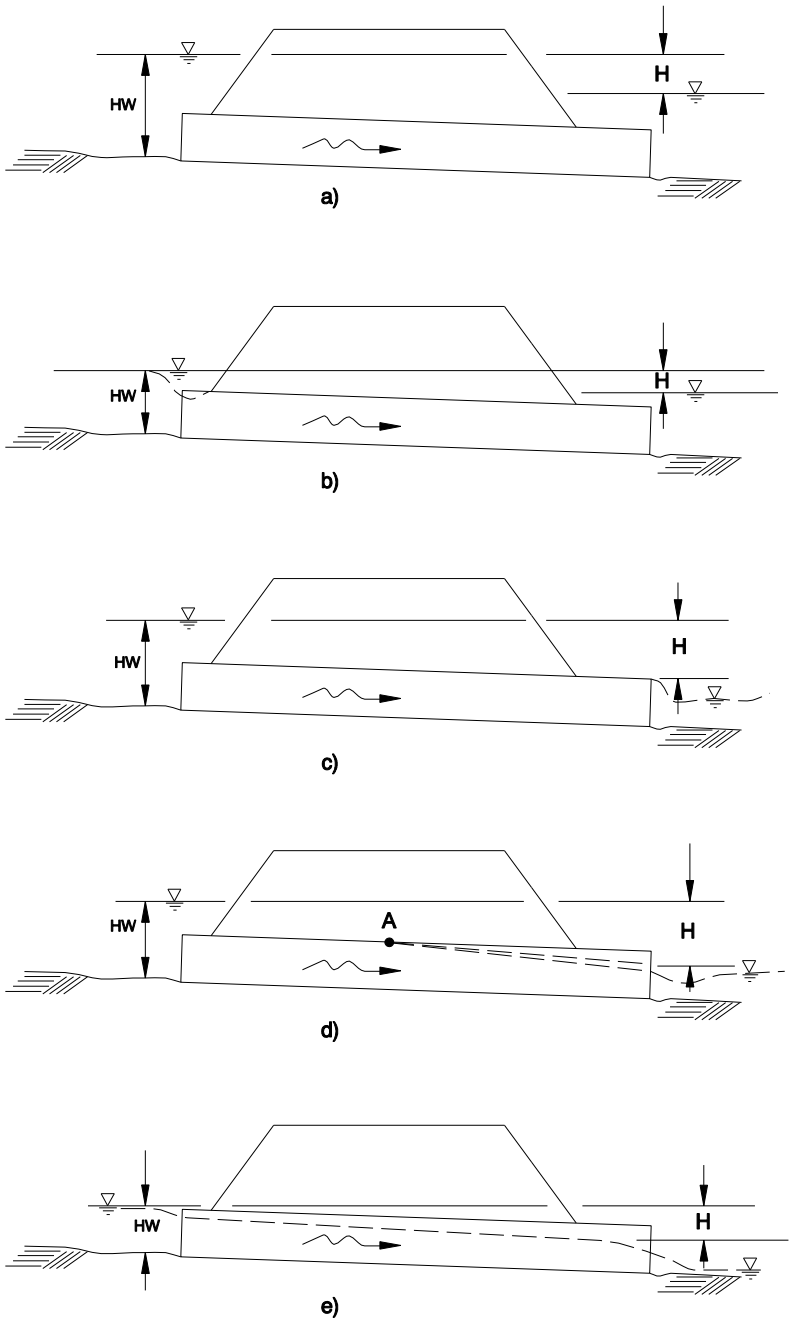



**Figure 9-14 Outlet Control Types**

Outlet control flow conditions are calculated based on the principle of conservation of energy. The total energy ($H_L$) required to pass the flow through the culvert barrel is shown in Figure 9-15 and expressed in Equation 9-5. It is made up of three major components. These components are usually expressed in feet of water (head) and include the entrance loss, the friction loss through the barrel, and the exit loss. Other losses, such as bend losses and junction losses, should be included as appropriate.

$$H_L = H_e + H_f + H_o + H_b + H_j \qquad \text{(Equation 9-5)}$$

Where:

$H_L$ = total energy head in feet,
$H_e$ = entrance loss in feet,
$H_f$ = friction loss through the barrel in feet,
$H_o$ = exit loss in feet,
$H_b$ = bend loss in feet, and
$H_j$ = junction loss in feet.

The entrance loss is a function of the velocity head in the barrel ($V^2/2g$) times an entrance loss coefficient $K_e$. Values of $K_e$ based on various inlet configurations are given in Table 9-3.

$$H_e = K_e \frac{V^2}{2g} \qquad \text{(Equation 9-6)}$$

Where:

$K_e$ = entrance loss coefficient,
$V$ = velocity of full flow in the culvert barrel in feet per second, and
$g$ = gravitational acceleration, 32.2 feet per second squared.

The friction loss in the barrel is also a function of the velocity head. Based on the Manning equation, the friction loss is:

$$H_f = \left(\frac{29n^2L}{R^{1.33}}\right)\left(\frac{V^2}{2g}\right) \qquad \text{(Equation 9-7)}$$

Where:

$n$ = Manning roughness coefficient (Chapter 8 Appendix A),
$L$ = length of the culvert barrel in feet,

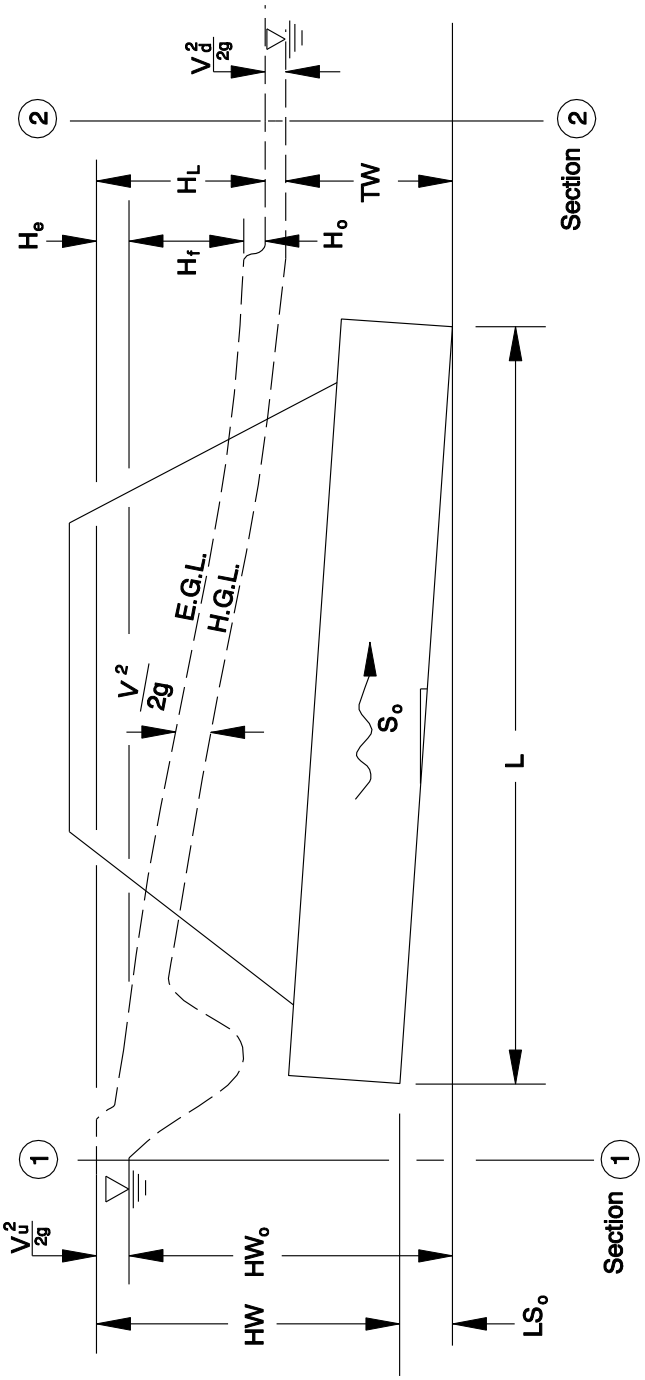



**Figure 9-15 Full Flow Energy and Hydraulic Grade Lines**

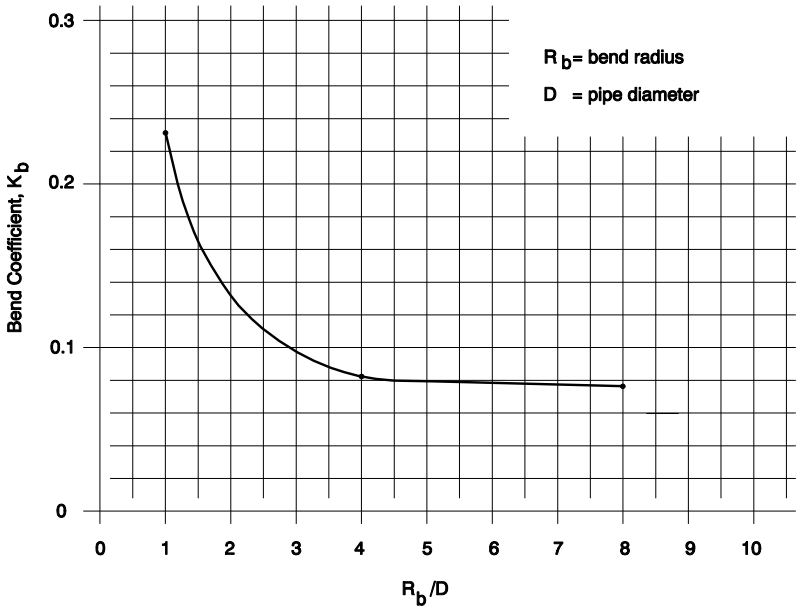
## TABLE 9-3 ENTRANCE LOSS COEFFICIENTS

Outlet control, full or partially full entrance head loss

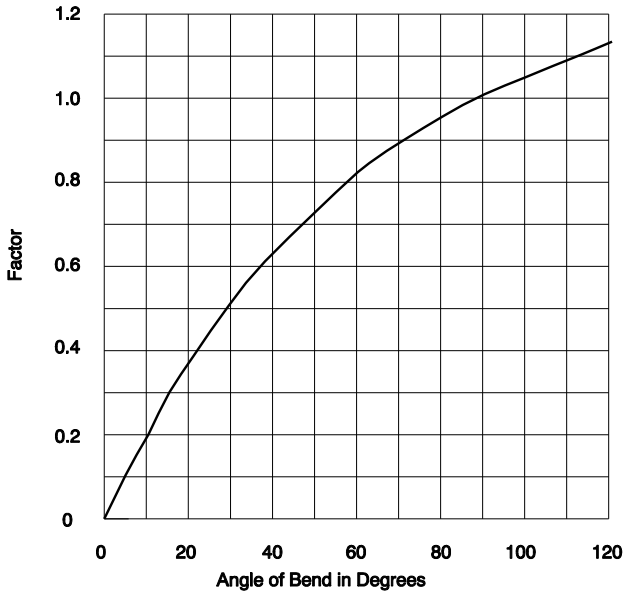
$H_e = K_e (V^2/2g)$

| Type of Structure and Design of Entrance | Coefficient $K_e$ |
|---|---|
| **Pipe, Concrete** | |
| Projecting from fill, socket end (groove-end) | 0.2 |
| Projecting from fill, square cut end | 0.5 |
| Headwall or headwall and wingwalls | |
| Socket end of pipe (groove end) | 0.2 |
| Square-edge | 0.5 |
| Rounded (radius = 1/12D) | 0.2 |
| Mitered to conform to fill slope | 0.7 |
| * End section conforming to fill slope | 0.5 |
| End section conforming to fill slope (see Figure 9-12), dia ≤ 54 in. | 0.35 |
| End section conforming to fill slope (see Figure 9-12), dia > 60 in. | 0.30 |
| Beveled edges, 33.7º or 45º bevels | 0.2 |
| Side- or slope-tapered inlet | 0.2 |
| **Pipe, or Pipe-Arch, Corrugated Metal** | |
| Projecting from fill (no headwall) | 0.9 |
| Headwall or headwall and wingwalls square-edge | 0.5 |
| Mitered to conform to fill slope, paved or unpaved slope | 0.7 |
| * End section conforming to fill slope | 0.5 |
| End section conforming to fill slope (see Figure 9-13), dia ≤ 54 in. | 0.35 |
| End section conforming to fill slope (see Figure 9-13), dia > 60 in. | 0.30 |
| Safety end section with safety bars, dia = 24 in | 0.65 |
| Safety end section with safety bars, dia = 60 in | 0.85 |
| Beveled edges, 33.7º or 45º bevels | 0.2 |
| Side- or slope-tapered inlet | 0.2 |
| **Box, Reinforced Concrete** | |
| Headwall parallel to embankment (no wingwalls) | |
| Square-edged on 3 edges | 0.5 |
| Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides. | 0.2 |
| Wingwalls at 30º to 75º to barrel | |
| Square-edged at crown | 0.4 |
| Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge | 0.2 |
| Wingwall at 10º to 25º to barrel | |
| Square-edged at crown | 0.5 |
| Wingwalls parallel (extension of sides) | |
| Square-edged at crown | 0.7 |
| Side- or slope-tapered inlet | 0.2 |

* Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. Limited hydraulic tests indicate they are equivalent in operation to a headwall in both inlet and outlet control.




a) Bend Coefficient for 90° bends




b) Factors for Other Than 90° Bends

**Figure 9-16 Bend Loss Coefficients**

R = hydraulic radius of the full culvert barrel, A/P, in feet (**Chapter 8 Appendix B**),
A = full cross-sectional area of the barrel in square feet (**Chapter 8 Appendix B**), and
P = wetted perimeter of the full barrel in feet (**Chapter 8 Appendix B**).

The exit loss is a function of the change in velocity heads between the outlet of the culvert barrel and the downstream channel. For a sudden expansion such as an endwall, the exit loss is:

$$H_o = 1.0\left(\frac{V^2}{2g} - \frac{V_d^2}{2g}\right) \qquad \text{(Equation 9-8)}$$

Where:

$V_d$ = channel velocity downstream of the culvert in feet per second

The downstream velocity is usually neglected, in which case the exit loss is equal to the full flow velocity head in the barrel. This depicts the case of a culvert discharging into a pool. The outlet loss equation based on this assumption is:

$$H_o = 1.0\frac{V^2}{2g} \qquad \text{(Equation 9-9)}$$

Bend losses are also a function of velocity head in the culvert barrel and a bend loss coefficient $K_b$. Values of $K_b$ are obtained from Figure 9-16. To determine $K_b$, first determine the bend coefficient for a 90º bend from Figure 9-16a, and then apply the appropriate adjustment factor from Figure 9-16b. Bend losses can be calculated using the following equation:

$$H_b = K_b\frac{V^2}{2g} \qquad \text{(Equation 9-10)}$$

Where:

$K_b$ = bend loss coefficient

Junction losses are considered where appropriate. Examples of junctions and methods to calculate junction losses are described in **Chapter 13**. Usually it is assumed the pipes entering and leaving the junction are full when calculating junction losses for outlet control.

Inserting the above relationships for entrance loss, friction loss, bend loss and exit loss into Equation 9-5, the following equation for head loss is obtained:

$$H_L = \left(1 + K_e + K + \frac{29n^2L}{R^{1.33}}\right)\left(\frac{V^2}{2g}\right) + H \qquad \text{(Equation 9-11)}$$

Since most culverts have no bend or junction losses, the equation can be reduced to:

$$H_L = \left(1 + K_e + \frac{29n^2L}{R^{1.33}}\right)\left(\frac{V^2}{2g}\right) \qquad \text{(Equation 9-12)}$$

Figure 9-15 shows the energy grade line and the hydraulic grade line for full flow in a culvert barrel. The energy grade line represents the total energy at any point along the culvert barrel. HW is the depth from the inlet invert to the energy grade line. The hydraulic grade line is the depth to which water would rise in vertical tubes connected to the sides of the culvert barrel. In full flow, the energy grade line and the hydraulic grade line are parallel straight lines separated by the velocity head except in the vicinity of the inlet where the flow passes through a contraction.

The headwater and tailwater condition as well as the entrance, friction, and exit losses are also shown in Figure 9-15. Equating the total energy at sections 1 and 2, upstream and downstream of the culvert barrel, the following relationship results:

$$HW_o + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + H_L \qquad \text{(Equation 9-13)}$$

Where:

$HW_o$ = headwater depth above the outlet invert in feet,
$V_u$ = approach velocity in feet per second,
TW = tailwater depth above the outlet invert in feet,
$V_d$ = channel velocity downstream of the culvert in feet per second, and
$H_L$ = sum of all losses in feet.

In most cases the approach velocity ($V_u$) and the downstream velocity ($V_d$) can be neglected. This depicts the case of a culvert discharging from pool to pool. When the upstream and downstream velocity head are assumed to be zero, the hydraulic gradeline and energy gradeline at Section 1 and Section 2 are coincident. Equation 9-13 then reduces to:

$$HW_o = TW + H_L \qquad \text{(Equation 9-14)}$$

When the approach velocity ($V_u$) is assumed to be zero

$$HW_o = HW + (L)(S) \qquad \text{(Equation 9-15)}$$

Where:

S = barrel slope in feet per foot

Substituting Equation 9-15 into Equation 9-14, the equation for headwater depth, HW, becomes

$$HW = TW + H_L - (L)(S) \qquad \text{(Equation 9-16)}$$

If it is desired to include the approach and/or downstream velocities, use Equation 9-8 for exit losses and Equation 9-13 instead of Equation 9-16 to calculate the headwater.

To accurately determine the headwater depth for the conditions shown in Figure 9-14d and 9-14e, the water surface profile through the culvert barrel must be calculated. Fortunately an approximate method of computation is available as explained below. The approximate method gives satisfactory results for headwater depths above 0.75D, where D is the height of culvert barrel.

For the condition shown in Figure 9-14d, the culvert must flow full for part of its length. The hydraulic gradeline for the portion of the length in full flow will pass through a point where the water breaks with the top of the culvert as represented by point A in Figure 9-14d. Backwater computations show that the hydraulic gradeline, if extended as a straight line, will cut the plane of the outlet cross section at a point above critical depth (water surface). This point is at a height approximately equal to one half the distance between critical depth and the crown of the culvert $[(d_c+D)/2]$. The elevation of this point can be used as an equivalent hydraulic gradeline; and $H_L$, as determined by Equation 9-12 or the nomographs, can be added to this elevation to find the water surface elevation of the headwater pool.

The full flow conditions for part of the barrel length, Figure 9-14d, will exist when: the headwater depth HW is equal to or greater than the quantity:

$$HW \geq D + (1 + K_e)\left(\frac{V^2}{2g}\right) \qquad \text{(Equation 9-17)}$$

Where:

HW = headwater depth in feet,
V = velocity of full flow in the culvert barrel in feet per second,
$K_e$ = entrance loss coefficient, and
D = inside height of the culvert in feet.

If the headwater is less than the above value, a free water surface as shown in Figure 9-14e, will extend through the culvert barrel.

Headwater depth HW can be expressed by a common equation for all outlet-control conditions, including all depths of tailwater. This is accomplished by designating the vertical dimension from the culvert invert at the outlet to the elevation from which H is measured as $h_o$. The headwater depth HW equation is:

$HW = H_L + h_o - (L)(S)$ (Equation 9-18)

Where:

$H_L$ = sum of all losses in feet computed by Equation 9-12 or from the nomographs,
$h_o$ = tailwater depth or $(d_c+D)/2$, in feet, which ever is greater (see Note),
L = length of the culvert barrel in feet, and
S = barrel slope in feet per foot.

*Note:*

*$h_o$ will equal the tailwater depth for conditions shown in Figure 9-14a, 9-14b, and 9-14c.*
*$h_o$ will either equal the tailwater depth or $(d_c+D)/2$, whichever is greater, for the conditions shown in Figure 9-14d and 9-14e.*

### 9.19.3 Outlet Control Nomographs

Outlet control nomographs solve the equation

$$H = \left(1 + K_e + \frac{29n^2L}{R^{1.33}}\right)\left(\frac{V^2}{2g}\right)$$

for head H when the culvert barrel flows full for its entire length. They are also used to determine head H for some part-full flow conditions with outlet control. These nomographs do not give a complete solution for finding headwater HW, since they only give $H_L$ as calculated in Equation 9-12. Headwater is calculated by incorporating H into Equation 9-18, as follows:

$HW = H_L + h_o - (L)(S)$ (see discussion for "Outlet Control Hydraulics").

Instructions for Use

1.0 To determine **head H** given a culvert size and a discharge Q.

a. Locate the appropriate nomograph in Appendix A for the type of culvert selected. Find $K_e$ for the entrance type in Table 9-3. Determine culvert barrel Manning's "n" value using **Chapter 8** Appendix A or methods in **Chapter 8**.

b. Begin in nomograph solution by locating the starting point on the length scale. To locate the proper starting point on the length scales, follow the instructions below:

(1) If the n value of the nomograph corresponds to that of the culvert being used, select the length curve for the proper $K_e$ and locate the starting point at the given culvert length. If a $K_e$ curve is not shown for the selected $K_e$, see (2) below. If the n value for the culvert selected differs from that of the nomograph, see (3) below.

(2) For the n of the nomograph and a $K_e$ intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two chart scales in proportion to the $K_e$ values.

(3) For a different roughness coefficient ($n_1$) than that of the chart n, use the length scales shown with an adjusted length $L_1$, calculated by the formula:

$$L_1 = L\left(\frac{n_1}{n}\right)^2$$

c. Using a straightedge, connect the point on the length scale to the size of the culvert barrel and mark the point of crossing on the "turning line". See Instruction 2.0 for size considerations for a rectangular box culvert.

d. Pivot the straightedge on this point on the turning line and connect with the discharge. Read the head in feet on the head (H) scale. For values beyond the limit of the chart scales, find H by solving Equation 9-12.

2.0 To use the box culvert nomograph for full-flow for other than square boxes.

a. Compute the cross-sectional area of the rectangular box.

b. Connect the proper point (see instruction 1.0) on the length scale to the barrel area and mark the point on the turning line. See Note.

*Note: The area scale on the nomograph is calculated for barrel cross-sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 1/2D. For other box proportions, use Equation 9-12 for more accurate results.*

c. Pivot the straightedge on this point on the turning line and connect with the discharge. Read the head in feet on the head (H) scale.

### 9.19.4 Critical Depth

Critical Depth ($d_c$) can be determined by using the mathematical equations in **Chapter 8** or by the critical depth charts in **Chapter 8** Appendix B. **Critical depth should never be greater than the diameter of the pipe or the rise of the box.**

## 9.20 Procedure for Selection of Culvert Size

**Step 1:** List the following data on the Culvert Design Sheet (Appendix B).

a. The hydrological data which includes the method used to compute the peak flow, the drainage area, the natural stream slope, the natural channel shape, the design flood and the check flood.

b. The allowable headwater depth, in feet, as discussed in Section 9.17.

c. The inlet and outlet elevations in feet.

d. The barrel slope in feet per foot.

e. The approximate barrel length in feet.

f. The type of culvert for first trial selection, including barrel material, barrel cross-sectional shape and entrance type.

**Step 2:** Determine the first trial size culvert.

Since the procedure given is one of trial and error, the initial trial size can be determined in several ways:

a. by arbitrary selection,

b. by using an approximating equation such as Q/10 = A (where A = end area) from which the trial culvert dimensions are determined, or

c. by using inlet control nomographs for the culvert type selected. If this method is used, an HW/D must be assumed, say HW/D = 1.5, and using the given Q, a trail size is determined.

If any trial size is too large in dimension because of limited height of embankment or availability of size, multiple culverts may be used by dividing the discharge equally between the number of barrels used. Raising the embankment height or the use of pipe-arch and box culverts with width greater than height should be considered. Final selection should be based on an economic analysis.

**Step 3:** Find headwater depth for trial size culvert.

a. Assuming INLET CONTROL

i. Using the trial size from Step 2, find the headwater depth (HW) by use of the appropriate inlet control nomograph. Tailwater (TW) conditions are to be neglected in this determination. HW in this case is found by multiplying HW/D obtained from the nomographs by the height of culvert D.

ii. If HW is greater or less than allowable, try another trial size until HW is acceptable for inlet control before computing HW for outlet control.

b. Assuming OUTLET CONTROL

i. Calculate the depth of tailwater TW, in feet, above the invert at the outlet for the design flood condition in the outlet channel. (See general discussion on tailwater)

ii For a tailwater elevation equal to or greater than the top of the culvert set $h_o$ equal to TW and HW by the following equation (Equation 9-18).

$$HW = H + h_o - LS$$

Where:

HW = vertical distance in feet from culvert invert (flow line) at entrance to the pool surface,
H = head loss in feet as determined from the appropriate nomograph or Equation 9-12,
$h_o$ = vertical distance in feet from culvert invert at outlet to the hydraulic grade line (in this case $h_o$ equals TW, measured in feet, above the culvert invert),
S = barrel slope in feet per foot, and
L = culvert barrel length in feet

iii. For tailwater TW elevations less than the top of the culvert, find headwater HW by Equation 14 as in b(ii) above except that

$$h_o = \frac{(d_C + D)}{2} \text{ or TW, whichever is greater.}$$

Where:

$d_c$ = critical depth in feet. Note: **$d_c$ cannot exceed D** and can be determined from the methods in **Chapter 8** or the charts in **Chapter 8** Appendix B.
D = Height of culvert opening in feet.

*Note: Headwater depth determined in Step3.b.iii becomes increasingly less accurate as the headwater computed by this method falls below the value D + (1 + $K_e$) $V^2/2g$. (See discussion under "Culvert Flowing Full with Outlet Control.")*

c. Compare the headwaters found in Step 3a and Step 3b (Inlet Control and Outlet Control). The higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected.

d. If outlet control governs and the HW is higher than is acceptable, select a larger trial size and find HW as instructed under Step 3b. (Inlet control need not be checked, since the smaller size was satisfactory for this control as determined under Step 3a).

**Step 4:** Try a culvert of another type or shape and determine size and HW by the above procedures.

**Step 5:** Compute outlet velocities for size and types to be considered in selection and determine need for channel protection. Use procedures in **Chapter 11**.

**Step 6:** Determine invert protection**.**

Use guidelines in **Chapter 5**.

**Step 7:** Record final selection of culvert with size, type, required headwater, outlet velocity, and economic justification.

## 9.21 Example

A culvert at a new roadway crossing must be designed to pass the 50-year event with a 100-year check flood. From the hydrologic analysis the peak flow is determined to be 100 cubic feet per second and the check flow is 120 cubic feet per second. From the field location survey, the roadway designer, and the Region Environmental Coordinator, the following information was obtained:

Design a circular pipe culvert. Consider alternate materials. These could be concrete, corrugated metal, or smooth metal pipe with outward helical spiral ribs. Plastic pipe could be used if the barrel diameter is equal to or less than 60 inches. Steel or aluminum metal pipes could be used. Corrugated metal pipe, if used, would have standard 2-2/3 by 1/2-inch corrugations.

Base the design headwater elevation on the subgrade elevation (507.0 feet) with two feet of freeboard (507.0 – 2.0 = 505.0 feet). Set the inlet at the natural streambed elevation. The overtopping elevation is the roadway surface. It should not be overtopped by the 100-year flood. Determine outlet velocities using methods in **Chapter 11** and the need for invert abrasion protection using the guidelines in **Chapter 5**. Fill out culvert Design Sheets for the alternate materials. These sheets are shown on Plates 1, 2, 3.

The size of a corrugated metal pipes is determined first, as shown on Plate 1. The initial selection is a 48-inch diameter pipe. It produces excessive headwater depths during inlet control. This size is too small, and no further calculations are made. A 54-inch diameter pipe is tried next. Inlet and outlet control elevations are calculated and compared for both the design and check floods. The 54-inch diameter pipe size is adequate for both floods and it flows under inlet control.

The need for invert abrasion protection is determined for the corrugated pipe, as shown in Plate 2. Methods in **Chapter 5** are used, and the culvert has a “Low” abrasion level. An additional increment of wall thickness or polymer coating is needed to resist abrasion. Additional protection may be needed to resist corrosion, and it should be considered before the culvert materials are listed on the Pipe Data Sheet. This is discussed in **Chapter 5**.

| | | |
|---|---|---|
| Inlet Elevation: | | 500.0 feet |
| Stream Slope: | | 0.02 feet per foot |
| Tailwater computed from Manning's: | | 50-year = 3.0 feet |
| | | 100-year = 3.4 feet |
| Approximate Culvert Length: | | 70 feet |
| Surface of Subgrade Elevation: | | 507.0 feet |
| Source of Discharge: | | Overland runoff and highway drainage |
| Waterway bed material: | | Silts, sands, and gravels up to ½ inch in diameter |
| Abrasion history of existing culvert: | | Galvanized CMP in-place for 45 years. Some invert perforation due to corrosion and abrasion |
| Fish passage | | Not needed |
| End treatment | | Mitered ends with slope paving, based on roadway design criteria |

Outlet velocities are calculated for the design and check floods using methods in **Chapter 11**. These are the "brink velocities" discussed in the chapter. The calculations are shown on Plate 2.

The size of a smooth walled pipe is determined next, as shown in Plate 3. Smooth walled pipes are concrete pipe, smooth walled metal pipe with outward helical spiral ribs, corrugated polyethylene Type D or S pipe, or smooth inner wall polyvinyl chloride pipe (SWPVCP).

Inlet control headwater elevation equations and nomographs are not included in Appendix A for mitered ends on smooth walled pipes. The nomograph scale (2) in Appendix A, Chart 3 for mitered ends on corrugated metal pipes will be used to evaluate the smooth walled pipes under inlet control. The outlet velocities calculations are the same as those for the corrugated metal pipe shown in Plate 2.

## 9.22 Specialized Culverts

The preceding sections of this chapter discuss the basics of culvert location and design. This section includes additional information about culverts used in special applications.

### 9.22.1 Culverts with Inlet Boxes and Access Holes

Inlet boxes and access holes can be used to reduce the barrel slope and lower outlet velocity. These features can also be used to provide clearance over the top of the barrel for utilities. This is shown in Figure 9-17.

The hydraulic losses caused by inlets and access holes should be calculated and considered in the design. Grate inlet, box, and access hole losses are discussed in **Chapter 13**. Sediment transport, debris passage, and fish passage should also be considered. These culverts do not provide passage for significant amounts of sediment or debris. They do not provide fish passage.

### 9.22.2 Fish Passage Culverts

It should be assumed that fish passage will be required at all proposed highway-stream crossing projects regardless of stream size unless told otherwise by the ODOT Region Environmental Coordinator. Fish passage policy is discussed in **Chapter 3**.

### 9.22.3 Open-Bottom Culverts

Open-bottom culverts are different than typical closed-conduit culverts such as circular pipes, arch-pipes, and boxes because they span the stream channel and they have no bottom. Their most common use is to provide fish passage because, if properly designed, the natural streambed in these culverts does not hinder fish movement. Another less common use, especially in the larger sizes, is to provide cost-effective alternatives to single-span bridges.

Open-bottom culverts are available in many configurations. One type is the bottomless semicircular arch shown in Figure 9-1. These arches are made from corrugated metal structural plates that are bolted together. They are available in span widths from 6 feet to 25 feet in steel, and span widths of 5 feet to 30 feet in aluminum.

Another type of open-bottom culvert is the 3-sided box shown in Figure 9-1. These boxes are made from reinforced concrete or corrugated metal structural plates bolted together. Steel 3-sided boxes are available in span widths from 9 feet 2 inches to 25 feet 5 inches, aluminum boxes are available in spans from 8 feet 9 inches to 25 feet 5 inches, and concrete 3-sided boxes are available in spans from 12 feet to 42 feet.

Open-bottom culverts of all sizes are “large culverts” as defined in Section 9.3. As a result, the hydraulic study and documentation requirements that apply to large culverts throughout this manual also apply to these culverts. Open-bottom culverts require the following additional documentation.

### 9.22.3.1 Hydraulic Study and Documentation

1. Crown and channel elevations at the culvert ends. These elevations are listed in lieu of the invert elevations in the description of a closed-conduit culvert. The crown is the highest point on the inside of the culvert, as shown in Figure 9-18.

2. Hydraulic data tables for each suitable shape. The hydraulic characteristics of open-bottom culverts of similar rise and span can vary considerably depending on the waterway shape.

3. A description of predicted scour types including elevations. If the channel is expected to degrade or aggrade during the design life of the structure, this should be noted. The expected change in the channel elevation should be listed.

4. If the channel is expected to degrade, the foundation recommendations should take this into consideration. If the channel is expected to aggrade, the crown elevations or the culvert size should consider this. These aspects of the design should be documented.

5. A description of the footing protection. If riprap is used, the class of riprap should be noted, whether or not it is to be grouted, and whether or not a filter blanket is needed under the riprap. A detail showing the approximate dimensions of the footing protection should be included. Typically, the footing elevations are not listed or shown. Usually these elevations are determined during the structural and/or foundation design.

### 9.22.3.2 Minimum Culvert Size

An open-bottomed culvert spans a section of the natural stream channel that is typically composed of loose material such as sands, gravels, etc. Some or all of this material may be scoured away and replaced during large floods. Typically, the loss of bed material occurs during the beginning (rising limb) of the flood and it is replaced as the flood recedes (falling limb).

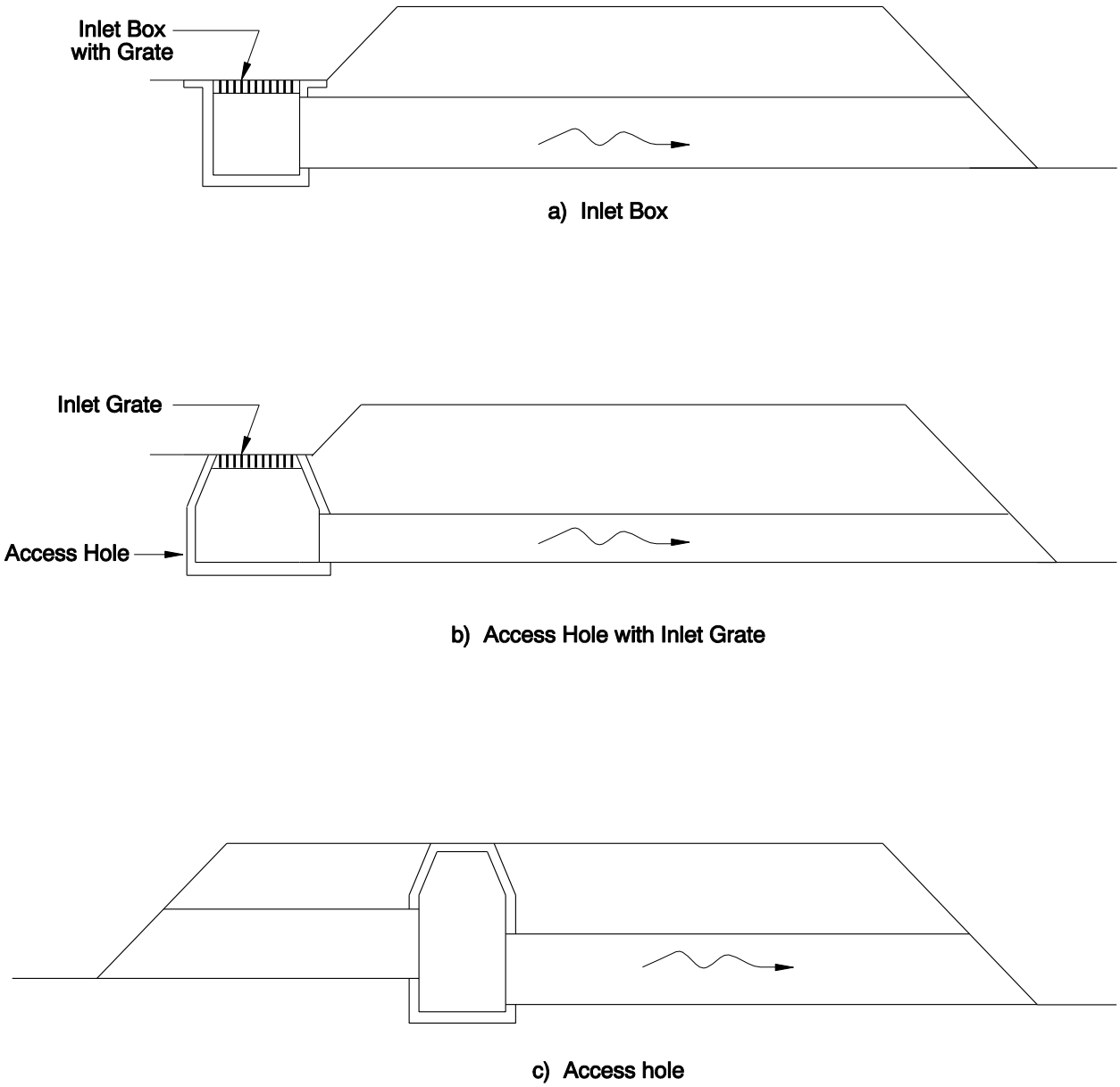



**Figure 9-17 Methods to Reduce Barrel Slope**

The stream may deposit the bed material upstream from the inlet rather than inside the barrel if the culvert is too small to span the active stream channel. In addition, excessive scour may occur inside the barrel. As a result, care must be taken to assure that the culvert meets or exceeds the minimum culvert size requirements listed in Section 9.12.

It is important that maintenance forces have access to the inside of the culvert for inspection or repair. In addition, revetment may be needed to protect the footings in some instances, and it needs to be inspected and replaced if it is scoured away. Clearance requirements should be requested from maintenance personnel on a project by project basis. Typical clearance requirements are an 8-foot by 8-foot minimum opening width to facilitate small equipment if a scour situation develops.

#### 9.22.3.3 End Treatment

There are many open-bottom culverts on the state highway system. Sloped ends with reinforced concrete collars or slope paving are the most common end treatment for metal open-bottom arches. Headwalls and wingwalls are also used. Headwalls with wingwalls are the most common end treatment for the reinforced concrete arches.

The end treatment requirements listed in Section 9.14 also apply to open-bottom culverts. In addition, skew cut sloped ends are not permitted on corrugated metal open-bottom arches unless the end is reinforced by a concrete collar, slope paving, or similar treatment. If an arch with unskewed ends is placed on a skew, the roadway embankment should be contour graded to match the ends as shown on Standard Drawing RD320.

#### 9.22.3.4 Hydraulic Modeling

The inlet and outlet control equations and nomographs described in Section 9.18 can be used to analyze open-bottom culverts if a scour analysis is not needed. This would apply to culverts whose footings are solidly keyed into non-erodible material and permanently protected from the scour associated with turbulent flow, high velocity flow, or pressure flow if they occur. Inlet and outlet control nomographs for many arch shapes are included in Appendix A, and hydraulic properties are listed in **Chapter 8** Appendix B. Hydraulic data for open-bottom culverts with other shapes are listed in manufacturer's trade literature.

A step-backwater analysis is needed if scour elevations must be considered. This applies to culverts whose footings are fully or partially founded on erodible material or soils and rock with an unknown erosion resistance.

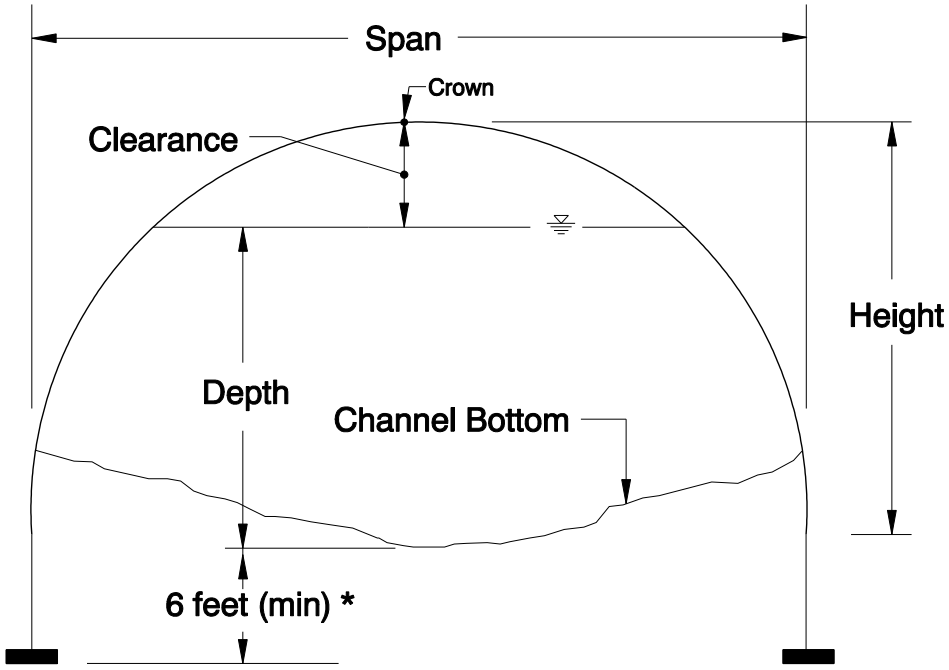



**Figure 9-18 Clearance for Open-Bottom Culvert**

The accuracy required of the hydraulic performance data also influences the study method. Open-bottom arch culverts almost always operate under outlet control, and the outlet control nomographs assume full flow within the barrel when calculating friction losses. This method tends to over estimate friction losses when the culvert barrel is flowing partially full. The step-backwater method can model and accurately calculate friction losses within a partially full barrel.

### 9.22.3.5 Scour

In general, open-bottom culverts with footings solidly keyed into non-erodible rock can withstand a temporary loss of streambed material due to scour during large floods. Open-bottom culverts with this type of foundation are preferred.

Occasionally open-bottom arch culverts are installed with footings on erodible material. This is done over canals, irrigation ditches, and other sites where all of these conditions can be met:

- the flows are regulated, or if not regulated, they can be predicted with certainty,
- the channel is stable in both vertical and horizontal alignment,
- debris passage is not a concern,
- there will be no hydraulic drops or jumps within the barrel or near the ends,
- clearance can be provided between the culvert crown and the water surface during the maximum predicted discharge, as shown in Figure 9-18,
- flow contraction into the culvert barrel is minimal and contraction scour is not a concern, and
- the footing bottoms are located at least six feet below the channel bottom, as shown in Figure 9-18.

Methods to predict scour in open-bottom culverts are being developed by the FHWA. Research indicates the deepest scour in open-bottom culverts occurs at the corners of the culvert inlet where the flow contracts as it enters the culvert barrel. Reports are available on the FHWA website: http://www.fhwa.dot.gov/engineering/hydraulics/scourtech/index.cfm. The conclusions and recommendations from these studies should be considered when estimating scour.

### 9.22.3.6 Footing Protection

Typically, the foundation of an open-bottom culvert will be one of these three types.

1. The footings for an existing culvert, the footings for an extension to an existing culvert, or the footings for a culvert proposed at a new location; are, or can be, keyed into solid non-erodible rock. Revetment is not needed to protect the footings in these cases.

2. The footings of an existing culvert are keyed into rock with questionable erosion resistance, or, it is not known with absolute certainty that the footings for a proposed extension or new culvert can be keyed into solid rock for their entire length. Cores, potholes, or other subsurface exploration indicates rock is present at footing depth. Revetment is needed in these cases to assure the footings are protected.

The footings are founded on erodible rock or loose material and the culvert meets the criteria listed in the previous subsection. In this case, the footings are protected by revetment.

A generic footing protection revetment detail is shown in Figure 9-19. The upper surface of the riprap is covered by native streambed material. This provides a natural channel bottom.

Footing protection is designed to resist hydraulic forces from the 25 or 50 year design event, or 100-year flood if the culvert is in a FEMA regulated floodway. The revetment is checked to verify it will remain intact during the more frequent of; the roadway overtopping flood or the 500-year flood.

*Note: Footing protection revetment for open-bottom culverts over regulated waterways such as irrigation ditches or canals should be designed using the operating flow. The revetment should be checked to verify it can withstand the greatest flow expected in the waterway. This maximum flow may occur during a flood or storm event if the waterway intercepts and collects runoff.*

Two methods of sizing revetment are recommended. They are as follows.

1. Use the tractive force method in **Chapter 15** with the following assumptions.

    a. The rock riprap specific gravity ($S_s$) is 2.65 unless laboratory tests on the rock indicate otherwise. Use laboratory test results if available.

    b. The angle of repose ($\phi$) for riprap produced to ODOT specifications is 41 degrees.

    c. The stability factor (SF) cannot be lower than 1.6

    d. The average flow velocity ($V_a$) and average flow depth ($d_{avg}$) are for flow within the culvert.

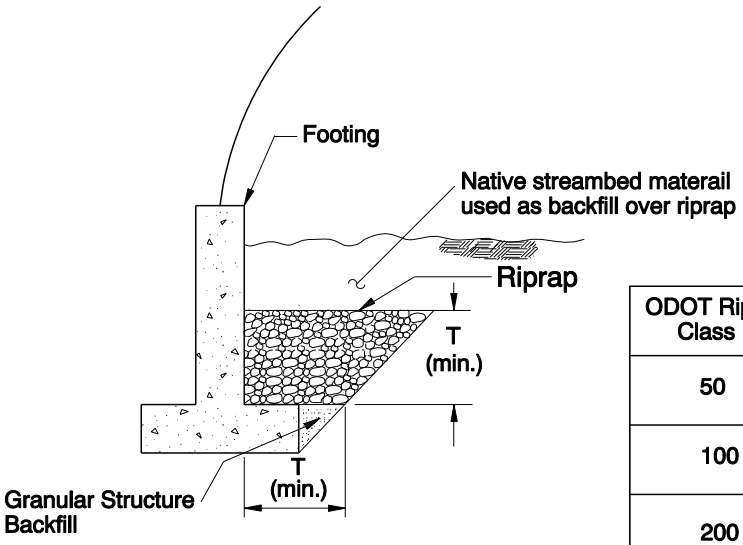
2. Use the ODOT velocity based method in **Chapter 15** with a velocity multiplier of 1.33.

3. Use the modified Isbash relationship in **Chapter 15** with the following assumptions.

    a. The coefficient “K” for vertical wall abutments is appropriate.

    b. The rock riprap specific gravity ($S_s$) is 2.65 unless laboratory tests on the rock indicate otherwise. Use test results if they are available.

Considerable judgment is needed in the application of these methods. The median diameter ($D_{50}$) of the riprap should be larger than the greater of the following: the appropriate calculated value, or two to three times bigger than the largest particles transported by the stream. ODOT riprap $D_{50}$ values are listed in **Chapter 15**.




| ODOT Riprap Class | T (ft) |
|---|---|
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |

**Figure 9-19 Typical Footing Protection Detail**

### 9.22.4 Sag (Inverted Siphon) Culverts

A sag culvert, sometimes called an inverted siphon, is a culvert with multiple sections, as shown in Figure 9-20. The end section inverts are at the elevations of the stream flowlines, and the central section is lower than the end sections. These culverts are used to convey flow under an obstruction such as a highway, railroad, or utility. These culverts are commonly used for conveying irrigation water under highways. In these instances, the elevation changes at each end of the culvert occur in reinforced concrete boxes called “siphon boxes” where the end pipes enter the box at higher elevations than the center pipe. These culverts can provide fish passage.

These culverts can collect sediment because they are the lowest points in the stream channel. A sediment trap may be needed upstream from the culvert if the waterway conveys sediment. There should be provisions for cleaning the culvert such as manholes to allow access to the depressed central section if a sediment trap cannot be provided. The culvert can be cleaned during the non-irrigation season if it is on a canal. Standing water may need to be pumped out to clean the culvert.

*Note: Difficulties in the inspection of these types of culverts maybe a concern to regulatory fish agencies. They are unlikely to be approved unless inspection access is provided to the depressed central section when fish passage is required. This could be accomplished by installing manholes to allow access to the depressed central section or providing provisions for underwater inspections. Consult with the Region Environmental Coordinator or project environmental contact.*

The design of sag culverts is beyond the scope of this manual. It is discussed in detail in the culverts chapter of the Association of State Highway and Transportation Officials (AASHTO) "Model Drainage Manual."

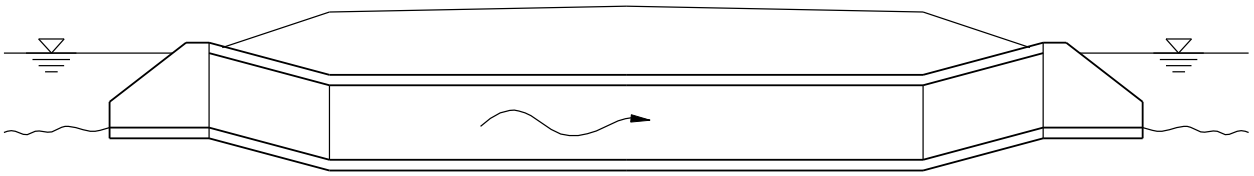


**Figure 9-20 Sag (Inverted Siphon) Culvert**

### 9.22.5 Detour Culverts, Construction Access Road Culverts, and Bypass Pipes

Detour culverts are conduits under temporary roads that carry public traffic while the highway construction is in progress. Construction access road culverts are temporary installations under roads used to get equipment to and from the site. Bypass pipes are conduits that carry flow through the construction site so work can be done "in the dry." Detour and access road culverts are usually larger pipes. Bypass pipes are often small diameter flexible pipes that can be moved as needed to keep them away from the construction activities.

*Note: Many environmental constraints that apply to permanent culverts also apply to temporary installations. Consult with the Region Environmental Coordinator or project environmental contact.*

The minimum end areas of detour and access road culverts are calculated by the agency and included in plans or specifications along with the period of the year when they can be used and any other requirements. Examples of typical recommendations are:

"Detour or access road culverts in place throughout the entire year should have inverts buried in the stream bottom at least one-third the culvert diameter or rise. The minimum net end area, after subtracting the area filled with streambed material, is ____ square feet."

or

"The minimum end area for a detour or access road culvert in place from the ___ of___ through the ___ of___ is ___ square feet."

During project development the hydraulics designer has little to no information about the detour or road access pipes. These are placed during construction by the contractor. The hydraulics designer uses "worst case" assumptions to estimate the pipe end area, as follows:

- the maximum headwater depth to diameter ratio (HW/D) is less than or equal to one,
- the tailwater depth is less than critical depth,
- the inlet will be a projecting end,
- the barrel will be corrugated metal, and
- the pipe slope will be nearly flat.

Temporary Water Management bypass pipes sizes are estimated by the hydraulics designer to develop the TWM plan and cost estimates. Guidance for the design of Temporary Water Management facilities are discusses in **Chapter 18.** The preceding worst case assumptions are often used. The pipe end areas are not listed on the contract documents. Instead, a table of estimated daily exceedance discharges is provided. Guidelines for exceedance discharge recurrence intervals are in **Chapter 3** and hydrology methods are discussed in **Chapter 7**.

## 9.23 Extending Existing Culverts

Often an existing culvert needs to be extended. This occurs most frequently when a roadway is to be widened. It should be verified that the extended culvert will provide adequate hydraulic performance, fish passage, and adequate service life when an extension is considered. Culvert extensions are subject to several requirements based on pipe materials, as described in **Chapter 5**.

The hydraulic performance of both the existing and extended culvert must be analyzed and compared. Often an inlet with increased hydraulic efficiency can be used to compensate for the increased friction losses caused by the longer barrel, and headwater elevations will not be increased by the extension.

An existing culvert meeting fish passage requirements can often be extended if the modified culvert also will meet passage requirements. Extension of a culvert, where the existing culvert must be modified to pass fish, is often considered to be a "retrofit." These are not always allowed by the regulatory agencies. Any culvert extension involving fish passage where the existing section must be modified to provide fish passage should be carefully reviewed with the project environmental contact in the early stages of the project, and prior to detailed design.

The existing culvert to be extended must have sufficient remaining service life to last for the design life of the proposed project. Often this will require a culvert replacement rather than extension, or the existing pipe will need to be rehabilitated. Trenchless methods are often used for pipe rehabilitation, as discussed in **Chapter 16**.

## 9.24 Scour at Culvert Outlets

Flow out of culverts is typically fast and shallow in comparison to flow in the downstream channel. In addition, there is often turbulence caused by hydraulic jumps or drops where culvert flow makes a transition to channel flow. These flow transitions can create scour holes in the channel bottom downstream from the outlet. This outlet scour should be considered in culvert design as follows.

1. Place riprap or construct an energy dissipator at the culvert outlet to prevent or minimize scour on ODOT right-of-way or adjacent property. This is the most common solution to outlet scour problems. Outlet scour protection is especially critical for relief culverts and other culverts placed in embankments at elevations higher than the stream flowline.

2. Do not provide scour protection. This can be done if the outlet channel is non-erodible or a scour hole can be tolerated. The scour hole must:

    - not undermine the culvert, embankment, structures, or cause unacceptable damage,
    - be confined to the ODOT right-of-way, or extend off of the right-of-way if suitable easements are obtained from adjacent landowners,
    - not cause intolerable environmental damage, and
    - not obstruct fish passage if fish passage is needed.

The need for energy dissipators should be determined early in the design process. These facilities can significantly increase the cost of the culvert and often require additional right-of-way.

Methods to estimate scour hole size, and procedures to design energy dissipators such as riprap blankets, riprap lined basins, or pipe tee outlet dissipators are presented in **Chapter 11**. Dissipators such as stilling wells, roughness rings inside the culvert barrel, or impact basins are discussed in the FHWA's Hydraulic Engineering Circular No. 14 (HEC-14), "Hydraulic Design

of Energy Dissipators for Culverts and Channels." Scour hole size estimates and the dissipators described in HEC-14.

## 9.25 Embankment Scour near Culverts

Flow into and out of culverts is often turbulent, especially during large floods. This flow can damage the highway embankments around the culvert ends. The locations where this damage is expected are "scour critical areas" and they are protected by revetment and/or a suitable biotechnical method.

The most common scour protection material is loose riprap, and other types are occasionally used, such as gabions or masonry walls. Vegetal bank protection generally is not adequate unless it is founded on a non-erodible underlayer such as riprap, a cable-reinforced articulating block mat, or an equivalent. Vegetation such as bushes are sometimes planted in riprap. Often the riprap is covered by soil before planting. Typically small willows are used and they are planted as close as 2 feet on centers in areas where their roots will be moist or wet during low flow periods. Large plants are not allowed within specified distances of the highway or near the culvert structure. The hydraulics designer should be contacted for guidance if plantings in riprap are considered.

A description of scour protection for critical areas is included in the Hydraulic Report. Typical embankment protection is shown in Figure 9-21. This protection should extend up the embankment to the lower of:

- the elevation shown in Figure 9-21, or
- the elevation of the bottom of the aggregate base.

A) SLOPED OR PROJECTING END

B) SLOPED END WITH SLOPE PAVING

C) HEADWALL AND WINGWALLS

B = Diameter Of Circular Barrel Or Span Of Pipe-Arch, Box, Or Open-Bottom Arch
D = Diameter Of Circular Barrel Or Rise Of Pipe-Arch, Box, Or Open-Bottom Arch
T = Thickness Of Riprap Blanket. See Table

| Riprap Class | "T" Distance |
|---|---|
| 50 | 12 inches |
| 100 | 18 inches |
| 200 | 24 inches* |
| 700 | 36 inches* |

* Riprap Backing Required Between Riprap And Embankment

NOTES: 1. Minimum Elevation Of Top Of Riprap At Inlet And Outlet Is One Diameter (D) Or 1 Foot Higher Than Design Headwater Or Tailwater Elevation, Respectively, Whichever Is Greater.
2. Suitable Vegetation Can Be Planted In Riprap. See Text.
3. Additional Protection May Be Needed Downstream From Culvert Outlet. See Chapter 11.
4. Riprap Required Around Cutoff Wall At Outlet. Riprap Optional Around Inlet Cutoff Wall Of Single Barrel Culvert. Riprap Required Around Inlet Cutoff Wall On Multiple Barrel Culverts.

**Figure 9-21 Typical Culvert Embankment Protection Details**

In general, Class 50 loose riprap is suitable for small and medium culverts, and Class 100 for large culverts. Larger rock is sometimes used when the energy dissipator requires larger riprap, and there may be a cost savings by using larger rock for both the embankment and the dissipator. In other cases, the larger rock may be needed to resist scour. An example is a culvert outlet on a lakeshore. Larger riprap may be needed to resist displacement from the waves on the lake.

Cutoff walls are often used at culvert ends to prevent undermining due to scour. Riprap is placed against the outlet wall. Revetment is also placed against the inlet cutoff wall of multiple barrel or multiple culverts. Riprap is optional around the inlet cutoff wall of single barrel installations. Cutoff wall protection details are provided in **Chapter 11**.

*Note: The typical details described in this subsection are applicable for most installations. Some culverts may require additional protection. It is the responsibility of the designer to evaluate the need for additional protection and to include it in the design.*

## 9.26 Debris Control

The potential for debris should be considered in the design. Accumulation of debris at the culvert inlet can result in the culvert not performing as designed and cause damage to the highway, culvert, and upstream property. The presence and extent of an existing debris passage problem can be determined by contacting maintenance personnel and neighboring landowners, examining the site, watershed, and nearby structures, and by reviewing maintenance records. Refer to Appendix C of this chapter for design guidelines when a debris control structure is necessary.

There are several options for coping with debris. Selection of an option should consider:

- the extent of damage that would occur if the culvert plugs in comparison to the cost of a structure which will pass the debris,
- ease of removing debris from the culvert,
- fish passage, if required, and
- preferences of maintenance personnel.

Options for coping with a debris problem are:

1. Retain the debris upstream of the culvert by installing a trash rack. Use of a trash rack is subject to approval by regulatory agencies reviewing the fish passage design if passage is required.

2. Do not put a trash rack on the main culvert and construct an overflow culvert higher on the embankment to handle flow if the main culvert plugs. A trash rack may be needed upstream from the overflow culvert.

3. Select a structure large enough to pass the debris. In some instances this may require a bridge rather than a culvert.

## 9.27 Piping

Piping is caused by seepage along a culvert barrel which removes fill material, forming a void similar to a pipe, hence the term piping. Fine soil particles are washed out freely along the void and can ultimately cause embankment failure. The water causing seepage typically enters the embankment fill from two locations. One location is an open pipe joint or a hole in the barrel.. The other location is the interface between the barrel and the embankment at the culvert inlet. Precautions against piping include:

- using watertight joints (especially important if culvert operates under pressure flow),
- constructing a headwall or slope paving with a minimum 3-foot deep cutoff wall at the inlet and outlet, or
- placing the culvert in an impermeable bedding.

*Note: It is important that cutoff walls are constructed according to plans. Observations of past projects indicate these walls are often not constructed to the specified depths. Cut-off walls are one of the best features to prevent piping.*

## 9.28 Flotation

Flotation is the failure of a culvert that is caused by buoyancy. Buoyancy is an uplifting force that is produced when the pressure outside the culvert is greater than the pressure inside the barrel. The resulting uplift may cause the culvert or the inlet end of the culvert to rise out of the embankment. Typically, when flotation occurs the inlet is submerged and the culvert is flowing partially full under inlet control. The causes of inlet control can be an inefficient inlet, debris blocking the inlet, or damage to the inlet.

Some types of culverts are more susceptible to flotation than others. Vulnerable culverts are usually:

- relatively large pipes with little depth of cover from the roadway embankment,
- thin walled, light, and flexible barrels such as corrugated metal or plastic,
- barrels with projecting ends or mitered ends without collars or slope paving (ends projecting unduly far out of the fill are especially vulnerable), or
- use a mitered end surrounded by large riprap or boulders.

Detour culverts or culverts under construction are typically the most susceptible to flotation because they usually have projecting ends and little cover. Especially vulnerable are culverts operating under inlet control which, due to stage construction or other reasons, project far upstream from the fill. Countermeasures to prevent flotation include:

- use slope paving, concrete collars, or headwalls and wingwalls to weight down and protect the ends of metal pipes,
- consider the possibility that high flows may occur during construction and assure that uncovered pipes do not project far upstream from the fill,
- use multiple culverts rather than a large single barrel to increase fill cover heights, or
- use heavy rigid barrels such as concrete pipe sections linked with tie bars.

## 9.29 Camber

Culverts installed under moderate to high fills can experience differential settlement. This occurs after the fill settles and compacts and the midsection of the culvert drops to a lower elevation than the outlet. To prevent this from occurring, the culvert is placed with a slightly elevated midsection and the pipe straightens when the fill settles. The increase in midsection elevation is called camber, and it is predicted by the Foundations Engineer. A profile showing the pipe camber needs to be included in the plans.

## 9.30 Flap Gates

A flap gate, often called tide gate, is sometimes used on the end of a culvert to limit backflow through the conduit. The most common use of a flap gate is to prevent the inundation of upland property on one side of the highway from a high tide or flood on the other side of the highway. Although a flap gate may be useful in certain circumstances, some factors to be considered before it is specified are listed below.

1. Hydraulic efficiency. The effect of a flap gate on the hydraulic efficiency of the culvert

should be considered in the design. Typically the effects are analyzed as “other losses,” as discussed in Subsection 9.19.2.

2. Fish passage. Fish passage requirements can preclude the use of a flap gate or require use of a "fish friendly" gate. The ODOT Region Environmental Coordinator or appropriate regulatory agencies should be contacted before use of a tide gate is considered on a fish bearing stream.

3. Environmental concerns. Installation of a flap gate in a coastal location may alter the circulation patterns of fresh, brackish, and salt water. These changes may affect the plants and wildlife in the vicinity. As a result, a flap gate should not be installed unless potential environmental effects are considered. It may not always prevent flooding upstream of the gate. If the gate stays closed too long and there is considerable runoff upstream, the gate could make the upstream flooding worse or provide no flood protection.

4. Flood control concerns. A flap gate should not be used to control flooding unless the effects on all affected landowners are considered. A gate may relieve flooding at one location and make it worse at another.

5. Maintenance. A flap gate may be held open by debris, it can be damaged, and is subject to wear. Periodic inspection and/or maintenance may be needed.

6. Embankment stability. The flow restriction caused by a flap gate may result in a difference in water surface elevations on either side of the highway. The ability of the roadway embankment to withstand the resulting hydrostatic pressure should be considered before a gate is specified.

**ODOT policy addressing tidegates is posted on the ODOT Business Services website. The designer should review this policy and assure the proposed design is in conformance.**

# PLATE 1

Oregon Department of Transportation (01/99)

## CULVERT DESIGN SHEET

| PROJECT Example Problem | STATION 1+00 | DESIGNER ADL | DATE 02/01/90 | SHEET 1 OF 3 |
|---|---|---|---|---|

### HYDROLOGICAL DATA

METHOD: Rational

DRAINAGE AREA: 50 (acres)

STREAM SLOPE: 0.02 (ft/ft)

CHANNEL SHAPE: Trapezoidal

DESIGN AND CHECK FLOWS / TAILWATER

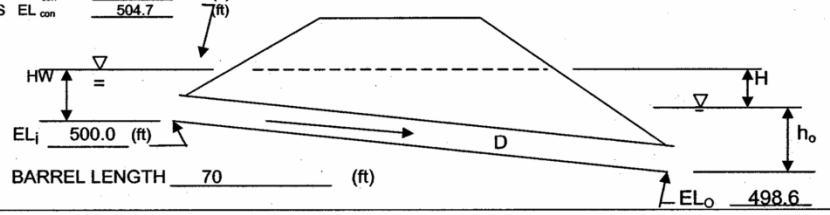
| | R.I. (years) | Q (cfs) | TW (ft) |
|---|---|---|---|
| DES | 50 | 100 | 3.0 |
| CHK | 100 | 120 | 3.4 |

CHECK FLOOD MAXIMUM ALLOWABLE HEADWATER ELEVATION 507.0 (ft)

DESIGN FLOOD MAXIMUM ALLOWABLE HEADWATER ELEVATION 505.0 (ft)

CHK $EL_{con}$ 505.6 (ft)

DES $EL_{con}$ 504.7 (ft)




TECHNICAL FOOTNOTES:

**(1)** USE INLET CONTROL NOMOGRAPH **(2)** $El_{hi} = HW + El_i$ **(3)** TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

**(4)** $h_o$= TW or $\frac{d_c+D}{2}$ (WHICHEVER IS GREATER) **(5)** $H = [1+ k_e +(29n^2L / R^{1.33})](V^2/2g)$ OR USE OUTLET CONTROL NOMOGRAPH

**(6)** $El_{ho} = El_o + H + h_o$ **(7)** USE HIGHER OF $El_{hi}$ or $El_{ho}$ FOR $EL_{con}$

| CULVERT DESCRIPTION<br>MATERIAL – SHAPE – SIZE – ENTRANCE | FLOW PER BARREL Q/N (cfs) | HEADWATER CALCULATIONS<br>INLET CONTROL<br>(1) HW/D | HW (ft) | (2) $EL_{hi}$ (ft) | OUTLET CONTROL<br>(3) TW (ft) | $d_c$ (ft) | $\frac{d_c+D}{2}$ (ft) | (4) $h_o$ (ft) | $k_e$ | (5) H (ft) | (6) $EL_{ho}$ (ft) | (7) $EL_{con}$ (ft) | V (fps) | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| C.M.P. Circ. – 48" – mitered | 100 | 1.4 | 5.6 | 505.6 | | | | | | | | | | H.W. too high |
| C.M.P. Circ. – 54" – mitered | 100 | 1.05 | 4.7 | 504.7 | 3.0 | 2.9 | 3.7 | 3.7 | 0.7 | 1.7 | 504.0 | 504.7 | 9.1 | O.K. for design flood |
| C.M.P. Circ. – 54" – mitered | 120 | 1.25 | 5.6 | 505.6 | 3.4 | 3.3 | 3.9 | 3.9 | 0.7 | 2.4 | 504.9 | 505.6 | 9.2 | O.K. for check flood |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |

COMMENTS/DISCUSSION:

See back of form for invert abrasion protection and outlet velocity calculations.

Slope paving required.

VARIABLE / SUBSCRIPT DEFINITIONS

D = Diameter or Rise of Barrel (ft)
$D_c$ = Critical Depth at Culvert Outlet (ft)
$EL_{con}$ = Control Headwater Elevation
$EL_{hd}$ = Design Headwater Elevation (ft)
$EL_{hi}$ = Inlet Control Headwater Elevation (ft)
$EL_{ho}$ = Outlet Control Headwater Elevation (ft)
$EL_i$ = Inlet Invert Elevation (ft)
$EL_o$ = Outlet Invert Elevation (ft)
G = Acceleration of Gravity (32.2 $ft/s^2$)
H = Total Outlet Control Head Loss (ft)
$h_o$(ft) See Technical Footnote 4
HW = Inlet Control Headwater Depth (ft)
$k_e$ = Outlet Control Entrance Loss Coefficient
N = Number of Barrels
n = Manning's Roughness Coefficient
Q = Discharge ($ft^3/s$)
R = Hydraulic Radius (ft)
R.I. = Recurrence Interval (Years)
TW = Tailwater Depth (ft)
V = Average Velocity in Barrel (ft/s)

INVERT ABRASION PROTECTION

■ YES
NO

TYPE Polymer coating or one increment increase metal thickness for spiral rip pipe. Additional protection may be needed to resist corrosion.

CULVERT BARREL SELECTED

SIZE 54" SHAPE: Circ.

MATERIAL: * n= 0.024 ENTRANCE: mitered

# PLATE 2



Invert abrasion protection calculations (See Chapter 5)

$Q_2$ = 2.6 cfs $Q_{6mo}$ = (0.66) ($Q_2$) = 1.7 cfs = invert abrasion protection design flow
Culvert will flow at normal depth for most of its length during the 1.7 cfs discharge.
Normal depth = 0.49 ft, normal velocity = 4.2 ft/s (see Chapter 8).
Bed materials as large as coarse gravels are present. The 1.7 cfs discharge can carry this material through pipe. Use polymer coating or 1 increment increase in barrel material thickness, as per Chapter 5 for "Low Abrasion" conditions.

Outlet velocity calculations (see Chapter 11) :
Normal flow depth in culvert during 100 cfs design flood = 2.7 ft TW depth = 3.0
Normal flow depth is less than TW depth. Case 2 flow occurs. Flow depth at outlet is TW depth.
d/D = 3.0/4.5 = 0.67 $A_{full} = \pi(4.5/2)^2 = 16$ ft$^2$ From hydraulics element chart in Chapter 8 Appendix B, $a/A_{full}$ = 0.70
a = 0.70 x 16 = 11 ft$^2$ V @ $Q_{100}$ = 100 cfs/11 ft$^2$ = 9.1 ft/s <=
During 120 cfs check flood, normal depth = 3.0 ft, TW depth = 3.4, Case 2 flow occurs
d/D = 3.4/4.5 = 0.75 From elements chart: $a/A_{full}$ = 0.81
a = 0.81 x 16 = 13 ft$^2$ V @ $Q_{120}$ = 120 cfs/13 ft$^2$ = 9.2 ft/s <=

# PLATE 3

Oregon Department of Transportation (01/99)

## CULVERT DESIGN SHEET

| PROJECT Example Problem | STATION 1+00 | DESIGNER ADL | DATE 02/01/90 | SHEET 3 OF 3 |
|---|---|---|---|---|

**HYDROLOGICAL DATA**

METHOD: Rational

DRAINAGE AREA: 50 (acres)

STREAM SLOPE: 0.02 (ft/ft)

CHANNEL SHAPE: Trapezoidal

DESIGN AND CHECK FLOWS / TAILWATER

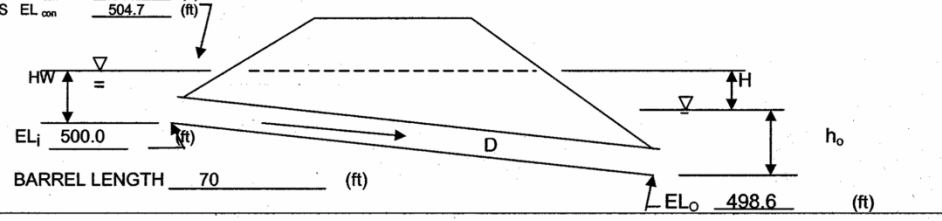
| | R.I. (years) | Q (cfs) | TW (ft) |
|---|---|---|---|
| DES | 50 | 100 | 3.0 |
| CHK | 100 | 120 | 3.4 |

CHECK FLOOD MAXIMUM ALLOWABLE HEADWATER ELEVATION 507.0 (ft)

DESIGN FLOOD MAXIMUM ALLOWABLE HEADWATER ELEVATION 505.0 (ft)

CHK $EL_{con}$ 505.6 (ft)

DES $EL_{con}$ 504.7 (ft)




TECHNICAL FOOTNOTES:

(1) USE INLET CONTROL NOMOGRAPH (2) $EL_{hi} = HW + EL_i$ (3) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(4) $h_o$= TW or $\frac{d_c+D}{2}$ (WHICHEVER IS GREATER) (5) $H = [1+ k_e +(29n^2L / R^{1.33})](V^2/2g)$ OR USE OUTLET CONTROL NOMOGRAPH

(6) $EL_{ho} = EL_o + H + h_o$ (7) USE HIGHER OF $EL_{hi}$ or $EL_{ho}$ FOR $EL_{con}$

| CULVERT DESCRIPTION: MATERIAL – SHAPE – SIZE – ENTRANCE | FLOW PER BARREL Q/N (cfs) | HEADWATER CALCULATIONS: INLET CONTROL (1) HW/D | HW (ft) | (2) $EL_{hi}$ (ft) | OUTLET CONTROL (3) TW (ft) | $d_c$ (ft) | $\frac{d_c+D}{2}$ (ft) | (4) $h_o$ (ft) | $k_e$ | (5) H (ft) | (6) $EL_{ho}$ (ft) | (7) $EL_{con}$ (ft) | V (fps) | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| * Circ. - 54" mitered | 100 | 1.05 | 4.7 | 504.7 | 3.0 | 2.9 | 3.7 | 3.7 | 0.7 | 1.3 | 503.6 | 504.7 | 9.1 | O.K. for design flood |
| * Circ. - 54" mitered | 120 | 1.25 | 5.6 | 505.6 | 3.4 | 3.3 | 3.9 | 3.9 | 0.7 | 1.9 | 504.4 | 505.6 | 9.2 | O.K. for check flood |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |

COMMENTS/DISCUSSION:

* Concrete, smooth wall spiral rib metal, or smooth wall poly (vinylchloride) pipe ASTM F 1803.

Slope paving required.

Headwater elevations same as CMP. Outlet velocities same as CMP. See page 2 for calculations.

VARIABLE / SUBSCRIPT DEFINITIONS

D = Diameter or Rise of Barrel (ft)
$D_c$ = Critical Depth at Culvert Outlet (ft)
$EL_{con}$ = Control Headwater Elevation
$EL_{hd}$ = Design Headwater Elevation (ft)
$EL_{hi}$ = Inlet Control Headwater Elevation (ft)
$EL_{ho}$ = Outlet Control Headwater Elevation (ft)
$EL_i$ = Inlet Invert Elevation (ft)
$EL_o$ = Outlet Invert Elevation (ft)
G = Acceleration of Gravity (32.2 ft/s$^2$)
H = Total Outlet Control Head Loss (ft)
$h_o$(ft) See Technical Footnote 4
HW = Inlet Control Headwater Depth (ft)
$k_e$ = Outlet Control Entrance Loss Coefficient
N = Number of Barrels
n = Manning's Roughness Coefficient
Q = Discharge (ft$^3$/s)
R = Hydraulic Radius (ft)
R.I. = Recurrence Interval (Years)
TW = Tailwater Depth (ft)
V = Average Velocity in Barrel (ft/s)

INVERT ABRASION PROTECTION

■ YES NO TYPE Polymer coating or one increment increase metal thickness for spiral rib pipe. Additional protection may be needed to resist corrosion

CULVERT BARREL SELECTED

SIZE 54" SHAPE: Circ

MATERIAL: * n= 0.012 ENTRANCE: mitered

# APPENDIX A - CULVERT PERFORMANCE CHARTS

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CHART 21 HEAD FOR C.M. ARCH CULVERTS FLOWING FULL EARTH BOTTOM ($n_b$ = 0.022) 0.3 ≤ RISE / SPAN <0.4 ..... 26

CHART 22 HEAD FOR C.M. ARCH CULVERTS FLOWING FULL IN OUTLET CONTROL EARTH BOTTOM ($n_b$ =0.022) 0.4 ≤ RISE / SPAN < 0.5 ..... 27

CHART 23 HEAD FOR C.M. ARCH CULVERTS FLOWING FULL IN OUTLET CONTROL EARTH BOTTOM ($n_b$=0.022) 0.5 ≤ RISE / SPAN ..... 28

These charts provide nomographs to determine inlet and outlet control headwater depths for common culvert shapes. Nomographs for additional culvert shapes are in FHWA Hydraulic Design Series No. 5 “Hydraulic Design of Highway Culverts.”

***Note:*** *The outlet control nomographs provide accurate estimates of headwater depths if there is full flow in the culvert barrel and a submerged outlet. The nomographs may slightly overestimate headwater depths if the barrel flows partially full.*

## INDEX TO CULVERT PERFORMANCE CHARTS

| Chart | Shape | Material | Control | Comments |
|---|---|---|---|---|
| 1 | Circular | Concrete | Inlet | Inlet projecting or in headwall |
| 2 | Circular | Concrete | Inlet | Prefabricated concrete end section |
| 3 | Circular | Metal | Inlet | Corrugated or structural plate pipe with inlet projecting, mitered, or in headwall. **(Use Scale 2 for ODOT sloped end with or without slope paving.)** |
| 4 | Circular | Metal | Inlet | Safety end section with bars **(Use for concrete or metal barrel.)** |
| 5 | Circular | Metal | Inlet | Prefabricated metal end section |
| 6 | Circular | Metal | Inlet | Reinforced concrete beveled ring around inlet |
| 7 | Circular | Concrete | Outlet | |
| 8 | Circular | Metal | Outlet | Corrugated metal pipe |

## INDEX TO CULVERT PERFORMANCE CHARTS, CONTD.

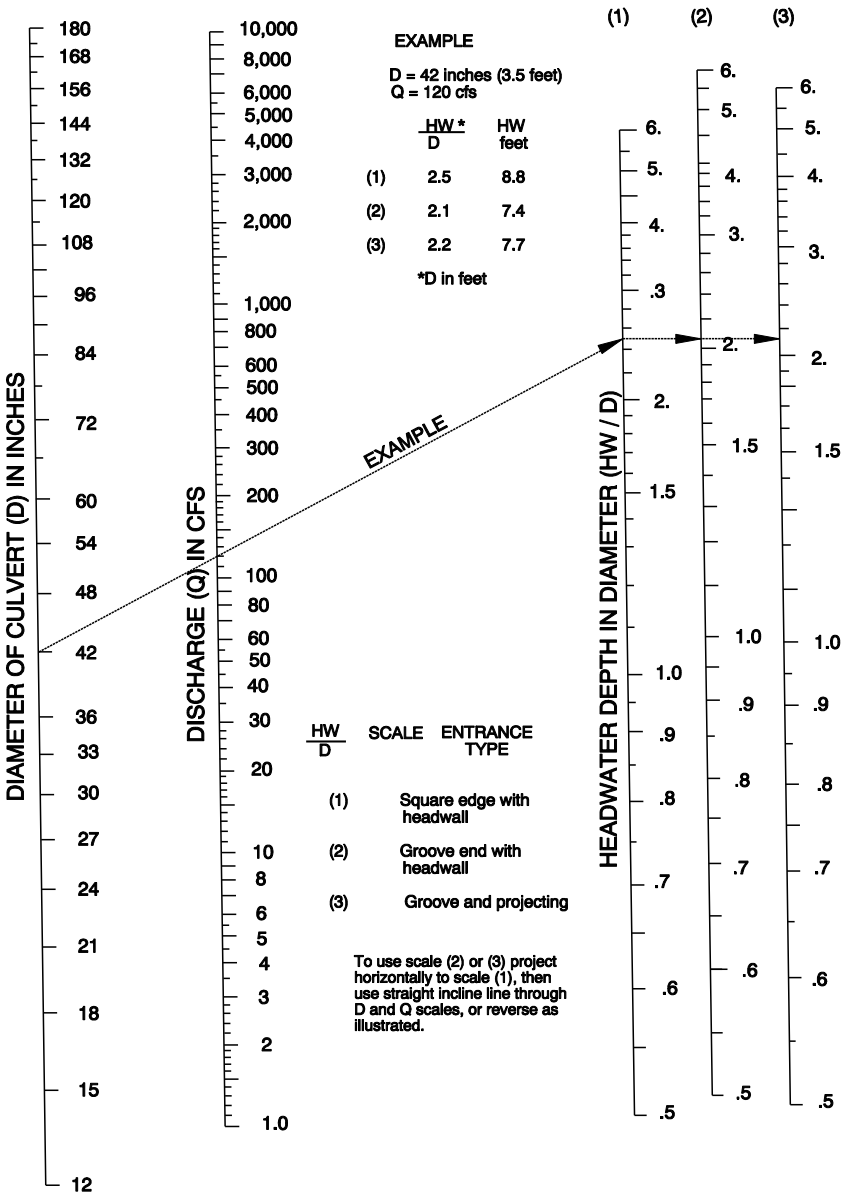
| Chart | Shape | Material | Control | Comments |
|---|---|---|---|---|
| 9 | Circular | Metal | Outlet | Structural plate pipe |
| 10 | Box | Concrete | Inlet | Top edge square with wingwalls |
| 11 | Box | Concrete | Inlet | Top edge beveled with wingwalls **(Use Scale 2 for box culvert shown on ODOT Standard Drawing BR 800.)** |
| 12 | Box | Concrete | Outlet | |
| 13 | Pipe-Arch | Metal | Inlet | Corrugated pipe-arch with inlet projecting, mitered, or in headwall **(Use Scale 2 for ODOT sloped end with or without slope paving.)** |
| 14 | Pipe-Arch | Metal | Inlet | Structural plate pipe-arch with inlet projecting, or in headwall with or without beveled edge and 18-inch corner radius |
| 15 | Pipe-Arch | Metal | Inlet | Structural plate pipe-arch with inlet projecting, or in headwall with or without beveled edge and 31-inch corner radius |
| 16 | Pipe-Arch | Metal | Outlet | Corrugated metal |
| 17 | Pipe-Arch | Metal | Outlet | Structural plate with 18-inch corner radius |
| 18 | Arch | Metal | Inlet | Structural plate arch with inlet projecting, mitered, or in headwall with $0.3 \leq \text{Rise/Span} < 0.4$ |

## INDEX TO CULVERT PERFORMANCE CHARTS, CONTD.

| Chart | Shape | Material | Control | Comments |
|---|---|---|---|---|
| 19 | Arch | Metal | Inlet | Structural plate arch with inlet projecting, mitered, or in headwall with $0.4 \leq$ Rise/Span $< 0.5$ |
| 20 | Arch | Metal | Inlet | Structural plate arch with inlet projecting, mitered, or in headwall with $0.5 \leq$ Rise/Span |
| 21 | Arch | Metal | Outlet | Structural plate arch with earth bottom and $0.3 \leq$ Rise/Span $< 0.4$ |
| 22 | Arch | Metal | Outlet | Structural plate arch with earth bottom and $0.4 \leq$ Rise/Span $< 0.5$ |
| 23 | Arch | Metal | Outlet | Structural plate arch with earth bottom and $0.5 <$ Rise/Span |

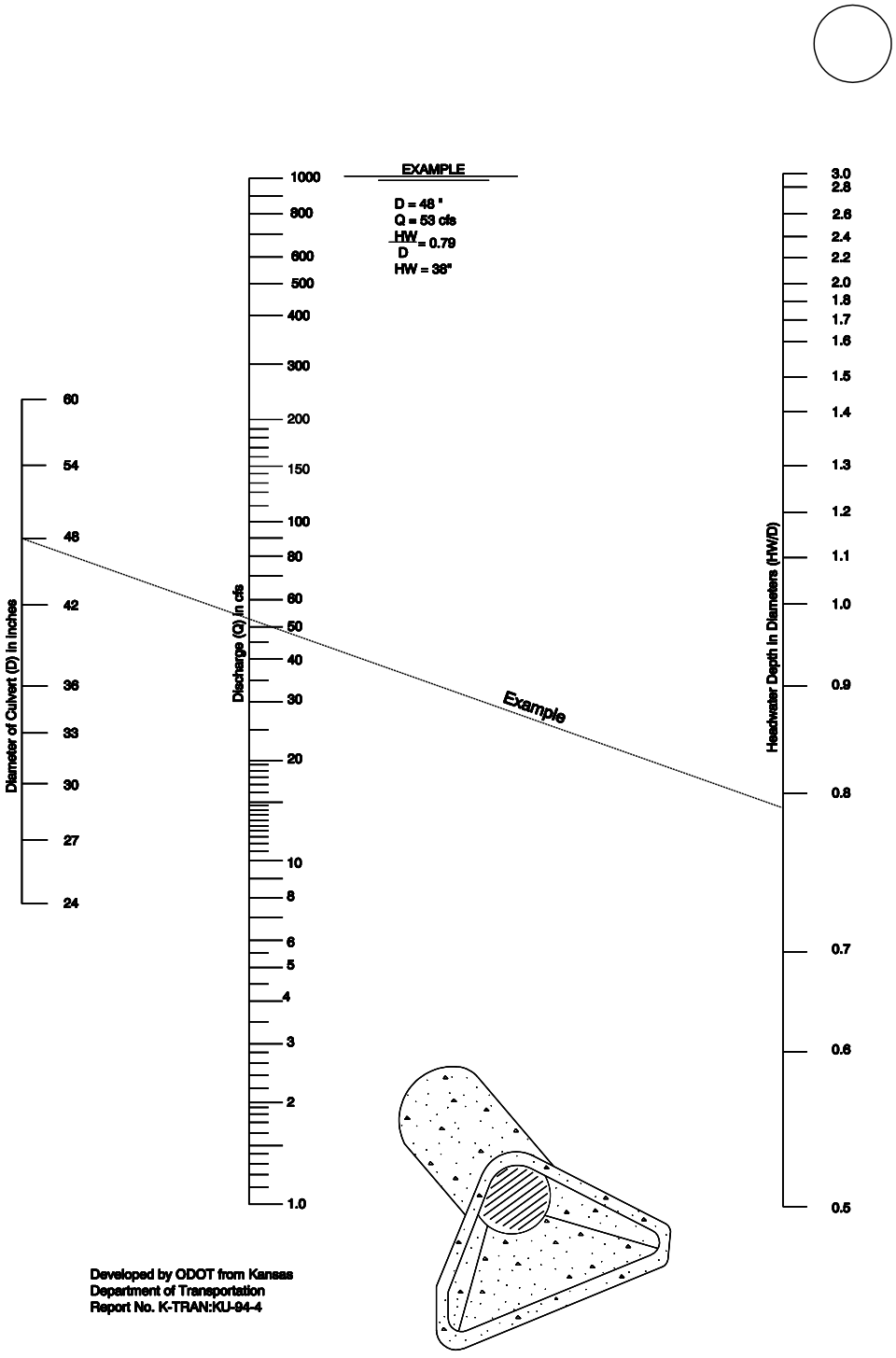
# CHART 1

## HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL



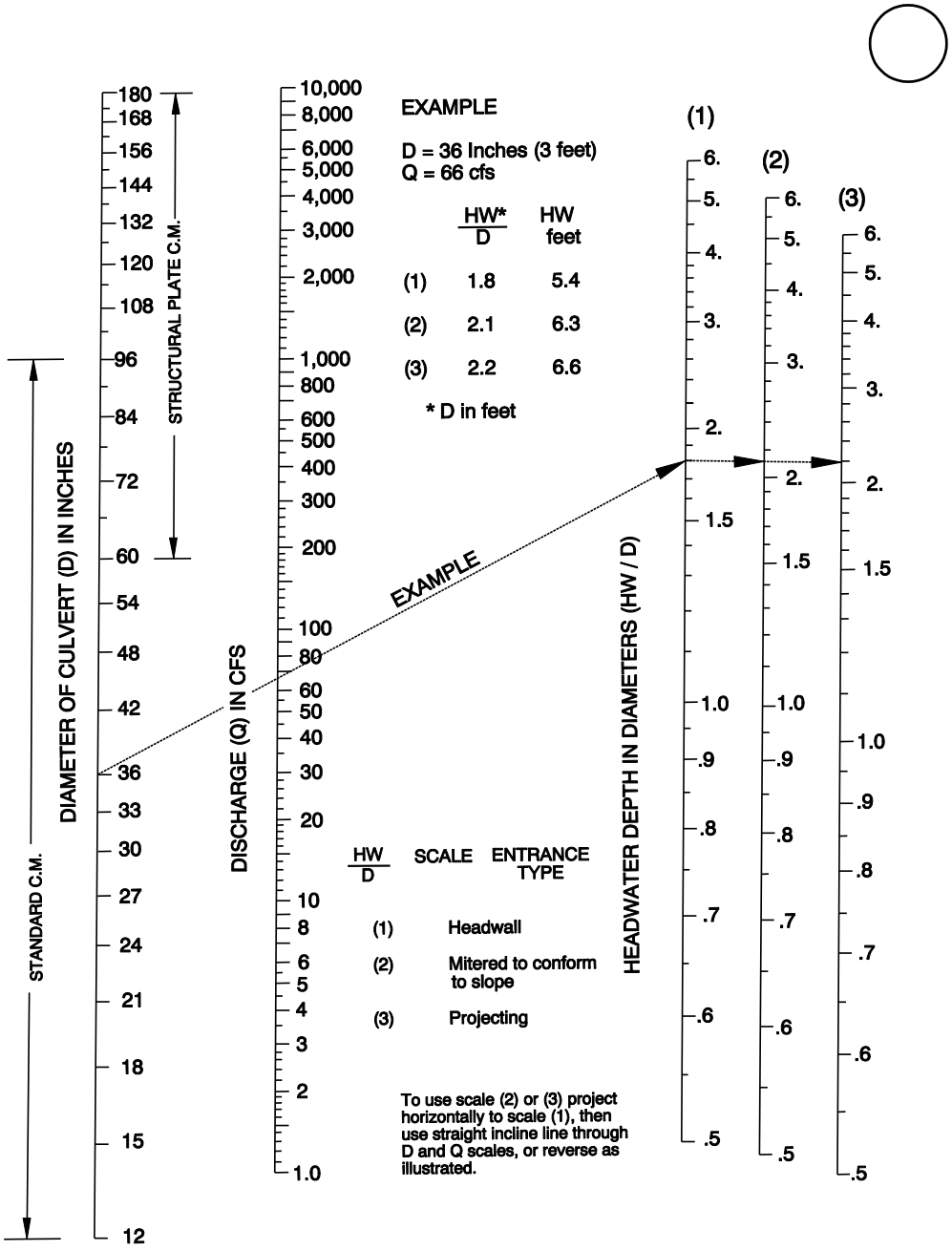
# CHART 2

## HEADWATER DEPTH FOR PREFABRICATED CONCRETE END SECTION IN INLET CONTROL



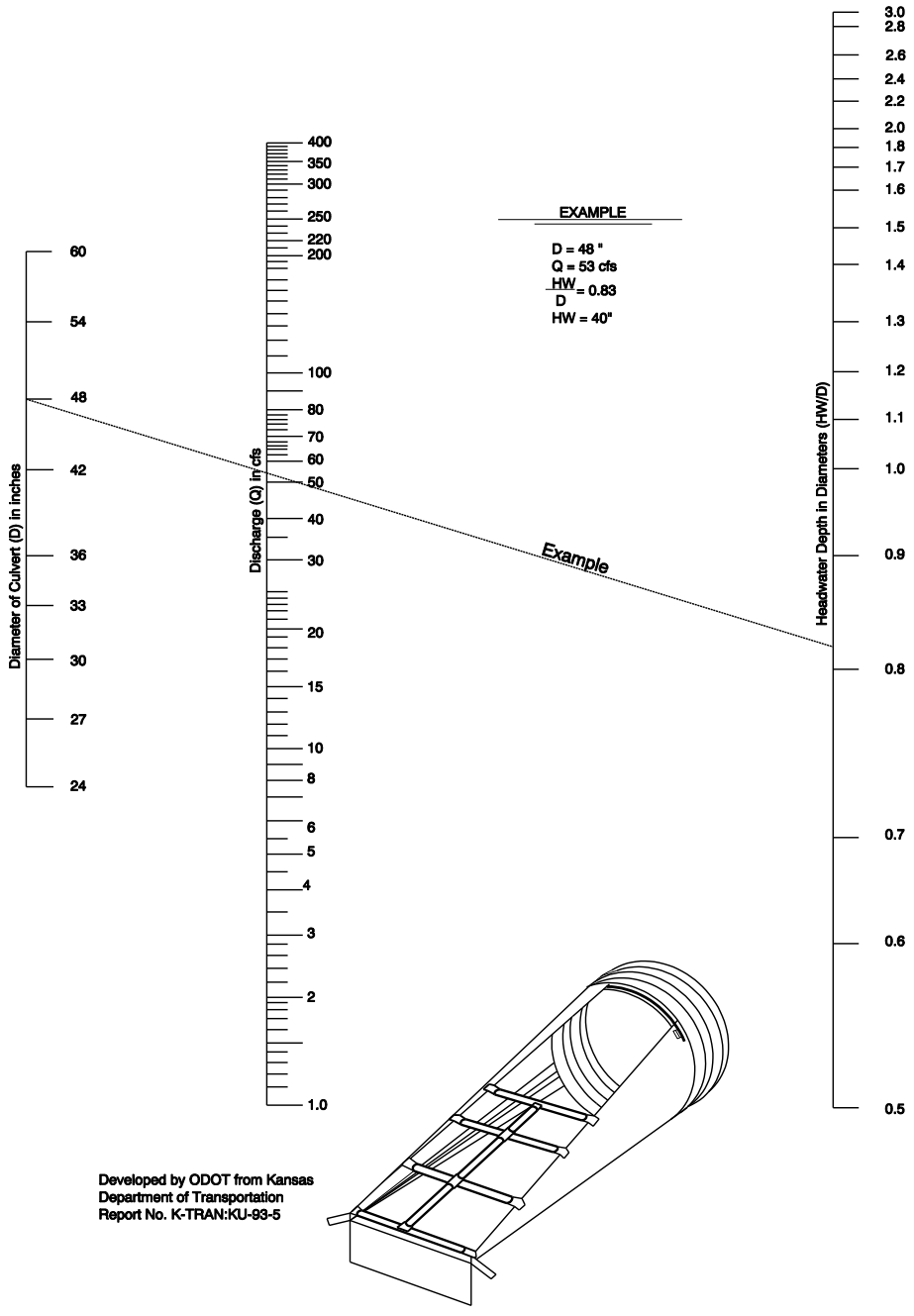
# CHART 3

## HEADWATER DEPTH FOR C.M. CULVERTS WITH INLET CONTROL

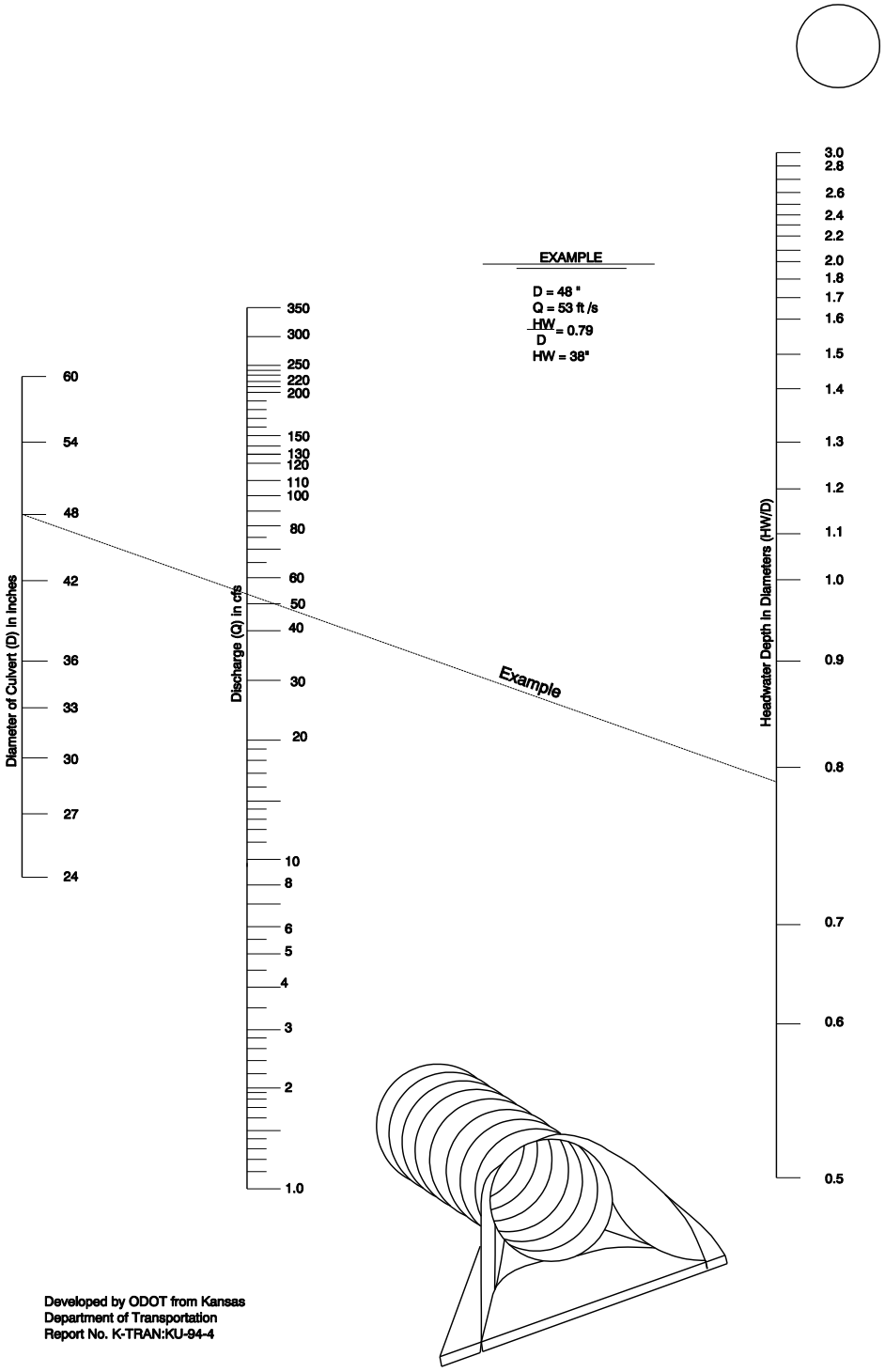


# CHART 4

## HEADWATER DEPTH FOR SAFETY END SECTIONS WITH SAFETY BARS IN INLET CONTROL

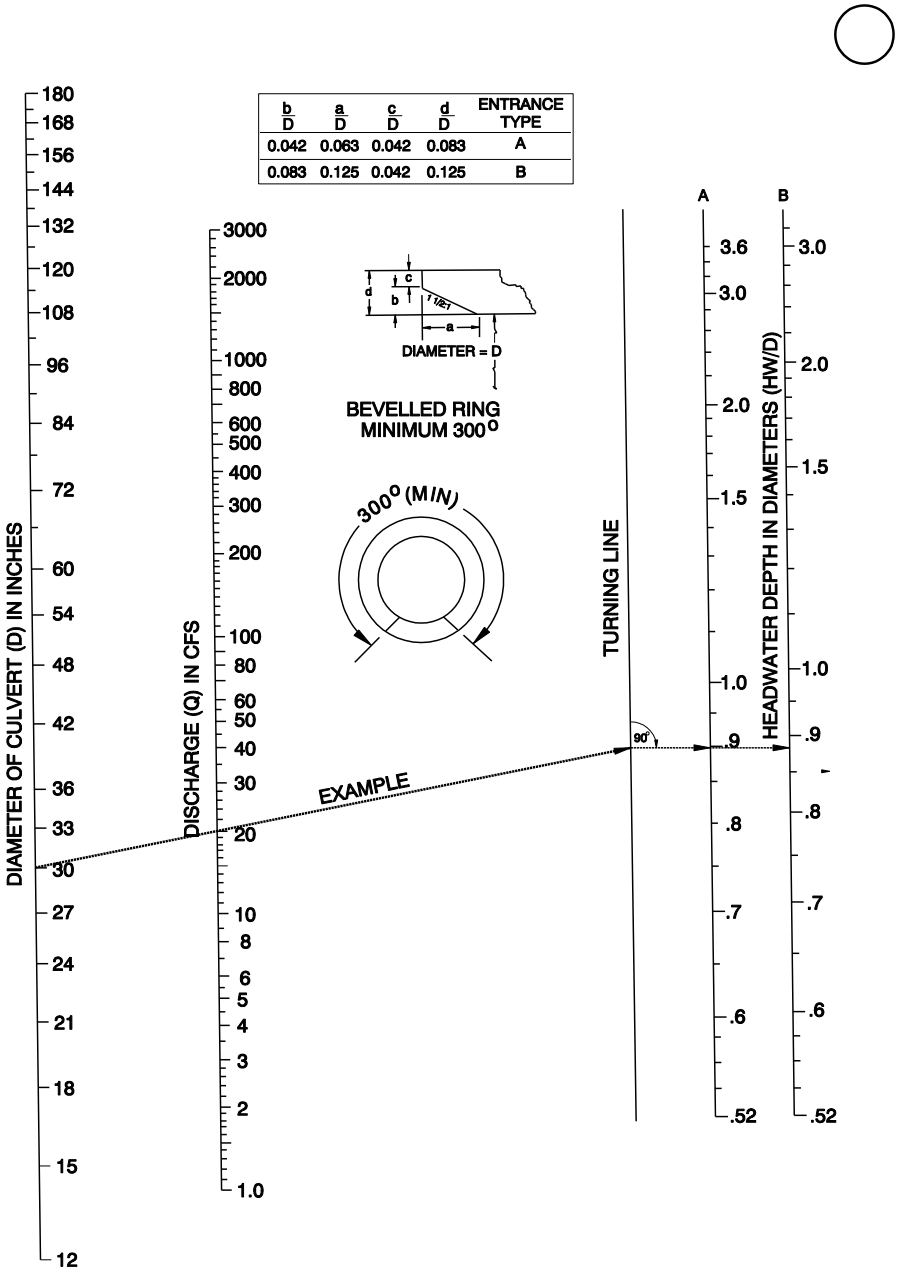


# CHART 5
# HEADWATER DEPTH FOR PREFABRICATED METAL END SECTIONS IN INLET CONTROL



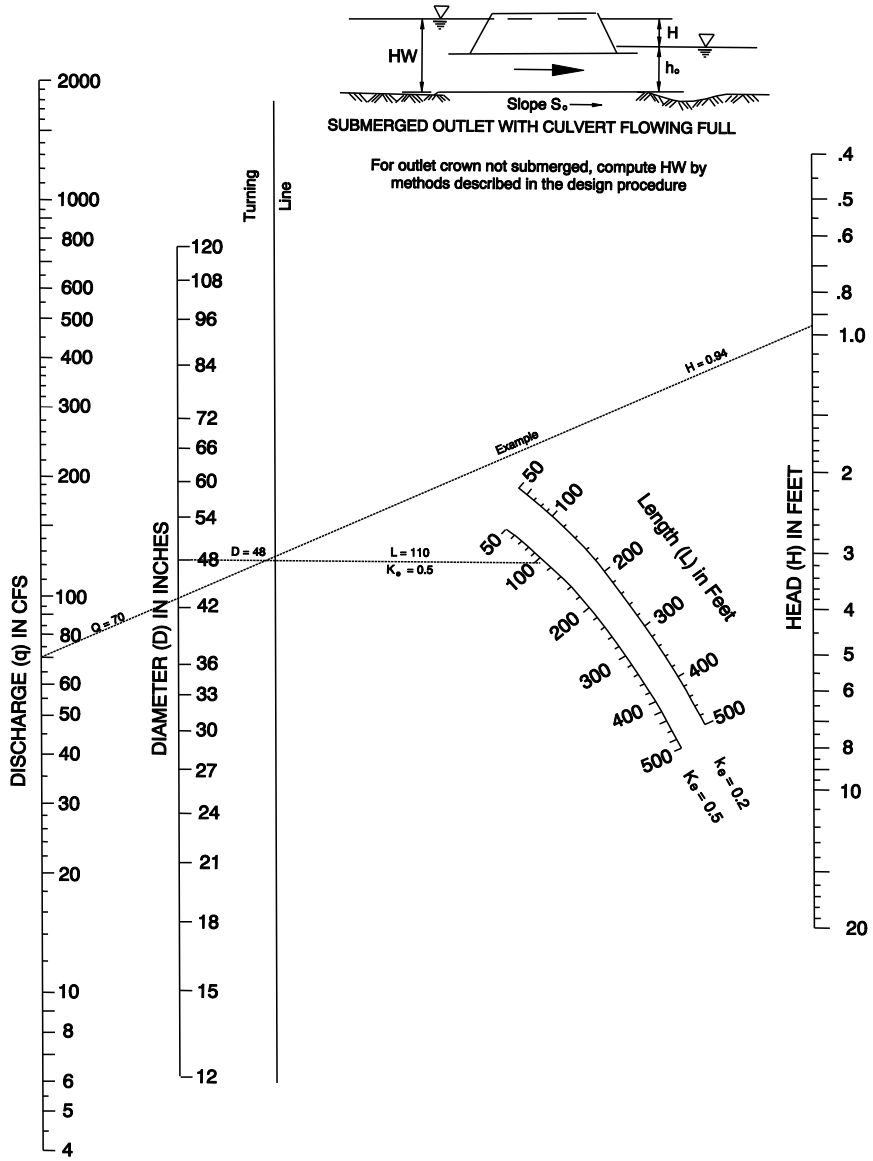
# CHART 6

## HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL



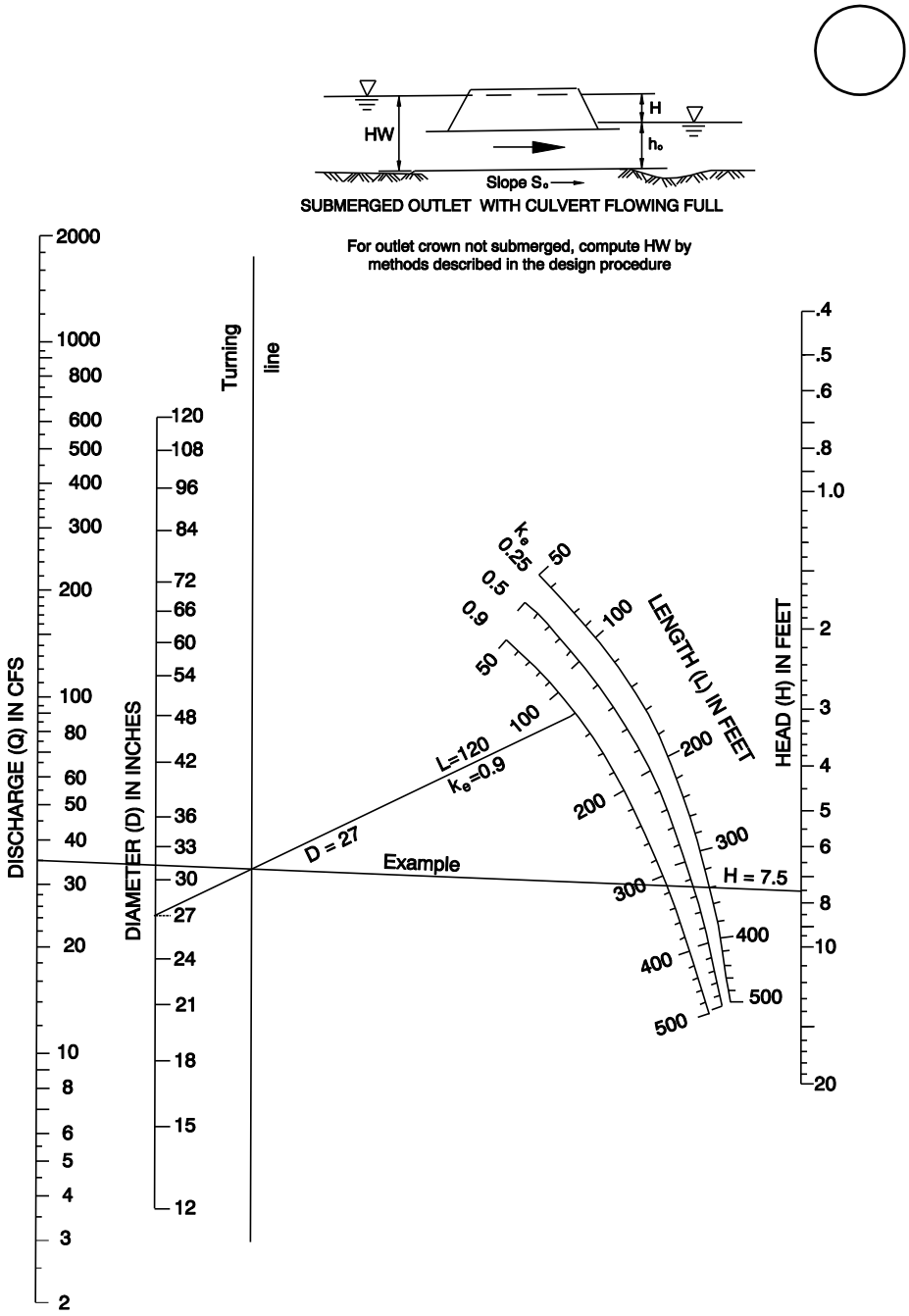
# CHART 7

## HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL IN OUTLET CONTROL n = 0.012



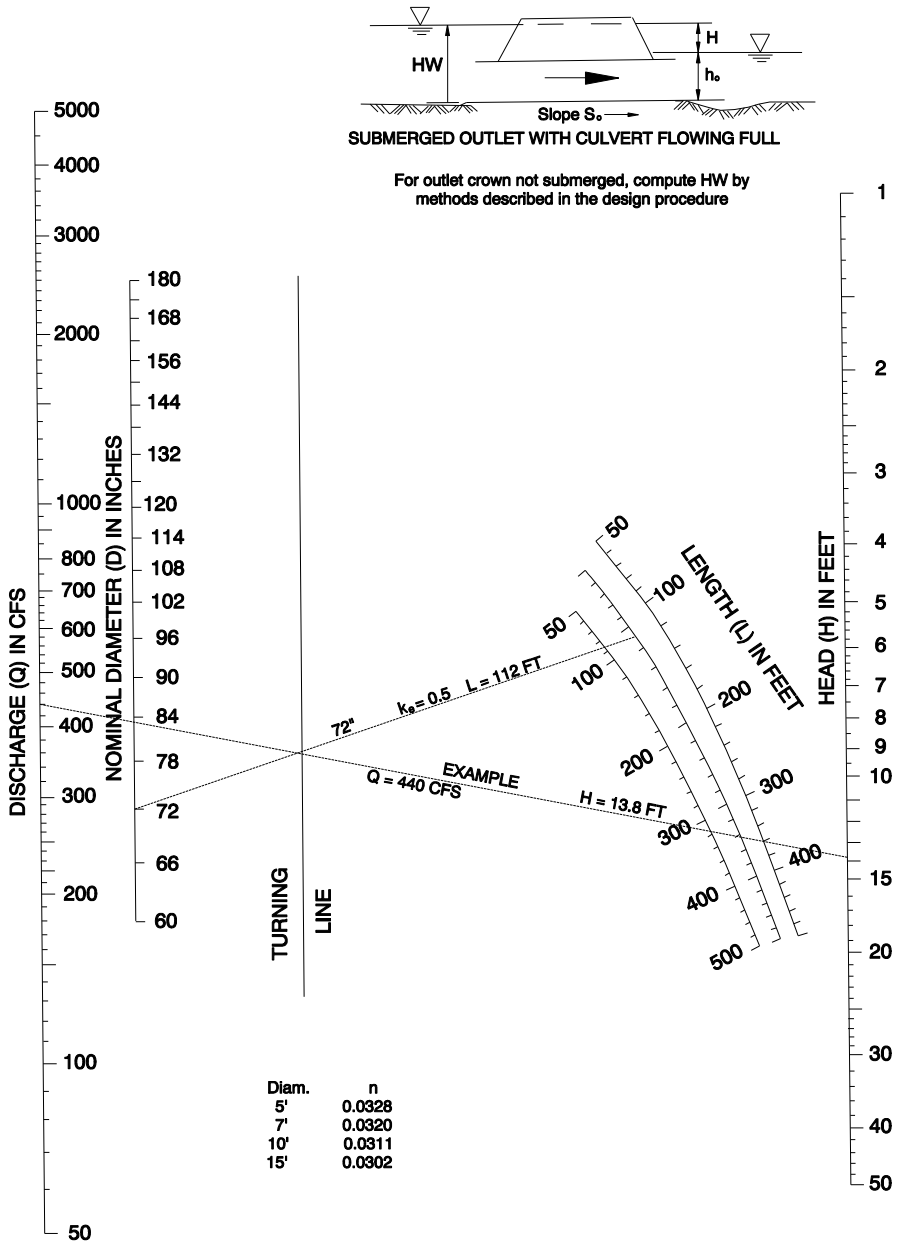
# CHART 8

## HEAD FOR STANDARD C.M. PIPE CULVERTS FLOWING FULL IN OUTLET CONTROL n = 0.024



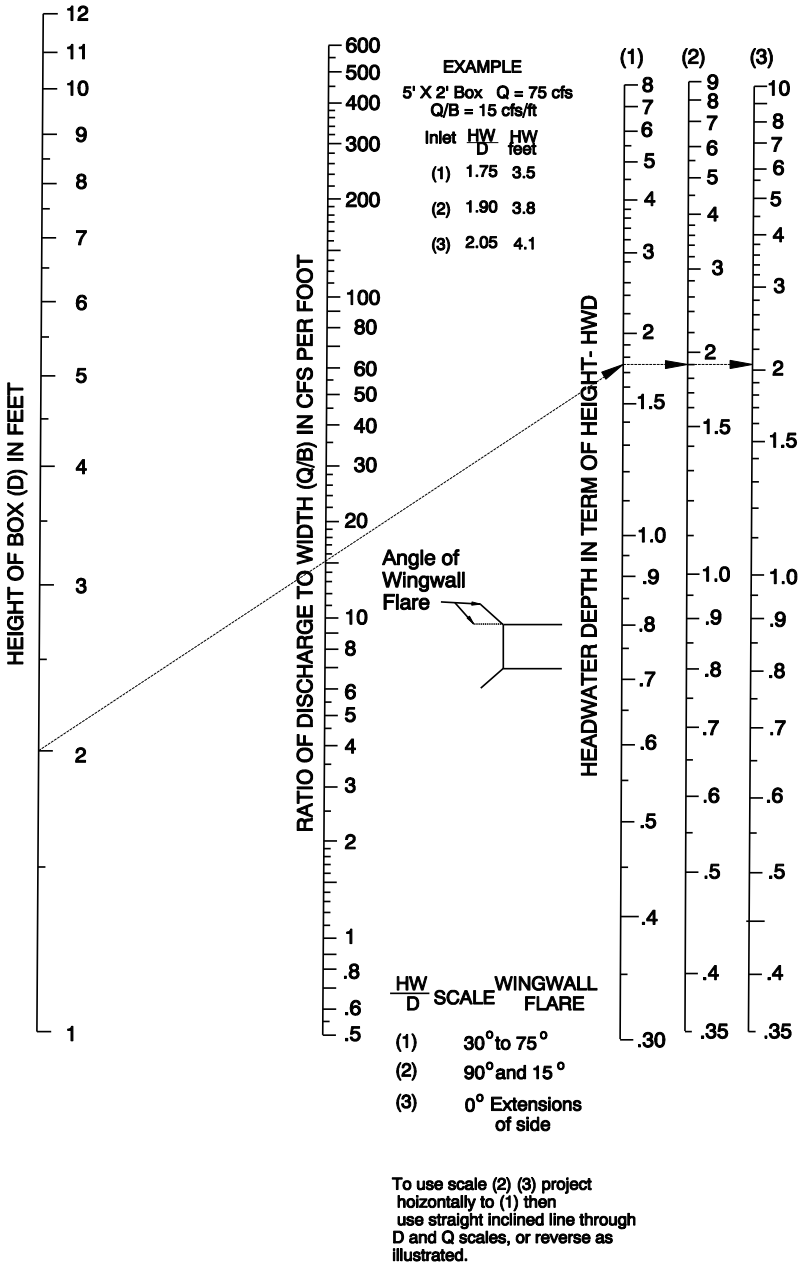
# CHART 9

## HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL IN OUTLET CONTROL N = 0.0328 TO 0.0302



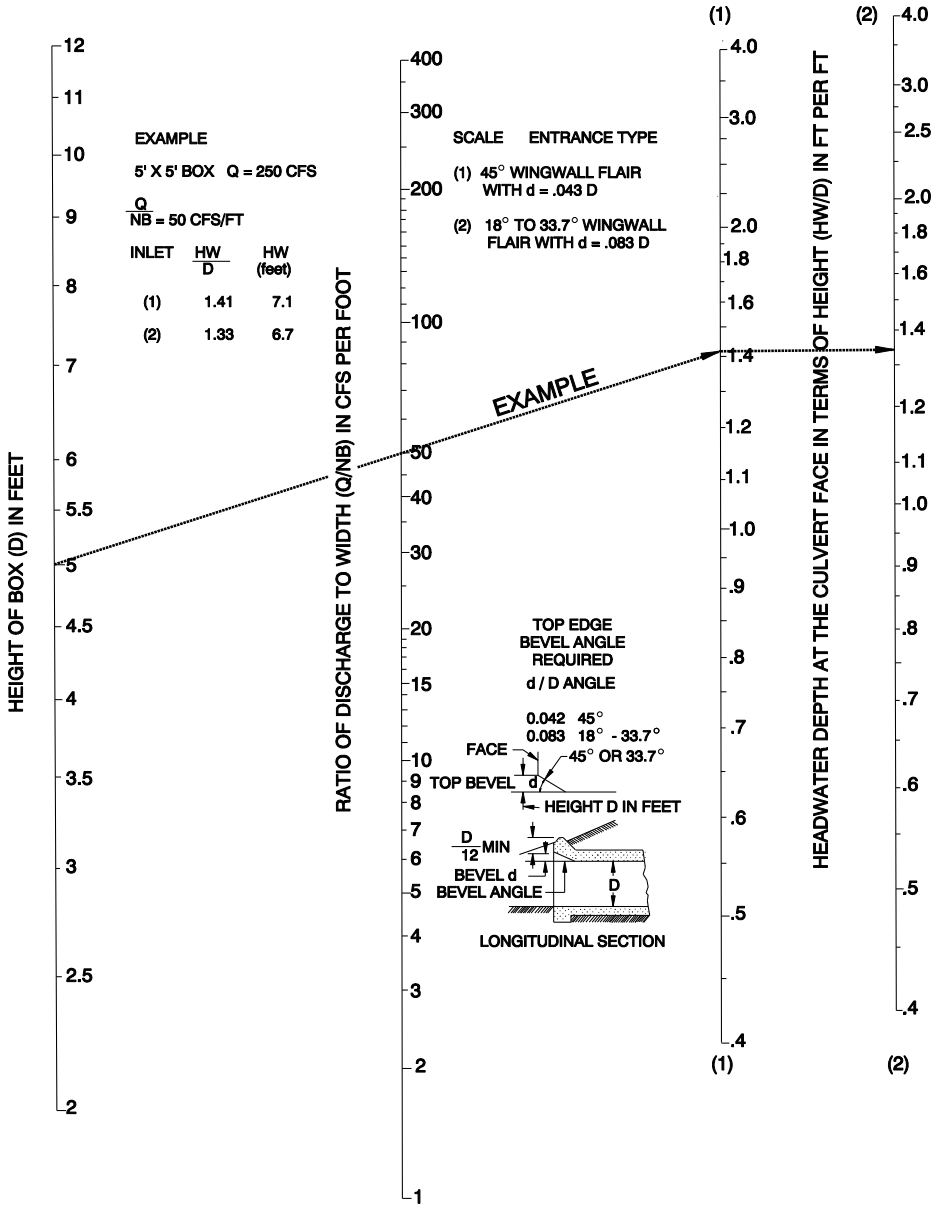
# CHART 10

## HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL



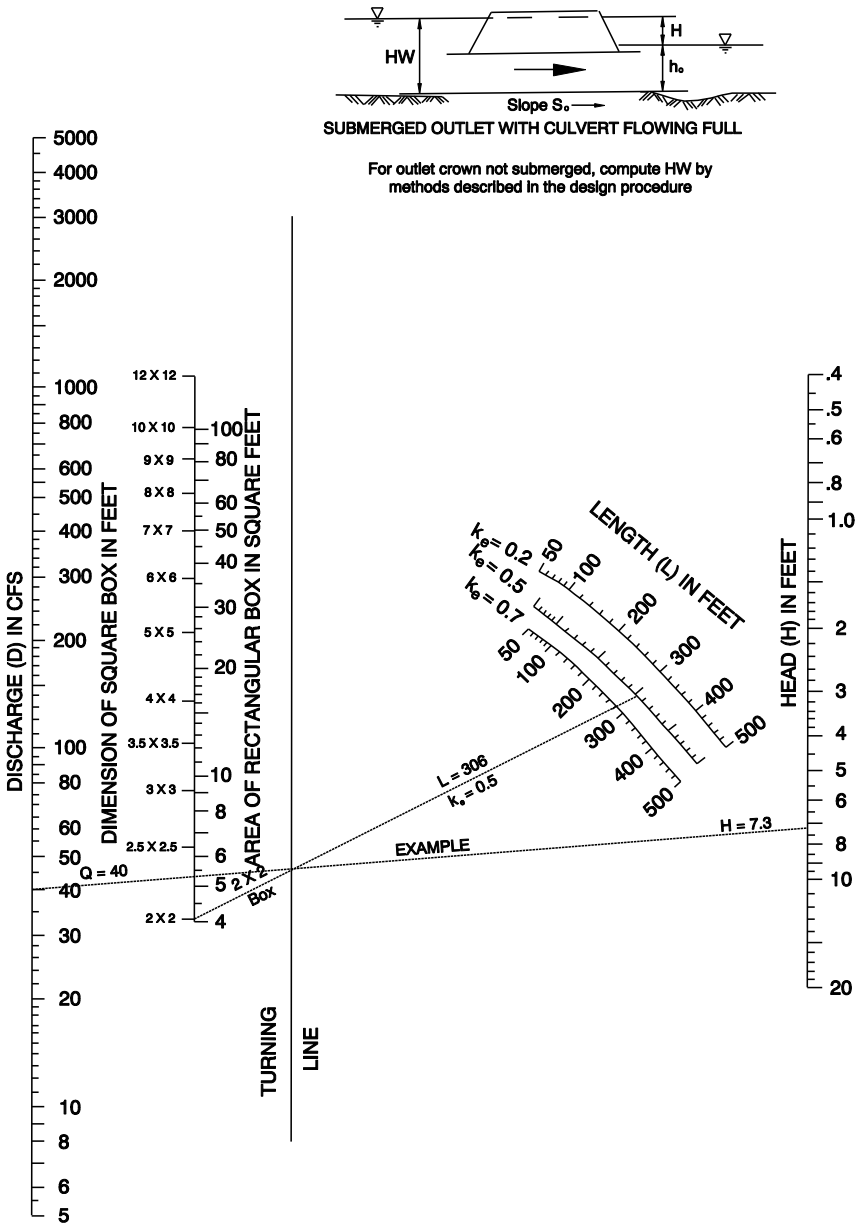
# CHART 11

## HEADWATER DEPTH FOR RECTANGULAR BOX CULVERTS WITH INLET CONTROL FLARED WINGWALLS 18° TO 33.7° AND 45° WITH BEVELED EDGE AT TOP OF INLET



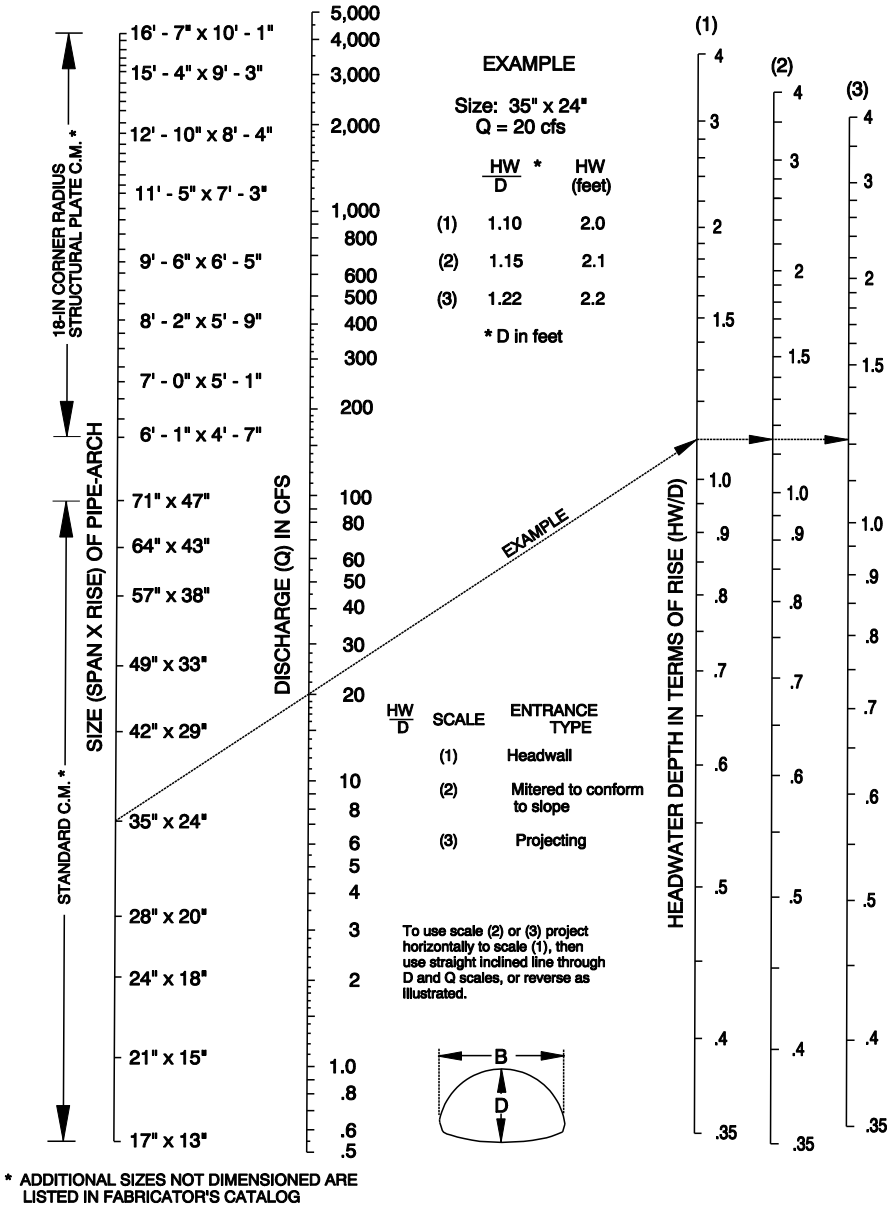
# CHART 12

## HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL IN OUTLET CONTROL n = 0.012



# CHART 13

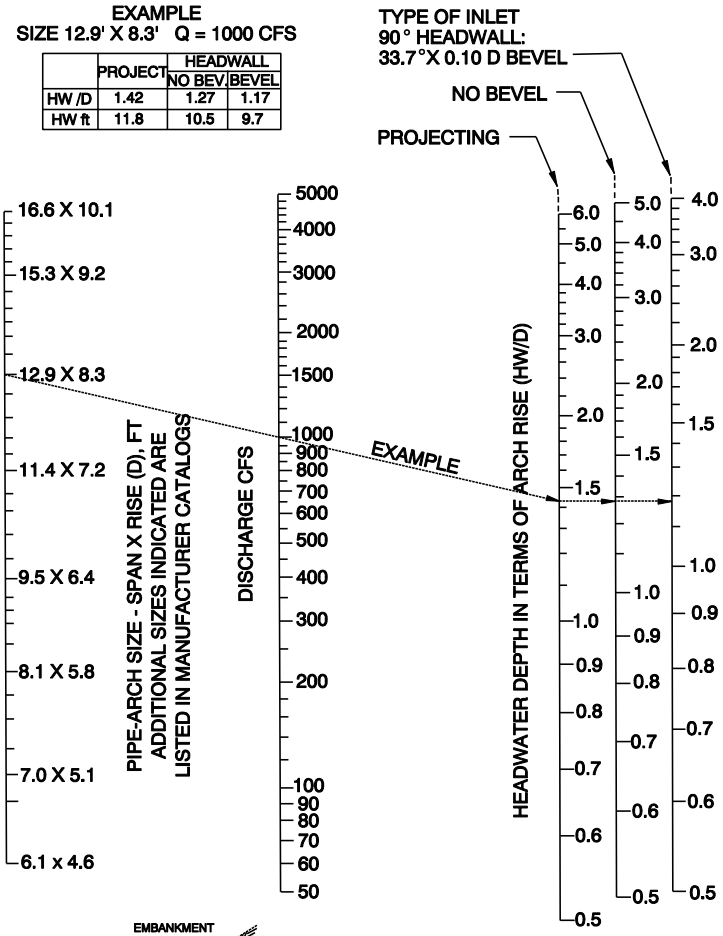
## HEADWATER DEPTH FOR C.M. PIPE-ARCH CULVERTS WITH INLET CONTROL

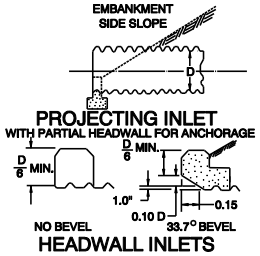


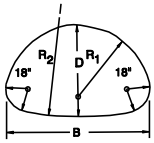

* ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

# CHART 14

## HEADWATER DEPTH FOR STRUCTURAL PLATE PIPE-ARCH CULVERTS WITH INLET CONTROL



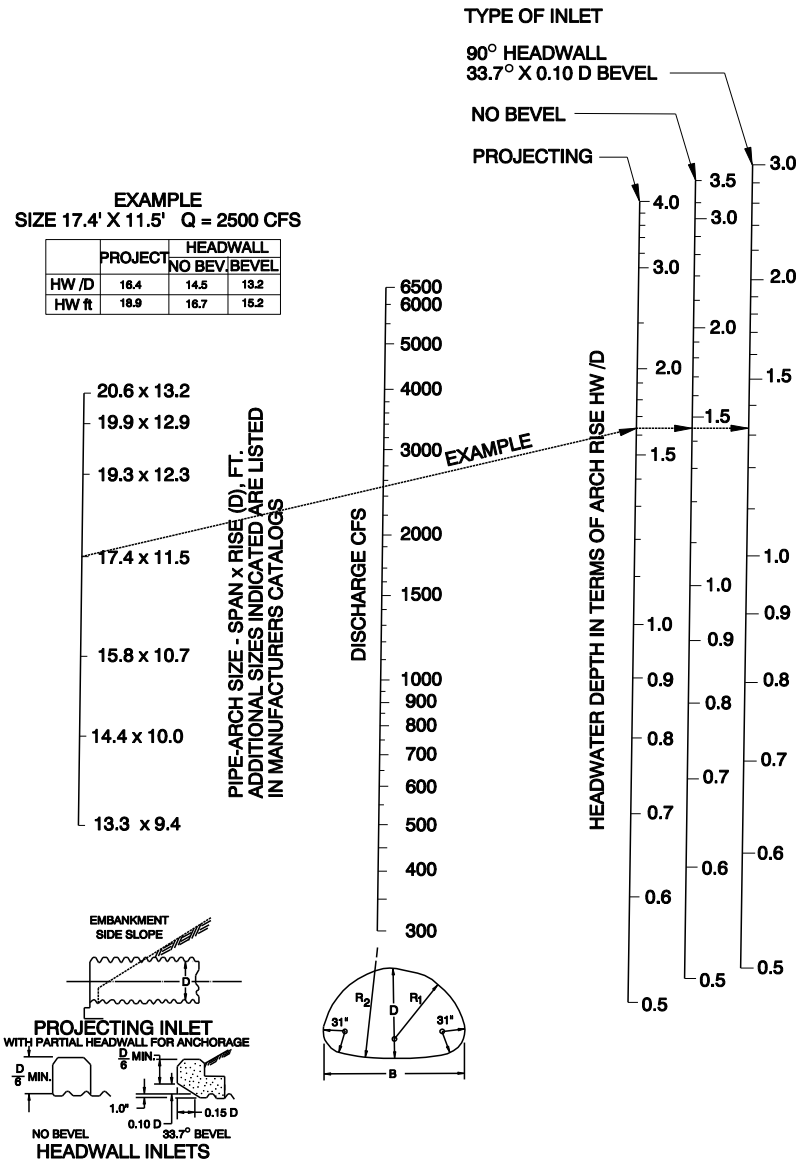




18-IN. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

# CHART 15

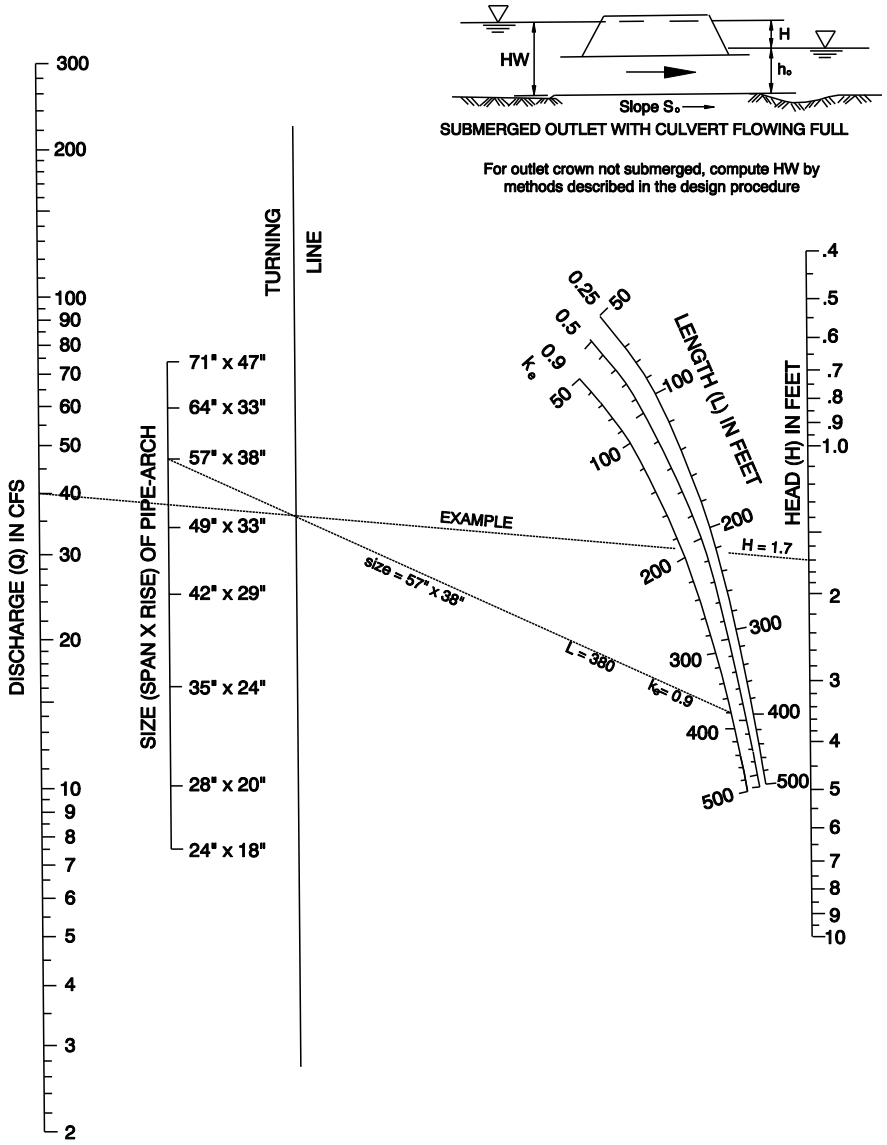
## HEADWATER DEPTH FOR STRUCTURAL PLATE PIPE-ARCH CULVERTS WITH INLET CONTROL




31-IN. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

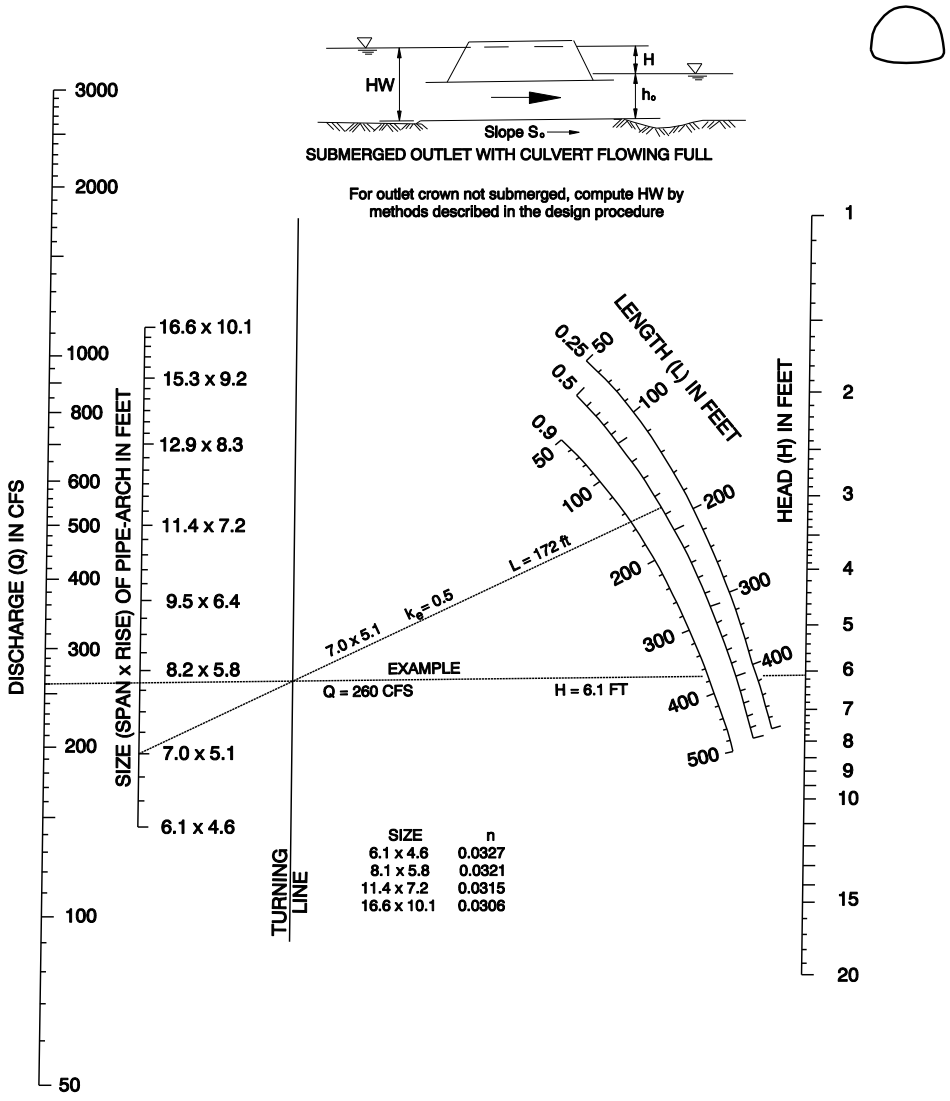
# CHART 16

## HEAD FOR STANDARD C.M. PIPE-ARCH CULVERTS FLOWING FULL IN OUTLET CONTROL n = 0.024



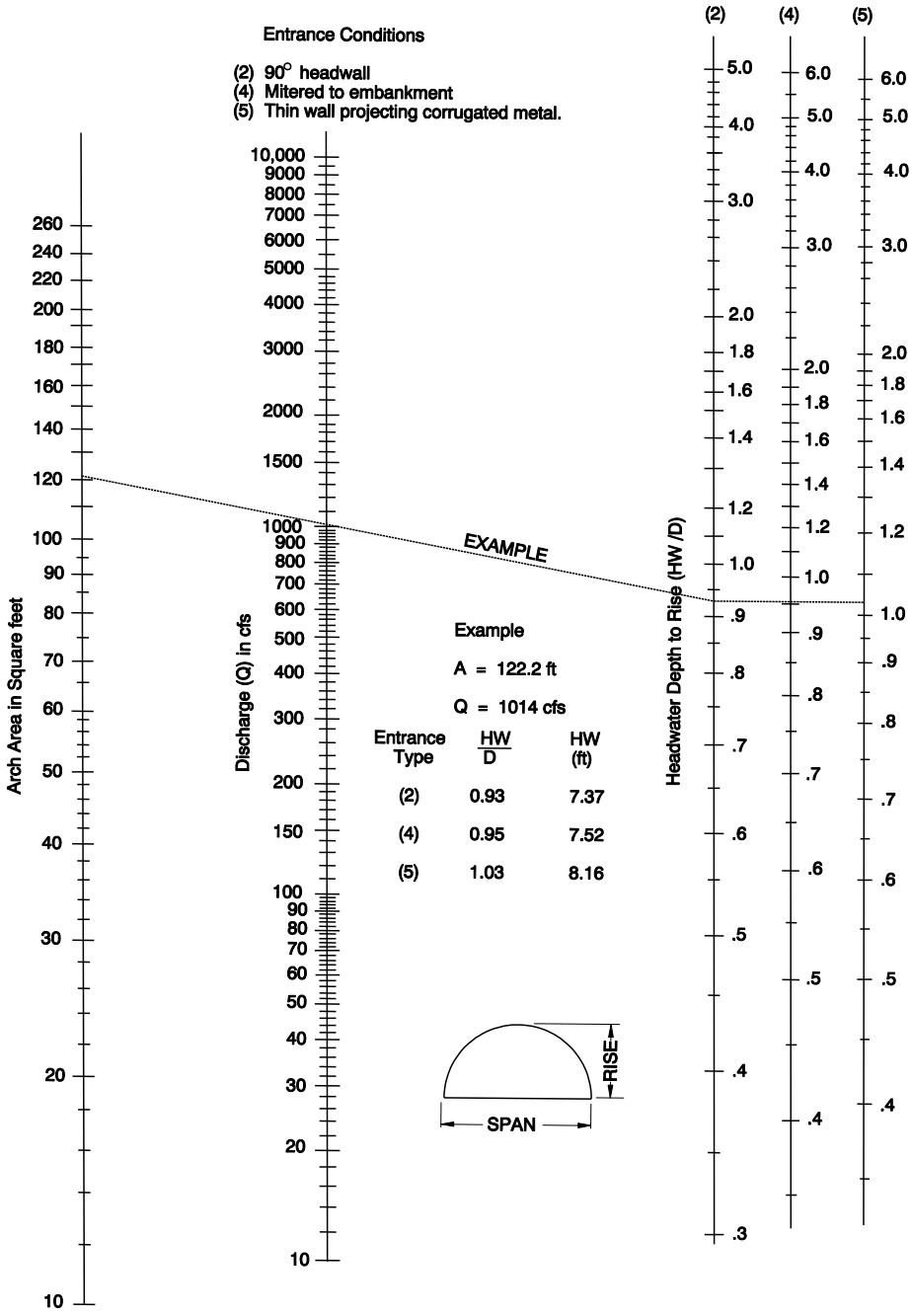
# CHART 17

## HEAD FOR STRUCTURAL PLATE CORRUGATED METAL PIPE ARCH CULVERTS 18 IN. CORNER RADIUS FLOWING FULL IN OUTLET CONTROL n= 0.0327 TO 0.0306



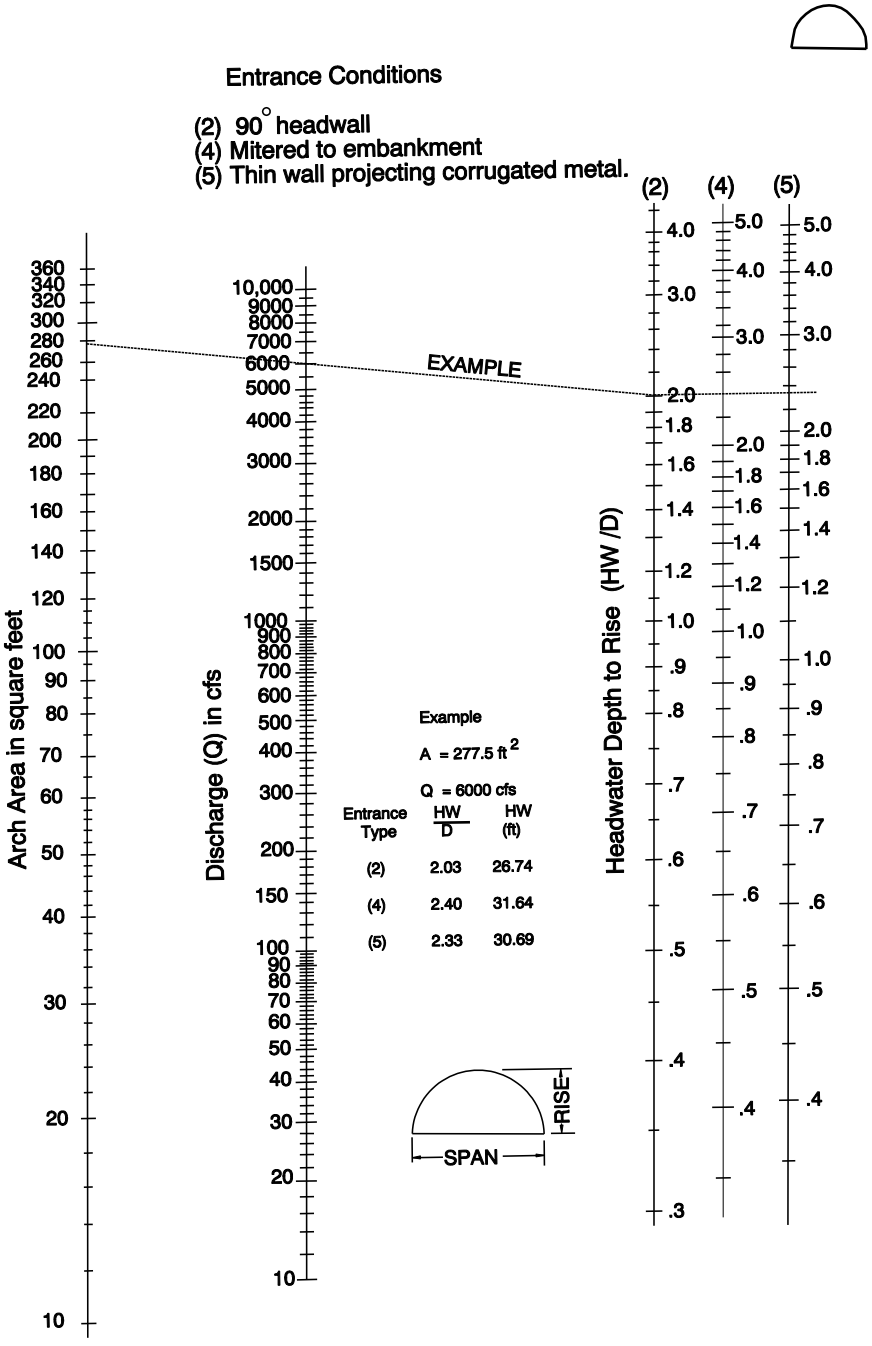
# CHART 18

## HEADWATER DEPTH FOR C.M. ARCH CULVERTS 0.3 ≤ RISE / SPAN <0.4 WITH INLET CONTROL



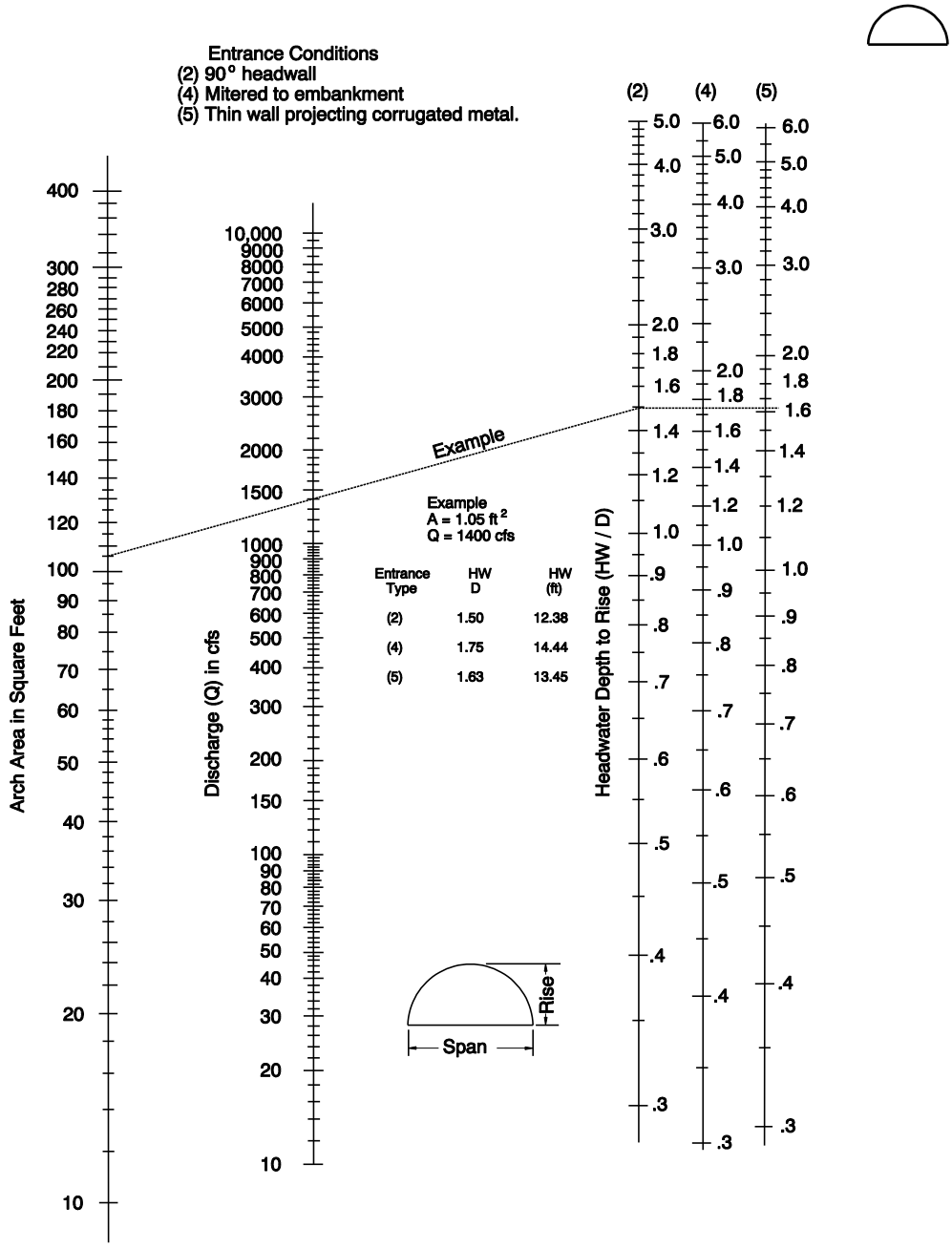
# CHART 19

## HEADWATER DEPTH FOR C.M. ARCH CULVERTS 0.4 ≤RISE / SPAN <0.5 WITH INLET CONTROL



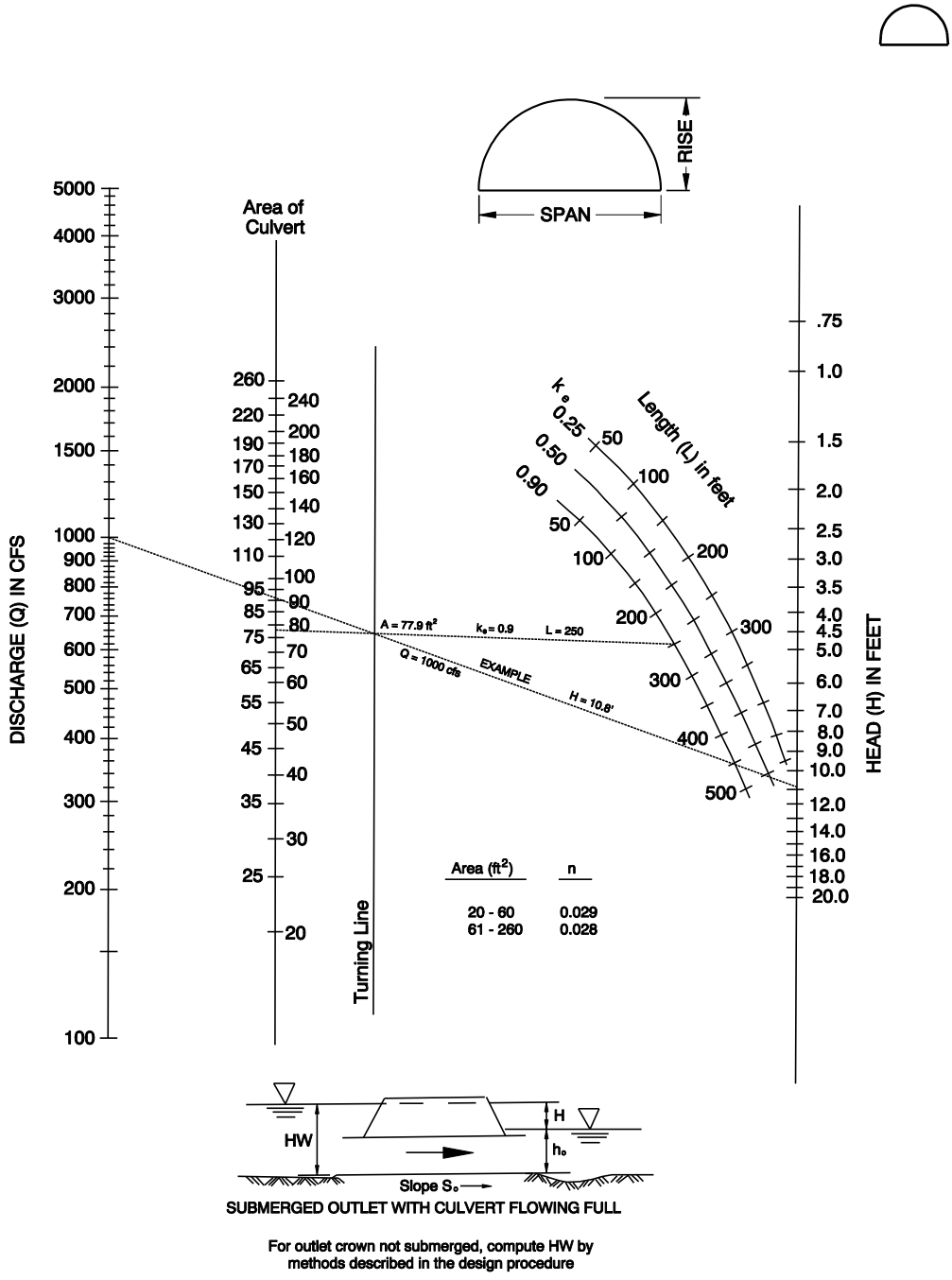
# CHART 20

## HEADWATER DEPTH FOR C.M. ARCH CULVERTS 0.5 ≤ RISE /SPAN WITH INLET CONTROL



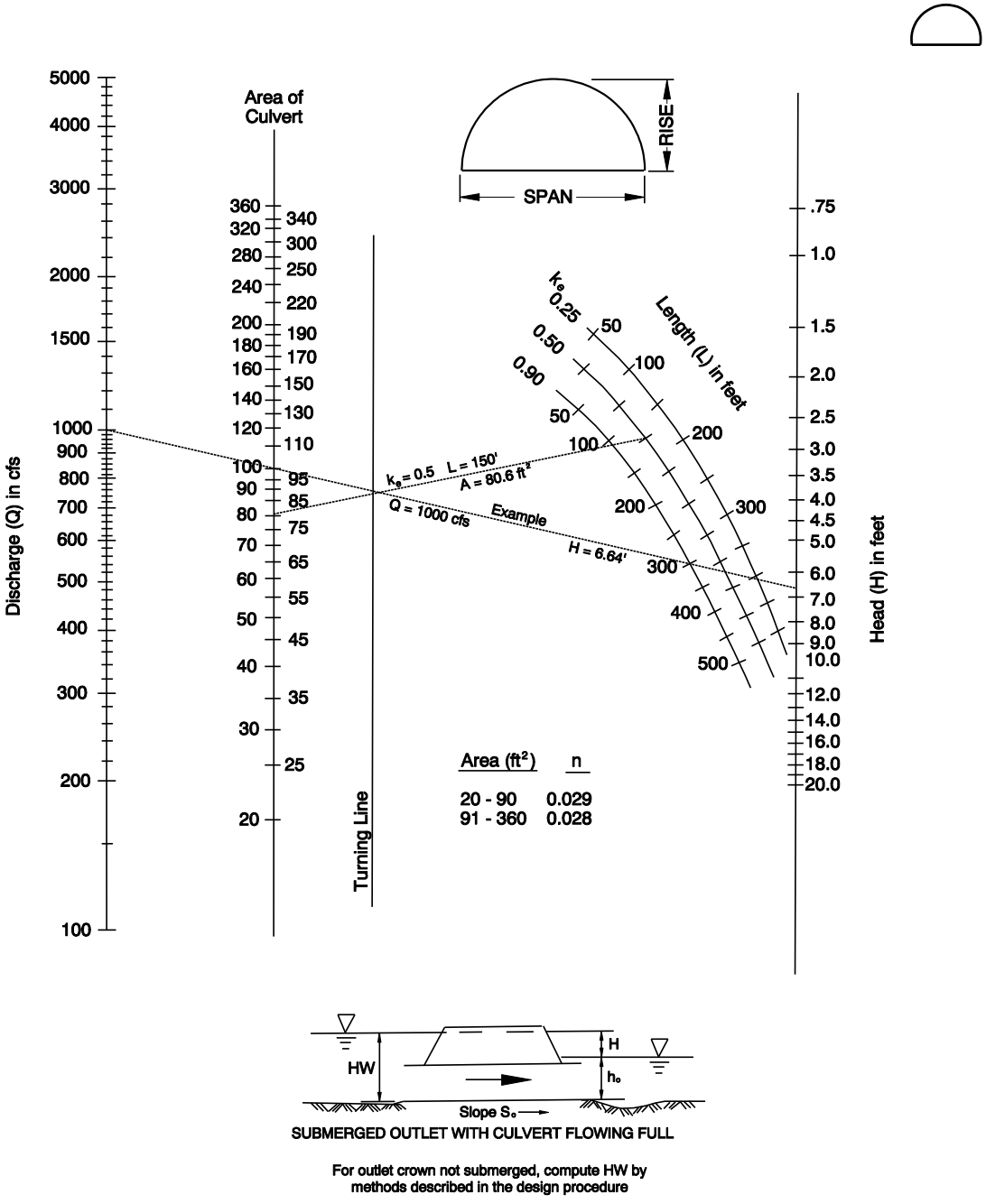
# CHART 21

## HEAD FOR C.M. ARCH CULVERTS FLOWING FULL EARTH BOTTOM ($n_b$ = 0.022) 0.3 ≤ RISE / SPAN <0.4



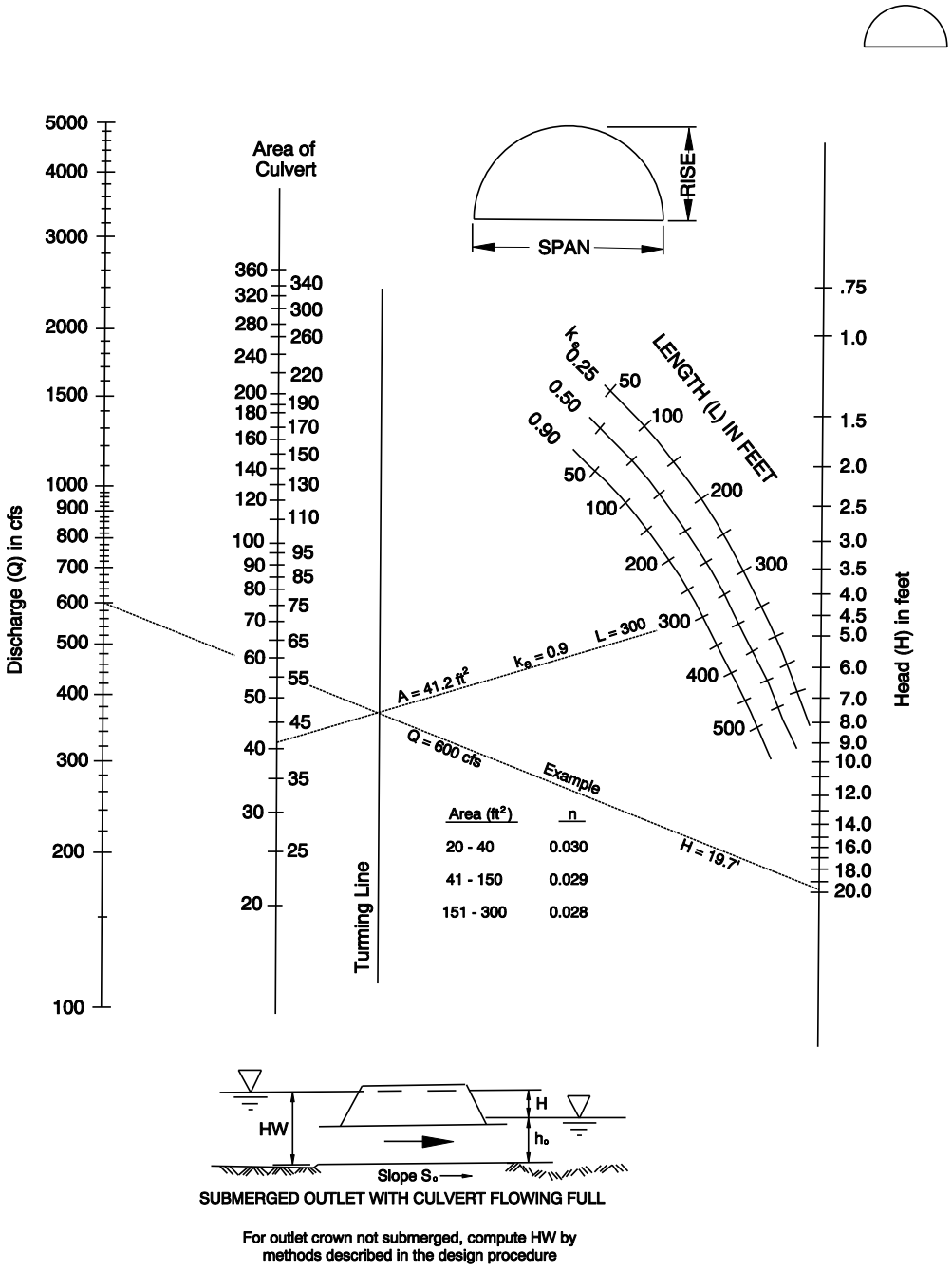
# CHART 22

## HEAD FOR C.M. ARCH CULVERTS FLOWING FULL IN OUTLET CONTROL EARTH BOTTOM ($n_b$ =0.022) 0.4 ≤ RISE / SPAN < 0.5



# CHART 23

## HEAD FOR C.M. ARCH CULVERTS FLOWING FULL IN OUTLET CONTROL EARTH BOTTOM ($n_b$=0.022) 0.5 ≤ RISE / SPAN



# APPENDIX B – CULVERT DESIGN SHEET

Oregon Department of Transportation (01/99)

# CULVERT DESIGN SHEET

| PROJECT | STATION | DESIGNER | DATE | SHEET OF |
|---|---|---|---|---|
| | | | | |

## HYDROLOGICAL DATA

METHOD: _____

DRAINAGE AREA: _____ (acres)

STREAM SLOPE: _____ (ft/ft)

CHANNEL SHAPE: _____

DESIGN AND CHECK FLOWS / TAILWATER

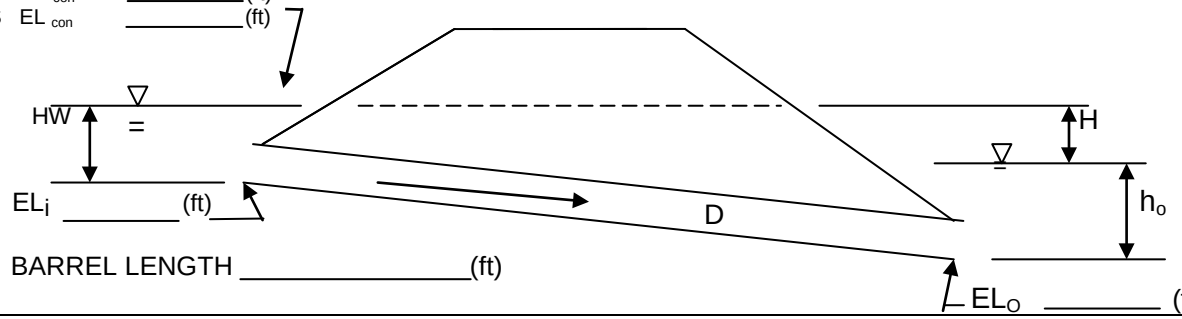
| | R.I. (years) | Q (cfs) | TW (ft) |
|---|---|---|---|
| DES | _____ | _____ | _____ |
| CHK | _____ | _____ | _____ |

CHECK FLOOD MAXIMUM ALLOWABLE HEADWATER ELEVATION _____ (ft)

DESIGN FLOOD MAXIMUM ALLOWABLE HEADWATER ELEVATION _____ (ft)

CHK $EL_{con}$ _____ (ft)

DES $EL_{con}$ _____ (ft)




TECHNICAL FOOTNOTES:

**(1)** USE INLET CONTROL NOMOGRAPH **(2)** $El_{hi} = HW + El_i$ **(3)** TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

**(4)** $h_o$= TW or $\frac{d_c+D}{2}$ (WHICHEVER IS GREATER) **(5)** $H = [1+ k_e +(29n^2L / R^{1.33})](V^2/2g)$ OR USE OUTLET CONTROL NOMOGRAPH

**(6)** $El_{ho} = El_o + H + h_o$ **(7)** USE HIGHER OF $El_{hi}$ or $El_{ho}$ FOR $EL_{con}$

| CULVERT DESCRIPTION<br>MATERIAL – SHAPE – SIZE – ENTRANCE | FLOW PER BARREL Q/N (cfs) | HEADWATER CALCULATIONS<br>INLET CONTROL | | | OUTLET CONTROL | | | | | | | | | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | (1) $\frac{HW}{D}$ | HW (ft) | (2) $EL_{hi}$ (ft) | (3) TW (ft) | $d_c$ (ft) | $\frac{d_c + D}{2}$ (ft) | (4) $h_o$ (ft) | $k_e$ | (5) H (ft) | (6) $EL_{ho}$ (ft) | (7) $EL_{con}$ (ft) | V (fps) | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |

COMMENTS/DISCUSSION:

VARIABLE / SUBSCRIPT DEFINITIONS

D = Diameter or Rise of Barrel (ft)
$D_c$ = Critical Depth at Culvert Outlet (ft)
$EL_{con}$ = Control Headwater Elevation
$EL_{hd}$ = Design Headwater Elevation (ft)
$EL_{hi}$ = Inlet Control Headwater Elevation (ft)
$EL_{ho}$ = Outlet Control Headwater Elevation (ft)
$EL_i$ = Inlet Invert Elevation (ft)
$EL_o$ = Outlet Invert Elevation (ft)
G = Acceleration of Gravity (32.2 $ft/s^2$)
H = Total Outlet Control Head Loss (ft)
$h_o$(ft) See Technical Footnote 4
HW = Inlet Control Headwater Depth (ft)
$k_e$ = Outlet Control Entrance Loss Coefficient
N = Number of Barrels
n = Manning's Roughness Coefficient
Q = Discharge ($ft^3/s$)
R = Hydraulic Radius (ft)
R.I. = Recurrence Interval (Years)
TW = Tailwater Depth (ft)
V = Average Velocity in Barrel (ft/s)

INVERT ABRASION PROTECTION

☐ YES TYPE _____

☐ NO _____

CULVERT BARREL SELECTED

SIZE _____ SHAPE: _____

MATERIAL: _____ n= _____ ENTRANCE: _____

# APPENDIX C – DEBRIS CONTROL STRUCTURES

## FHWA Hydraulics Engineering Circular #9 Debris-Control Structures

This circular provides information on debris accumulation and the various debris control countermeasures available for culvert and bridge structures. This circular presents various problems associated with debris accumulation at culvert and bridge structures and provides general guidelines for analyzing and modeling debris accumulation on a bridge structure to determine the impacts that the debris would have on the water surface profile through the bridge structure and the hydraulic loading on the structure. Various types of debris countermeasures for culvert and bridge structures are discussed with in this circular. General criteria for selection of these countermeasures and general design guidelines for some of the structural measures are also included in this FHWA manual.

Note: Project environmental personnel should be contacted if it is proposed to install these structures when fish passage is a concern. They can tell you if they are allowed and if there are any environmental concerns.

The following web address is the Federal Highway Administration's Publications site which has the Circular "Debris Control Structures, HEC 9" (latest edition).

http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm

# CHAPTER 10

# BRIDGES

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## 10.1 Introduction

This chapter provides information for the planning and hydraulic design of highway bridges. The methodology is intended for those with an understanding of basic hydrologic and hydraulic analysis methods and some experience in the design of hydraulic structures.

Bridges are often the most expensive and complex highway structures, and considerable diligence and care are used in the hydraulic aspects of their design. These hydraulic aspects include, but are not limited to:

- changes in water surface profiles during floods due to the presence of the bridge,
- the passage of streambed material, ice, or debris,
- the potential for scour and erosion around the bridge foundation,
- clearance under the bridge deck for navigation, and
- drainage of stormwater runoff from the bridge deck.

The preceding aspects of bridge hydraulics are used in many activities, such as planning, location, design, and maintenance. In the case of moveable bridges, these aspects are also considered in bridge operation. This chapter addresses the first four of these aspects. The last aspect, bridge deck drainage, is covered in **Chapter 13**.

## 10.2 Policy

General ODOT policies that pertain to highway facilities such as bridges are listed in **Chapter 3**.

## 10.3 Bridges, Bridge Components, and Dimension Estimates

Occasionally the hydraulic designer will need to estimate the type, size, and location of a small bridge. In addition, the hydraulic designer often recommends the size or the shape of the waterway opening. To do either of these tasks, the designer should be familiar with structure types, the comparative costs of commonly used structures, their components, their advantages and limitations, and methods of estimating their dimensions.

There are many types of bridges, as shown in Figure 10-1. They range in complexity from simple single-span structures to complex multiple span girder bridges or trusses. The majority of highway bridges over water are relatively short structures such as open-bottom culverts, single or

multiple span prestressed slab bridges, or single or multiple span prestressed box beam bridges. Methods of estimating the dimensions of these smaller structures are discussed in detail in this section. A detailed discussion of the more complex structures is beyond the scope of this chapter.

### 10.3.1 Single Versus Multiple Span Bridges

Many considerations govern the selection of a structure, such as hydraulic performance, foundation conditions, cost, aesthetics, etc. One of the most important requirements is the need to span an obstruction. From a hydraulic standpoint, this obstruction could be a waterway that has to be kept clear for navigation, the passage of floodwaters, or a number of other concerns.

In general, bridges use two means to cross obstructions. One means is to span the obstruction entirely with a single span. The other method is to use two or more shorter spans with bents between the spans. An advantage of the single-span method is the lack of interior bents and their associated costs. This cost advantage is offset to some degree by the added expense of the longer span because the cost of a span is often proportional to its length. Another advantage of a single-span is to eliminate any scour or debris related problems associated with piers. The advantage of the multi-span method can be a reduction in span costs, but this is offset by the expense of the interior bents. The hydraulic designer will often examine both of these alternatives at a site, if they are both viable.

### 10.3.2 Comparative Costs

Although many types of structures can be used at most crossings, only a limited number of types are cost-effective. In order to help in the selection of alternatives for hydraulic analysis, the following table has been adapted from the ODOT Bridge Section *Bridge Design and Drafting Manual*, the ODOT Bridge Standard Drawings, and trade literature. It lists common structure types in order of increased costs, and it also lists the ranges of typical span lengths. These are broad guidelines, and they are recommended for selecting alternatives for further study, only. The selection of a crossing type is usually made by the structural designer after a thorough study of hydraulic, aesthetic, geotechnical, environmental, and other considerations.

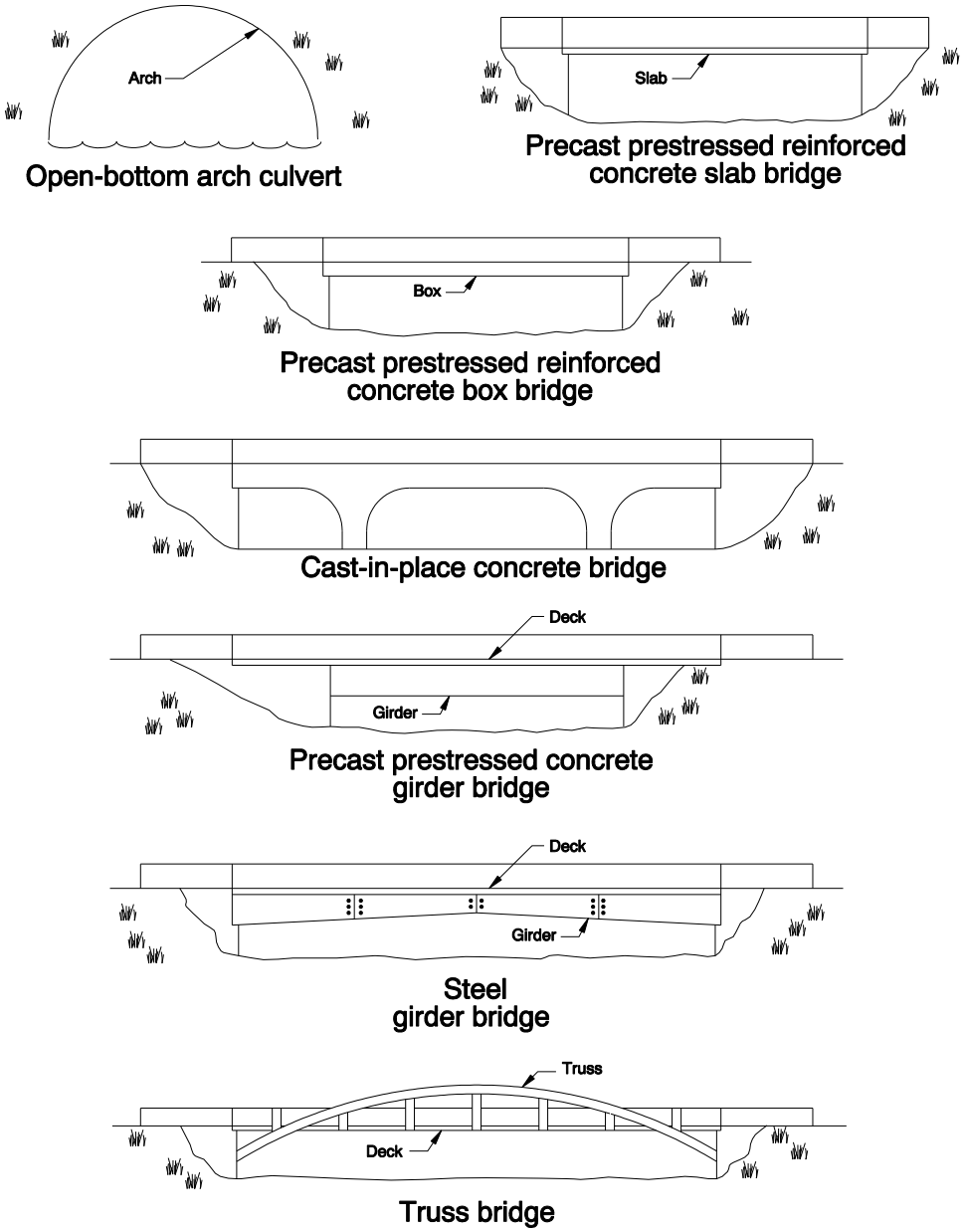



**Figure 10-1 Structure Types**

**Table 10-1 Comparative Structure Costs**
(Structures are listed in order of increasing cost.)

| Structure Type | Span Lengths |
|---|---|
| Open-bottom culvert* | 6 to 30 feet |
| 3-sided reinforced concrete rigid frames | 12 to 42 feet |
| Precast prestressed concrete slab | 15 to 83 feet |
| Precast prestressed concrete box | 54 to 116 feet |
| Cast-in-place concrete slab | up to 50-66-50 feet (3-span bridge) |
| Precast prestressed integral deck concrete girder | up to 130 feet |
| Precast prestressed concrete girder | 71 to 164 feet |
| Cast-in-place box girder | ** |
| Cast-in-place post-tensioned box girder | ** |
| Steel girder or truss | ** |

* Open-bottom culverts can cost more than slab span bridges at many sites.
**Normally used for long single-span and longer multi-span bridges.

### 10.3.3 Open-Bottom Culverts

Open-bottom culverts are available in a wide range of span lengths from approximately 6 to 30 feet. These structures have characteristics of both bridges and culverts, and they are discussed in detail in Chapter 9. They are also discussed in this chapter because scour depth estimates, and in some cases, hydraulic performance analyses, are done using the same methods as bridges. Like bridges, they:

- have a waterway opening with a bottom composed of natural streambed materials,
- have footings that must be protected from scour, and
- in the larger sizes, have hydraulic characteristics more similar to bridges than culverts.

In a manner similar to culverts, they

- do not have a deck (the roadway crosses over the culvert on a layer of earth and/or aggregate fill), and
- in the smaller sizes, have hydraulic characteristics more similar to culverts than bridges.

The hydraulic designer should pay particular attention to the following when locating or performing a hydraulic study on an open-bottom culvert:

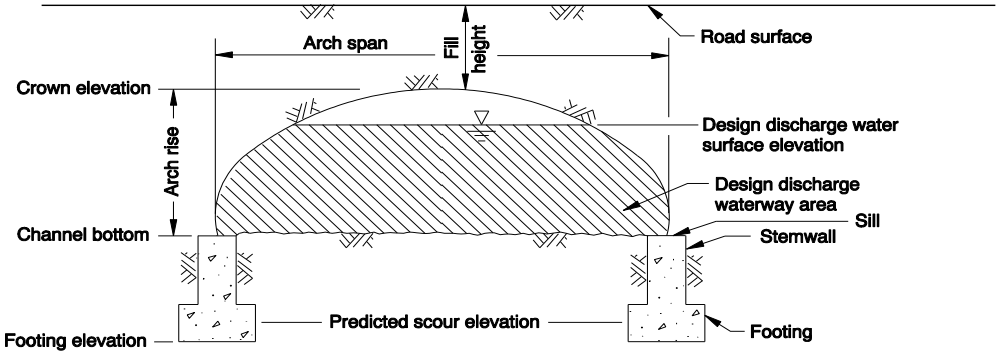
**Foundations** - Open-bottom culverts are almost always supported by spread footings, as shown in Figure 10-2. The bottoms of the footings are placed below the predicted scour elevation unless they

are keyed into non-erodible rock. Sometimes it is necessary to locate the footings considerably deeper than the elevation of the bottom of an arch if the predicted scour is fairly deep. Stemwalls are used in these instances to connect the arch to the footings, as shown in Figures 10-2a and 10-2b. Stemwalls can also be used to increase the waterway area by elevating the arch above the channel bottom. Stemwalls are not used with the precast 3-sided rigid frame box culverts shown in Figure 10-2c. These culverts rest directly on the footings.

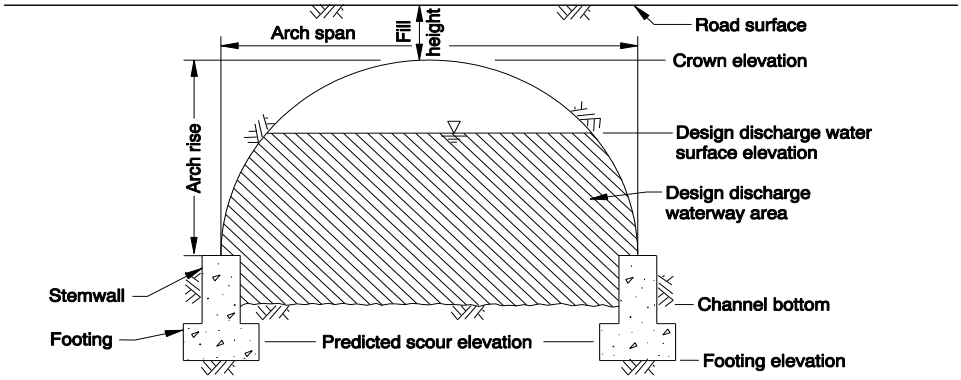
Footings, stemwalls, and the requisite excavation can often be quite expensive, and experience has shown the cost of a scour resistant foundation is usually the critical factor in determining whether an open-bottom culvert or a bridge is the most cost-effective structure. In general, the open-bottom culvert is an economical structure where the footings can be protected from scour without an extensive and costly foundation. Usually this occurs when the footings can be keyed into non-erodible rock, and the rock is at or near the ground surface. The other application is a crossing over regulated waterway such as a canal where the scour depth is limited and predictable, and the footings can be placed below the predicted scour elevations with reasonable cost. This subject is discussed in more detail in **Chapter 9**.

**Fill Heights -** Open-bottom culverts, unlike bridges, usually have a layer of fill between the crown of the culvert and the roadway surface, as shown in Figure 10-2. The height of this fill is critical, and it must be thick enough to distribute the live load and prevent the culvert from being distorted, or possibly failing, due to the pressures exerted by traffic. At the same time, the fill height must not be excessive, so as to prevent structural failure of the culvert due to the weight of the surrounding fill. The ranges of allowable fill heights are limited for many types of arches, in particular metal open-bottom arches. As a result, fill heights are often a critical factor in the decision about whether or not to use an arch and the selection of the shape of the arch. This subject is addressed in more detail in **Chapter 5** and **Chapter 9**.

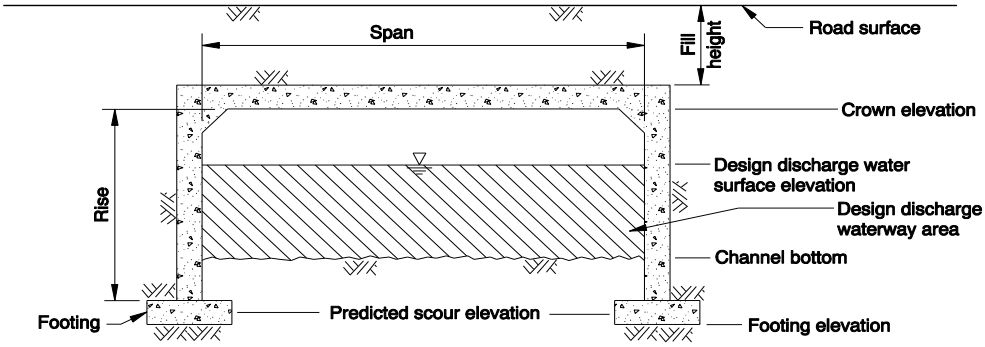
*Note: Rigid frames can be designed so traffic crosses over pavement on the top of the frame. This is not often done, and a special design is needed.*




a) Low profile structural plate metal arch




b) Semicircular structural plate metal arch




c) 3-Sided rigid frame reinforced concrete box

**Figure 10-2 Open-Bottom Culverts**

### 10.3.4 Precast Prestressed Slab or Box Beam Bridges

Structures with spans of precast prestressed slabs and boxes are the most common short and medium length highway bridges. These spans are self-supporting. In other words, they do not need the additional support from a superstructure such as a truss or girders. Precast prestressed slabs are used for single-span and multi-span structures with span lengths of approximately 15 to over 83 feet, and precast prestressed boxes can be used for spans with lengths from approximately 54 to 116 feet. Slabs and boxes built to ODOT Standard Plans can be used for spans with skew angles from 0 degree (an unskewed bridge) to a maximum skew angle of 45 degrees. Skew angles over 45 degrees can be made using special designs. These slabs and boxes are shown in Figure 10-3 and ODOT Standard Drawings BR400 through BR465.

A hydraulic performance advantage of these bridge spans is their shallow depth between the pavement surface and the bottom of the box or slab. These depths are considerably less than many of the other structure types, and this is an important asset when vertical clearance over the water or channel bottom is a critical factor. Another advantage of these spans is their relatively smooth under surface. This can be a benefit for structures that pass ice or large floating debris.

**Waterway Opening Dimensions -** The waterway opening dimensions of slab or box span bridges can be estimated using the following procedures. These estimates are usually adequate for the hydraulic modeling of the recommended alternatives. Later in the design process a recommended alternative will be selected. It will often have a waterway with different dimensions than the waterways of the alternatives described in the hydraulic study. If this occurs, and the proposed waterway is smaller than hydraulic report recommendations, it is recommended that the proposed waterway be modeled in order to verify that it has adequate hydraulic performance.

The clear span distance is the width of the structure opening, and it is the distance between the inside faces of the abutments. A single-span structure spans the entire clear span distance. Multi-span structures have one or more interior bents within the clear span distance, as shown in Figure 10-3a.

The span length of a slab or box is the distance between the centerlines of the bearing pads, as shown in Figure 10-3b. The centerline of each bearing pad is approximately 1.5 feet outside of the face of the end bent. As a result, the span length of the typical single-span structure is greater than the clear span distance. The span length for a given clear span distance can be estimated for a typical ODOT slab or box bridge by the following equation:

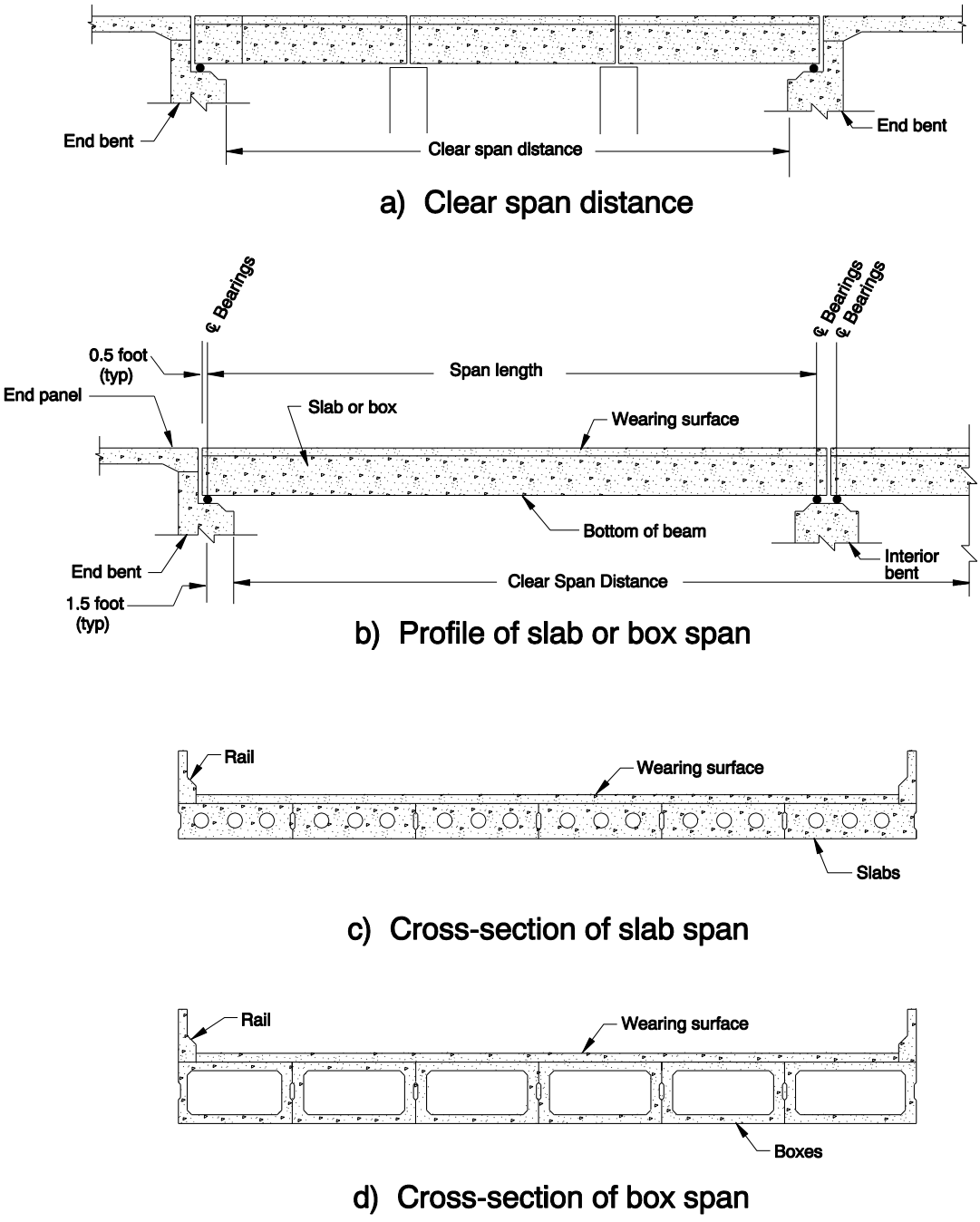



**Figure 10-3 Prestressed Concrete Slab and Box Spans**

$$\text{Span Length}_{\text{Single-Span}} = \text{Distance}_{\text{Clear Span}} + 3 \qquad \text{(Equation 10-1)}$$

Where:

$\text{Span Length}_{\text{Single-Span}}$ = Estimated minimum length of a single span between bearing centers in feet

$\text{Distance}_{\text{Clear span}}$ = Distance between the inner faces of the end bents in feet

The end of the span is 0.5 foot beyond the centerline of the bearing. It is assumed that both ends of the span abut each other at each interior bent for estimating purposes. Consequently, the minimum lengths of multiple spans that can cross a clear span distance is:

$$\text{Span Length}_{\text{Multi-Span}} = \frac{\left(\text{Distance}_{\text{Clear Span}} + 3\right) - [(1.0)(\#_{\text{interior bents}})]}{\#_{\text{Spans}}} \qquad \text{(Equation 10 - 2)}$$

Where:

$\text{Span Length}_{\text{Multi-Span}}$ = Estimated length of each of the multiple spans between centers of bearing in feet

$\text{Distance}_{\text{Clear Span}}$ = Distance between the inner faces of the end bents in feet

$\#_{\text{Interior Bents}}$ = Number of interior bents

$\#_{\text{Spans}}$ = Number of spans

The preceding equation assumes all spans have equal length. The designer will usually try to do this because it is most economical. Sometimes unequal span lengths are used to clear obstacles and for other purposes.

The span width must be sufficient to accommodate the roadway, bridge rails, and sidewalks, if used. Information on the roadway width, rail type, and sidewalk requirements can be obtained from the roadway or bridge designer. Typically the span width is a multiple of 4 feet because the individual slabs and boxes are this wide. The slabs and boxes do not fit together exactly, so it is customary to add an extra 0.042 foot (1/2 inch) of deck width for each slab or box used, as follows:

$$\text{Width}_{\text{Span}} = (4.042)\ (\#_{\text{Slabs or Boxes}}) \qquad \text{(Equation 10-3)}$$

Where:

$\text{Width}_{\text{Span}}$ = Estimated total width of the span in feet

$\#_{\text{Slabs or Boxes}}$ = Number of slabs or boxes in each span

The total depth of the span can be estimated by adding the thickness of the surfacing to the depth of the slab or box. The surfacing thickness can be provided by the bridge designer. If this information is not available and the bridge is on a road with a constant gradient, a thickness of 2 inches can be assumed at midspan and a thickness of 3 inches at the span ends. The surfacing is thicker at the ends

of the span because the slab or box has a slight upward arch shape due to camber. If the bridge is on a vertical curve, this assumption cannot be made and the bridge designer should be contacted for an elevation estimate.

The depth of the slab and box can be estimated using the tables in Standard Drawings BR450 and BR460, respectively. The name of each table gives the span or box depth. For example, "SLAB 12" represents a slab with a depth of 12 inches in Drawing BR450, and "Box 33" represents a box with a depth of 33 inches in Drawing BR460. The following equation can be used:

$$Depth_{Total} = Depth_{Slab\ or\ Box} + Thickness_{Surfacing} \quad \text{(Equation 10-4)}$$

Where:

| | | |
|---|---|---|
| $Depth_{Total}$ | = | Estimated total depth of span in feet |
| $Depth_{Slab\ or\ Box}$ | = | Depth of slab or box in feet |
| $Thickness_{Surfacing}$ | = | Thickness of surfacing in feet |

The elevations of the bottom of the slab or box can be estimated with either the centerline profile and cross-slope, or 3-line profile. This information can be provided by the bridge or roadway designer. In most cases, the roadway cross-slope will also be the cross-slopes of the upper and lower surfaces of the bridge deck. An exception occurs when the roadway has gutters. Bridge decks generally do not have gutters, and the cross-slope of the road and bridge may differ. The bridge designer should be contacted for assistance. The following formula can be used in most applications to determine the bottom of beam elevations:

$$EL_{BOB} = EL_{CL} + [(Distance)\ (Cross\text{-}slope)] - Depth_{Total} \quad \text{(Equation 10-5)}$$

Where:

| | | |
|---|---|---|
| $EL_{BOB}$ | = | Estimated bottom-of-beam elevation in feet |
| $EL_{CL}$ | = | Elevation of road centerline in feet |
| Distance | = | Distance from centerline to deck edge in feet |
| Cross-slope | = | Cross-slope of roadway or deck surface in feet per foot (This is a positive value if the edge of the deck is higher than the highway centerline, and it is a negative value if the edge of the deck is lower than the centerline.) |
| $Depth_{Total}$ | = | Depth of span including surfacing in feet (Use Equation 10-4) |

### 10.3.4.1 Example - Estimating Bridge Span Dimensions

A slab or box bridge will be included in a hydraulic model. The clear span distance and roadway elevations are known. Estimates are needed of the deck width and the bottom of slab or box elevations.

The bridge is on a road with a constant 0.5 percent grade. It is also on a horizontal curve where the roadway is superelevated and it has a constant 0.025 foot per foot cross-slope across its entire width. The clear span distance must be at least 95 feet and the waterway opening is not skewed in relation to the road centerline. An interior bent can be located within that clear span distance, if needed. The road centerline elevations at the ends of the clear span distance are 95.26 and 95.73 feet. The total roadway width is 45 feet. The deck must accommodate this roadway and provide sufficient width for the bridge rails. A width of 1.33 feet is assumed for the rail. This is the width of the commonly used "Type F" bridge rail.

A single span would have to be longer than the 95-foot clear distance, as shown in Figure 10-4. Using Equation 10-1, the span length from centerline of bearing to centerline of bearing would be:

$\text{Span Length}_{\text{Single-Span}} = 95.0 + 3.0 = 98.0$ feet

This distance can be spanned by a single reinforced concrete box. A two-span bridge would be needed if slabs are used. Using equation 10-2, the span lengths between the bearing centerlines of a two-span bridge would be:

$$\text{Span Length}_{\text{Multi-Span}} = \frac{(95.0 + 3.0) - [(1.0)(1)]}{2} = 48.5 \text{ feet}$$

The number of slabs or boxes can be calculated by dividing needed roadway and rail width by 4.042 as follows:

$$\frac{45 + 1.33 + 1.33}{4.042} = 11.8 \text{ slabs or boxes}$$

It could be assumed that 12 slabs or boxes would be adequate if the bridge was on a tangent. This bridge is located on a horizontal curve, and some added width will be needed to accommodate the curved roadway, as shown in Figure 10-4. The bridge and end panels were sketched to see how many extra slabs or boxes would be needed. One extra slab or box is sufficient, for a total of 13 slabs or boxes. The deck width is estimated by Equation 10-3 to be:

$\text{Width}_{\text{Span}} = (4.042)\ (13) = 52.5$ feet

The depth of the box for the single-span bridge is estimated from Standard Drawing BR460 to be 4 feet (48 inches). The thickness of the surfacing is assumed to be 0.25 feet (3 inches) at the ends of

the bridge, as discussed in the previous section. Using Equation 10-4, the total depth at the ends of a single span is:

$Depth_{Total}$ = 4.0 + 0.25 = 4.25 feet (for a single-span bridge)

*Note: A 42-inch deep box can also be used. It is able to handle span lengths up to 102 feet. This box, however, is near its design length limit with a 95-foot span. The 48-inch deep box can be used for spans up to 116 feet. This depth is selected for estimating purposes because it is not near its design length limit and it has reserve capacity for more weight, such as a pavement overlay. The practice of selecting a box or slab depth providing extra capacity is recommended for waterway dimension estimating purposes.*

The slab depth of the two-span bridge is estimated from Drawing BR450 to be 1.75 feet (21 inches). This depth is adequate for spans up to 55 feet long. Using Equation 10-4, the total depth of a span for a two-span bridge is:

$Depth_{Total}$ = 1.75 + 0.25 = 2.00 feet (for two spans)

The bottom-of-box or bottom-of-slab elevations are estimated for all four corners of each of the two bridges using Equation 10-5. The distances from the road centerline to the edges of the deck are scaled from the drawing in Figure 10-4. The elevation of the bottom of the box at the upstream left corner of the single-span bridge is determined by a typical calculation, as follows:

$EL_{BOB}$ = 95.26 + [(52.5/2) (0.025)] – 4.25 = 91.67 feet

In general, the dimensions used in hydraulic modeling of bridge decks abutments and bents are:

- elevations rounded to the nearest 0.01 foot, and
- horizontal measurements to the nearest 0.1 foot.

The bridge opening dimensions, rounded according to the preceding guidelines, are:

- clear span distance = 95.0 feet, and
- span width = 52.5 feet

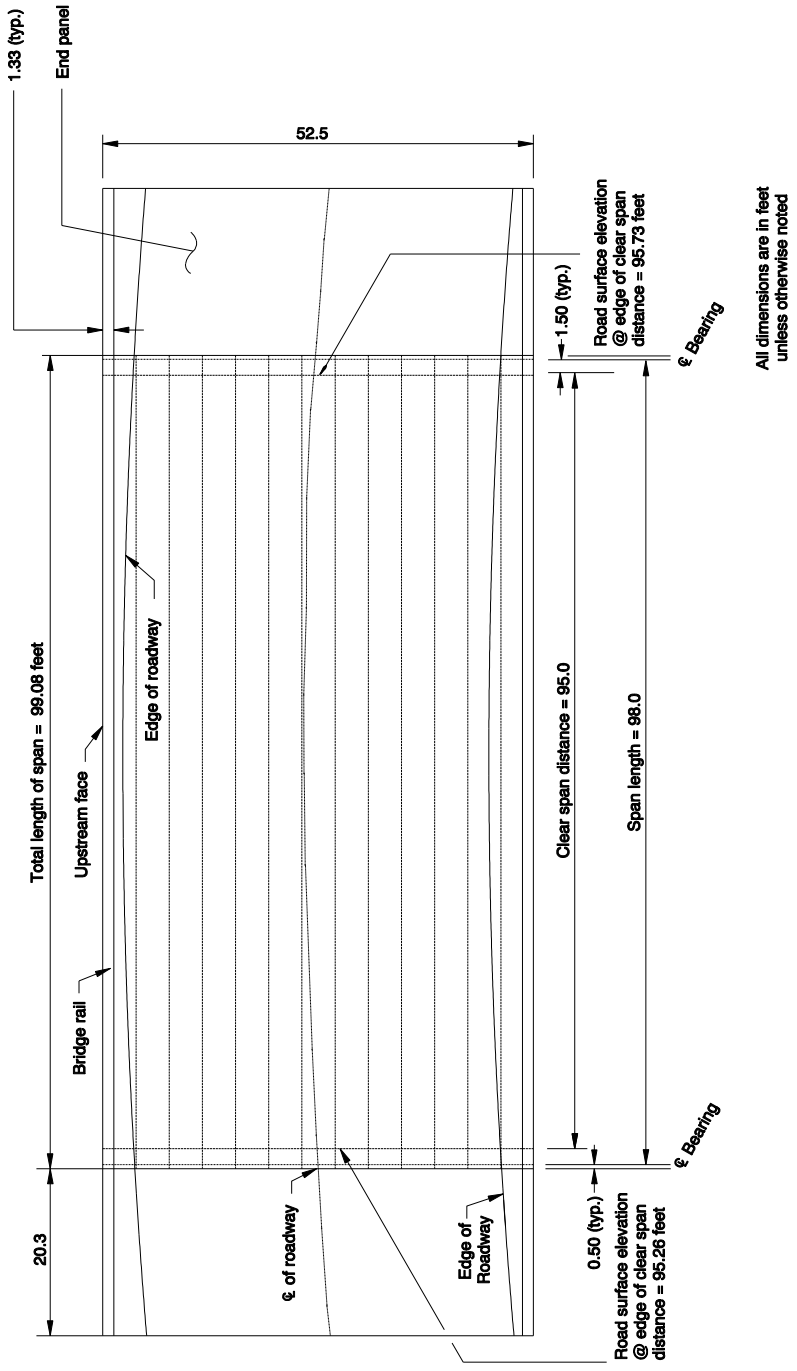



**Figure 10-4 Plan View of Span in Example**

The elevations of the bottom of the boxes or slabs at the corners of the end bents are:

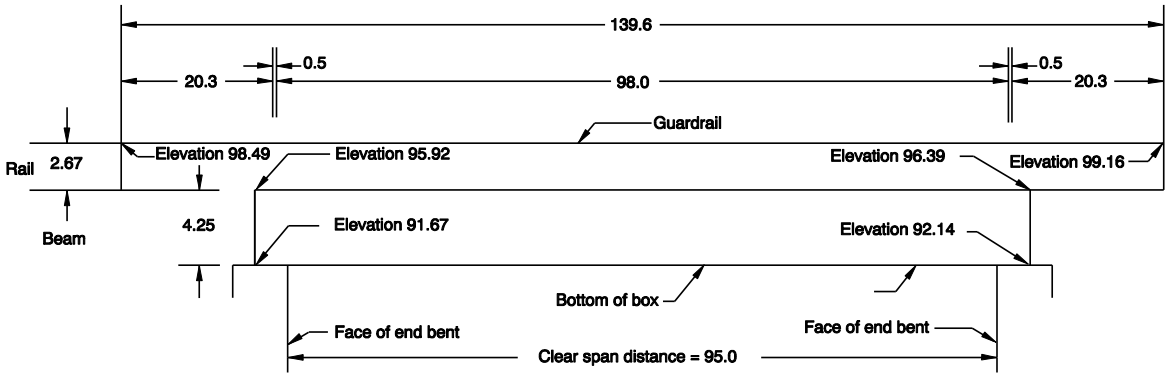
| Bridge | Centerline Elevation (feet) | Distance to Deck Edge (feet) | Total Cross-slope (foot per foot) | Span Depth (feet) | Corner Location | Corner Elevation (feet) |
|---|---|---|---|---|---|---|
| Single-span | 95.26 | 26.3 | + 0.025 | 4.25 | Upst. Left | 91.67 |
| | 95.73 | 26.3 | + 0.025 | 4.25 | Upst. Right | 92.14 |
| | 95.26 | 26.3 | - 0.025 | 4.25 | Dnst. Left | 90.35 |
| | 95.73 | 26.3 | - 0.025 | 4.25 | Dnst. Right | 90.82 |
| Two-span | 95.26 | 26.3 | + 0.025 | 2.00 | Upst. Left | 93.92 |
| | 95.73 | 26.3 | + 0.025 | 2.00 | Upst. Right | 94.39 |
| | 95.26 | 26.3 | - 0.025 | 2.00 | Dnst. Left | 92.60 |
| | 95.73 | 26.3 | - 0.025 | 2.00 | Dnst. Right | 93.07 |

The opening dimensions of the upstream face of a single-span bridge are shown in Figure 10-5a, and the dimensions of the downstream face of a two-span bridge are shown in Figure 10-5b. The elevations and dimensions should be verified by the bridge designer.

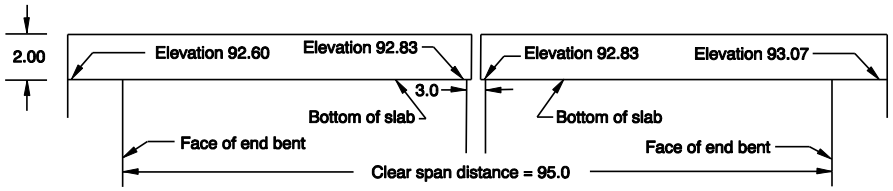
The area obstructed by the Type “F” bridge rail is shown in Figure 10-5a. It is assumed the rail will be 2.67 feet tall and it will extend the entire length of the bridge, including end panels. The area obstructed by the rail must be included in the hydraulic model if roadway overtopping occurs. Typical rails are shown in the ODOT bridge and roadway standard drawings. An interior bent with a 3 foot width is shown in Figure 10-5b. This width is a commonly used estimate of the interior bent width for a slab or box span bridge.

### 10.3.5 Prestressed Beam, Cast-in-Place, Girder, Truss, and Other Bridges

Precast beam bridges, cast-in-place bridges, girder bridges, truss bridges and other types of relatively long or complex structures are used in many applications. These bridges are often designed for a specific site, and their dimensions cannot be estimated by the relatively simple procedures used for slab and box bridges. The structural designer can provide the necessary dimensions for the hydraulic modeling of these bridges. Additional information about these bridges is in publications such as the ODOT Bridge Design and Drafting Manual, the publications of the American Association of State Highway and Transportation Officials (AASHTO), and the many textbooks on the subject. Methods for estimating the dimensions of these structures are not included in this chapter.




a.) Upstream face of single-span box bridge opening
(facing upstream)




b.) Downstream face of double-span slab bridge opening
(facing upstream)

All dimensions in feet

**Figure 10-5 Estimated Waterway Opening Dimensions for Example**

### 10.3.6 Interior Bents (Piers)

An interior bent is an intermediate support for a multi-span structure. Interior bents in the water are often called piers, and both terms are used interchangeably in this chapter. The hydraulic designer should be familiar with the different types of bents and their hydraulic characteristics. The types, sizes, and locations of the interior bents are usually determined by the structural designer after consultation with the geotechnical and hydraulic designers. Many factors are considered, such as structural needs, aesthetics, environmental concerns, seismic safety, costs, hydraulic characteristics, the potential for debris accumulation, and foundation requirements.

Many foundation types are used to support interior bents. Piling is often an economical foundation in softer soils. Drilled shafts are often used in rock or in soils that are too rocky to accommodate piling. Spread footings can be used on rock or on erodible materials in locations where the footings are protected from scour or founded below scour depth. The foundation type is usually recommended by the geotechnical designer after consultation with the structural and hydraulic designers.

There are many types of interior bents, and the more common types and their foundations are discussed in this chapter and shown in Figure 10-6. The dimensions of interior bents are difficult to estimate without experience in structural design, and it is recommended that a structural designer be consulted for the needed dimensions.

#### 10.3.6.1 Single Column Bents

These bents are comprised of a single column, and they are often supported by piling with caps, drilled shafts, or spread footings, as shown in Figure 10-6a. These bents are sometimes called cantilever or hammerhead bents because of their shape.

Single column bents have hydraulic advantages. They are relatively easy to keep clear of debris. There is only one face that can catch and retain debris, and usually this face is accessible for cleaning. In addition, single column bents can sometimes be made with round or nearly round shapes, and scour depth is the same for all flow attack angles. In other words, the scour depths are not strongly influenced by the direction of the flow. This characteristic can make single column bents desirable where unskewed structures are used at skewed road/stream crossings, where the flow can attack the bent from varying angles, or where the attack angle is likely to change during the life of the structure.

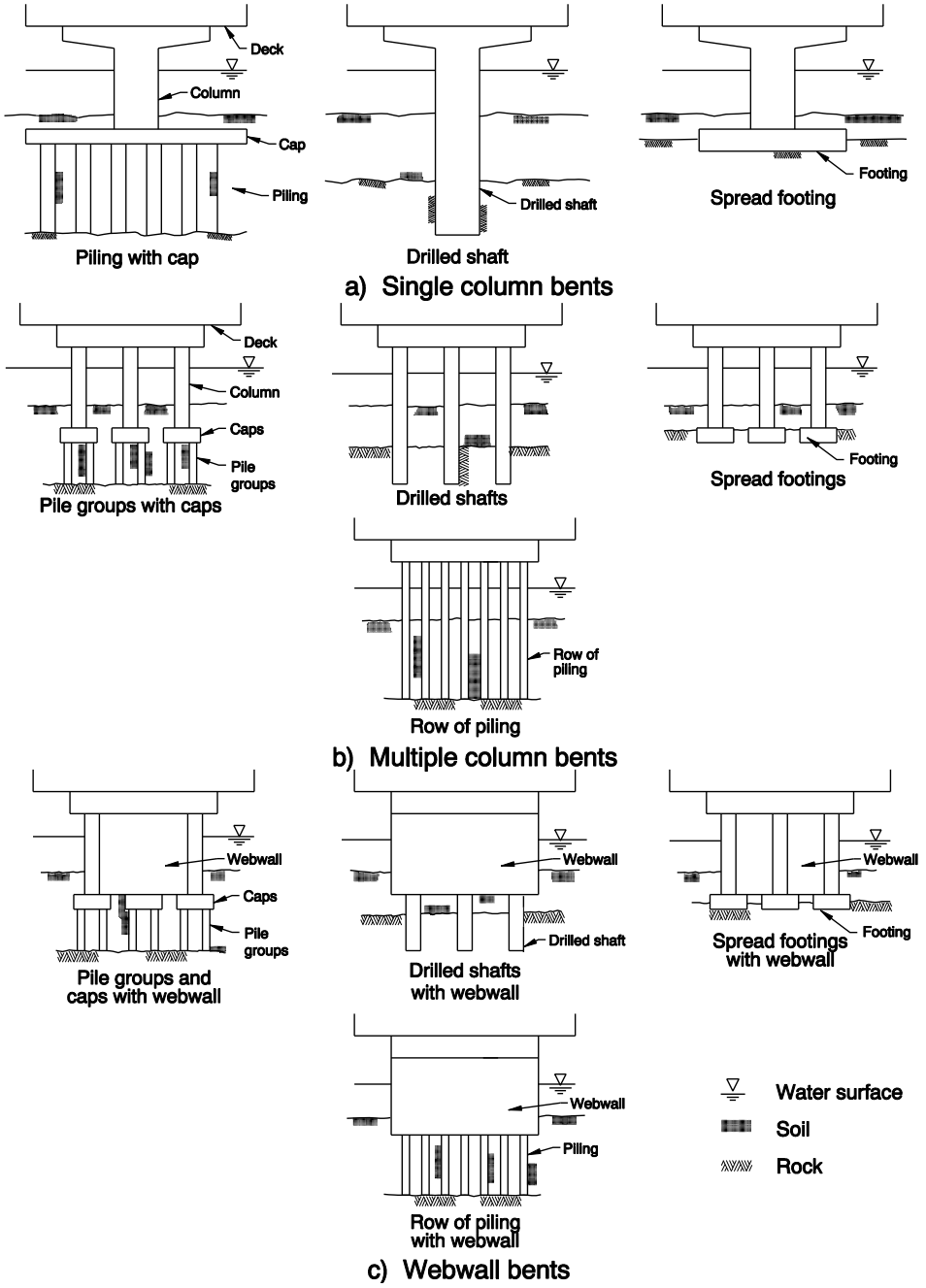



**Figure 10-6 Interior Bents**

A hydraulic disadvantage of single column bents can be their relatively deep scour depths. These depths are due to the relatively large width of the column, and they can be deeper than the scour depths associated with the thinner columns of multiple column bents.

#### 10.3.6.2 Multiple Column Bents

These bents are comprised of a row of columns, and they can be supported by pile groups with caps, drilled shafts, spread footings, or rows of piles, as shown in Figure 10-6b.

Multiple column bents have several hydraulic advantages. These bents often have relatively shallow scour depths. Often the scour depths are proportional to the widths of the columns, and the smaller columns of multiple column bents often have shallower depths than the large columns of single column bents. Also, like single column bents, multiple column bents with widely spaced round or square columns often have similar scour depths for varying flow attack angles.

A hydraulic disadvantage of multiple column bents is the possibility that they can catch and retain debris between the columns. This debris can be relatively inaccessible and difficult to remove - especially during flood events.

#### 10.3.6.3 Multiple Column Bents with Webwalls

These bents are comprised of columns with a connecting webwall between the columns. These bents are sometimes called "wall bents," and they are supported by the same foundations as multiple column bents without webwalls, as shown in Figure 10-6c.

The addition of a webwall to a multiple column bent has a considerable effect on its hydraulic characteristics. A bent with a webwall can have relatively shallow scour depths if the direction of the flow is parallel to the bent centerline. Scour depths can increase considerably if the flow attacks the bent at an angle. As a result, these bents perform best if they are aligned with the flow and the flow always comes from the same direction. Another advantage of these bents is their ability to pass debris. The webwalls prevent debris from lodging between the columns.

### 10.3.7 Abutments (End Bents)

Almost all bridges require support at the ends to retain the approach embankments and carry the vertical and horizontal loads from the superstructure. These supports are often called "abutments" or "end bents." "Abutment" is the most commonly used term in the field of hydraulic engineering, and "end bent" is often preferred by structural designers. Both terms are used in this chapter with the following distinctions. The term "abutment" applies to the entire supporting structure, including earth fill, the structural members, the foundation, and the revetment needed to provide scour and erosion protection. The term "end bent" is used to describe the load bearing structural members that support the span, such as the pilings, footings, drilled shafts, pilecaps, wingwalls, abutment walls, etc.

There are many different types of abutments, and the hydraulic designer should be familiar with their components, their advantages and disadvantages, and methods of estimating their dimensions. The choice of the type, size, and location of the abutment is made by the structural designer after consultation with the foundation and hydraulic designers. The more common abutments are discussed in this chapter.

#### 10.3.7.1 Spillthrough Abutments

This abutment has a fill slope on the channel side of the end bent, as shown in Figure 10-7. This abutment is often used where the bent is supported by piles or drilled shafts, and in almost all cases, the face of the abutment is protected from scour or erosion by a layer of riprap. As a general rule, a spillthrough abutment is more cost-effective than a vertical abutment where the height of the vertical abutment is greater than 10 feet. There are three types of spillthrough abutments defined in the ODOT Bridge Design and Drafting Manual, Options A, B, and C. Cross-sections of these abutment options are shown in Figure 10-8.

**Option A -** This spillthrough abutment option does not have wingwalls to retain the fill around the end bent, as shown in Figure 10-8a. This is the abutment with the lowest cost because there are no wingwalls. This cost advantage is often offset by the added expense of the bridge span, because a longer structure is needed to provide an adequate waterway opening. Of the three options, this type has the smallest waterway opening area for any given span length.

**Option B -** This option uses short wingwalls to retain the embankment fill, as shown in Figure 10-8b. This option is often cost-effective because the wingwalls allow the use of a shorter span. This option is used more frequently than the other two. For this reason, it is recommended that it be used for the initial modeling of a spillthrough abutment bridge.

**Option C -** This option uses taller wingwalls and an extended pile cap to retain the embankment fill, as shown in Figure 10-8c. It is often a cost-effective abutment in some applications because it can allow the use of a shorter span. A disadvantage of this option is the greater longitudinal forces from lateral soil loads that must be resisted by the end bent. The added structural strength needed to resist these forces can add to the cost of the abutment. This option is often used if the additional cost of the abutment can be offset by a reduction in the cost of the span.

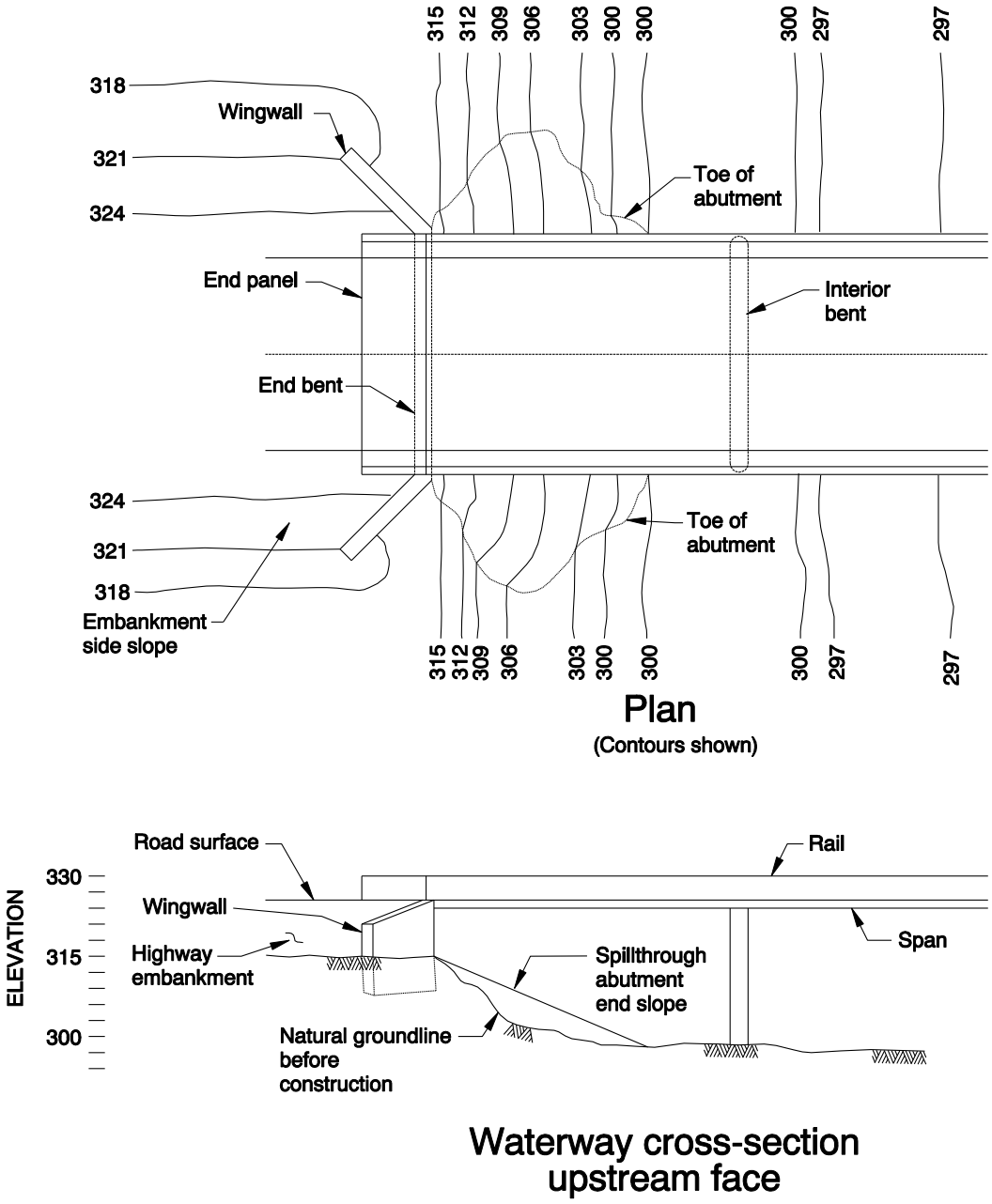
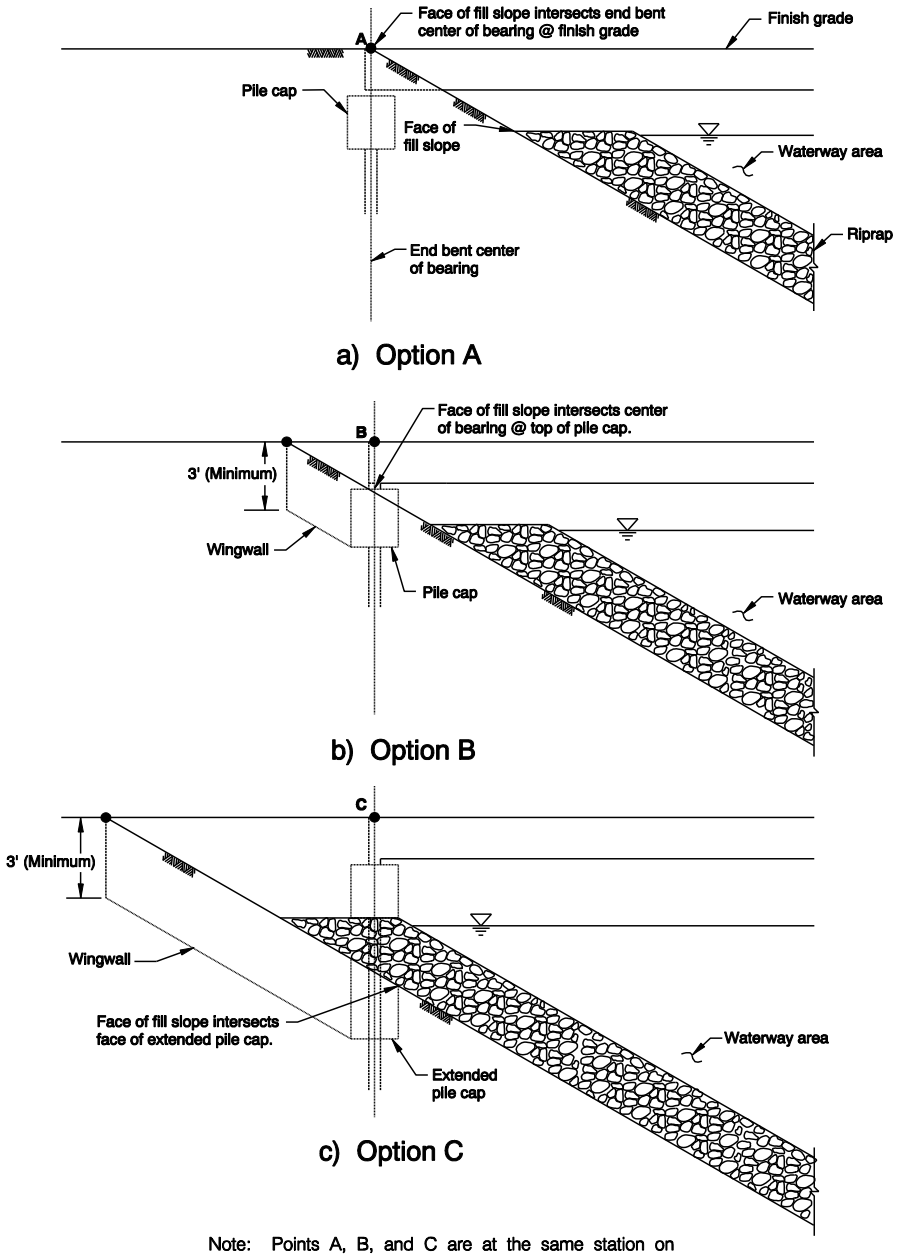



**Figure 10-7 Spillthrough Abutment with Wingwalls**




Note: Points A, B, and C are at the same station on Centerline. Figure illustrates how waterway opening areas can be increased by using different abutment configurations for bridges with equal span lengths. A = B = C

**Figure 10-8 Spillthrough Abutment Options**

**Abutment Dimensions -** The hydraulic modeling of a spillthrough abutment bridge waterway opening requires an estimate of its dimensions. The revetment protection on the abutment faces must be included in the hydraulic model if it obstructs a portion of the opening. Methods to estimate spillthrough abutment dimensions are included in this section. Methods to calculate revetment blanket thickness are in **Chapter 15**. Methods to estimate the clear span distance and the span dimensions are described previously in this subsection.

Abutment end slopes and embankment side slopes are shown in Figure 10-7. The end slope can be assumed to be 2 units of vertical distance to 1 unit of horizontal distance (1V:2H) for scoping estimates and the initial waterway opening hydraulic modeling. In almost all cases, the fill and revetment will be stable at this slope. Steeper end slopes are often desired to reduce structure costs and for other reasons. The end slope should not be steeper than 1V: 1-1/2H if it will be protected by loose riprap. The embankment side slope can be estimated for scoping and initial estimates from the guidelines in the ODOT Highway Design Manual. Regardless of the abutment or embankment slopes chosen, the stability of the fill and revetment should be verified in the geotechnical and hydraulic designs, respectively.

The location of the face of the abutment fill slope is needed for the hydraulic modeling of the structure. The face of the abutment fill slope in Option A can be approximated by extending a line downward from the end bent centerline of bearing on the top of the span, as shown in the Figure 10-8a. The face of the fill slope for Option B can be approximated by extending a line downward from the end bent centerline of bearing on the top of the pile cap, as shown in the Figure 10-8b. The fill slope for Option C can be approximated by extending a line downward from a point on the face of the pile cap, as shown in the Figure 10-8c.

### 10.3.7.2 Vertical Abutments

This abutment type differs from a spillthrough abutment because it does not have a fill slope on the channel side. The embankment fill is retained behind an abutment wall and wingwalls, as shown in Figure 10-9. These abutments are often supported by spread footings protected by loose riprap.

**Abutment Dimensions -** The dimensions of a vertical abutment waterway opening are relatively easy to estimate. The clear span distance can be estimated using Equation 10-1, and the faces of the abutment walls can be assumed to project vertically downward from the ends of the clear span. The wingwalls usually project out from the abutment wall at an angle. This angle is generally half the angle between the centerlines of the road and the stream, and the walls extend outward to the toes of the embankment fill, as shown in Figure 10-9. An exception is the “U” abutment where the wingwalls are parallel to the road centerline.

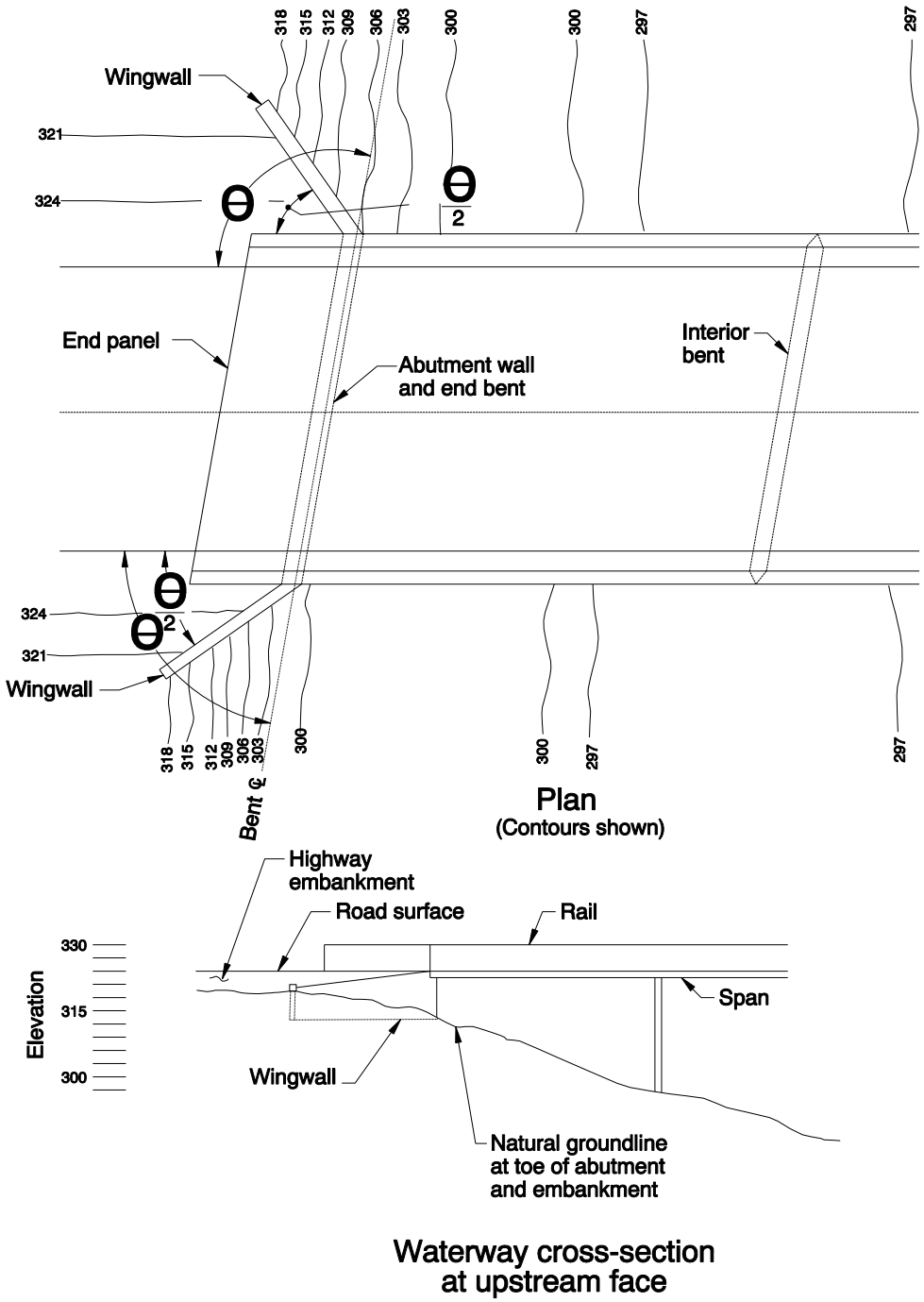



**Figure 10-9 Vertical Abutment**

In most cases, the revetment protecting the abutment wall toes does not reduce the cross-sectional area of the bridge opening, and it does not have to be included in the hydraulic model. The revetment protecting the abutment is usually placed in a trench adjacent to the wall face above the wall toe.

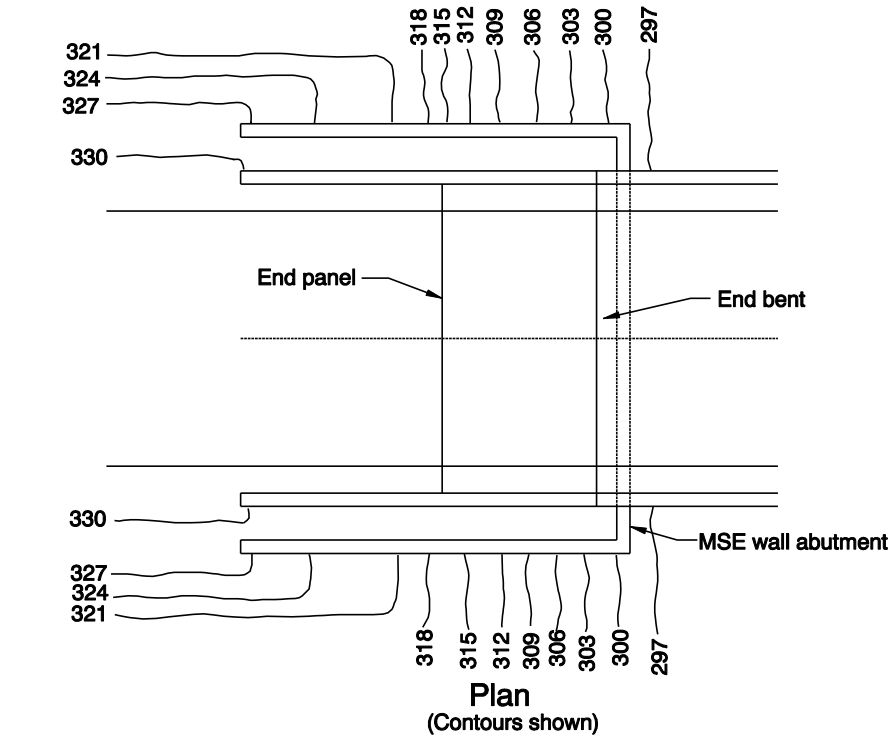
#### 10.3.7.3 Mechanically Stabilized Earth (MSE) Wall Abutments

The mechanically stabilized earth (MSE) wall abutment is a variation of the vertical abutment. This abutment is often a cost-effective alternative to a tall conventional vertical abutment, or a conventional abutment founded on a material with a low bearing capacity. Unlike the vertical abutment described in the previous subsection, the MSE abutment does not convey substantial concentrated loads to the underlaying earth or rock. The embankment fill is retained by walls on the outside of the abutment. The structural loads from the bridge are transferred to the fill within the abutment by a shallow footing under the end bent, and the structural loads are transmitted to the underlaying material by the abutment fill. A MSE wall abutment is in Figure 10-10.

MSE wall abutments are different than the typical conventional vertical abutments. Unlike conventional abutments which can be skewed up to 45 degrees or more, MSE wall abutments are usually unskewed, and if they are skewed, the maximum skew angle is $20^{o}$. In order to avoid a skewed configuration, they are often placed a distance back from the main channel at skewed highway/waterway crossings. This results in a longer bridge. In addition, unlike the angled wingwalls used on most conventional vertical abutments, wingwalls parallel to the road centerline are often preferred for MSE wall abutments.

The MSE wall abutment has been used in Oregon in a limited number of recent applications. Almost all of the abutments have been constructed away from the water on the streambank with little or no in-water work. In addition, most of the bridge spans and supporting footings have been located above the elevation of the check flood. The designer should use caution when considering a MSE wall abutment for a site where the abutment will need to be built in the water or where floodwaters are expected to contact the footings or the bottom of the span.

**Abutment Dimensions -** The dimensions of MSE wall abutments and the spans supported by these abutments should be estimated by a structural designer. These structures are often used on large and complex girder bridges, and the dimension estimating guidelines in this chapter may not apply.



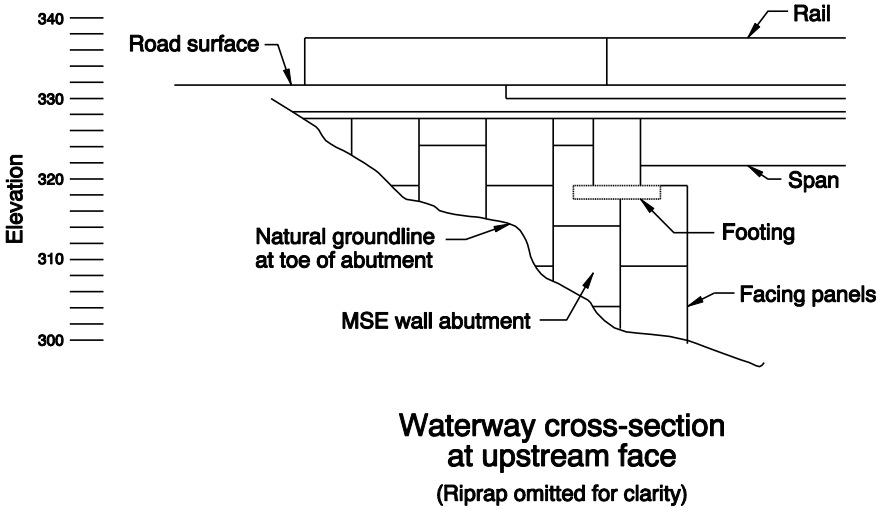




**Figure 10-10 Mechanically Stabilized Earth (MSE) Wall Abutment**

#### 10.3.7.4 Set Back Abutments

The set back abutment is a spillthrough, vertical, or mechanically stabilized earth abutment located at a distance away from the waterway. Spillthrough abutment set back distance is the horizontal distance between the toe of the abutment fill and the edge of the stream or other water body during ordinary high water, as shown in Figure 10-11. Vertical abutment or mechanically stabilized earth abutment set back is the horizontal distance between the inside face of the abutment and the edge of the waterway during the ordinary high water, as shown in Figure 10-12.

It is important to define the location on the abutment where the set back is measured. Abutment set back can seldom be defined by a single distance. A single distance could be used if the bridge crossed a prismatic waterway at a right angle. This rarely occurs. As a result, set back distances often vary at different locations on the abutment fill toe or the end bent.

Set back abutments are used for many purposes, as follows.

- Many streams and rivers have significant overbank flow beyond the limits of the ordinary high water during large floods. Set back abutments can enlarge the waterway opening to provide adequate hydraulic capacity and to reduce contraction scour depths.
- Abutments can be set back to provide pedestrian and animal passage on the stream banks under the bridge.
- Set back abutments can be used to reduce or eliminate the need for revetment or other scour protection.

*Note: The use of set back distance as scour protection must only be done after an engineering analysis proves it is viable. This is discussed in the scour protection section of this chapter.*

### 10.3.8 End Panels

An end panel is a reinforced concrete slab across the approach fill at the end of the bridge that absorbs the impact from traffic entering or leaving the bridge deck. It prevents the fill from settling or the pavement from cracking near the end bent. This panel is often called an "impact panel," and it is used on state highway bridges and many bridges built for local agencies. End panels are shown in Figures 10-7, 10-9, and 10-10.

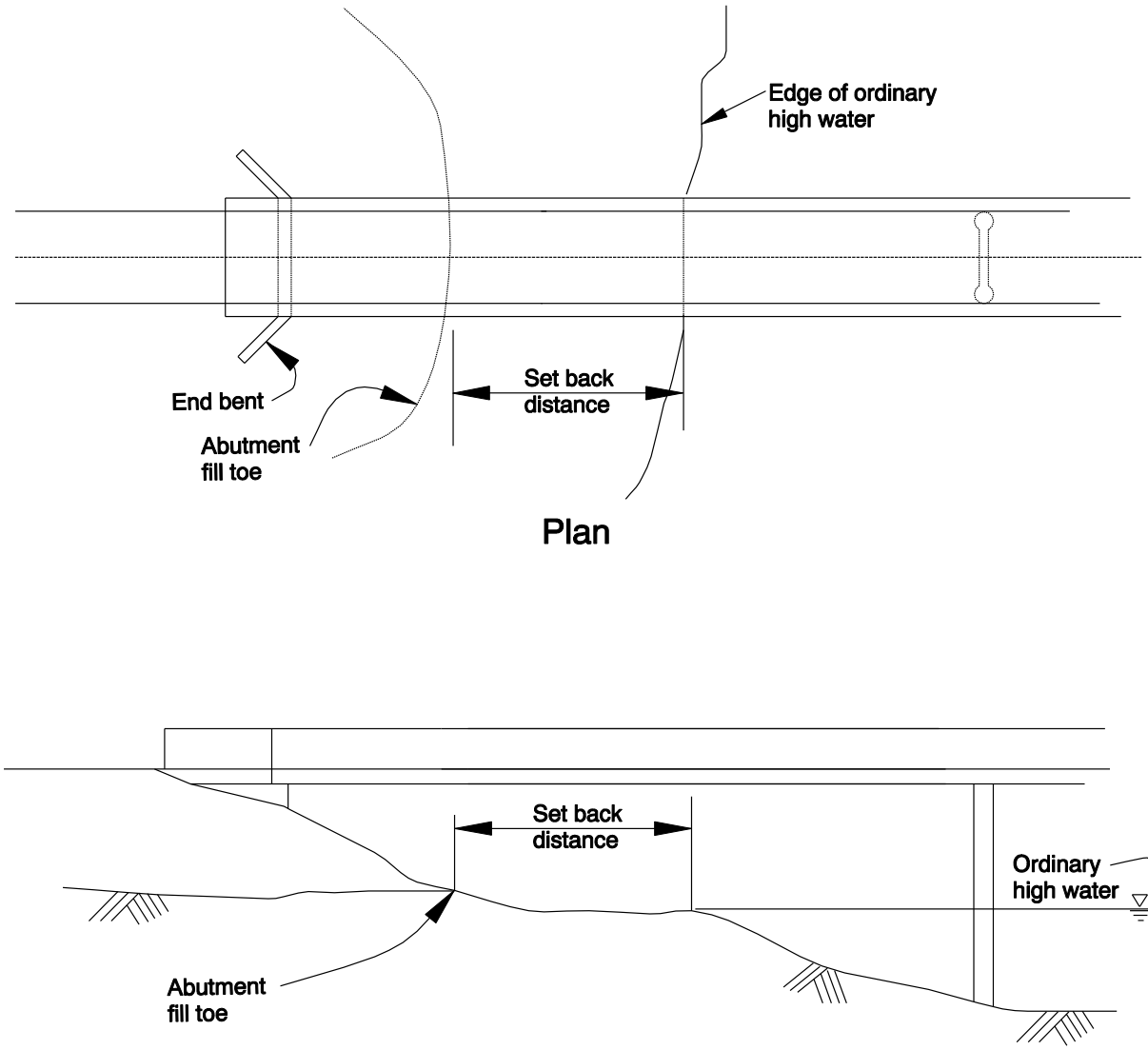



**Figure 10-11 Set Back Distance for Spillthrough Abutment Bridge**

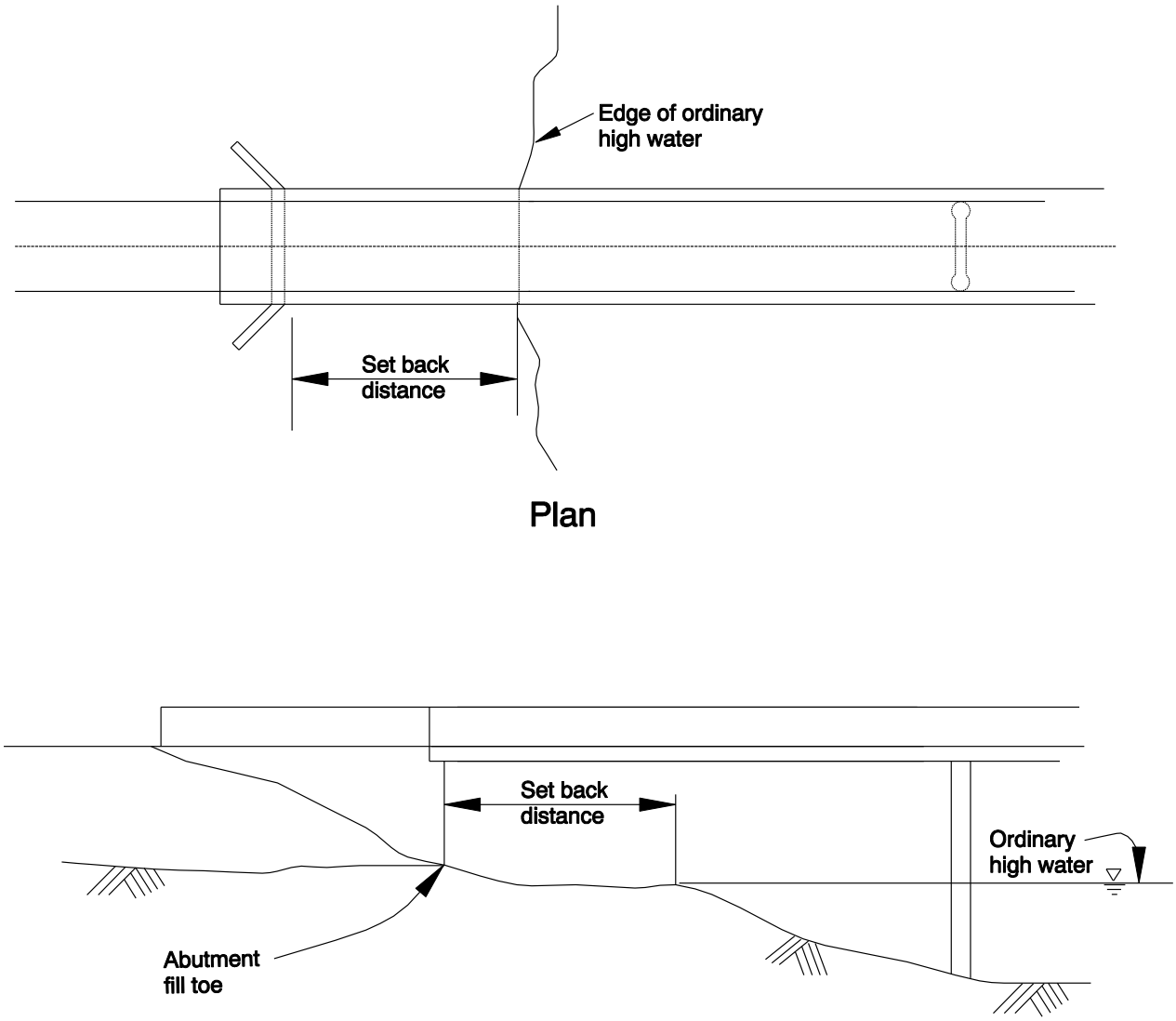



**Figure 10-12 Set Back Distance for Vertical and Mechanically Stabilized Earth Abutments**

Occasionally it is necessary to estimate the length of the end panel in order to design or specify erosion protection for the abutment supporting the panel. As general guidance, the end panel is 30 feet long for a bridge on a major highway, an abutment fill where excessive settlement is anticipated, a bridge with a deep abutment, or a bridge with severely skewed end bents. The end panel is 20 feet long if the foregoing situations do not occur.

### 10.3.9 Bridge Railing

In most instances the bridge railing is above the flood elevations, and its type, size, and location are of no concern to the hydraulics designer. In some instances, however, the bridge railing is within the path of flowing water and its hydraulic characteristics are of concern. Methods to model the hydraulic characteristics of rails that obstruct or retard flow are discussed in the user's manuals for the bridge analysis computer programs.

In general, most bridge rail types can be classified into two groups for hydraulic purposes, solid rail and flow-through rail. Solid rail includes solid concrete parapets and the often used ODOT Type F bridge rail shown in ODOT Standard Drawing BR200. These rails completely obstruct the flow unless the water goes over or around them. Flow through rails are also modeled as solid rails at sites where they may clog with floating debris.

Flow-through rail includes sheet metal guardrail mounted on posts and the tubular rails. Although these flow-through rails do not completely obstruct flow, they differ in their ability to retard flow. One of the most restrictive rails is the thrie beam rail in Standard Drawing BR233 and one of the least restrictive rails is the 2-tube side mount rail in Drawing BR226.

## 10.4 Design Criteria

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This section includes guidance for the hydraulic aspects of bridge waterway opening sizing and location. Design and check flood discharges are listed in **Chapter 3**, guidance for hydraulic report writing is in **Chapter 4**, guidelines for riprap sizing are in **Chapter 15**.

### 10.4.1 Discharges and Tailwater Elevations

Several discharges are used for different purposes in bridge hydraulic studies. The criteria for discharge selection are in Section 10.8. Nearby downstream water bodies may considerably influence the hydraulic characteristics at the site, and these influences are often called "tailwater effects." Tailwater effects are addressed in Section 10.9.

### 10.4.2 Backwater

Backwater is the increase in water surface elevation due to the constriction caused by the bridge or approach roadway. It is the increase above the water surface elevation if the bridge or roadway were not in place. This elevation change is measured at the approach section. This section is located one waterway opening width upstream from the upstream face of the bridge constriction, as shown in Figure 10-13. Backwater is determined by a comparison of these two water surface profiles:

- the crossing without the subject structure and the approaches to the structure that occupy the floodplain, often called the "natural channel" conditions (this condition may occasionally include backwater from other existing downstream structures), and
- the crossing with the existing or proposed structure, approaches, and channel modifications, often called the "with bridge" conditions.

*Note: Changes in riparian vegetation at the crossing site should be considered in the backwater analysis. The comparison should reflect the vegetation in its mature state. As an example, groups of willows planted as a riparian habitat enhancement can significantly increase backwater depths. The vegetation effects are accounted for in the selection of Manning's roughness coefficient "n."*

Backwater depths are determined for both the existing and proposed crossings if the crossing is a replacement of an existing structure. The backwater depths should not be excessive. General criteria follow.

**Road Overtopping -** The backwater from the bridge or approaches should not cause water to overtop the road more frequently than the design flood recurrence interval. Excessively frequent overtopping cannot always be prevented by enlarging the bridge opening. The roadway may need to be raised at some locations to satisfy overtopping criteria. Minimum overtopping recurrence intervals are listed in **Chapter 3**.

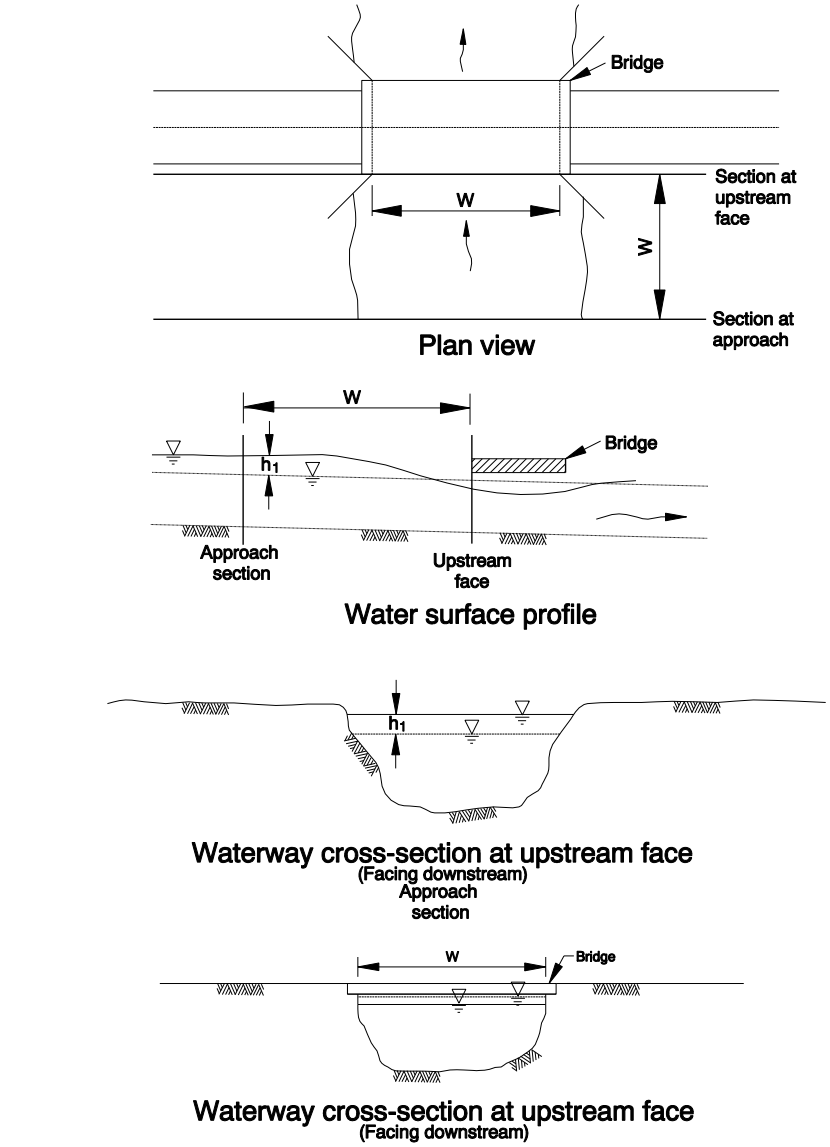


**Figure 10-13 Backwater**

In some circumstances, the bridge required to pass the design flood would be excessively long or high. If this occurs, a combination of flow through the bridge waterway opening and overtopping at low points in the adjacent roadway may be an alternative to accommodating the entire design flood under the bridge. This may be acceptable and allowable in certain circumstances, such as the following.

- The overflow does not damage adjacent property.
- The overflow does not travel across adjacent properties in paths it did not use before, or in significantly greater or lesser quantities than it did before, unless a drainage easement is acquired or the property is purchased.
- The approach roadway is also overtopped in other locations and traffic cannot travel to the bridge.
- Alternate routes are available to traverse or to evacuate the area.
- The road is useable when overtopped (shallow overtopping).

**Crossings at Locations of Existing Structures -** In general, backwater elevations should not be increased when replacing existing structures. A reduction in backwater should be considered if there is a flooding problem upstream from the existing crossing, or there is a scour problem at the existing bridge that can be alleviated by reducing the backwater.

**Crossings at New Locations -** In general, backwater elevations should not be higher than the site can tolerate. This is often governed by the road overtopping criteria. Backwater should not be increased on nearby upstream properties unless the needed drainage easements or right-of-way have been acquired. See **Chapter 3**.

**Crossings in Regulatory Floodplains or Over Regulatory Floodways -** Regulatory floodplains or floodways often have restrictions on the depth of the backwater that can be produced by a structure or its approaches. See **Chapter 2**. All proposed changes to water crossings should be evaluated to determine if they are in a regulatory floodplain or floodway, and it should be verified that they meet all applicable regulations. The ODOT Region Technical center hydraulic staff should be consulted about crossings in or over floodways subject to the federal Flood Insurance Program administered by the Federal Emergency Management Agency (FEMA). The ODOT Geo-Environmental Section's Engineering and Asset Management Unit or the FEMA website has floodway maps that can be used to determine if a crossing is in a floodway.

### 10.4.3 Clearance

The vertical distance between the design flood water surface and the bottom of the bridge slab, box or beam is often called "clearance." The bottom of the bridge span is considered to be the elevation of the lowest structural member. This is the bottom of the lowest slab, box, girder, or beam.

The normal minimum clearance is 1.0 foot above the design flood elevation or 3 feet if drift or debris is a concern. The appropriate design flood recurrence intervals are listed in **Chapter 3**. If practical, 1 foot of clearance above the 100-year flood is provided.

Exceptions to the clearance requirement can be obtained with ODOT approval. An exception is for city or county bridges whose approaches are overtopped more frequently than once every ten years. The minimum bottom-of-beam elevation for these situations is 1.0 foot above the 10-year flood elevation. Under rare circumstances such as a park setting or where other controls on grade lines make it necessary, high water above the beam bottoms or over the deck may be allowed.

Many of the larger rivers, as well as bays, lakes, and estuaries in Oregon are considered to be navigable. The appropriate ODOT permit liaison officer should be contacted to determine if a crossing is on a navigable waterway and the needed navigational requirements. In addition, canals and irrigation ditches frequently have clearance requirements and the appropriate operators or irrigation districts should be contacted.

Clearance distances should be maintained or increased on crossings over streams that carry debris or ice during floods. Maintenance personnel should be contacted, and maintenance records reviewed, to see if debris or ice passage has been a problem. Three to four feet of clearance may be needed at some sites to accommodate debris or ice passage.

### 10.4.4 Waterway Alignment

At a minimum, the waterway opening should span the channel. The channel is the portion of the waterway that:

- conveys the moving bed material during large floods,
- typically is scoured of vegetation during the flood season, and
- often has steep and well defined banks.

The channel on many waterways can be defined as the area that is subject to flow every year. This is the waterway under the Ordinary High Water (OHW) elevation. The OHW is defined and discussed in **Chapter 6**.

The waterway opening should discharge flow with a direction and velocity that minimizes or eliminates damage to downstream facilities or property. This will often require a skewed structure at a location where the highway crosses the stream at an angle. The abutments should be skewed to match the direction of flow in the channel if the stream and roadway centerlines are skewed and the abutment faces are near the edges of the channel. An unskewed structure (i.e., bents normal to the roadway centerline) can often be used at a skewed crossing if the abutments are set well behind the channel in areas with low flow velocities. In instances where low flow and flood flow pass through the crossing in different directions, the waterway opening should be aligned to pass the flood flow.

The waterway opening should provide adequate performance throughout its design life with anticipated changes in streambed profile and alignment. As an example, additional streambed to bottom-of-span clearance may be needed over a waterway with a history of streambed aggradation, or a longer bridge might be needed to accommodate meander migration.

The waterway opening size and shape should consider construction and maintenance practices. As an example, additional deck to streambed clearance may be needed for maintenance equipment at a crossing where debris may need to be removed from under the bridge.

### 10.4.5 Environmental Concerns

Bridges and bridge approaches are often located in environmentally sensitive waterways and riparian areas. The need to minimize disturbance to these areas can often govern the location of the abutments and the interior bents, and consequently, the waterway opening size and alignment. The Region Environmental Coordinator or project team environmental representative should be contacted early in the design process for a description of the environmental concerns and requirements.

### 10.4.6 Scour Elevations and Countermeasures

The proposed structure and approaches need to resist scour damage. In order to do this, the flow depths and velocities through the structure must not be in excess of those that the scour countermeasures can withstand. The need to keep these depths and velocities at acceptable levels can often govern the size and alignment of the waterway opening and the elevation of the bottom of the bridge span.

## 10.5 Design Procedures

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The hydraulic design of a stream crossing is a complex procedure that involves many tasks. The design procedures for state highway system bridges are in this chapter. Design tasks for local agency bridges are in the current ODOT Local Agency Project Manual. The typical bridge hydraulic design includes many, if not all, of the following tasks;

- visit the site to provide scoping assistance, investigate vertical and lateral channel stability, and request specialized hydraulic survey data,
- determine or assist in determining the Ordinary High Water (OHW) elevations,
- compile data,
- determine the hydrology for facility design and temporary water management (TWM),
- determine if variable tailwater elevations occur,
- create a hydraulic model of the existing crossing, if one is present,

- calibrate the hydraulic model of the existing crossing so the modeled performance reflects the historical performance,
- determine the "existing bridge" hydraulic performance,
- create a hydraulic model of the crossing with no bridge or approaches in place to determine the "natural channel" hydraulic performance,
- create a hydraulic model of the probable alternatives and analyze the alternatives,
- design the scour protection countermeasures,
- reanalyze the proposed alternatives with the scour countermeasures in place, if needed,
- analyze the OHW flood through the proposed alternatives,
- analyze the detour or temporary crossing, if used,
- compile and submit the Hydraulics Report,
- provide hydraulic assistance for the deck drainage design,
- review hydraulic aspects of the design as shown on plans during various stages of the design,
- develop TWM plans and specifications,
- provide data and support to permit specialist in order to obtain permits required for the project, and
- provide hydraulic design assistance to the project team, as needed.

Not all tasks need to be performed for every situation, as shown in the following list. These are generalities, and engineering judgment should be used when planning the hydraulic study procedure for a specific location.

1. A backwater analysis may not be possible or needed for bridges over pooled or slowly flowing waters (less than 3 feet per second during the check flood), such as crossings over sloughs, wetlands, lakes, or tidal areas such as mudflats. At these crossings environmental concerns rather than hydraulics often determine the required bridge length.

   The hydraulic study:

   - should provide the water surface elevation in the waterway opening,
   - should include revetment designed to resist wave action, if applicable, and
   - it does not need to provide approach section data, backwater depths, velocities, or scour elevations.

2. An existing channel water surface profile, only, (i.e. bridge does not need to be included in the hydraulic model) may need to be calculated for bridges over flowing water where:

   - the abutments or piers do not contact the water during the 500-year flood,
   - the waterway is in a fixed location where it cannot move and contact the structure, and
   - revetment is not needed as a protection from flowing water.

3. A scour analysis is not required if the structure is solidly founded on non-erodible rock and no revetment is needed.

4. Calculation of hydraulic data may not be needed if a previous hydraulic analysis provides the needed information. This previously analyzed information should be used with care. The data that is used should reflect current conditions and have sufficient accuracy. The Hydraulics Report should mention which data is from previous studies, and it should also list the sources.

5. The proposed work on the structure does not involve the waterway. Bridge rail replacement and deck rehabilitation are examples. Bridge widening may not need a hydraulic study if it does not affect the hydraulic performance of the structure, and scour elevations or revetment sizes are not needed for the design.

The design steps are described in detail in the following sections. Guidance on the information to be included in the Hydraulics Report is in **Chapter 4**.

## 10.6 Scoping and Preliminary Structure Estimate

A preliminary design task is to assist the project team with the preliminary structure type, size, and location estimate. This task is called "scoping", and it usually requires a site inspection. The site visit is a convenient time to determine the specialized survey data for the hydraulic study. The survey data is requested during this visit or shortly thereafter. The site visit is also a good time to investigate channel stability because this factor will affect the preliminary design recommendations. Hydraulic study data collection is discussed in detail in **Chapter 6**.
The preliminary structure type, size, and location estimate is made during or shortly after the site visit, and the hydraulic designer provides input from the hydraulic perspective. The following general guidelines can be used.

1. The structure should span the active stream channel, at a minimum. Regardless of the bridge type, the abutments should not intrude into the channel. This requirement often results in a spillthrough structure alternative that is longer than a vertical abutment structure. Abutment face slopes of 1V: 2H can be used for preliminary estimates.

2. The structure may need to span both the main channel and floodplain overbank areas at sites where significant contraction scour would occur if the bridge spans the channel, only.

3. The structure should be skewed, as needed, to match the flood flow direction. In some cases an unskewed structure can be used. The decision to use an unskewed structure at a skewed crossing is usually made later in the design process.

4. The most likely structure type is used for the preliminary estimate and it is determined by the bridge designer. In absence of this information, vertical abutment bridges are often used where there is non-erodible rock at or near the ground surface at the abutment locations, and the abutment heights are less than 10 feet. Spillthrough abutment bridges are often used at other locations.

5. Environmental setbacks and the nature of allowable in-stream work should be addressed at this time. These requirements can often greatly influence structure type, size, and location.

6. Regulatory floodway requirements should be addressed at this time if the structure, roadway embankments, riparian enhancement, or riparian mitigation are in a floodway. These requirements can control the structure type and location, the roadway alignment, the landscaping and riparian habitat modifications, the choice of sidewalks, guardrail, etc. A hydraulics designer should be consulted if any part of the project is within a floodway. The floodway is defined in **Chapter 2**.

## 10.7 Compiling Project Design Data

Data compilation for a hydraulic study includes collecting the preliminary data, as discussed in the previous section and Chapter 6. It also includes compiling project design information such as the survey and utility location, and datum verification.

An accurate site survey and resulting design information is especially critical for structures such as bridges and large culverts. The typical survey is done using computerized electronic equipment and must be processed prior to use in design. This data reduction is typically done in the survey office. The survey office verifies the quality of the survey and the information submitted to the designers. Experience has shown it is important for the hydraulic designer to also carefully review the design information and to verify it realistically represents the hydraulic structures. The following are some of the more common hydraulic survey errors.

- The terrain survey extends down to the water surface, only. The terrain model produced by the survey erroneously shows a flat channel bottom at the water surface elevation during the survey.
- The terrain model extends down to the edge of the water, and the only underwater data collected are the channel thalweg locations and elevations. The survey terrain model will represent the underwater channel as “V” shaped, although this might not be the case.
- The culvert invert elevations are measured at the surface of the sediment layer covering the invert, rather than on the pipe itself. This will represent the culvert being at a higher than actual elevation.

There are several methods to verify survey accuracy. Many, such as statistical analysis of confidence points, are complex and done by people experienced with surveying. One of the more simple methods is also the most effective. It is to "cut' cross-sections from the terrain model at critical locations, to take these sections to the site, and to visually compare their shape and elevation to actual features. Errors in the survey or data reduction are often clearly visible.

The utility location information should also be verified. Visible utilities such as poles should be shown on the terrain model and be seen in the field. The site should be inspected for signs or other markers indicating underground utilities are present. These utilities should also be shown on the terrain model.

The project survey elevation datum should be noted and compared to the elevation datums used for the reference studies, reports, and publications. This is especially critical where there are regulatory floodplains and floodways. Several different vertical control reference datums have been used throughout the state since the beginning of the highway system, and many of these datums are in use today. The elevation differences between these datums are often several feet. It is essential to convert all elevations from other sources to the project datum before they are used in design.

## 10.8 Hydrology

One of the most important steps in bridge hydraulic design is to determine the study discharges, or "hydrology." Hydrologic methods are provided in **Chapter 7** and the data needed in the hydrology section of a hydraulics report are listed in **Chapter 4**. The discharges used in bridge design are:

- the design discharge (the 25-year, 50-year, or roadway overtopping discharge),
- the base flood (100-year) discharge,
- the check discharge (the road overtopping or 500-year discharge),
- scour calculation discharges,
- discharges needed for environmental and permit application data, such as the average annual (2-year) discharge or the ordinary high water (1.5-year discharge), and
- monthly peak or daily discharges for temporary water management during construction.

Changes in sea level during the bridge design life are considered in coastal structure design.

### 10.8.1 Design Discharge

Design discharges for bridges in various applications are listed in **Chapter 3**. The design discharge is used to compare the hydraulic characteristics of the existing and proposed crossings. It is also used to determine the minimum recommended waterway area and the minimum bottom-of-span elevation, and to design the revetment. The 25 or 50-year flows are the most common

design discharges. A 100-year flood design discharge is used in regulatory floodways and floodplains. The road overtopping flood is the design discharge if overtopping occurs more frequently than the design flood recurrence interval listed in **Chapter 3**. The hydraulic performance during the design discharge is reported for both the existing and proposed structures.

### 10.8.2 Base Flood

This is a discharge with a 100-year recurrence interval. It is used to compare the hydraulic performance of the existing and proposed crossings, to design the revetment, and to calculate scour elevations. The base flood is often used to assess the effects of inundation caused by the proposed structure and to compare the effects of inundation caused by the existing and proposed structures. The base flood is the design flood in regulatory floodplains and floodways. The hydraulic performance during the base flood discharge is reported for both the existing and proposed structures.

### 10.8.3 Check Discharge

The check discharge is used to compare the hydraulic performance of the existing and proposed crossings, to check the stability of the structure and revetment, and to estimate the scour elevations. The check discharge is either the 500-year flood or the roadway overtopping flood. The check discharges are reported for both the existing and proposed crossings. It is common to have different check discharges for the existing and proposed structure if the road overtopping floods are the check discharges. Hydraulic performance during the check flood is reported for the existing and proposed structures.

500-Year Flood - This flood is expected to occur, on the average, once every 500 years. This is the check discharge if road overtopping occurs less frequently than once in 500 years.

Road Overtopping Discharge - The road overtopping discharge is the flood when either the road overtops, the bridge overtops, or another area near the upstream side of the bridge overtops and the water enters an adjacent drainage. In general, this occurs when the energy grade line elevation at the approach section is at the same elevation as the point of overtopping. The energy grade line represents the surface elevation of water pooled on the upstream side of the crossing. The road overtopping flood is not analyzed if it occurs less frequently than once every 500-years.

The hydraulic stresses caused by an overtopping flood with a recurrence interval less than 500-years will often create greater hydraulic stresses than the 500-year flood. This is not always the case. Occasionally a 500-year flood with overtopping will create greater stresses than a more frequent overtopping flood. It is good practice to always analyze scour and waterway opening velocities caused by the 500-year flood. The flood causing the deepest scour and highest waterway opening velocities should be reported.

### 10.8.4 Scour Calculation Discharges

Scour is calculated for the 100-year flood, the 500-year flood, and the roadway overtopping flood. The scour depths are compared to determine the design and check discharges for scour estimates. The scour design and check discharges depend on the site hydraulic characteristics, as follows.

**Case 1:** The incipient roadway overtopping flood occurs more frequently than the 100-year flood and creates deeper scour than the 100-year flood. The overtopping flood is the design flood and the check flood is either the 100-year flood or the 500-year flood, whichever creates the deepest scour.

**Case 2:** The incipient roadway overtopping flood occurs more frequently than the 100-year flood and creates shallower scour than the 100-year flood. The 100-year flood is the design flood and the check flood is either the overtopping flood or the 500-year flood, whichever creates the deepest scour.

**Case 3:** The incipient roadway overtopping flood occurs less frequently than the 100-year flood but more frequently than the 500-year flood. The 100-year flood is the design flood and the check flood is either the overtopping flood or the 500-year flood, whichever creates the deepest scour.

**Case 4:** The incipient roadway overtopping flood occurs less frequently than the 500-year flood. The 100-year flood is the design flood and the 500-year flood is the check flood.

### 10.8.5 Average Annual Discharge and Ordinary High Water Discharge

The average annual discharge has a 2.33-year recurrence interval. It is the flood that occurs at least once every two years on the average. This flood was often used to determine the riparian area for regulatory purposes. It is still used by some agencies.

Experience has shown that the 2-year flood can often overestimate the riparian area. Many regulatory agencies have adopted the ordinary high water (OHW) discharge as an indicator of the riparian area. This is the flood expected to occur every year, on the average. The OHW elevation is often determined by field marks, as discussed in **Chapter 6**. OHW elevation estimates based on field marks are often uncertain and difficult, especially after floods. It is good practice to verify the OHW elevations using a hydraulic model. The OHW flood is approximately the 1.5-year event.

### 10.8.6 Temporary Water Management Discharges

Temporary Water Management (TWM) is used to keep water from being contaminated by construction activities, to keep water from interfering with construction operations, and in the case of streams, to ensure there is an uninterrupted flow of water through the work site.

Several discharges are used in TWM. Mean daily exceedance discharges are often used to estimate pump sizes, bypass pipe diameters, and other TWM features. Maximum predicted discharges are often used to design critical temporary structures such as cofferdams. Methods to calculate TWM discharges are in **Chapter 7**.

### 10.8.7 Sea Level Change

Sea level change should be considered in coastal structure design. This change is primarily a combination of global sea level rise combined with local vertical land movement. Trends in sea level change have been calculated for three tide level gaging stations on the Oregon Coast and a nearby gage in California by the National Ocean Service (NOS) in cooperation with the National Oceanic and Atmospheric Administration (NOAA). A positive trend means the sea level appears to be rising in relation to the land. A negative trend means the opposite. The following summaries are based on information from NOAA.

**Gage 9419750 Crescent City, California (near Brookings) –** "The mean sea level trend is -0.16 feet per century (- 0.48 millimeters per year) with a standard error of 0.009 inches (0.23 millimeters) per year based on monthly mean sea level data from 1933 to 1999.

**Gage 9432780 Charleston, Oregon (near Coos Bay) –** "The mean sea level trend is +0.57 feet per century (+1.74 millimeters per year) with a standard error of 0.034 inches (0.87 millimeters) per year based on monthly mean sea level data from 1970 to 1999.

**Gage 9435380 South Beach, Oregon (near Newport) –** "The mean sea level trend is +1.15 feet per century (+3.51 millimeters per year) with a standard error of 0.029 inches (0.73 millimeters) per year based on monthly mean sea level data from 1967 to 1999.

**Gage 9439040 Astoria, Oregon –** "The mean sea level trend is (-0.05 feet per century (–0.16 millimeters per year) with a standard error of 0.009 inches (0.24 millimeters) per year based on monthly mean sea level data from 1925 to 1999.

Sea level change can also be estimated using this chart developed by P. Vincent for a 1989 Masters of Science Dissertation at the University of Oregon, Eugene, Oregon.

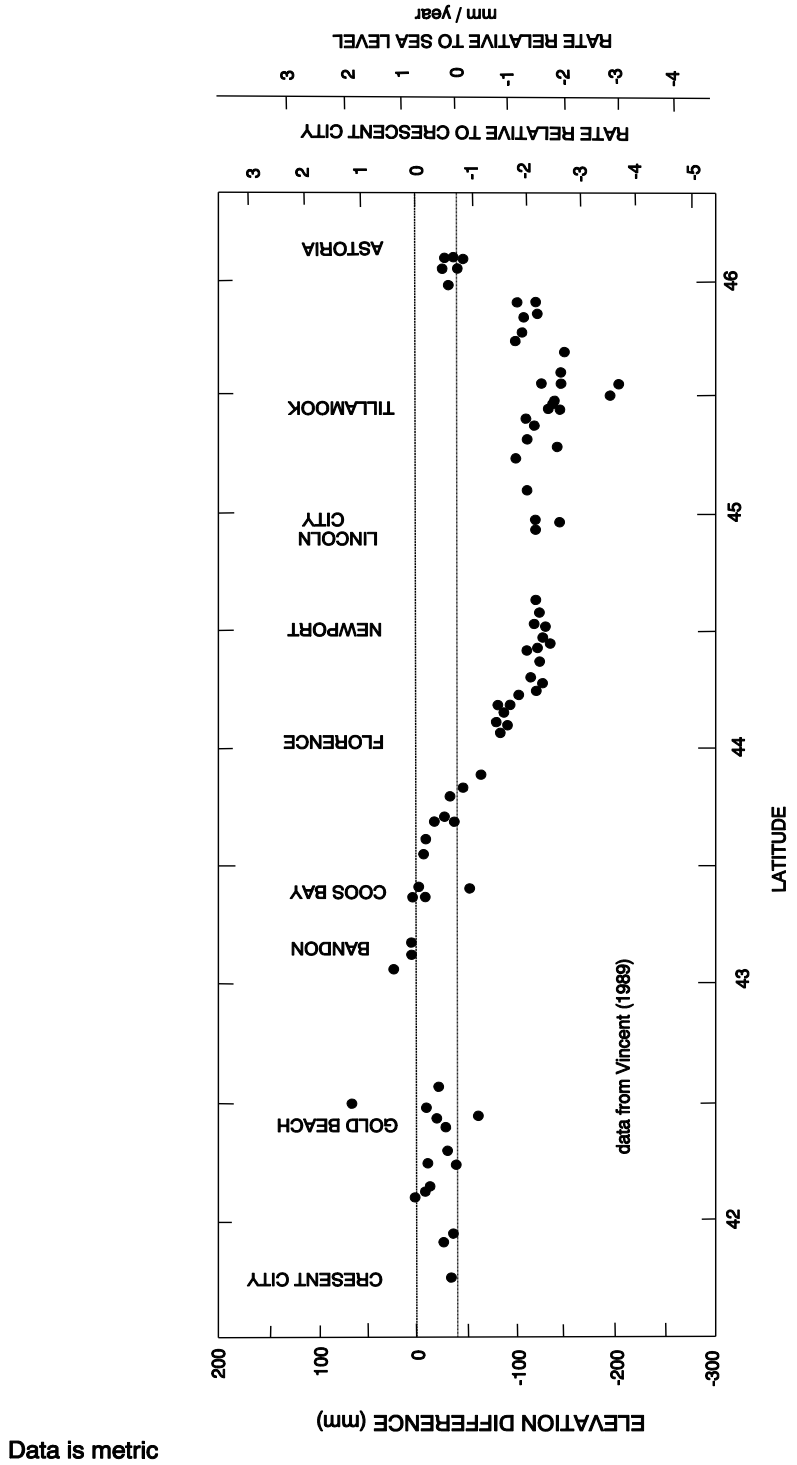


**Figure 10-14 Sea Level Change on the Oregon Coast**

## 10.9 Tailwater Effects

Many bridges cross streams and rivers upstream from their confluence with other streams and rivers, lakes, estuaries, bays, or the ocean. The water surface elevation in the downstream water body should be considered during the hydraulic design when it can influence the flow velocity and water surface elevation at the bridge. Examples of different situations follow.

Situation 1- The bridge is on a stream or river, the downstream water body is also a stream or river, and neither watercourse has significant regulation or flow diversion upstream from their confluence. In addition, both the tributary and downstream water body watersheds should respond to the same hydrological events. In this situation, it may be possible that floods with different recurrence intervals can occur simultaneously on either watercourse. The difference in flood recurrence interval increases with the ratio of watershed size, and the relationship is shown in the "Frequencies for Coincidental Occurrence" table in **Chapter 13**.

The crossing should be analyzed with the given flood passing through the tributary and a coincident flood in the downstream water body. The crossing should also be analyzed with the given flood in the downstream water body and a coincident flood in the tributary. The hydraulic characteristics should be reported for the combination(s) that result in the highest water surface elevation at the bridge and the highest velocity through the waterway opening. These velocities and elevations will be used in the hydraulic and structural designs.

Situation 2- The bridge is at a site where the coincidental occurrence relationship in Situation 1 does not apply. This could occur when the crossing is over a regulated stream, the downstream water body is a regulated stream or reservoir, or the tributary crossed by the structure and the downstream water body does not respond to the same hydrologic events. An example where Situation 2 applies is a flooding stream that discharges into a downstream reservoir that has been previously drained to provide flood storage. Unlike the coincident floods that would occur during Situation 1, in this instance there would be no tailwater effect from the downstream water body.

In Situation 2, the crossing should be analyzed when the given flood passes through the bridge and the flow is at normal depth in the downstream channel (there is no backwater from the neighboring water body). This combination will usually result in the highest velocity through the bridge opening. The crossing should also be analyzed when the given flood passes through the opening and there is a flood of identical recurrence interval in the downstream water body. This combination will usually result in the highest water surface elevation. The hydraulic data for both combinations of discharge and backwater should be reported.

Situation 3 - The bridge is on a stream or river and the elevation of the downstream water body is influenced by the tides. This occurs in estuaries and bays, near the ocean, and on large tidally influenced rivers such as the lower reaches of the Columbia River. The highest velocity is

expected when the given flood passes through the bridge opening, and there is mean lower low tide in the downstream water body. The highest water surface elevation at the bridge is expected when the given flood passes through the bridge opening and there is mean higher high tide in the downstream water body.

Situation 4 - The bridge is at a site where the tailwater elevations are influenced by another structure, such as a weir, a dam, or another bridge. The structure should be analyzed with and without the downstream structure in place if there is any chance the structure will be removed or wash out during the design life of the proposed structure. (In the case of a nearby bridge, it is always assumed that the structure will be removed.) The highest water surface elevations typically occur when the downstream structure is in-place, and the greatest flow velocities usually happen after it is removed.

## 10.10 Hydraulic Modeling

Hydraulic modeling is the process to determine the water surface profiles through the bridge site. It is also used to estimate flow velocities and to calculate scour depths.

Water flowing through a bridge is a complex process. In most instances, the predominate flow direction is in the longitudinal (upstream-downstream) direction, and there is limited flow in the vertical (up-down) direction, or in the transverse (side-to-side) direction. The hydraulic characteristics of this flow are modeled by one-dimensional analysis, which recognizes flow in the longitudinal direction and ignores flow in other directions. These one-dimensional analyses are often made using the following computer programs:

- U.S. Corps of Engineers Hydraulic Engineering Center - River Analysis System (HEC-RAS), or
- Federal Highway Administration (FHWA) Water Surface Profile (WSPRO). This program is a module in the FHWA HYDRAIN software package.

The HEC-RAS program is preferred by ODOT.

Sometimes, transverse flow across the waterway is predominate and the one-dimensional method may not be appropriate. In these instances, more accurate answers may be provided using a two-dimensional modeling method. Two dimensional (2-D) models simulate flow in the longitudinal and transverse directions at a series of user defined node points. Flow in the vertical direction is assumed to be negligible. These models can account for transverse flow due to lateral velocities and water surface gradients that cannot be accounted for with one-dimensional models. A 2-D model should be considered for major projects with complex flow patterns that one-dimensional models cannot adequately analyze, such as the following.

- Wide floodplains with multiple openings, particularly on skewed embankments.
- Floodplains with significant variations in roughness or complex geometry, such as ineffective flow areas, flow around islands, or multiple channels.
- Sites where more accurate flow patterns and velocities are needed to design better and more cost-effective countermeasures such as riprap along embankments and/or abutments
- Tidally affected river crossings of tidal inlets, bays, and estuaries.
- High risk or sensitive locations where potential losses or liability costs are high.

Two commonly used 2-D computer programs are the U.S. Corps of Engineers (USCOE) RMA-2 and the Federal Highway Administration (FHWA) Finite Element Surface-Water Modeling System: Two-Dimensional Flow in a Horizontal Plane (FESWMS-2DH). Both RMA-2 and FESWMS model steady and unsteady flow. FESWMS is recommended for highway crossings of rivers and floodplains because it supports both super and subcritical flow analysis, and it can analyze weirs (roadway overtopping), culverts, and bridges.

The Surface Water Modeling System (SMS) developed by Brigham Young University in a cooperative project with the USCOE and the FHWA can be used to develop the finite element mesh and associated boundary conditions necessary for RMA2 and FESWMS. The solution files for both programs, which contain surface elevation, velocity, or other functional data at each mesh node, can be read into SMS to generate vector plots, color-shaded contour plots, time variant curve plots, and dynamic animation sequences.

*Note: The RMA2 and FESWMS-2DH methods require considerable input data and user expertise, and as a consequence, it can be an expensive analysis. It is recommended that the ODOT Region Technical Center hydraulics staff be contacted before this method is used on ODOT projects.*

### 10.10.1 Existing Structure

The first step in the hydraulic modeling is to analyze the existing structure, if one is present. The terrain data is provided by the field survey, and the structural data is based on the survey data and the plans for the existing structure.

The predicted existing crossing hydraulic performance should be compared to historical flood records and observed highwater data, if it is available. The hydraulic model may need to be adjusted if the predicted flood frequency relationship is significantly different than the flooding history. This adjustment is called "calibration" and it is usually made by adjusting the waterway or structure Manning's roughness coefficients. It is also made by designating ineffective flow areas in the cross-section.

Backwater depths for the existing structure are based on a comparison between the existing structure and natural channel water surface profiles.

### 10.10.2 Natural Channel

The water surface profile computed with the natural channel hydraulic model provides the baseline of comparison for the hydraulic performance of both the existing (if present) and proposed structures. The natural channel model is the same for both structures, and it represents the hydraulic conditions (i.e. highwater elevations, flow velocities, flow distributions, etc) at the site without the roadway, roadway embankment, or structure.

### 10.10.3 Proposed Structures

The proposed structure or alternative structure geometries are included in these hydraulic models. The structural designer can often list and describe the possible alternatives for these analyses. In absence of this information, it is customary to report data on spillthrough abutment and vertical abutment bridge alternatives. Throughout the modeling process it is important to verify that the modeled structure is a viable option. As an example, a hydraulic analysis of a single-span bridge is of little use if a multi-span structure is needed.

Special care should be used when modeling crossings where pressure flow or roadway overtopping occurs. The hydraulic performance of these sites are often controlled by features such as the bottom of beam elevations, the bridge deck profile, the roadway profile, and the flow resistance of the guardrails, curbs, and road surface. Guidance on the hydraulic modeling of these sites is included in Section 10.3 and the user's manual for the analysis program.

The proposed structure(s) should be modeled for both the post-construction conditions and future conditions that may occur during the structure design life. An example is a pavement overlay that blocks the roadway overtopping flow. In this case the structure should be analyzed with and without the overlay in place. The highest upstream water surface elevations and velocities through the bridge opening may occur after the road grade is overlaid. Another example is the case where the roadway grade may be raised in the future to eliminate overtopping. It may be necessary to construct a longer bridge to accommodate a future grade raise.

Backwater depths for the proposed structure(s) are based on a comparison between the proposed structure(s) and natural channel water surface profiles.

## 10.11 Scour and Erosion Protection

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Bridges in riverine and marine environments are often exposed to the scouring action of flowing water and the erosive action of waves on standing water. As a result, scour and erosion countermeasures are used to protect the structure from these destructive forces. In this section the term "scour protection" applies to countermeasures against both erosion and scour damage.

This section discusses countermeasures to protect bridges and abutments from scour caused by river and stream discharges and flows in estuaries due to tidal fluctuations. These countermeasures can also be used on inland waterways where flow velocities are low and wave action is the dominant cause of erosion. This could occur on deep bodies of water such as tidal estuaries, bays, lakes, and large rivers. In these cases, it is recommended that the revetment be designed to resist wave action and checked to verify that it will withstand the forces caused by flowing water. Methods of sizing riprap to resist wave damage are presented in **Chapter 15**.

Countermeasures to protect structures from ocean wave action are beyond the scope of this manual. Coastal engineering design methods should be used. Specialized publications on coastal engineering are available from several sources. The United States Corps of Engineers "Coastal Engineering Manual" is a widely used reference. The design method for coastal protection on an ODOT structure should be discussed with the Region Technical Center hydraulics staff before it is used in design. ODOT must verify that it is an acceptable procedure for the application.

Erosion damage to spillthrough abutments can be caused by roadway or bridge deck runoff flowing down the fill slope. This damage should be prevented, and methods to do this are presented in **Chapter 13**.

### 10.11.1 Extent of Scour Protection

The scour protection should prevent damage to the structure or supporting embankments from streamflow and wave action during floods of lesser or equal magnitude to the design event. The protection should survive the check flood with minimal damage to the structure or supporting embankment. Design and check floods are discussed in Section 10.8.

The protection should last throughout the design life of the structure without major maintenance. Structure design life is typically between 75 and 120 years. The structural designer can provide this information.

Scour protection with a reduced design life can be used in some circumstances. This should be clearly stated in the Hydraulic Report if it is the case. The frequency, type, and extent of anticipated repair or replacement should be described. Access requirements to repair or replace the scour protection should be described. This includes physical access, easements, and possibly added right-of-way. Responsible parties should be aware of, and agree to, the anticipated repairs or replacement.

### 10.11.2 Scour or Erosion Protection Countermeasure Preference and Selection

Almost all bridges are protected by one or more countermeasures. Countermeasures for each abutment and pier should be analyzed and preferred methods used if they will provide the required protection. As an example, riprap revetment may be the only practical protection for an abutment

located on an eroding bank at the outside of a river bend. Set back without revetment may be the practical and preferred scour protection for the other abutment located behind the inside edge of the bend. The following list includes common scour protection methods in the general order of preference.

Avoiding areas subject to scour. The structure and supporting abutments are located away from scour prone areas, in the horizontal and/or vertical directions. Set back abutments, deep foundations, or retaining walls are often used to avoid scour prone areas.

**Keying critical foundation members into scour and erosion resistant materials.** The most commonly used methods are keying spread footings, drilling shafts, or driving pilings into erosion resistant materials. These methods cannot be used for spillthrough abutment embankments.

**Biotechnical protection.** These methods include planted embankments, vegetated biodegradable geogrids, live cribwalls, willow fascines, etc. These methods rely solely on vegetation for erosion protection. Many of these methods are discussed in **Chapter 15**. Biotechnical protection may be a suitable alternative for less scour critical areas with suitable growing conditions where it can be periodically maintained and replaced.

Biologically enhanced revetment. This includes riprap, concrete cribs, and concrete jacks planted with vegetation; articulated concrete block mats with vegetal cover; riprap with rootwads; and boulder toes. A common characteristic of enhanced revetment is vegetation backed or supported by an erosion resistant and durable layer. It does not depend solely on the vegetation to provide adequate scour protection.

**Revetment.** The most commonly used revetment material is loose rock riprap. Other revetment types, such as concrete cribs or jacks are also used. Revetment is the most common protection in scour critical areas where other more preferred methods are not practical.

**Relocating the waterway.** This includes relocating the channel bank or banks, or use of spur fields, barbs, or rootwads in riprap to move the waterway away from the structure. This is often the most practical method for protecting existing structures from scour damage.

These factors should be considered when selecting a scour protection method. The most preferred method should be used unless it is impractical due to one or more factors. These factors cannot be rated in importance. Any or all may be critical to select a scour protection type.

The factors are as follows.

**Public safety.** The consequences of scour protection failure should be considered. This includes the danger to the public during a failure, both to the traveling public and to people in the vicinity of the failure.

**Expense.** Structure repair or replacement can be very costly. There is the direct cost to the agency of the repair or replacement, and the cost to the public due to the inconvenience of interrupted commerce and travel. There is also the cost of liability from damage claims.

**Ease of repair or replacement if the scour protection is damaged or fails.** Many forms of scour protection are extremely difficult to repair or replace after the bridge deck is placed or after construction access roads are removed.

**Environmental impacts of failure.** Failure of scour protection can cause considerable environmental damage. Typical damage is accelerated embankment erosion and streambed contamination from embankment soils. This usually occurs during the winter when sensitive fish species are in the stream. Bridges often carry utilities across waterways. A critical concern can be a spill from a petroleum pipeline, gas pipeline, sewage pipeline, or other conduit that could rupture if the structure collapses.

**Road closure.** Scour protection failure often results in a road closure. This could be a prolonged closure if a complicated structural repair or replacement is needed. This is especially critical in Oregon. The largest floods in the state are almost always regional events that affect most of the bridges in the vicinity. Adequate scour protection of structures on critical highways is especially important during these events.

**Certainty of design.** Hydraulic conditions are often difficult to predict with certainty. Scour protection methods with extra strength and durability are often desired in applications where future conditions are uncertain.

### 10.11.3 Definition of Structure

The definition of a structure, for scour protection design is:

- the structure itself, including end bents, interior bents, end panels, and
- the abutment fills as needed to prevent structural damage.

In some cases, additional facilities are located away from the bridge to prevent damage to the structure from scour or erosion. These items should also be designed to the same standards as the structural protection. Examples include:

- weirs or check dams arresting headcuts that would undermine the bridge foundation if they were allowed to progress through the waterway opening (headcuts are described in **Chapter 9**), or
- spurs or embankment protection that direct the water through the bridge opening if structural damage could occur if the items were not in place and functioning as designed (spurs and embankment protection are discussed in **Chapter 15**.)

### 10.11.4 Variable Tailwater Elevations

Scour protection at sites with variable tailwater elevations should be checked to verify it is adequate for the range of expected conditions. Typically the greatest design and check flood waterway opening velocities occur when the tailwater elevation is at its lowest. This usually determines the hydraulic shear forces the scour protection must resist. This would be the rock size, using riprap for an example. Conversely, the highest design and check flood elevations usually occur when the tailwater elevation is at its highest. This combination of discharge and tailwater often governs the elevation of the top of the revetment protection. This is discussed in more detail in Section 10.9.

## 10.12 Scour Protection Design

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A scour protection design is made for each application. Standard materials and designs are used where they provide adequate protection. As an example, riprap revetment would be of a standard ODOT classification unless circumstances require a special size or gradation. Several design methods for scour protection are discussed in the following subsections.

### 10.12.1 Scour Depths for Scour Protection Design

The primary means of scour protection is to have sufficient foundation burial into the supporting materials to be stable if scour occurs. The total scour depths discussed in Section 10.13 are used in the scour protection design. Shallower foundations can be used if they are protected by revetment adequate to resist damage from the check flood, or by sufficient set back distance.

Tops of pier footings are to be lower than the base flood scour elevations. Bottoms of pier footings are to be lower than the check flood scour elevations, or 6 feet below the thalweg, whichever is lower. Shallower footing burial is acceptable where the footings are keyed into solid erosion resistant rock.

Abutments that project into shallow and calm water do not need to be protected unless wave action is a concern or a new unprotected abutment fill has just been constructed. Shallow and calm water is assumed to be less than 3 feet deep with velocities less than 3 feet per second during the check flood.

Scour depths are to be calculated without considering nearby structures as scour protection, unless they will be intact and maintained throughout the structure design life. As an example, a new structure will be built immediately upstream from an existing bridge. The new structure should be designed so it is not damaged by scour if the existing structure is removed.

Additional information about foundations is in the ODOT Technical Services Foundations Manual and the Bridge Design and Drafting Manual.

### 10.12.2 Set Back Abutment Design

Set back abutments are placed in a location on the streambank or shore where they will not be undermined by scour or erosion, as shown in Section 10.3. The design is to assure the abutment is far enough from the waterway to not be undermined during the structure design life. In some cases, revetment, bioprotection, or river training structures are needed in addition to set back to prevent undermining.

#### 10.12.2.1 Set Back Abutment Preliminary Estimates

It is often necessary to estimate if set back abutments are feasible early in project development when design data is not available. Set back abutments without additional protection may be feasible if all of these conditions occur during events of lesser or equal magnitude to the scour protection check flood.

- Pressure flow does not occur due to water contacting the bridge superstructure, ice jammed against the superstructure, or debris lodged against the superstructure.
- Piers and abutments are located out of areas with flowing water, i.e. water depths less than 3 feet and flow velocities less than 3 feet per second.
- Erosive wave action does not occur.
- The waterway banks or shore are not expected to recede toward the abutment.
- It is unlikely future development within the floodplain during the structure design life will direct the river or stream toward the abutment.

Additional scour protection should be planned for abutments where all of the foregoing conditions do not occur. Set back abutments should not be selected based on preliminary estimates alone. An abutment design must be made to assure the preliminary estimates are valid.

#### 10.12.2.2 Set back Abutment Design Information

The following information is required for set back abutment design.

- The structure design life.
- The site hydraulics, including stages and velocities for floods up to and including the revetment check flood.
- The ordinary high water elevation.
- The proposed foundation type, size, and location.
- The superstructure bottom-of-beam elevation.
- The toe location, soil angle of repose, end erodibility for earthen embankments.

- The geological characteristics of the soils and rocks supporting the foundation or embankment in scour prone areas.
- The streambank or shore recession rate if lateral erosion or scour is a concern. The rate of streambed degradation if vertical scour or erosion is expected.

Often detailed information is not available and assumed conditions are used in the design. All assumed conditions should be biased towards the structure safety. For example, if rock or soil conditions are not known, it should be assumed the supporting material is erodible.

#### 10.12.2.3 Set Back Abutment Design Sequence

The following procedure is used to design the typical set back abutment. This procedure can also be used to evaluate and design urgent and emergency repairs to existing structures. Cross-sections of the structure and streambank are the primary tool for this analysis.

**Step 1 -** Locate the proposed foundations and end panels in relation to the scour prone areas on the cross-sections. The foundation is considered to be structural items such as footings, pilings, and retaining walls. Locate placed embankments supporting the foundation and end panels.

**Step 2 -** Locate the ordinary high water on the cross-section. Set back is the distance between the foundation and the ordinary high water, as shown in figures 10-11 and 10-12. Almost always the set back distances will be different at various locations on the structure.

**Step 3 -** Locate the active scour areas. These are locations where scour has occurred, is occurring, or has the potential to occur during the structure design life, such as:

- scour described in historical data such as maintenance files, bridge inspection reports, flood photos, and other records,
- scour observed during a site inspection,
- scour caused by the presence of the structure in the waterway (contraction, abutment, pier, and pressure flow scour), and
- potential scour due to changes in future conditions.

Changes in future conditions can be:

- removal of objects that restrain waterway movement such as foundations of obsolete structures,
- the potential for floodplain development that would direct the stream or river toward the structure,
- debris or ice collection on a proposed pier or superstructure,
- the change in sea level associated with climate change.

**Step 4 -** Determine the rate of streambank recession if it will occur. This rate is often based on observed scour. Stream movement is discussed in detail in the Federal Highway Administration Hydraulic Design Series Number 6, "River Engineering for Highway Encroachments." Procedures to predict movement in meandering streams are in the National Cooperative Highway Research Program Report 533 "Handbook for Predicting Stream Meander Migration."

**Step 5 -** The urgent repair process is triggered when observed scour is threatening the foundation or end panel, and the repair can wait until the in-water work period. Often it will take a year or two to design and permit an urgent repair. Draw the threshold for an urgent repair on the cross-sections. This is shown in Figure 10-15a.

**Step 6 -** The emergency repair process is triggered when scour is endangering or damaging the foundation or end panel, and the repair must be done immediately to save the structure. Draw the threshold for an emergency repair on the cross-sections. This is shown in Figure 10-15b.

**Step 7 -** Measure the distances between the scour prone areas and the threshold lines for urgent and emergency repairs. These distances are $D_{urgent}$ and $D_{emergency}$ shown in Figure 10-15.

**Step 8 -** Divide the distances calculated in the previous step by the rates of recession. This will estimate time intervals until an urgent or emergency repair is needed.

**Structures should not be designed with urgent or emergency repairs anticipated to occur within their design lives.** Any exceptions to this practice should be approved by the people responsible for the structure.

Increased set back distance or scour protection should be used if the setback distance is insufficient to protect an unshielded foundation or end panel, as shown in Figure 10-16. Scour protection methods discussed in this chapter can be used.

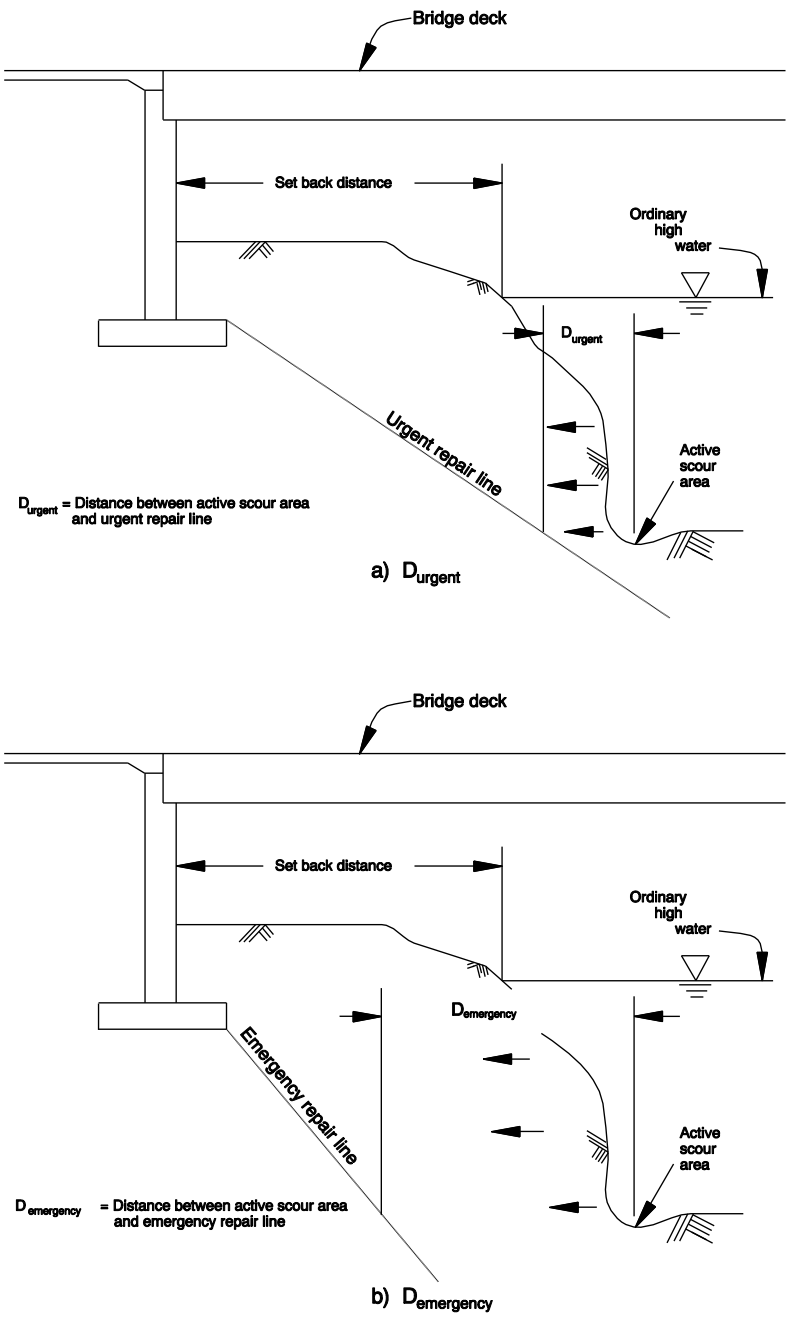



**Figure 10-15 $D_{urgent}$ and $D_{emergency}$**

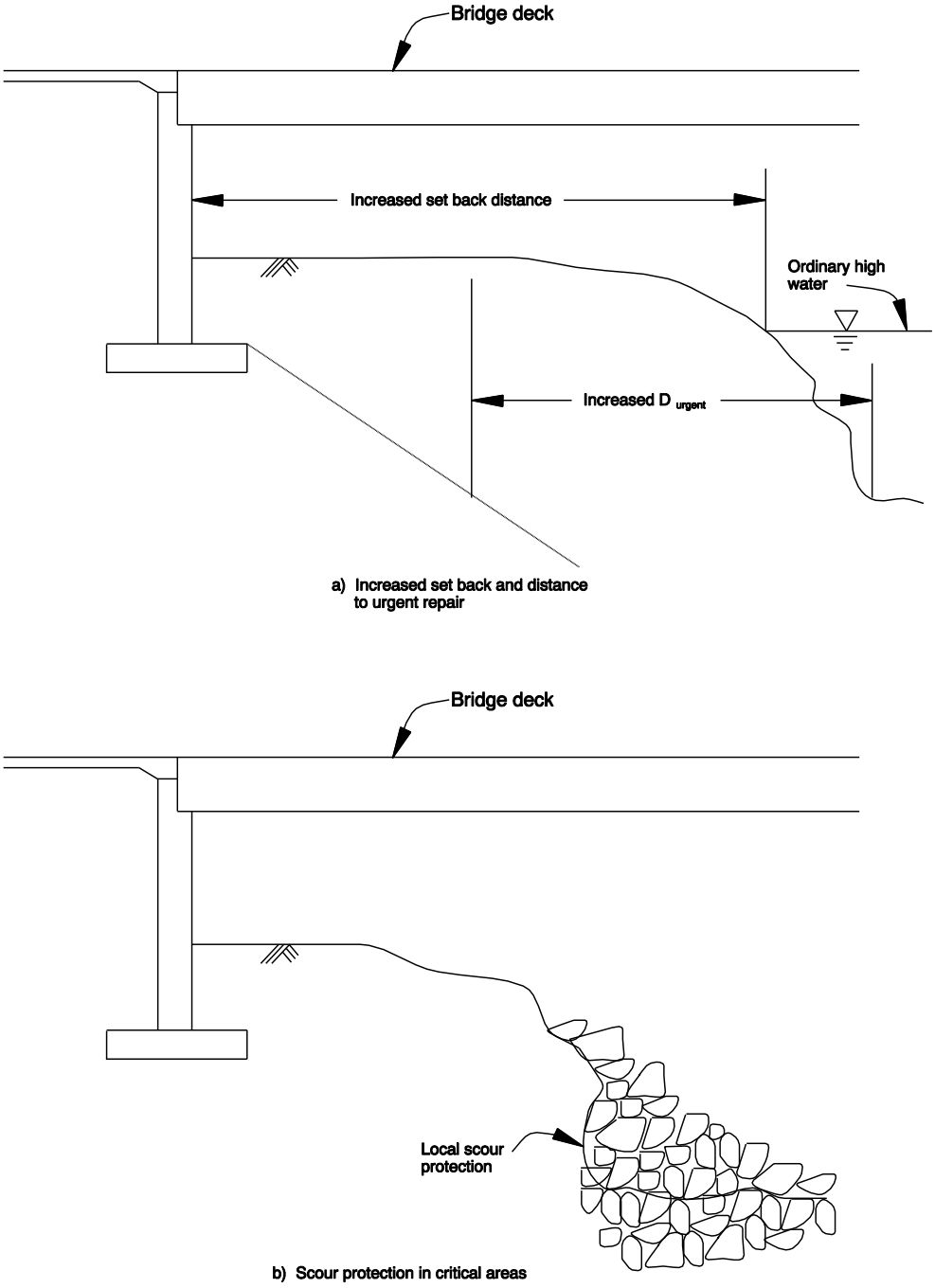



**Figure 10-16 Increased Setback and Scour Protection**

### 10.12.3 Biotechnical Protection and Habitat Enhancement

Crossing structures are often located in environmentally sensitive riparian areas. In most of these locations it is desired that the structure have minimal impact on the riparian corridor. This is often accomplished by locating the abutments on the streambanks away from the channel, minimizing the use of revetment in the waterway through use of biotechnical stabilization, and planting native vegetation to enhance the riparian environment. This subsection provides some broad guidelines on biotechnical issues. **Chapter 15** and personnel that specialize in the biological disciplines can provide more specific guidance.

#### 10.12.3.1 Growing Conditions

The root and stem structure provided by healthy plants is essential to biotechnical protection. In general, plants grow best in areas where they are protected from destructive forces, have adequate room to grow, enough sunlight and water, and sufficient soil of the right type. These requirements vary greatly among the plant species. The hydraulic forces caused by extreme winter flows, for example, could destroy a western red cedar and be harmless to a sandbar willow. Often a careful examination of the existing vegetation at the site and their locations can provide insight on the appropriate plants for biological enhancement. Qualified environmental personnel and landscape designers can often provide information on this subject.

These habitat requirements influence the choice of plants at a bridge site. Small bushes and similar plants are usually used within a few yards of the structure. Large plants such as trees should be used with caution. The root systems of many large trees can damage the structure foundation. The damage can be caused by the expansion of the roots as the tree grows, or the forces exerted on the foundation by the tree as it sways in the wind. In other instances, the root system of the plant cannot fully develop in the presence of the nearby structure, and the plant will have inadequate support to resist toppling in the wind. Locating large trees near the bridge deck is also avoided because of clearance problems, and plants of all types have limited success growing under the bridge deck because it blocks both light and precipitation.

#### 10.12.3.2 Site Use and Maintenance

In addition to environmental requirements of plant growth, the vegetation should also be compatible with site use and planned maintenance activities. The mature plants should not cause damage to the structure, utilities, or roadway. In addition, they should not obstruct the driver's vision or prevent maintenance access to the site. Landscape maintenance in most rural and many undeveloped urban locations is limited. Watering, for example, is only practical in landscaped areas with irrigation systems. Another example is pruning. The size and shape of the mature plant should be considered so that periodic trimming or pruning is minimal or not needed.

Local roadway, landscape, and bridge maintenance personnel should be contacted for input before the plants and planting locations are selected, and areas where plants are preferred and discouraged should be discussed. In these discussions, it is important to address that maintenance will be provided after construction.

#### 10.12.3.3 Environmental Concerns

Environmental concerns and agreements can often influence plant choice. Himalayan blackberry and scotch broom are examples of plants that are not specified for riparian enhancements because of environmental concerns about exotic (non-native) species. Environmental personnel should be contacted for input before plants and their locations are selected.

#### 10.12.3.4 Hydraulic Concerns

The presence of vegetation in the floodplain can greatly influence the hydraulic characteristics of the waterway, and these effects are usually the greatest for smaller structures. As a result, the hydraulic aspects of vegetation should be considered before the plants and their locations are selected. These effects should also be considered when reviewing the planting plans developed by others. In both cases, the roughness effects of the vegetation should be included in the hydraulic model of the site, and it should be verified that the hydraulic performance will be satisfactory with the vegetation mature and in place.

In general, plants increase the waterway roughness ("n" value). This can have two affects on hydraulic characteristics, it increases the depth of flow, and it can change the distribution of flow velocity in the stream cross-section. In both cases, the step-backwater software programs commonly used to analyze bridge crossings can also model the hydraulic effects of the plants. A typical procedure to analyze the hydraulic effects of vegetation in the waterway follows. It assumes a step-backwater analysis was made previously to model the bridge without plantings.

**Step 1 -** Add cross-sections, as necessary, to model the planted area. It is best to plant vegetation in ineffective flow areas.

**Step 2 -** Subdivide the waterway cross-sections at the planting locations, based on varying Manning's roughness coefficients. A good reference for the roughness coefficients is the Federal Highway Administration Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains, Report No. FHWA-TS-84-204 (FHWA: Washington D.C., 1984).

**Step 3 -** Execute the program.

**Step 4 -** Examine the water surface profiles to determine any changes in backwater elevation.

**Step 5 -** Verify that the backwater with the proposed vegetation is within the acceptable limit. If not, modify the planting plan as needed.

**Step 6 -** Examine the cross-section velocity distributions to determine any changes in the velocity or the distribution of velocity in the channel.

**Step 7 -** Verify that any changes in the velocities or velocity distribution do not affect scour depths. If scour depths are affected, include the effects in the scour depth calculations.

**Step 8 -** Verify that the changes in velocities or the velocity distribution do not require changes in the revetment design. If the revetment design is affected, revise the design or modify the planting plan.

**Step 9 -** Verify that changes in the velocity distribution do not increase velocities in erosion prone areas such as channel banks. If increased erosion is predicted, determine if the erosion is tolerable. If it is not, protect the erodible area or modify the planting plan.

### 10.12.3.5 Choice of Biological Enhancement Methods

Plants should not be solely relied upon to provide bridge scour protection. It has been difficult to accurately predict the degree of protection plants provide. This does not mean plants cannot be used. Plants can provide biological enhancement of the bridge site, and they can also be an element in the scour and erosion protection. The enhanced protection should provide the same degree of protection required in Section 10.11. The enhanced protection should not depend on the plants in order to be effective. A backup system should be incorporated to provide adequate protection if the plants do not grow or if they die. Recent experience with biotechnical bank protection indicates that unforeseen damage from animals, insects, or diseases can decimate the plantings.

The most common method of biological enhancement in current use at bridge sites is to plant desirable trees and shrubs in suitable locations, and not to rely on them for scour and erosion protection. Often these plantings include willows in the riprap revetment. In most cases, bush size willows are used in areas where their roots will receive adequate moisture. Often other bushy plants are used higher on the embankments, such as red-osier dogwoods and snowberries. Bush alders of different varieties are also used in riparian habitats.

Willows and dogwoods are often planted as cuttings inserted into soil placed over and within the riprap. These cuttings are most successful when:

- they are cut and inserted during their dormant season (usually December and January),
- the lower end of the cutting is in an area with moist or saturated soil,
- 75 to 95 percent of the cutting length is buried, and
- the correct polarity is used. In other words, the top end of the cutting on the host plant is the top end of the installed cutting.

### 10.12.4 Standard Revetment Designs

Revetment protection is designed for each individual structure at each site, and the typical procedure is to modify a standard revetment design to fit the application. This section includes standard revetment designs for spillthrough abutments, vertical abutments, mechanically stabilized earth wall (MSE wall) abutments, and interior bents. Additional information about revetment, including gradation and properties, is discussed in **Chapter 15**. Biological enhancement of revetment is discussed in the previous subsection.

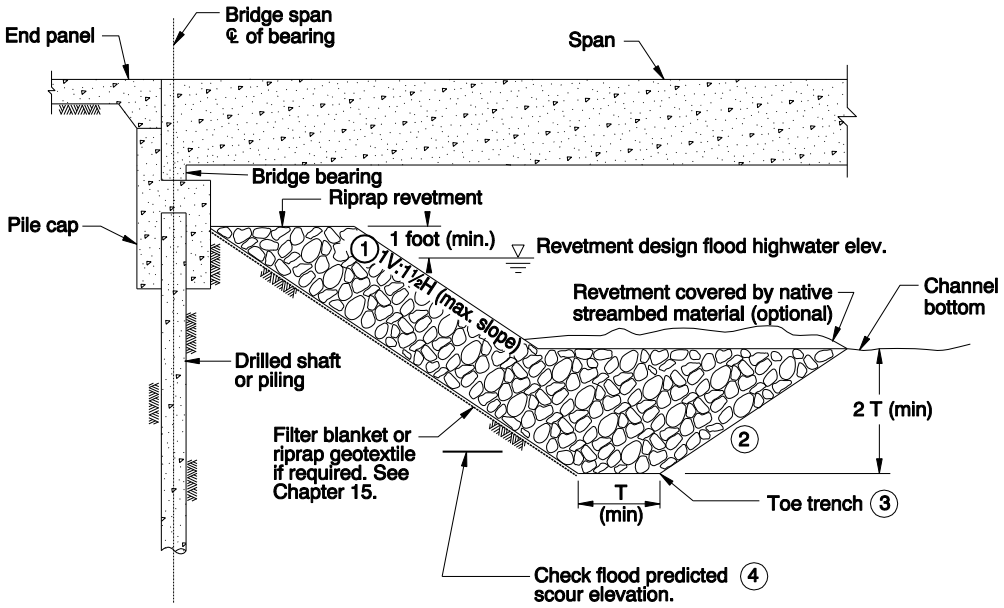
Revetment or equivalent protection should be used if there is flowing water against the abutment during events up to and including the revetment check flood. Flowing water is considered to be water more than 3 feet deep or having flow velocities in excess of 3 feet per second.

Estimated scour depths and elevations are an essential tool for revetment design. The scour types used in revetment design are discussed in Section 10.13.

#### 10.12.4.1 Spillthrough Abutment Revetment

The objective of this revetment is to protect the abutment fill from scour damage during events up to and including the revetment design flood. Typical cross-sections of spillthrough abutment revetment are shown in Figures 10-7 and 10-8, and details are shown in Figure 10-17. The rock size is determined by methods in **Chapter 15** using the highest velocity though the bridge opening. This velocity usually occurs at the downstream face. The maximum elevation of the revetment protection is determined by the highest water surface elevation in the bridge opening plus 1 foot, and this is often the energy grade line elevation at the upstream faces of the abutments. An exception occurs if water is ponded at the upstream faces of the abutments during the revetment design flood. In this case, the elevation of the energy grade line at the approach section should be used. Wave action on the ponded water should also be considered, if present, and guidelines are included in **Chapter 15** for sizing rock at inland locations. The rock should extend upward on the abutment face as needed to prevent wave damage.

The revetment should wrap around the sides of the bridge abutments and protect the fresh embankment slopes below the revetment design flood elevation. A toe trench may be needed on the upstream side of the bridge abutments if there is significant flow contraction. The revetment should extend along the embankments a distance sufficient to protect the bridge end panels. This distance can be approximated by the guidelines in Subsection 10.3.8. An example of an end panel undermined by wave action from a flooding river is shown in Figure 10-18. This end panel was supported by stone embankment material with little scour resistance. A layer of revetment would have prevented this damage.




| TABLE | | |
|---|---|---|
| Riprap Class | T (feet) | Filter Blanket or Riprap Geotextile |
| 50 | 1.0 | No |
| 100 | 1.5 | No |
| 200 | 2.0 | Yes |
| 700 | 3.0 | Yes |
| 2000 | 4.0 | Yes |

Notes:

1. The maximum slope of an abutment with riprap protection is 1 unit vertical to 1½ units horizontal. Steeper abutment faces require special scour protection such as paved end slopes or rock filled gabion baskets.

2. The maximum slope of the inside of the toe trench is the angle of repose of the streambed material. A flatter slope may be needed in con-cohesive materials such as saturated sands or gravels. A 1 unit horizontal to 1 unit vertical slope can be used for quantity estimates if site specific information is not available.

3. The bottom of the riprap filled toe trench must be below the lower of:
   - the elevation of predicted scour during the check flood,
   - the elevation of the channel thalweg minus the toe trench depth.

4. Predicted total scour based on Subsection 10.13.7.

**Figure 10-17 Spillthrough Abutment Revetment Details**

**Figure 10-18 Undermined End Panel**

### 10.12.4.2 Pile or Shaft Supported Vertical Abutment

Vertical abutments can be supported by piling or drilled shafts. An essential part of this design is to extend the bottom of the pile cap down to an elevation lower than the bottom of the adjacent riprap filled toe trench and the check flood scour elevation, as shown in Figure 10-19. This extended cap often requires structural modifications to withstand the lateral earth pressure behind the abutment face.

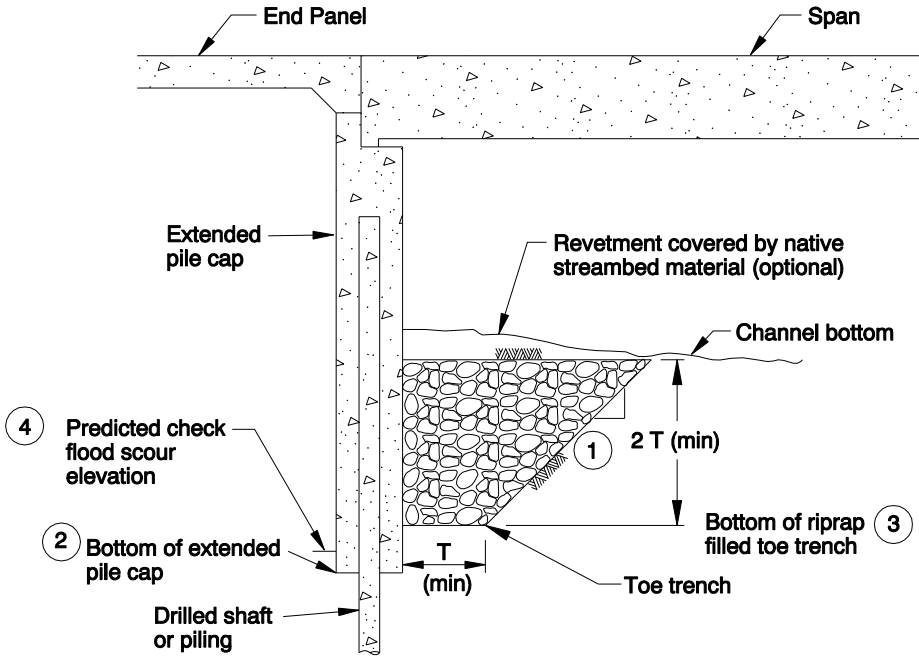
The embankment wrapping around the edges of the wingwalls should be protected from erosion if significant wave action occurs. Guidelines for sizing riprap to resist wave action are in **Chapter 15**. The riprap should extend upward to a high enough elevation to prevent wave damage.

#### 10.12.4.3 Footing Supported Vertical Abutment and MSE Wall Abutment Revetment

The primary objective of the revetment on these abutments is to protect the footing or bottom edge of the facing panel from scour damage during events up to and including the revetment check flood. Typical revetment cross-sections for footing supported vertical and MSE wall abutments are shown in Figures 10-20 and 10-21, respectively. Like spillthrough abutments, the rock size is determined by methods in **Chapter 15** using the highest velocity though the bridge opening, which is usually at the downstream face. The maximum elevation of the revetment protection is determined by the highest water surface elevation in the bridge opening, and this is often the energy grade line elevation at the upstream face of the abutment. An exception occurs if water is ponded at the upstream faces of the abutments during the revetment design flood. In this case, the elevation of the energy grade line at the approach section should be used.

The embankment wrapping around the edges of the wingwalls should be protected from erosion if significant flow velocity or wave action occurs. Guidelines for sizing riprap to resist wave action are in **Chapter 15**. The riprap should extend upward to a high enough elevation to prevent wave damage.

*Note: The MSE wall abutment is a unique design with limited use in Oregon. Almost all of the abutments have been constructed away from the water on the streambank with little or no in-water work. In addition, most of the bridge spans and supporting footings have been located above the elevation of the check flood. Caution should be used when considering MSE wall abutments for sites where the abutment will need to be built in the water or where floodwaters are expected to contact the footings or the bottom of the span.*




| TABLE | |
|---|---|
| Riprap Class | T (feet) |
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |

Notes:

1. The maximum slope of the inside of the toe trench is the angle of the repose of the streambed material. A flatter slope may be needed in non-cohesive materials such as saturated sands or gravels. A 1 unit horizontal to 1 unit vertical slope can be used for quantity estimates if site specific information is not available.

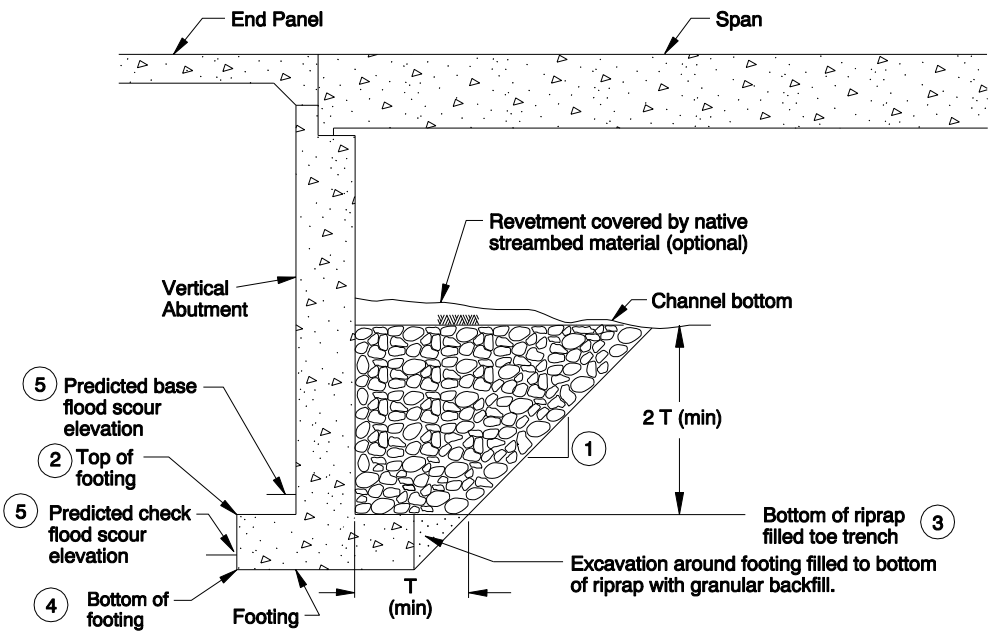
2. The bottom of the extended pile cap must be below the bottom of the riprap filled toe trench.

3. The bottom of the riprap filled toe trench must be below the lower of:
   - the elevation of predicted scour during the check flood, or
   - the elevation of the channel thalweg minus the toe trench depth.

   Higher elevations may be permitted or the revetment eliminated if the bottom of the extended pile cap is keyed into solid rock.

4. Predicted total scour based on Subsection 10.13.7.

**Figure 10-19 Pile or Shaft Supported Vertical Abutment Revetment Details**




| TABLE | |
|---|---|
| Riprap Class | T (feet) |
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |

Notes:

1. The maximum slope of the inside of the toe trench is the angle of repose of the streambed material. A flatter slope may be needed in non-cohesive materials such as saturated sands or gravels. A 1 unit horizontal to 1 unit vertical slope can be used for quantity estimates if site specific information is not available.

2. The top of the footing must be below the lower of, the bottom of the riprap filled toe trench, or the base flood scour elevation.

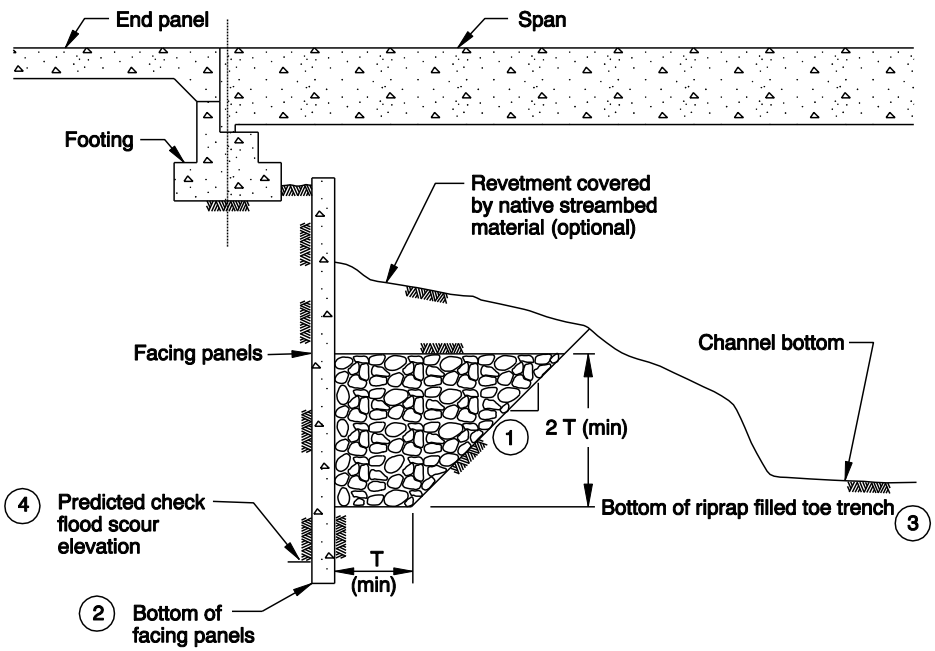
3. The bottom of the riprap filled toe trench must be below the lower of:
   - the elevation of predicted scour during the base (100-year) flood, or
   - the elevation of the channel thalweg minus the toe trench depth.

   Higher elevations may be permitted or the revetment eliminated, if the footings are keyed into non-erodible rock.

4. The bottom of the footing must be below the elevation of predicted scour during the check flood or 6 feet below the elevation of the thalweg, whichever is lower.

5. Predicted total scour based on Subsection 10.13.7.

**Figure 10-20 Footing Supported Vertical Abutment Revetment Details**




| TABLE | |
|---|---|
| Riprap Class | T (feet) |
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |

1. The maximum slope of the inside of the toe trench is the angle of repose of the streambed material. A flatter slope may be needed in non-cohesive materials such as saturated sands or gravels. A 1 unit horizontal to 1 unit vertical slope can be used for quantity estimates if site specific information is not available.

2. The bottoms of the facing panels must be below the elevation of the bottom of the riprap fill toe trench.

3. The bottom of the riprap filled toe trench must be below the lower of:
   - the elevation of predicted scour during the check flood, or
   - the elevation of the channel thalweg minus the toe trench depth.

   Higher elevations may be permitted, or the revetment eliminated, if the bottoms of the facing panels are keyed into non-erodible rock.

4. Predicted total scour based on Subsection 10.13.7.

**Figure 10-21 MSE Wall Abutment Revetment Details**

#### 10.12.4.4 Interior Bent (Pier) Revetment

The interior bent of a bridge, often called a pier, is supported by the underlaying soil or rock, and scour of these underlaying materials will reduce the strength of the bent. The desired practice is to embed the foundation sufficiently deep to have adequate strength after estimated scour occurs. In some cases, this may not be practical, and it may be necessary to protect the foundation with riprap, articulated block mats, or other means. This is most often done when protecting existing structures. This should not be done for new structures unless approved by the ODOT Region Technical Center hydraulics staff. Riprap, in most cases, is considered to be temporary scour protection for piers.

Footing bases on new bridges are designed to be below the estimated check flood scour elevation, as described in Subsection 10.12.1. Debris, if it is anticipated to be present, should be considered in the scour calculations. Many designers also include revetment around the pier as an additional countermeasure against unanticipated scour. A typical detail for pier revetment is shown in Figure 10-22. Pier revetment size is based on the approach flow velocity upstream from the pier. Methods to calculate pier riprap size are in **Chapter 15**.

## 10.13 Scour

Scour at bridges is a complex phenomenon, and it often has multiple and interrelated causes. In general, there are two categories of scour. One category is general scour, and this is a change in elevation over most or all of the stream bottom. The other category is local scour, and it occurs at specific locations in the waterway opening. The major causes of scour are:

- long-term changes in channel profile or location, typically called "aggradation" (raising), "degradation" (lowering), or lateral shifting "plan form changes" of the channel bottom,
- general scour due to the contraction of flow as the discharge passes through the bridge opening, called "contraction scour,"
- local scour adjacent to the faces of the abutments, called "abutment scour,"
- local scour around the interior bents, called "pier scour," and
- other types of pier scour, such as the scour caused by a buildup of debris on a nearby part of the structure.

The primary purpose for scour calculations is bridge foundation and scour protection design. The scour types to be calculated are discussed in section 10.12. Each of the listed scour types is described in more detail in the remainder of this section. Bridge scour and calculation methods are described in detail in the Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 18 Evaluating Scour at Bridges.

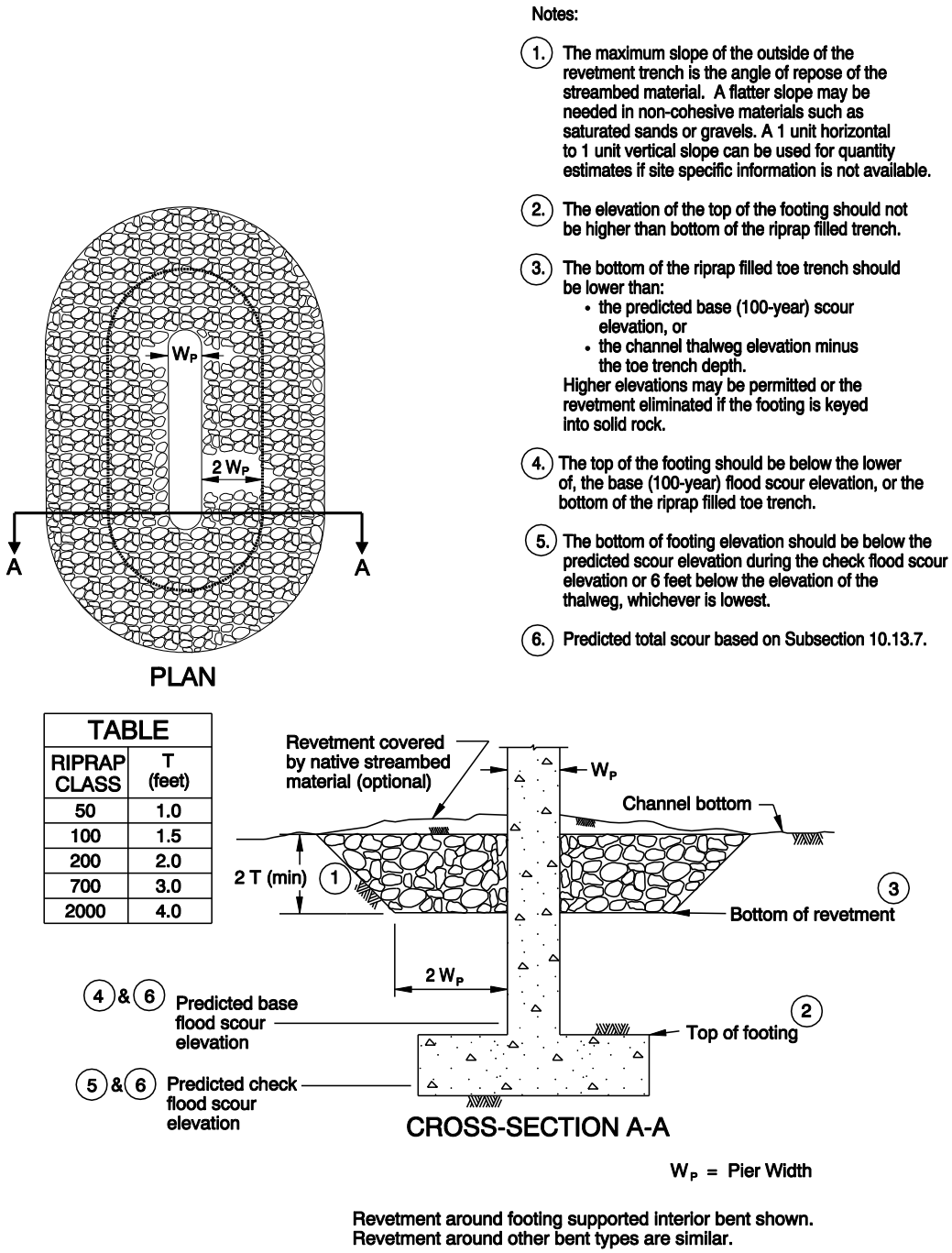



**Figure 10-22 Interior Bent Revetment Details**

### 10.13.1 Discharges and Tailwater Effects Used in Scour Analyses

The scour elevations are reported for the scour design and check discharges. The scour elevations are calculated for the combinations of discharges and tailwater elevations that create the deepest scour depths in the bridge opening. If changes to the hydrology, structure, waterway, roadway, or other items are expected within the design life of the structure that may cause deeper scour depths, scour elevations based on these deeper depths should be reported. Guidance on selecting discharges and tailwater effects are included in Sections 10.8 and 10.9, respectively.

### 10.13.2 Aggradation and Degradation and Plan Form Changes

Aggradation and degradation are long-term channel profile changes caused by the buildup or removal, respectively, of bed material on the channel bottom. These changes are usually considered to be permanent, and they are often caused by changes in the stream discharge, the amount or size of the moving bed load, or a change in the energy grade line profile. These channel profile changes are especially critical in the design of fish passage culverts, and they are discussed in **Chapter 9**. A useful reference on this subject is the FHWA Hydraulic Engineering Circular No. 20 Stream Stability at Highway Structures.

Plan form changes are changes in location of the channel banks in the horizontal plane. Plan form changes of concern to hydraulic designers are usually one or more of two types, meander migration or channel widening.

Meander migration is a shifting in the lateral and downstream direction the lowest point of the channel cross-section, often called the channel "thalweg." This lateral instability can occur on streams that have meandering paths. It can also occur on streams with straight channels, such as previously meandering streams that have been straightened by channel changes.

Channel widening occurs when the banks scour away and the channel widens. This widening can be caused by many sources, such as flow from a large flood, an increase in the sediment discharge, livestock or land use practices eliminating bank vegetation, or many other sources.
Plan form changes are difficult to predict on a long-term basis. As a result, in the hydraulic design it should be assumed that the channel thalweg or banks can shift laterally unless they are physically constrained.

Considerable survey data is available for estimating changes in waterway cross-section and profile at ODOT bridges and local agency bridges that are included in the ODOT bridge inspection program. This information is helpful when estimating aggradation or degradation. Regardless of the source of the survey data, all elevations should be converted to a common datum before they are compared. Survey data is available from the following sources:

- waterway profiles and cross-sections are almost always made before a bridge installation or replacement, and this data is in the bridge construction files (ODOT Bridge Section)

or the hydraulic study files (ODOT Geo-Environmental Section's Engineering and Asset Management Unit),
- waterway profiles and cross-sections are often made before a detailed scour study, and this data is in the scour reports (ODOT Bridge Section), and
- waterway cross-sections are made periodically during bridge inspections, and this data is in the bridge inspection reports (ODOT Bridge Section or ODOT Region Bridge Inspectors).

Aggradation should be estimated at bridge sites where it can be predicted, and it should not be included in the predicted total scour. The estimated channel elevation after aggradation should be mentioned separately in the report with a cautionary note to the structural designer. The bridge waterway opening may need to be enlarged to accommodate the anticipated aggradation.

Degradation, if it can be predicted, should be included as a component of the total predicted scour.

### 10.13.3 Contraction Scour

Contraction scour is general scour caused by increased flow velocities within the bridge opening in comparison to the slower velocities in the upstream and downstream waterway. Contraction scour can occur in the bridge opening due to the constriction caused by the bridge abutments and/or internal bents. This type of scour can also be caused by contraction due to constrictions in the natural channel. This is not uncommon because bridges are often placed across natural constrictions.

Equations in HEC-18 are used to calculate contraction scour in most applications, and detailed instructions on their use is included in the publication. One equation, the clearwater scour equation, is used where there is no live-bed movement, such as:

- the transport of bed material from upstream of the contraction is small in quantity or composed of fine material that washes through the contraction in suspension, or
- coarse sediments are present that may armor the channel bottom and limit the depth of live-bed contraction scour.

The other equation, the live-bed scour equation, is used where substantial amounts of bed material are washed into and out of the contracted area during floods, and the clearwater equation is not applicable.

Both equations should be used to determine the potential contraction scour and the lowest elevation reported at sites where the scour type cannot be predicted with certainty, or locations where both types of scour may occur.

ODOT practice is to assume the channel thalweg can change location and the predicted contraction scour depth can occur anywhere in the bridge opening. Exceptions to this general rule

occur at sites where the waterway has a fixed alignment and movement is not possible. Examples are canals, channels incised in solid rock, channels restrained by spur dikes or levees, etc.

### 10.13.4 Abutment Scour

Abutment scour is local scour that occurs at the faces of abutments that project into the waterway or floodplain. The obstruction causes flow vortexes to form at the toe of the abutment, and this turbulent flow scours away the underlaying bed material. At present, equations to predict abutment scour are mainly based on laboratory data and they tend to predict conservative scour depths. In other words, it is likely the actual abutment scour will be less than the predicted value, and unlikely the abutment scour will be greater than the prediction.

ODOT recommended practice is to protect the toe of the abutment with revetment in lieu of including abutment scour in the predicted scour elevation. An exception occurs when revetment protection is omitted from the face of the abutment and the toe of the abutment is not solidly keyed into non-erodible rock. In this case, abutment scour is calculated and included in the predicted total scour elevation.

### 10.13.5 Pier Scour

Pier scour is a form of local scour that occurs around interior bents that are exposed to flow. Methods in HEC-18 are recommended to calculate pier scour. The calculation of pier scour should consider the effects of varying angles of flow attack, possible movement of the channel thalweg, and if present, the effects of pressure flow and debris accumulation.

ODOT practice is to assume the channel thalweg can change location within the bridge opening, and the varying thalweg locations can change the directions from which the flow approaches the pier. The flow direction can influence the predicted scour depth, and the maximum pier scour that results from the various possible flow directions should be reported. In addition, it is assumed that due to a moving thalweg, the deepest predicted pier scour depth can occur at any pier in the bridge opening. Exceptions to this general rule occur at sites where the waterway has a fixed alignment and movement is not possible. Examples are canals, channels incised in solid rock, channels restrained by spur dikes or levees, etc.

The pier scour depth can also be influenced by occurrence of pressure flow during a flood event. Pressure flow scour occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the elevation of the low chord of the bridge superstructure. Pressure flow causes additional pier scour when the water that collects at the upstream face of the bridge and plunges downward into the bridge opening. The additional pier scour due to pressure flow can be estimated by the procedures in HEC-18.

### 10.13.6 Pier Scour Due to Other Causes

Increased scour can occur at piers due to the collection of debris on the piers or superstructure, the presence of ice, or numerous other causes. Procedures in HEC-18 can be used to estimate pier scour due to debris. It is necessary to estimate the size of the debris buildup in order to use this procedure. Maintenance records, bridge inspection reports, photographs of floods, and recollections of witnesses can all be used to estimate the debris buildup during large floods.

### 10.13.7 Total Scour

Total scour elevation at a structure where the abutment toes are protected from scour, and the scour protection is designed to withstand the check flood:

- for piers, the thalweg elevation – pier scour – contraction scour – channel degradation, and
- for abutments, the thalweg elevation – contraction scour – channel degradation.

Total scour elevation at a structure where the abutment toes are not protected from scour, or the scour protection is inadequate to withstand the check flood:

- for piers, the thalweg elevation – pier scour – contraction scour – channel degradation,
- for piers within the abutment scour zone, or abutments having piers within the abutment scour zone, the thalweg elevation – abutment scour – pier scour – contraction scour – channel degradation,
- for abutments, the thalweg elevation – abutment scour - contraction scour – channel degradation, and
- for abutments having piers within the abutment scour zone, the thalweg elevation – abutment scour – pier scour – contraction scour – channel degradation.

The abutment scour zone is an anticipated scour cavity adjacent to the abutment toe. It extends downward to the total scour elevation. The top width of the cavity is twice the total scour depth. Pressure flow and debris components should be included in the scour calculations if they occur.

Not all forms of scour may be present at all crossings. Scour depths may be limited by the presence of non-erodible rock. The erodibility of the rock can be determined by the foundation or geotechnical designer.

## 10.14 Temporary Crossings

Temporary crossings such as construction bridges and detour bridges are often used during the installation, replacement, and rehabilitation of structures. These bridges are often designed by the contractors or subcontractors who do the construction of the permanent crossing. The locations and lengths of these structures are often governed by available right-of-way or construction easements, environmental considerations, navigational requirements, as well as hydraulic needs.

The typical design information provided for a temporary structure by a hydraulic designer includes the predicted low flow discharge during the construction season, the predicted 5-year flood discharge for structures in place through the flood season, and flood elevations. Instructions for calculating the discharges are included in **Chapter 7** and flood elevations are often determined from the hydraulic model for the permanent structure.

Temporary structures across floodways subject to Federal Emergency Management Agency requirements need special consideration. These temporary structures must meet additional hydraulic requirements if they are in place across the floodway between November 1 and May 31. The Region Technical Center hydraulics staff should be contacted for assistance as soon as possible during the design process if a temporary structure will be needed across a floodway during the aforementioned period.

# CHAPTER 11

# ENERGY DISSIPATORS

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## 11.1 Introduction

This chapter provides information for the planning and design of energy dissipators at the outlets of open-channels or closed conduits such as culverts or storm sewers. The methodology is intended for those with an understanding of basic hydrologic and hydraulic methods and some experience in the design of hydraulic structures. Most of this chapter is based on information and methodology in the Federal Highway Administration (FHWA) Hydraulic Engineering Circular Number 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels" (HEC-14).

Flowing water, due to its movement, possesses kinetic energy. This energy is often expressed in hydraulic engineering as "velocity head" and it is related to the velocity of the fluid as follows:

$$H_{velocity} = \frac{V^2}{2g}$$ **(Equation 11-1)**

Where:

$H_{velocity}$ = Velocity head in feet
V = flow velocity in feet per second
g = the acceleration of gravity (32.2 feet per second squared, or $ft/s^2$)

As shown by the preceding equation, the kinetic energy of water is highly dependent on its velocity, and the faster the water travels the greater its velocity head and kinetic energy.

The kinetic energy of a flow is directly related to its ability to move particles, and the faster the flow, the larger the particles that it can move. This ability to move material gives water the means to erode and scour channels and conduits. As a result, a durable hydraulic conveyance system should have flow velocities that do not exceed the maximum velocities the conduit and channel lining materials can withstand.

The ability of a lining material to withstand scour and erosion damage varies considerably. The hard materials used to line conduits and artificial channels such as metal, concrete, steel, plastic, wood and rock can withstand fairly high velocities, as discussed in **Chapter 5**. Conversely, the softer materials that line typical natural channels like soil and vegetation can withstand much slower velocities, as discussed in **Chapter 8**.

Energy dissipators are used to reduce the velocity, and consequently, the erosion potential of flowing water. Their most common use is to reduce the outlet flow velocities from conduits that discharge onto embankments, into natural or unlined channels, or into drainage swales. Occasionally they are also used to slow the flow out of lined open channels such as spillways, ditches, and other hydraulic structures.

## 11.2 Policy and Practice

General policies of the Federal government and ODOT pertaining to hydraulic design are discussed in **Chapter 3** of this manual. General Geo-Environmental Section practice specific to energy dissipator selection and design include:

- coordination with other Federal, state, and local agencies concerned with water resources planning will have a high priority,
- safety of the general public and protection of property is an important consideration,
- the passage of ice or debris, if present, shall be considered,
- the frequency and type of maintenance and allowances for the access of maintenance equipment and personnel shall be considered,
- sediment deposition and scour shall be considered and minimized when possible,
- environmental impacts such as fish passage and disturbance of fish habitat, wetlands, and riparian areas shall be considered,
- the hydraulic performance of the dissipator shall consider the class of the roadway, consequences of scour and erosion, economics, and local site conditions.
- The dissipator should reduce the velocity to or below that which exists in the natural channel

## 11.3 Sources of Information

The type, source, and amount of data needed for a dissipator design will vary depending on the size and complexity of the dissipator. Large or complex dissipators typically require data similar to a large culvert location survey and the needed information is usually supplied with the culvert location data. If the dissipator is to be used on a storm sewer outlet or other application, a special survey is often needed to provide the needed information. Large or complex dissipator designs typically require:

- a comprehensive contour map or terrain model of the site providing the centerlines of the upstream and downstream conduits and/or channels, the roadway, the locations of utilities and developed property such as buildings, the edges of agency right-of-way and drainage easements,

- cross-sections and profiles of the conduits and/or channels and the roadway,
- description of the streambed or other items to be protected, including soil and geological information,
- photographs of the site,
- navigation, environmental, and fish passage requirements, and
- known history of the site, including recollections of past floods and damages, damage complaints, and names and phone numbers of involved agencies and neighboring landowners.

Simple or small dissipators can often be designed using the roadway location data. The dissipator design requires calculation of the site hydrology/hydraulics and additional information is often needed, such as the watershed area and the type of land cover (forest, suburban lots, etc). An inspection of the site and interviews with maintenance personnel can often be a valuable supplement to the location survey.

## 11.4 Results of Dissipator Design Studies

A dissipator at the end of a conduit is usually designed at the same time as the facility that provides the incoming flow. In these cases the information required from the dissipator design is listed in the guidelines in **Chapter 4** of this manual for culverts or storm sewers. Occasionally a dissipator is designed separately from the upstream channel or conduit or the dissipator is designed to handle flow from a source other than a culvert or a storm sewer. In this case the following information is needed:

- location of the dissipator including highway, milepoint, etc,
- design flow and recurrence interval,
- dimensions of the dissipator, including a sketch,
- recommended materials,
- details of channel changes, if needed,
- descriptions of special design considerations including drawings as needed,
- fish passage and other environmental data as needed, and

- descriptions of maintenance needs if they will be atypical or unusually frequent.

## 11.5 Types and Selection of Energy Dissipators

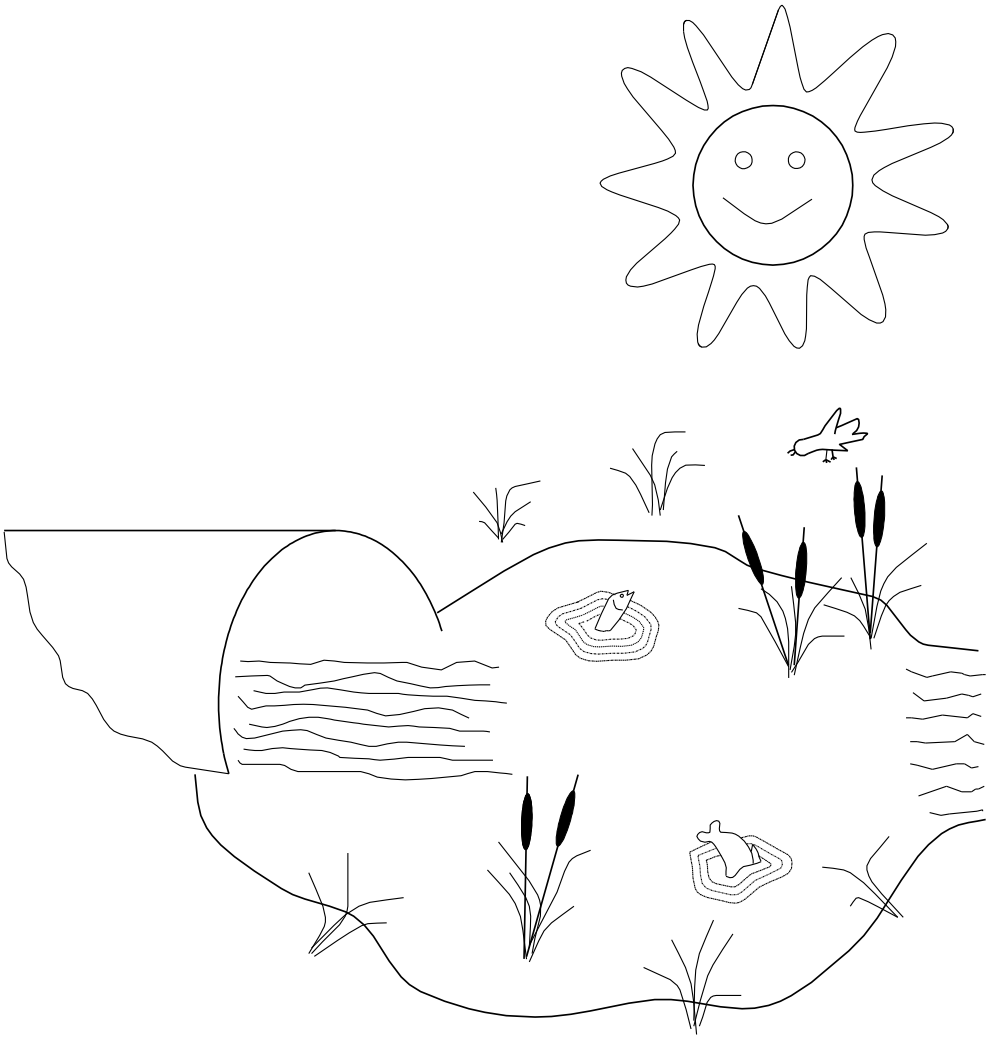
There are many kinds of energy dissipators, and most are either internal or external. Internal dissipators are located within the upstream conduit and they slow the flow before it leaves the conduit. External dissipators are located at the outlet end of conduits or channels. Some of the more common types are listed in this section with guidance on their selection.

**Unlined Pools -** These external dissipators consist of water filled pools, as shown in Figure 11-1. These pools can be located at the outlets of conduits or channels. The sides and bottom of the pool are not completely covered by revetment, and hence the name "unlined pool." Energy is dissipated as turbulence when the higher velocity flow from the upstream conduit or channel collides with the water that is ponded in the pool.

The pool can be formed by the scouring forces from the higher velocity flow that exits the upstream conduit or channel. This type of pool is commonly called a "natural scour hole," and its size can be predicted by methods in Section 11.8 of this chapter and its design is discussed in Section 11.9. Considerable sediment can be washed downstream when this pool forms, and the effects of this material on the downstream channel should be considered. The banks of these pools are often composed of the soils and rocks that underlie the site with a covering of the vegetation that naturally establishes itself over time.

The natural scour hole can be excavated by construction equipment and personnel instead of natural forces. This excavation reduces the amount of sediment that is carried downstream. At a minimum, these pools usually have the shape and size of a naturally occurring scour hole. The banks can be planted with vegetation and sometimes revetment is used on sections of the bank to protect critical areas from erosion.

These basins can be designed to provide fish habitat and passage. Typically an excavated scour hole is used in this application because there is less chance that a mound around the exterior of the pool may form that can obstruct fish passage.



Note: The designer must make provisions to insure that the stream and the pool do not degrade to a point where fish passage is blocked. This may be accomplished by providing a stable liner for the outlet channel or the pool

**Figure 11-1 Unlined Pool**

*Note: An unlined pool in many circumstances can be considered to be a wetland by resource management agencies and regulations may prevent or limit future modifications or maintenance. These factors should be considered before this type of energy dissipator is selected.*

Unlined pools are often used where:

- fish passage and/or wetland habitat are desired,
- the site has enough room to accommodate the pool and it is located within agency right-of-way or drainage easements,
- the outlet of the upstream channel or conduit, the highway facilities, and neighboring property can be protected from undermining or damage due to scour or erosion, and
- the pool is not a nuisance or safety hazard to the highway users or to the general public.

**Internal Dissipators -** This dissipator is used inside a closed conduit to slow the flow to an acceptable velocity before it enters the downstream channel. One type, roughness rings, uses concentric rings inside the conduit to slow the flow, and energy losses are created by the turbulence at the interface where the water contacts the rings. An example is shown in Figure 11-2a. Another type of internal dissipator is the "pipe tee" shown in Figure 11-2b. This device dissipates hydraulic energy by splitting the flow and abruptly changing its direction of travel. Regardless of the type of internal dissipator, riprap pads are usually placed at the outlets to further spread the flow and reduce the velocity to prevent scour of the channel or embankment. The design of these dissipators is discussed in Section 11.10.

Internal dissipators are often used where:

- their cost is justified by the ability to use a smaller, less costly, or more environmentally friendly type of external dissipator,
- ice or debris passage is not a problem,
- moderate velocity reduction is needed, and
- fish passage is not required.
- right-of-way is limited

**Riprap Pad -** This is the most common type of external energy dissipator for smaller conduits or channels. When this dissipator is used the flow crosses over the riprap before it enters the downstream channel. The roughness of the riprap creates turbulence in the flow and considerable hydraulic energy is dissipated. An example of a pad downstream from a conduit is

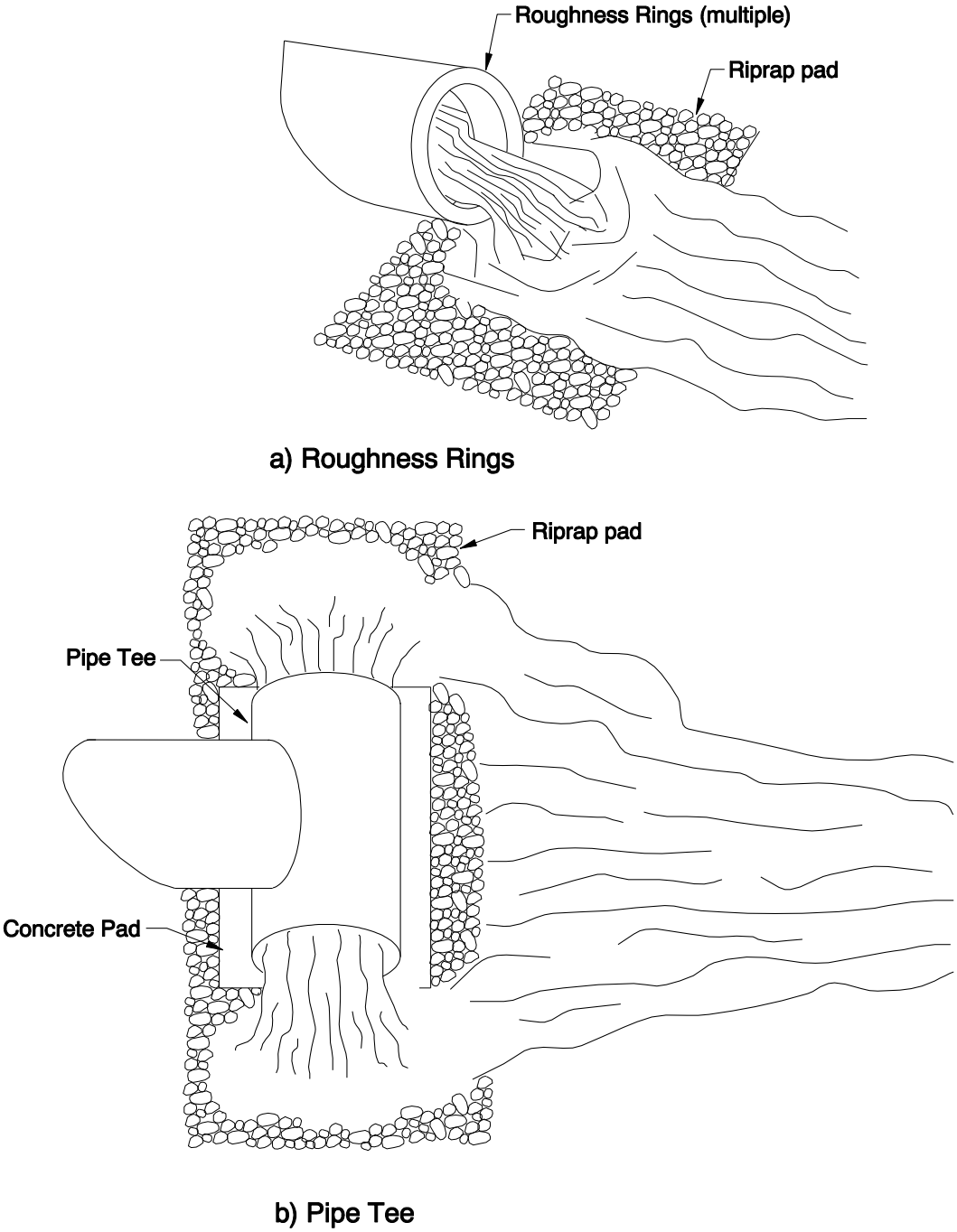



**Figure 11-2 Internal Energy Dissipators**

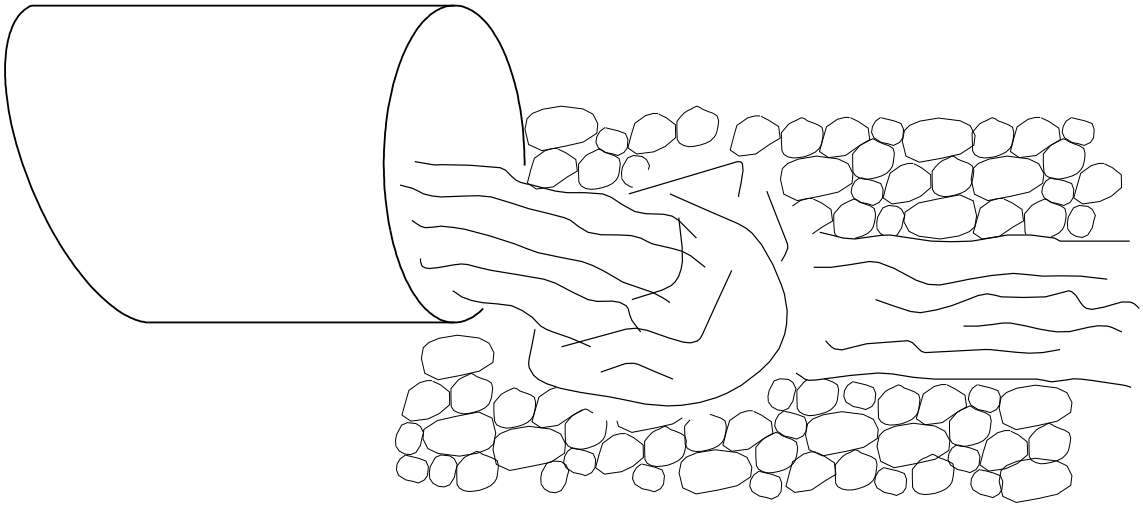


**Figure 11-3 Riprap Pad**

shown in Figure 11-3. These pads are also used in conjunction with many other types of dissipators and they are located immediately downstream from the dissipator outlet. The design of these dissipators is discussed in Section 11.11.

These pads are best suited for conduits with outlet flows having specific characteristics. The outlet should be free-flowing, in other words, the outlet should not be submerged by tailwater. The effectiveness of the riprap in creating turbulence is greatest at shallow flow depths and it lessens as the flow deepens. In addition, the outlet velocities should not be excessively high. Fast flow can displace the riprap and wash it downstream.

Riprap pads are often used where:

- a low-cost and easily constructed dissipator is needed,
- flow from the outlet of the conduit has moderate to low velocity and depth, and
- fish passage is not required.

**Riprap Lined Basins -** These external dissipators are most often used at the outlets of larger conduits or channels. They consist of a pool known as a preformed scour hole that is lined with riprap, as shown in Figure 11-4. Like an unlined pool, the velocity flow entering these dissipators collides with the relatively tranquil water in a pool and considerable hydraulic energy

is dissipated. The riprap lining protects the surrounding soil from the scour and erosion caused by the turbulent flow in the basin. The design of these dissipators is presented in Section 11.12.

*Note: The depth and dimensions of the preformed scour hole depend on the size and quality of the rock riprap available.*

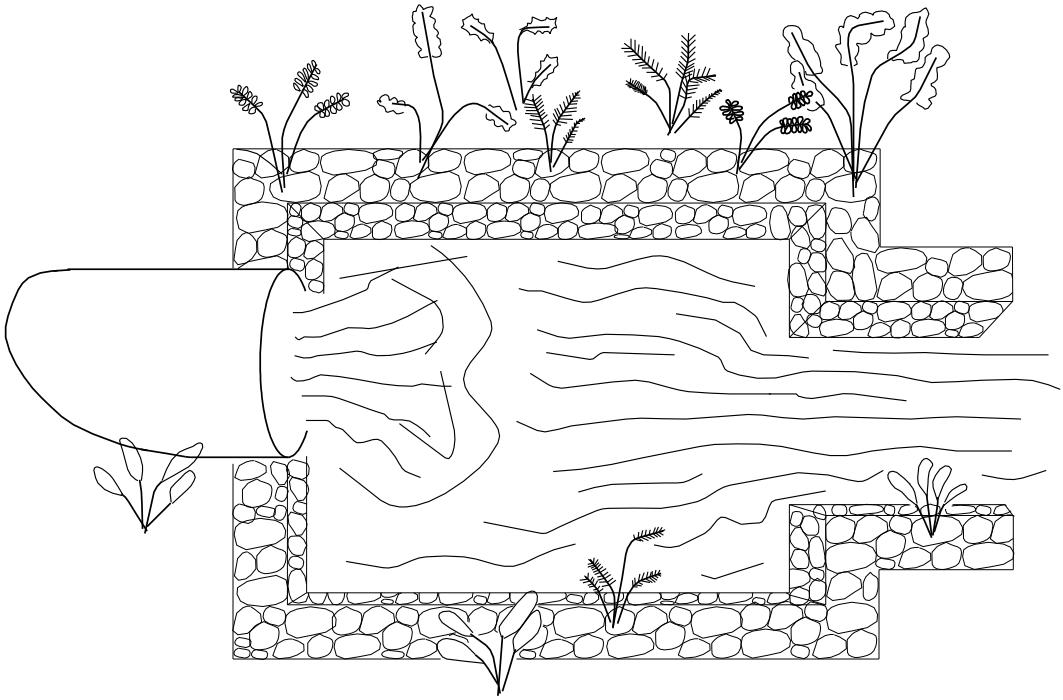


**Figure 11-4 Riprap Lined Basin**

These basins can be designed to pass fish and riparian plantings can be installed in the riprap at the basin edges in some locations. As a result, with the exception of unlined basins, these basins are some of the most environmentally friendly dissipators.

Riprap lined basins are often used where:

- fish passage and/or wetland habitat are desired and the site cannot accommodate an unlined pool,
- flow depths and velocities are too high for a riprap pad, and
- the pool is not a nuisance or safety hazard to the highway users or to the general public.

**Drop Structures -** In general, the flow velocity in a conduit is proportional to its slope, and the steeper the conduit, the higher the velocity. Drop structures are often used to flatten the slopes of the conduits in a drainage system, and consequently, they can reduce the flow velocities in the system and at the outlet. An example is shown in Figure 11-5.

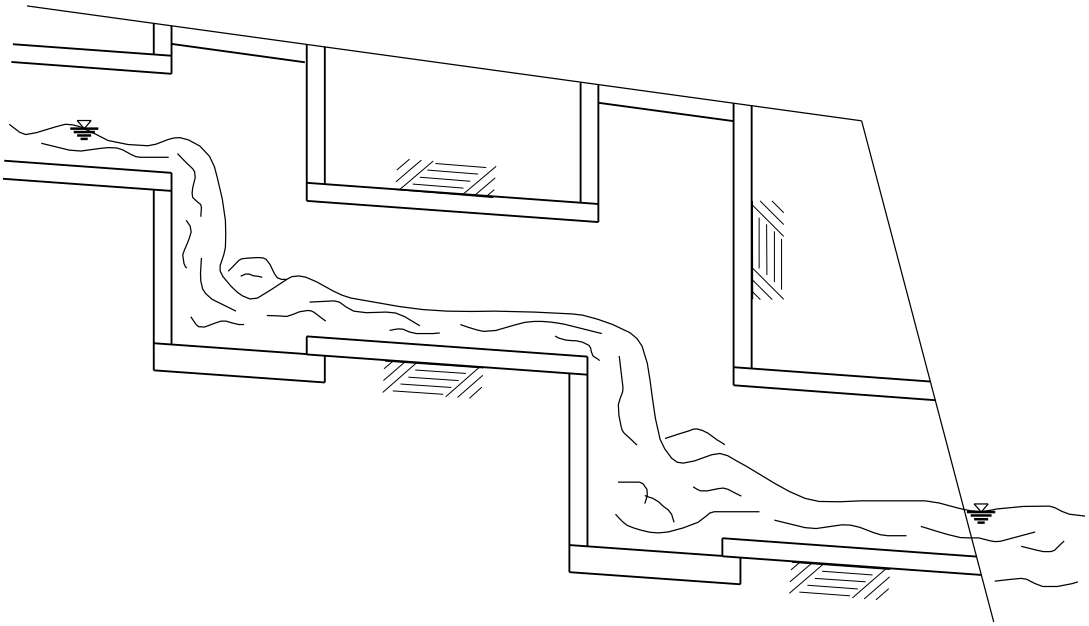
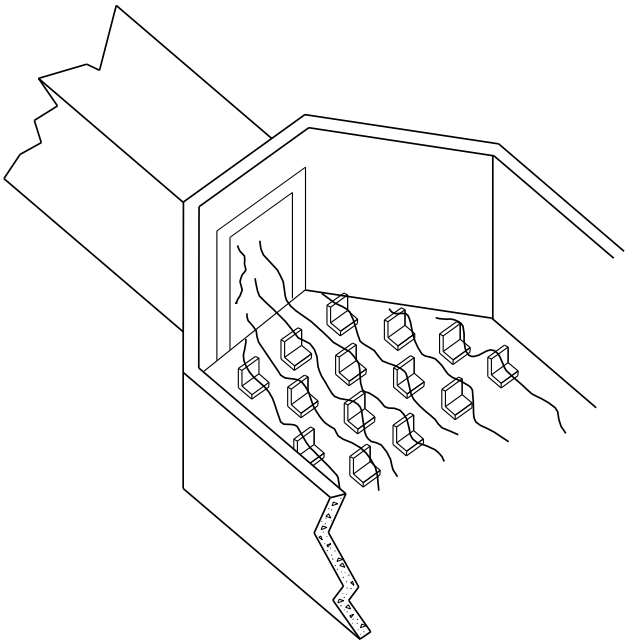


**Figure 11-5 Drop Structures in a Storm Sewer**

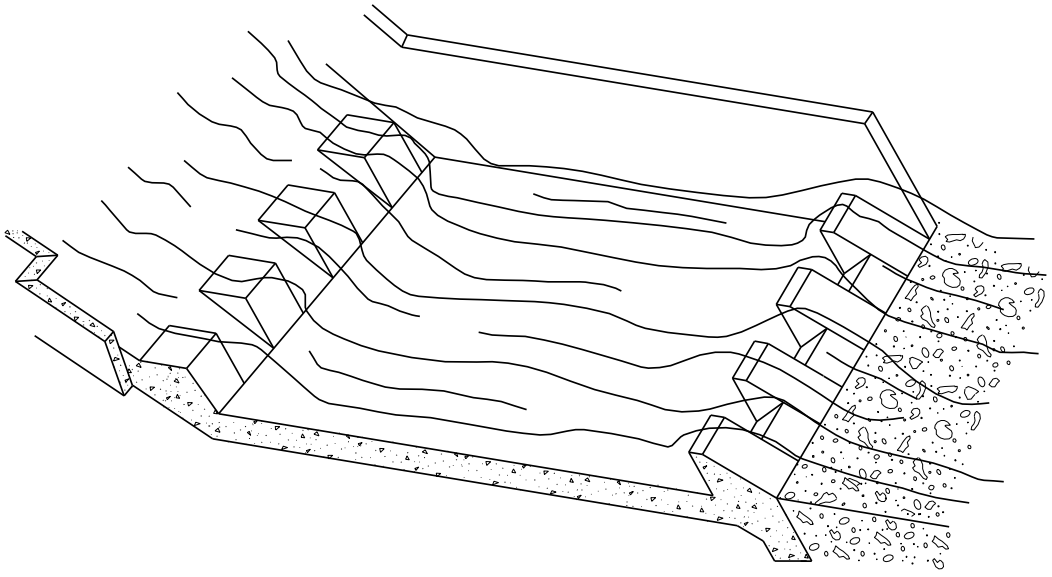
Drop structures are often used where:

- it is desired to reduce the velocities in conduits leading to the outlet as well as the velocity at the outlet,
- their cost is justified by the ability to use a smaller, less costly, or more environmentally friendly type of external dissipator,
- ice or debris passage is not a problem, and
- fish passage is not required.

**Complex Dissipators -** These external dissipators are used at the outlets of both conduits and channels. Most types use blocks, sills, walls, or other obstructions to slow the flow, they are made of cast-in-place reinforced concrete, and they are specifically designed for each site. They are called complex dissipators because they require considerable effort to design and build.



Colorado State University Rigid Boundary Basin



United States Bureau of Reclamation Type II Basin

**Figure 11-6 Complex Dissipators**

Examples of two of the many types of complex dissipators are shown in Figure 11-6. The design of these dissipators is not presented in this manual. The topic is presented in detail in the Federal Highway Administration Hydraulic Engineering Circular No. 14: "Hydraulic Design of Energy Dissipators for Culverts and Channels" and the American Association of State Highway and Transportation Officials "Model Drainage Manual."

Complex dissipators are often used where:

- the outlet velocities or discharge volumes are too great for other types of dissipators,
- the areas occupied by other types of dissipators would be too large for the site, and
- normally, fish passage is not required, but there are circumstances where complex dissipators could be designed to incorporate and even assist fish passage.

## 11.6 Dissipator Design and Check Discharges

An essential step in the dissipator design is to determine the design and check discharges. Design and check flows for various types of drainage facilities are listed in **Chapter 3** and the hydrologic methods of calculating these discharges are presented in **Chapter 7**.

The dissipator should adequately dissipate the energy from the design flow and it should not be damaged by this flow. In most cases, the design discharges for the dissipator and the facility that discharges into the dissipator are the same. As an example, a culvert under a main highway has a 50-year design discharge. The dissipator would also have a 50-year design flow.

Dissipator design discharges that are larger than the upstream facility design flows are allowed in certain circumstances, as follows.

1. The failure of the dissipator or the inability of the dissipator to reduce flow energy could result in a catastrophic failure of a roadway or adjacent facility that would threaten human life or cause extensive property damage. As an example, the abutment of a railroad bridge needs to be protected from the discharge out of an ODOT culvert and a 25-year design flow was used to size the culvert. In this case a larger design flow may be justified to assure the bridge will not be damaged, such as the 50-year, 100-year, 500-year, or overtopping flow (see Note at end of this section).

2. Local ordinances, requirements of regulatory agencies, or agreements with neighboring property owners require a larger design flow than ODOT standards.

Check dissipators for a range of discharges as notes in table 3.1, **Chapter 3**. The dissipator should protect structures from undermining or other scour related damage during the check

discharge. Structures include, but are not limited to: the outlet of the conduit if the diameter or span is 72-inch or larger, the dissipator itself if it is a complex dissipator, or a nearby structure. Typically culvert outlets, complex dissipators, and similar structures are designed to provide a 75-year life without structural failure. Designing the dissipator to prevent damage during the 100-year flood helps to assure the structure will last as long as a 75 year design life.

*Note: In some instances a lesser and more frequent flow than the design or check discharge can cause greater hydraulic stresses on the dissipator. This happens most often when road overtopping occurs more frequently than the design or check flow. If a more frequent flood causes greater hydraulic stresses than the design or check flood, the dissipator should be designed to dissipate the energy from, and withstand damage from, the more frequent flood.*

## 11.7 Hydraulics of Conduit and Channel Outlets

The hydraulic characteristics of the flow at the outlet of a conduit or channel are the major factors that determine the type and size of the energy dissipator. The outlet of the conduit or channel is often called the "brink" and flow characteristics are calculated at this location. Several terms are used in energy dissipator design to describe this flow, as follows.

**Brink Depth -** There are two types of brink depths, actual and equivalent. The actual brink depth, y, is the actual depth of the flow at the brink.

y = Brink depth in feet

The equivalent brink depth, $y_e$, is a hypothetical depth of flow at the brink. It is used in certain hydraulic calculations to represent the brink depth in non-rectangular conduits or channels. It is based on the following equation:

$$y_e = \left(\frac{a}{2}\right)^{0.5} \qquad \textbf{(Equation 11-2)}$$

Where:

$y_e$ = Equivalent brink depth in feet
a = Cross-sectional area of flow in square feet

**Froude Number -** The Froude number is a dimensionless expression of the ratio of the inertial force of moving water to the force of gravity. Two forms of this expression are commonly used in dissipator design. One expression, Fr, is used to describe the flow out of rectangular shaped conduits or channels, as follows:

$$Fr = \frac{v}{(32.2y)^{0.5}} \qquad \textbf{(Equation 11-3)}$$

Where:

Fr = Froude number of the flow at brink of rectangular conduit or channel, flowing full or partially full

V = Average flow velocity in feet per second

y = Brink depth in feet

The Froude number calculated by the preceding equation is applicable to rectangular conduits or channels flowing full or partially full. In most instances it does not provide realistic Froude numbers for non-rectangular sections. In some applications where non-rectangular conduits or channels flow full or partially full, an equivalent Froude number is used in dissipator design, and it is calculated as follows:

$$Fr_e = \frac{v}{(32.2y_e)^{0.5}} \qquad \textbf{(Equation 11-4)}$$

Where:

$Fr_e$ = Equivalent Froude number of the flow at the brink of a non-rectangular conduit or channel, flowing full or partially full

v = Average flow velocity in feet per second

$y_e$ = Equivalent brink depth in feet

The flow characteristics at the brink of a conduit or channel usually depend on the magnitude of the flow, the slope of the conduit or channel, and the tailwater depth. Descriptions of several commonly encountered outlet conditions follow. In these descriptions there are many references to the terms and concepts of conduit and open-channel hydraulics. Concepts such as determining the properties of normal flow and critical depth in conduits are discussed in detail in **Chapter 9**. Useful information such as critical depth charts and the hydraulic properties of common conduits are presented in Appendix B, of **Chapter 8**. Concepts related to open-channel flow are discussed in **Chapter 8**, and Appendix A of **Chapter 8** lists the roughness values of many common conduits and channels.

**Case 1: Partially Full Conduit or Channel on a Steep Slope with a Free Flowing Outlet -** This flow type occurs when a conduit or channel is on a steep slope and the outlet is free flowing. In other words, there is supercritical flow in the conduit or channel and the tailwater depth does not influence the depth of the flow at the outlet. An example of a culvert with this profile is shown in Figure 9-11a. The most common occurrences of this profile are in culverts under inlet control with low tailwater or storm sewers and channels on steep slopes.

In Case 1 flow:

- the brink depth is the depth of normal flow,
- the equivalent brink depth is based on the area of normal flow, and
- the brink velocity is the velocity of normal flow.

**Case 2: Partially Full Conduit on a Steep Slope with a Submerged or Partially Submerged Outlet -** This type of flow occurs at the outlets of conduits when there is supercritical flow at normal depth upstream from the outlet and the outlet is submerged or partially submerged. A culvert with this type of flow is shown in Figure 9-11b. The most common occurrence of this flow profile is in culverts under inlet control and storm sewers on steep slopes.

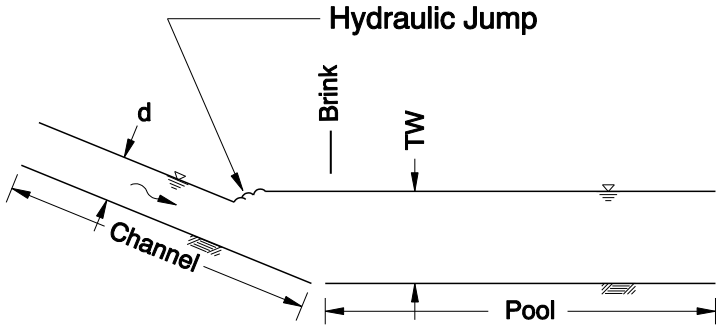
In Case 2 flow:

- the brink depth is the diameter or rise of the conduit if the outlet is fully submerged,
- the brink depth is the depth of the tailwater above the outlet invert, if this depth is greater than the depth of normal flow in the conduit,
- the brink depth is the depth of normal flow in the conduit, if this distance is greater than the depth of the tailwater above the outlet invert,
- the equivalent brink depths are based on the flow areas corresponding to the brink depths previously listed, and
- the brink velocities are based on the flow areas corresponding to the brink depths previously listed.

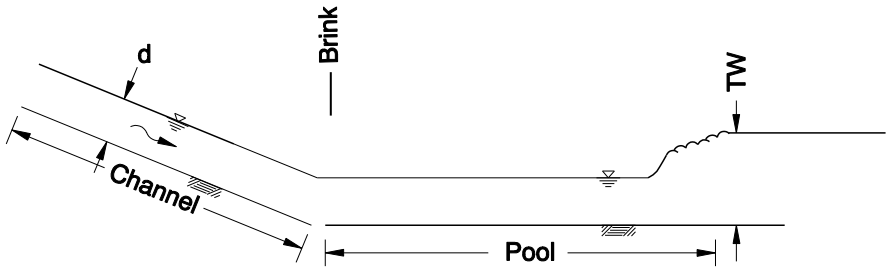
**Case 3: Channel on a Steep Slope with Pooled Water at the Outlet -** This flow type occurs in channels when there is supercritical flow at normal depth upstream from the outlet and there is slower moving subcritical flow or ponded water at the outlet. This profile often occurs at the outlets of lined ditches or channels that empty into ponds, canals, or other relatively slow moving or stagnant water bodies.

In Case 3 flow:

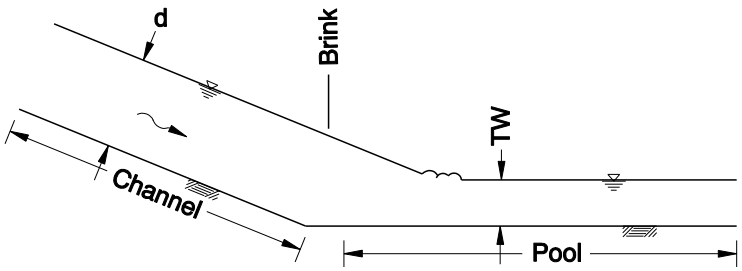
- the brink depth is the depth of the tailwater above the bottom of the upstream channel at the outlet, if this depth is greater than the depth of normal flow in the upstream channel, as shown in Figure 11-7a, unless the tailwater depth is greater than the resultant depth for a hydraulic jump, the brink depth will be normal depth because the jump will not occur here as shown in figure 11-7b.




a) Tailwater depth greater than normal depth




b) Tailwater depth greater than normal depth




c) Normal depth greater than tailwater depth

*d = Depth of normal flow in upstream channel*
*TW = Depth of tailwater*

**Figure 11-7 Flow in a Steep Channel Entering a Pool**

- the brink depth is the depth of normal flow in the upstream channel, if this depth is greater than the depth of the tailwater above the bottom of the upstream channel at the outlet, as shown in Figure 11-7c,

- the equivalent brink depths are based on the flow areas corresponding to the brink depths previously listed, and

- the brink velocities are based on the flow areas corresponding to the brink depths previously listed.

**Case 4: Full Conduit on a Mild Slope with a Submerged Outlet -** This type of flow occurs at the outlet of a conduit when there is full flow in the conduit and the outlet is submerged. This type of flow in a culvert is shown in Figure 9-14a. The most common occurrence of this flow profile is in culverts or storm sewers on mild slopes that discharge into other waterways.

In Case 4 flow:

- the brink depth is the diameter or rise of the conduit,

- the equivalent brink depths are based on the full flow area of the conduit, and

- the brink velocities are based on the flow areas corresponding to the brink depths previously listed.

**Case 5: Full Conduit Under Surcharge with a Submerged, Partially Submerged, or Free-Flowing Outlet -** This type of flow occurs when the conduit is full throughout its entire length and the outlet is partially submerged or free-flowing, as shown in Figure 9-14b. In this case the critical depth of the flow in the conduit is at least as large as the conduit diameter or rise. This type flow commonly occurs in culverts and storm sewers operating under surcharge.

In Case 5 flow:

- the brink depth is the diameter or rise of the conduit,

- the equivalent brink depth is based on the cross-sectional area of the full conduit, and

- the brink velocities are based on the flow areas of the brink depths previously listed.

*Note: Outlet velocities in Case 5 flow can be extremely high. Complex dissipators are often needed to handle this type of flow.*

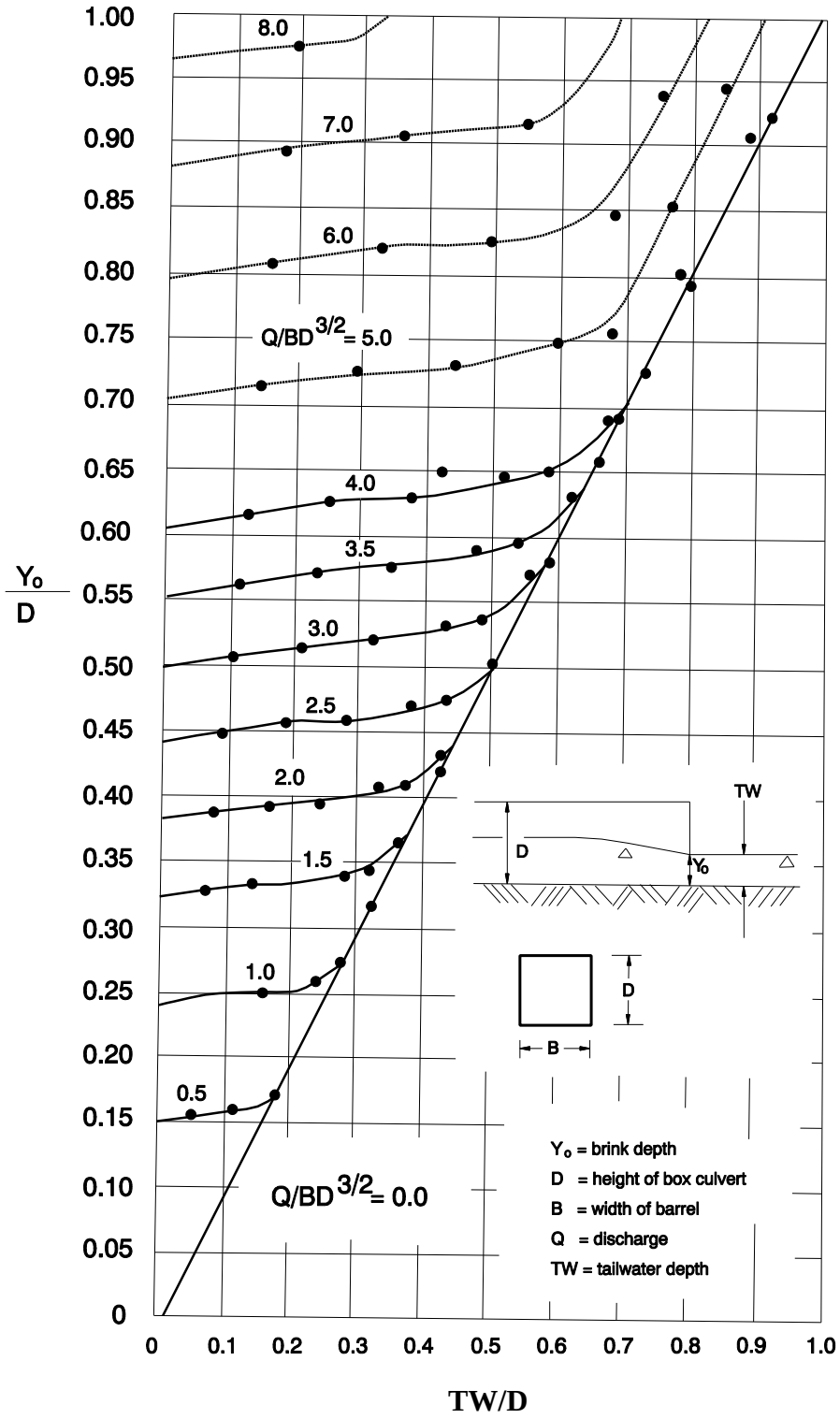



**Figure 11-8 Brink Depth Rating Curves for Rectangular Conduits on Mild Slopes**

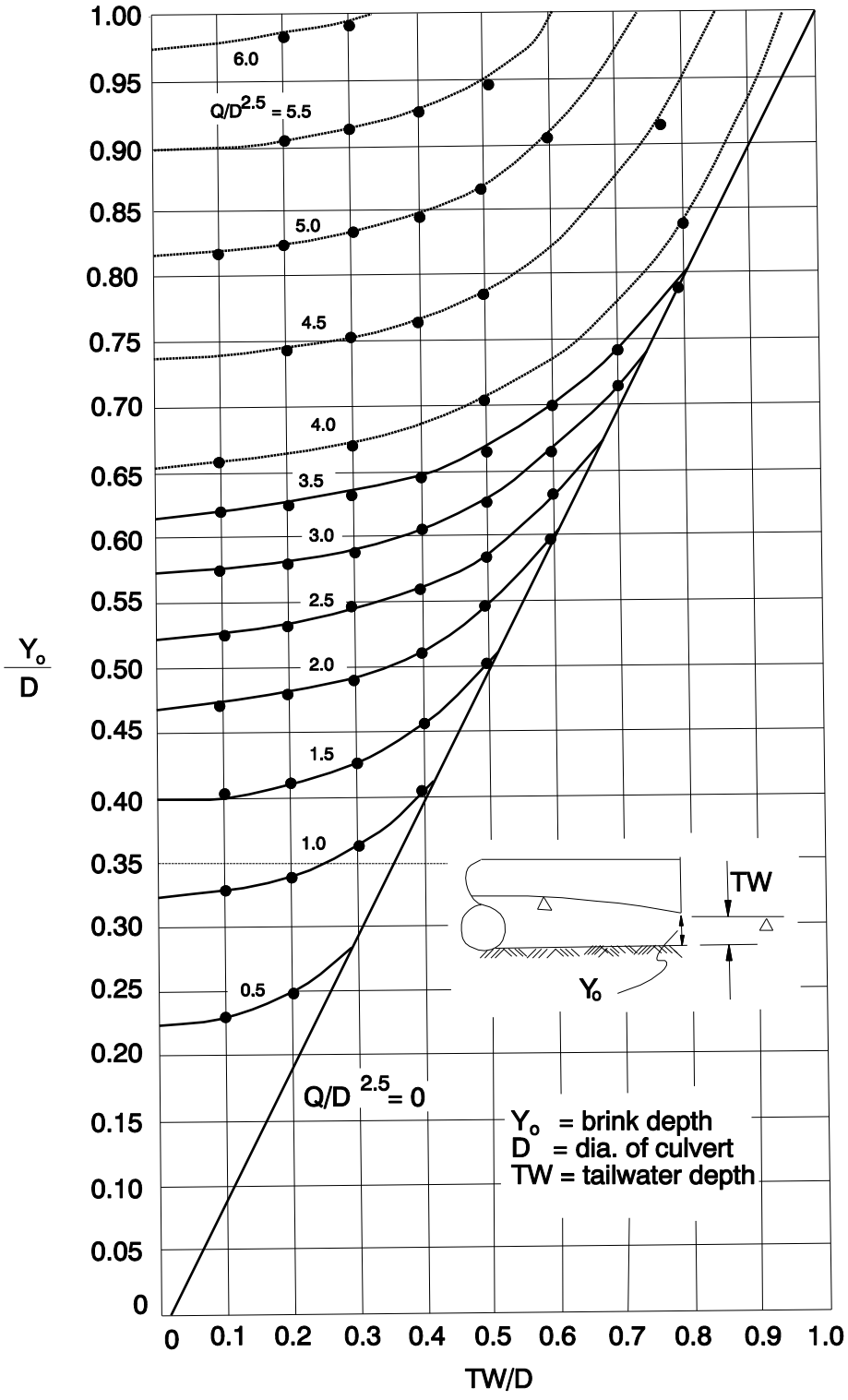



**Figure 11-9 Brink Depth Rating Curves for Circular Conduits on Mild Slopes**

**Case 6: Full or Partially Full Conduit or Channel on a Mild Slope with a Partially Submerged or Free Flowing Outlet**

This is the most common type of flow at the outlet of both conduits and channels, and it occurs when there is subcritical flow just upstream from the outlet and the outlet is not submerged. Culverts with this flow type are shown in Figures 9-14c and 9-14d, and these profiles can occur in other closed conduits such as storm sewers. Open-channels with this type of flow have profiles similar to Figure 9-14d.

During Case 6 flow the depth of flow at the brink is influenced by the amount of the discharge, the conduit or channel size and shape, and the tailwater elevation. The brink depths for rectangular and circular conduits can be determined from the rating curves in Figures 11-8 and 11-9, respectively. The brink depths for conduits and channels with other shapes can be approximated as follows:

- if the depth of the tailwater above the outlet invert is greater than critical depth in the upstream conduit or channel, assume the brink depth is the depth of the tailwater above the invert, or

- if critical depth in the upstream conduit or channel is greater than the depth of the tailwater above the outlet invert, assume the brink depth is the critical depth in the upstream conduit or channel.

### 11.7.1 Example: Hydraulic Characteristics of Flow at the Brink of a Box

The brink depth, brink velocity, and Froude number are needed for an old box culvert. The 50-year design flow is 459 cubic feet per second, the box has a 7 foot span and a 6 foot rise, and the culvert has a 0.01 foot per foot slope. Tailwater depths are assumed to be negligible.

**Step 1:** The first step is to determine the type of flow at the outlet. Since there is negligible tailwater, the outlet is free-flowing, and Case 1, Case 5, or Case 6 flow types can occur. In order to determine whether the conduit is flowing full with surcharge or partially full, the discharge that occurs when the conduit is full but not surcharged is calculated by Manning's equation for discharge (**Chapter 8**). The Manning's roughness coefficient, n, is assumed to be 0.015 because the box is made from worn concrete (Table 8-A-3 of Appendix A, **Chapter 8**). The cross-sectional area of the full conduit multiplied by the hydraulic radius to the 2/3 power, $AR^{2/3}$, is 57.8. Using Manning's equation:

$$Q_{FULL} = \left(\frac{1.49}{0.015}\right)(57.8)\left(0.01^{0.5}\right) = 574 \text{ cubic feet per second}$$

Q = 459 cubic feet per second and Q is less than or equal to $Q_{FULL}$. Consequently, the conduit is partially full and Case 5 flow cannot exist.

A hydraulics elements chart for a 7 foot span by 6 foot rise box (Chart 11 of Appendix B, **Chapter 8**) is used to get the depth of normal flow, as follows:

$$\frac{Q}{Q_{FULL}} = \frac{q}{Q_{FULL}} = \frac{459}{574} = 0.80$$ From the hydraulic elements chart, $$\frac{d}{D_{FULL}} = 0.70$$

D = 6 feet (the rise of the box). d = (0.70) (6.0) = 4.2 feet

In order to determine if the conduit is on a mild or steep slope the critical depth, $D_c$, is calculated. If the normal depth, d, is greater than $D_c$, there is subcritical flow in the barrel and the conduit is on a mild slope. Conversely, if $D_c$ is greater than d, there is supercritical flow in the barrel and the conduit is on a steep slope. The critical depth chart for rectangular sections (Chart 16 of Appendix B, **Chapter 8**) is used to find $D_c$, as follows:

Q = 459 cubic feet per second and B = 7.0 feet (the span of the box).

$$\frac{Q}{B} = \frac{459}{7} = 65.6 \text{ cubic feet per second per foot}$$

From the critical depth chart, $D_c$ = 5 feet. Since the d less than or equal to $D_c$, there is supercritical flow in the conduit. Now the flow type at the outlet can be determined. Case 1 flow occurs because the conduit is partially full, it is on a steep slope, and the outlet is free flowing.

**Step 2:** In this step the brink depth and brink velocity are calculated. If the conduit were a shape other than rectangular, an equivalent brink depth would be calculated. The brink depth, y, is assumed to be the depth of normal flow, d, calculated in the previous step.

At the brink, y = d = 4.2 feet.

The brink velocity is calculated using the hydraulic elements chart as follows:

$A_{FULL}$ = 42 square feet (Table 9-B-2). Using the continuity equation (**Chapter 8**):

$$V_{FULL} = \frac{Q_{FULL}}{A_{FULL}} = \frac{574}{42} = 13.7 \text{ feet per second}$$

$\frac{q}{Q_{FULL}} = 0.80$ From the hydraulic elements chart (Chart 11, Appendix B, **Chapter 8**),

$\frac{v}{V_{FULL}}$ = 1.11, and v = (1.11) (13.7) = 15.07 feet per second.

At the brink, v = 15.07 feet per second.

**Step 3:** The last step is to calculate the Froude number at the brink using Equation 11-3. The brink depth and velocity were calculated in the previous step, and the Froude number is:

$$Fr = \frac{15.07}{[(32.2)(4.2)]^{0.5}} = 1.3$$

At the brink: Fr = 1.3

### 11.7.2 Example: Hydraulic Characteristics of Flow at the Brink of an Arch

The equivalent brink depth, brink velocity, and equivalent Froude number are needed for an arch culvert that discharges directly into a pool. The 50-year design flow is 70 cubic feet per second, the structural plate arch has a 10 foot span and a 5.25 foot rise and an earth bottom, and the slope of the arch is 0.005 foot per feet. The tailwater depth is 1.6 feet during the design flood.

**Step 1:** Since there is tailwater and the outlet is partially submerged, Case 2, Case 5, or Case 6 flows can occur. In order to determine if the culvert is flowing full, Manning's Equation is used. The Manning's roughness coefficients, n, for arches with various bottoms and cross-sectional areas are listed on the outlet control nomographs (Appendix A, **Chapter 9**). The cross-sectional area of the conduit flowing full (Table 4, Appendix B, **Chapter 8**) is: A = 41 square feet. The rise/span ratio is: 5.25 / 10 = 0.52 From Chart 23 for an arch culvert with an earth bottom, a rise/span ratio ≥ 0.5, and a cross-sectional area from 4.1 to 150 square feet: n = 0.029 From Table 4: Appendix B, **Chapter 8** $(41)(1.60)^{2/3} = 56.1$ Using Manning's equation for discharge (Equation 8-15):

$$Q_{FULL} = \left(\frac{1.49}{0.029}\right)(56.1)\left(0.005^{0.5}\right) = 204 \text{ cubic feet a second}$$

Q = 70 cubic feet per second and Q is less than or equal to $Q_{FULL}$. The conduit is partially full and Case 5 flow cannot exist.

The hydraulics element chart for a semicircular arch (Chart 3 of Appendix B, **Chapter 8**) is used to get d, as follows:

$\frac{Q}{Q_{FULL}} = \frac{q}{Q_{FULL}} = \frac{70}{204} = 0.34$ From the hydraulic elements chart, $\frac{d}{D_{FULL}} = 0.32$

D = 5.25 feet (the rise of the arch). d = (0.32) (5.25) = 1.68 ft

$D_c$ is calculated by the iterative method using Equation 8-19 in **Chapter 8**, and $D_c$ = 0.03 feet. Appendix B, **Chapter 8**, the critical depth chart for corrugated metal arch culverts, can also be used to find $D_c$. Since the d is more than or equal to $D_c$, the barrel is on a mild slope. There is Case 6 flow at the outlet because the barrel is partially full, it is on a mild slope, and the outlet is submerged or free-flowing.

**Step 2:** In this step the equivalent brink depth and brink velocity are calculated. The flow area at the brink is assumed to be the cross-sectional area of the arch below the tailwater elevation because the 1.6 foot tailwater depth is greater than the 0.03 foot critical depth. This flow area is calculated using the hydraulic elements chart (Chart 3, Appendix B, **Chapter 8**) as follows:

d = y = TW = 1.6 feet D = 5.25 feet $A_{FULL}$ = 41 $ft^2$ $\frac{d}{D} = \frac{1.6}{5.25} = 0.31$

From the hydraulic elements chart, $\frac{a}{A_{FULL}} = 0.42$ and $a = (0.42)(41) = 17.2$ square feet

The equivalent brink depth is calculated using Equation 11-2 as follows:

$$y_e = \left(\frac{17.2}{2}\right)^{0.5} = 2.9 \text{ feet}$$

The brink velocity is calculated using the continuity equation (**Chapter 8**) and the flow area below the tailwater elevation, as follows:

$$v = \frac{Q}{a} = \frac{70}{17.2} = 4.1 \text{ feet per second}$$

At the brink: $y_e$ = 2.9 feet and v = 4.1 feet per second

**Step 3:** The equivalent Froude number at the brink is calculated using Equation 11-4. The equivalent brink depth and brink velocity were calculated in the previous step, and the equivalent Froude number is:

$$Fr_e = \frac{4.1}{[(32.2)(2.9)]^{0.5}} = 0.42$$

At the brink: $Fr_e$ = 0.42

## 11.8 Estimating the Size of an Unlined Scour Hole

In some cases it is necessary to estimate the size and shape of the scour hole that would occur if there were no scour or erosion protection beyond the conduit or channel outlet. Methods have been developed for predicting the size of scour holes and they are discussed in detail in the FHWA Hydraulic Engineering Circular No. 14 "Hydraulic Design of Energy Dissipators for Culverts and Channels" September 1983 with March 1996 Amendment (HEC-14). A computer program that can calculate scour hole dimensions is included in the HY-8 module of the FHWA's HYDRAIN software package.

A simplified version of the method of predicting scour holes in non-cohesive soils is presented in this manual. This method is suitable for the most applications because the majority of streambeds are composed of cohesionless materials. Based on the agency's experience this method tends to be fairly conservative. In other words, the scour holes that actually occur are rarely larger than the holes predicted by the equations.

A method of predicting scour hole dimensions in cohesive soils is not given in this manual, but it is included in HEC-14 and HY-8. It is applicable for sandy clays with a Plasticity Index based on Atterburg Limits (PI) not lower than 5 or higher than 16. Prior to using this design method, laboratory tests are needed on representative samples of the soils at the location of the scour hole, such as the PI (AASHTO T 90-96) and the unconfined compressive strength (AASHTO T 208-96). The Geo-Environmental Section can provide details about the needed tests.

Predicting the exact dimensions of a scour hole is difficult because of the many factors involved, some of which are: discharge, conduit shape, soil type, flow duration, conduit gradient, height of the drop from the conduit outlet to the channel bed, and the tailwater depth. In addition, the dimensions of the scour hole can change over time. If the stream transports sediment the hole may be quite large during the time of peak flow and it may fill with sediment during periods of lower flows. This occurs more often with culverts and very rarely with storm sewers. A typical scour hole is shown in Figure 11-10a, and in general, research has shown:

- the scour hole geometry varies with tailwater conditions,
- the largest scour holes occur when the tailwater depths are less than half of the conduit rise, and
- the maximum scour depth occurs at a distance of $(0.4)(L_s)$ downstream from the conduit outlet, where $L_s$ is the length of the scour hole.

The following equations are used to calculate the dimensions of scour holes in non-cohesive soils such as silts, sands, or gravels. They are intended to be used along with the maintenance history

and the results of a site reconnaissance to estimate the scour potential of the site. The maximum dimensions of the scour hole can be estimated as follows:

$$d_s = C_g\, C_h\, R\left(\frac{2.27}{\sigma^{0.33}}\right)\left(\frac{0.176\,Q}{R^{2.5}}\right)^{0.39}\left(\frac{t}{316}\right)^{0.06} \qquad \text{(Equation 11-5)}$$

$$w_s = C_g\, C_h\, R\left(\frac{6.94}{\sigma^{0.33}}\right)\left(\frac{0.176\,Q}{R^{2.5}}\right)^{0.53}\left(\frac{t}{316}\right)^{0.08} \qquad \text{(Equation 11-6)}$$

$$l_s = C_g\, C_h\, R\left(\frac{17.10}{\sigma^{0.33}}\right)\left(\frac{0.176\,Q}{R^{2.5}}\right)^{0.47}\left(\frac{t}{316}\right)^{0.10} \qquad \text{(Equation 11-7)}$$

$$v_s = C_g\, C_h\, \left(R^3\right)\left(\frac{127.08}{\sigma^{0.33}}\right)\left(\frac{0.176\,Q}{R^{2.5}}\right)^{1.24}\left(\frac{t}{316}\right)^{0.18} \qquad \text{(Equation 11-8)}$$

Where:

$d_s$ = Maximum depth of scour hole in cohesionless soil in feet
$w_s$ = Maximum width of scour hole in cohesionless soil in feet
$l_s$ = Maximum length of scour hole in cohesionless soil in feet
$v_s$ = Maximum volume of scour hole in cohesionless soil in cubic feet
$C_g$ = Dimensionless coefficient to account for the gradient of the conduit. See Table 11-1a and Figure 11-10b.
$C_h$ = Dimensionless coefficient to account for the ratio of the drop height to the conduit diameter, $H_s$ / D. See Table 11-1b and Figure 11-10c.

Where:

$H_s$ = Vertical distance between the conduit invert at the outlet and the streambed prior to scour, in feet.
D = Diameter of conduit in feet.
R = Hydraulic radius of the flow at the brink in feet. This is the flow area of the brink in square feet divided by the wetted perimeter of the brink in feet. See Appendix B of **Chapter 8** for tables listing hydraulic radii of common conduits flowing full. If the conduit is flowing partially full, the hydraulic radius can be determined using the hydraulic element charts in Appendix B of **Chapter 8**.
σ = Standard deviation of the bed-material grain size distribution using the following formula:

$$\sigma = \left(\frac{d_{84}}{d_{16}}\right)^{0.5} \qquad \text{(Equation 11-9)}$$

The values $d_{84}$ and $d_{16}$ are based on a distribution curve determined from a particle size analysis of the soil at the location of the scour hole (AASHTO T 88-97). A σ value

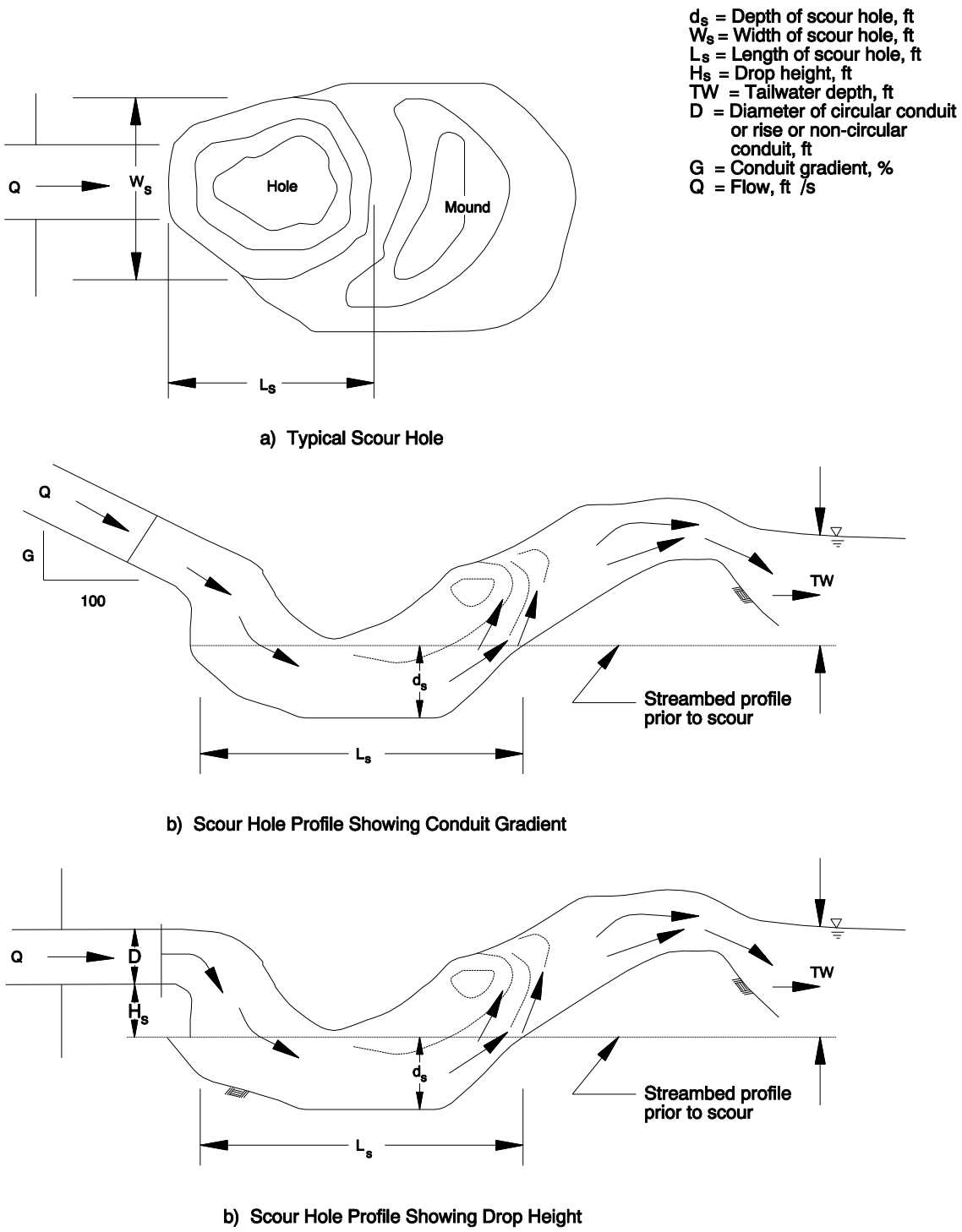



**Figure 11-10 Scour Hole Geometry**

based on test results is preferred. If these results are not available, it can be assumed that σ is 2.10 for gravel and 1.87 for sand.

Q = Discharge in cubic feet per second

t = 30 minutes or the time of concentration, if longer. If t is not known a value of 30 minutes should be used.

### 11.8.1 Example: Estimating the Size of an Unlined Scour Hole

The Wirfsville storm sewer outfall is to be relocated. The 36-inch diameter corrugated metal pipe (n = 0.021) is at a 0.20 percent grade, the design discharge is 15 cubic feet per second, and the outfall will be onto fairly uniformly graded beach sand with a σ value of 1.2. There will be a 1.6 foot drop between the outlet invert and the channel bed. Peak flow is estimated to occur when the time of concentration is 120 minutes. The size and shape of the scour hole is needed.

The $C_g$ coefficients for a 0.20 percent gradient were interpolated for depth, width, length, and volume using the values in Table 11-1a, as follows: $C_g$ (Depth) = 1.00, $C_g$ (Width) = 1.03, $C_g$ (Length) = 1.02, and $C_g$ (Volume) = 1.03

The drop height to conduit diameter ratio is: 1.6 feet drop / 3 feet diameter = 0.53. The $C_h$ coefficients for a ratio of 0.53 were interpolated for depth, width, length, and volume using the values in Table 11-1b, as follows: $C_h$ (Depth) = 1.12, $C_h$ (Width) = 1.28, $C_h$ (Length) = 0.85, and $C_h$ (Volume) = 1.15

The hydraulic radius at the brink is needed. To calculate this radius, it is necessary to determine the characteristics of the flow at the outlet. This procedure is presented in detail in Section 7 and it is summarized in this example.

Full flow without surcharge using Manning's equation for discharge and an $AR^{2/3}$ value of 5.8:

**Table 11-1 Coefficients for Unlined Scour Holes in Cohesionless Materials**

**a) Culvert Gradient Coefficient, $C_g$**

| Conduit Gradient, G, in Percent | Coefficient $C_g$ for Conduit Gradient: $C_g$ for Depth Equation | $C_g$ for Width Equation | $C_g$ for Length Equation | $C_g$ for Volume Equation |
|---|---|---|---|---|
| 0 | 1.00 | 1.00 | 1.00 | 1.00 |
| 2 | 1.03 | 1.28 | 1.17 | 1.30 |
| 5 | 1.08 | 1.28 | 1.17 | 1.30 |
| > 7 | 1.12 | 1.28 | 1.17 | 1.30 |

**b) Drop Height to Diameter Ratio Coefficient $C_h$**

| Drop Height to Diameter Ratio, $H_s$ / D | Coefficient $C_h$ for Outlets Above the Streambed: $C_h$ for Depth Equation | $C_h$ for Width Equation | $C_h$ for Length Equation | $C_h$ for Volume Equation |
|---|---|---|---|---|
| 0 | 1.00 | 1.00 | 1.00 | 1.00 |
| 1 | 1.22 | 1.51 | 0.73 | 1.28 |
| 2 | 1.26 | 1.54 | 0.73 | 1.47 |
| 4 | 1.34 | 1.66 | 0.73 | 1.55 |

$$Q_{FULL} = \left(\frac{1.49}{0.021}\right)(5.8)\left(0.002^{0.5}\right) = 18.4 \text{ cubic feet per second}$$

Depth of normal flow in the conduit using the hydraulic elements chart for circular pipe:

$\frac{q}{Q_{FULL}} = \frac{15}{18.4} = 0.82$ From Chart 1, Appendix B, **Chapter 8**, $\frac{d}{D_{FULL}} = 0.69$

d = (0.69) (3) = 2.07 ft

Q = 15 cubic feet per second, depth of critical flow using the critical depth chart for circular pipe:

$D_c$ = 1.2 feet

Since q less than $Q_{FULL}$, d more than $D_c$, and TW less than D there is Case 6 flow at the outlet and the brink depth can be determined using Figure 11-9. Using the figure, y / D = 0.32 and y = (0.32) (3) = .96 feet.

The ratio of the brink depth to the diameter of the conduit is used with the hydraulic elements chart (Chart 1 of Appendix B of **Chapter 8**), and the hydraulic radius of the conduit at full flow (Table 1 of Appendix B of **Chapter 8**), to determine the hydraulic radius at the brink, as follows:

$$\frac{y}{D} = 0.32 \quad \text{From Chart 8:} \quad \frac{r}{R_{FULL}} = 0.74 \quad \text{From Table 8-B-1:} \quad R_{FULL} = 0.75 \text{ feet}$$

r = R = (0.74) (.75) = .56 ft

σ = 1.2 (see given data)

Q = 15 cubic feet per second (see given data)

t = 120 minutes (see given data). If t was less than 30 minutes, or if t was not known, a value of 30 minutes would be used.

The maximum dimensions of the scour hole are estimated using Equations 11-5 through 11-8 to be:

$$d_s = (1.00)(1.12)(.56)\left(\frac{2.27}{1.2^{0.33}}\right)\left[\frac{(0.176)(15)}{.56^{2.5}}\right]^{0.39}\left(\frac{120}{316}\right)^{0.06} = 3.25 \text{ feet}$$

$$w_s = (1.03)(1.28)(.56)\left(\frac{6.94}{1.2^{0.33}}\right)\left[\frac{(0.176)(15)}{.56^{2.5}}\right]^{0.53}\left(\frac{120}{316}\right)^{0.08} = 16.1 \text{ feet}$$

$$l_s = (1.02)(0.85)(.56)\left(\frac{17.10}{1.2^{0.33}}\right)\left[\frac{(0.176)(15)}{.56^{2.5}}\right]^{0.47}\left(\frac{120}{316}\right)^{0.10} = 22.1 \text{ feet}$$

$$v_s = (1.03)(1.15)(.56^3)\left(\frac{127.08}{1.2^{0.33}}\right)\left[\frac{(0.176)(15)}{.56^{2.5}}\right]^{1.24}\left(\frac{120}{316}\right)^{0.18} = 421 \text{ cubic feet}$$

The maximum depth is estimated to occur at a distance of 8.9 feet downstream from the end of the conduit, based on the following: 0.4 x $L_s$ = 0.4 x 22.3 = 8.9 feet

## 11.9 Design of Unlined Pools

The unlined pool is the simplest form of energy dissipator. The main design tasks are to assure that the highway embankment, culvert outlet, and other facilities will not be damaged by scour or erosion, and that the pool will be within ODOT right of way or drainage easements. Further

discussion about this dissipator and guidance on its selection is in Section 11.5 and a pool is shown in Figure 11-1.

The outlet should have a cutoff wall to prevent scour damage to the conduit or the lining of the upstream channel. The cutoff wall should be designed to resist undermining and this is typically done using one of the following three methods.

1. The cutoff wall is keyed into non-erodible material as shown in Figure 11-11a. As additional and optional scour protection, the excavation that is made on the stream side of the wall, in order to place the wall, can be backfilled with loose riprap. ODOT Class 50 or Class 100 loose riprap is recommended.

2. The cutoff wall extends down into erodible material; and scour depth is not calculated, or the scour depth is calculated and the wall does not extend below the predicted scour elevation. In this case riprap should be placed around the stream side of the wall, as shown in Figure 11-11b. The riprap size should be determined by the methods used to size the rock in riprap pads, as discussed in Section 11.11.

3. The cutoff wall extends down into erodible material, the scour depth has been calculated, and the cutoff wall extends to an elevation below the predicted scour depth, as shown in Figure 11-11c. As additional and optional scour protection, the excavation that was made on the stream side of the wall, in order to place the wall, can be backfilled with loose riprap. ODOT Class 50 or Class 100 loose riprap is recommended.

The unlined pool can be excavated during construction or it can form due to scour from floods that occur subsequent to construction. Regardless of the method used, if the streambed or other material at the outlet is non-cohesive, the unlined pool may eventually resemble the scour hole shown in Figure 11-10 and its dimensions can be predicted by the equations in Section 11.8.

The right-of way or drainage easements should be of adequate size to contain the pool and the mound downstream from the pool. This mound is most likely to occur when scour from floods is used to create the pool, or when the pool is downstream from a conduit or channel that carries sediment or other bed load during peak flows. The mound is less likely to occur if the pool is excavated during construction, or if the flow carries little sediment, such as the discharge from a storm sewer. Critical structures and other features that may be undermined or damaged should be protected. Methods of bank protection are discussed in **Chapter 15** of this manual.

Vegetation can be planted in and around the embankment protection, as discussed in **Chapter 9**. Vegetation can also be planted in and around the outlet of the conduit or channel and along the edges of the pool. The vegetation should not obstruct the discharge from the outlet unless the effects of this obstruction are considered in the hydraulic design. Large vegetation should not be planted too close to any structure, such as a conduit, channel lining, or end treatment. The plants could grow large enough to damage the structure. As a general rule, large trees such as pacific

willows, red alders, western red cedars, etc should not be planted within 10 feet from the structure, and large bushes such as sitka willows, pacific dogwoods, serviceberries, etc should not be planted within 3 feet from the structure. A structural designer should be consulted if more detailed guidance is needed.

## 11.10 Design of Internal Energy Dissipators

The internal dissipator is contained within the upstream conduit. Two of the more common types of internal dissipators are roughness rings and tees. Guidance on the selection of these dissipators is provided in Section 11.5 and they are shown in Figure 11-2. The design of roughness rings is not presented in this manual. It is included in the FHWA's Hydraulic Engineering Circular No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels."

The design of tee dissipators is addressed in this section. The most common use of this dissipator is at the outlet of a culvert, storm sewer, inlet box, gutter, or other water collection facility where flow must be conveyed down a steep embankment slope. The dissipators perform the dual functions of conveying the flow down the slope without causing embankment erosion and they dissipate much of the flow energy at the outlet.

A standard design is presented in this section for tee dissipators that convey design flows up to and including 16 cubic feet per second. If a dissipator is needed for greater flows or for situations where the standard design cannot be used, the Geo-Environmental Unit should be contacted for assistance. The components of the dissipator are as follows.

**Upstream Drainage System -** The tee dissipator is located at the outfall of an upstream drainage system. This outfall could be a culvert, a storm sewer, an inlet box, or a flared or sloped inlet located behind a gap in a drainage curb. Examples of these outfalls are shown in Figure 11-12, and detailed guidance on the design of inlet boxes, gutters, and other types of roadway drainage is provided in **Chapter 13**.

The maximum allowable design discharges from the outfall are listed in the table in Figure 11-12. The discharge should not contain large debris or large amounts of sediment because these dissipators are susceptible to clogging. Trash racks or grates should be installed at all inlets where water entering the upstream drainage system carries debris. The openings in the rack or grate should be small enough to retain any debris that is large enough to clog the smallest pipe in the downstream system. A settling basin is recommended upstream from an inlet if the water entering the inlet carries a large amount of sediment.

**Lateral Connection -** The upstream drainage system attaches to the pipe going down the slope and to the stub to the cleanout gate using a lateral connection, as shown in Figure 11-12. The connection to the cleanout stub should have the same diameter as the outlet of the upstream

drainage system. The connection to the pipe going down the slope should have the same diameter as the outlet of the upstream drainage system if debris can enter the system. If debris cannot get into the system, the pipe going down the slope can be reduced one size in comparison to the upstream pipes. The reduction in pipe size should be at the lateral connection.

**Cleanout Gate -** A stub with a cleanout hatch or gate is attached to the lateral connection to provide access for debris removal, as shown in Figures 11-12 and 11-13. This access should be provided whether or not the inlets to the system are covered by grates or trash racks. Typically the access hatch is latched or locked shut when not in use. This prevents water pressure from opening the hatch.

**Slope Pipe -** The pipe down the slope is usually buried within the embankment as shown in Figure 11-12, 11-13, and 11-14. The maximum difference in elevation between the inlet and outlet of the slope pipe depends on the slope pipe diameter. These maximum differences are listed in the table in Figure 11-13. Typically the slope pipe as well as the other components of

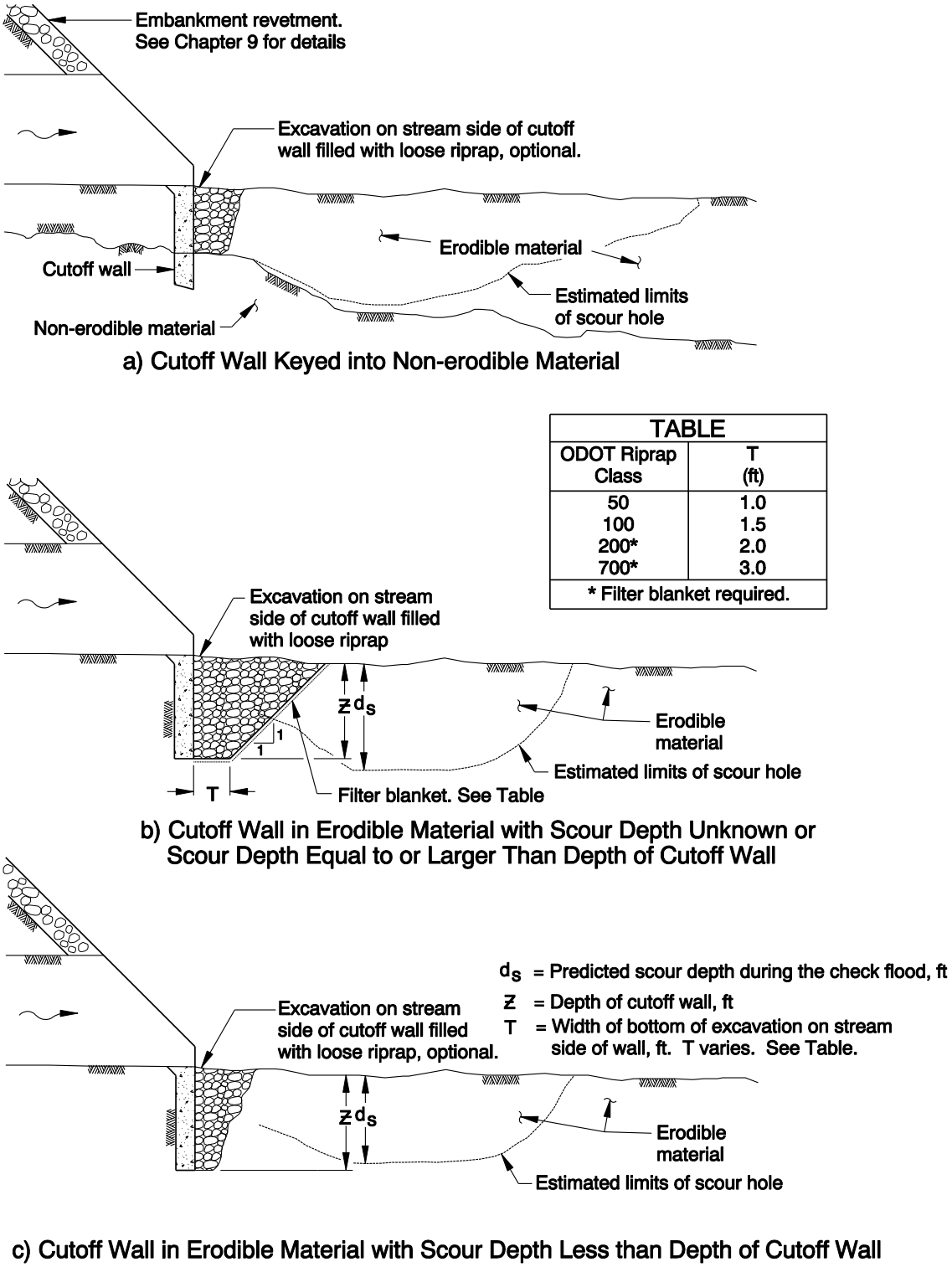



**Figure 11-11 Outlet Into Unlined Pool**

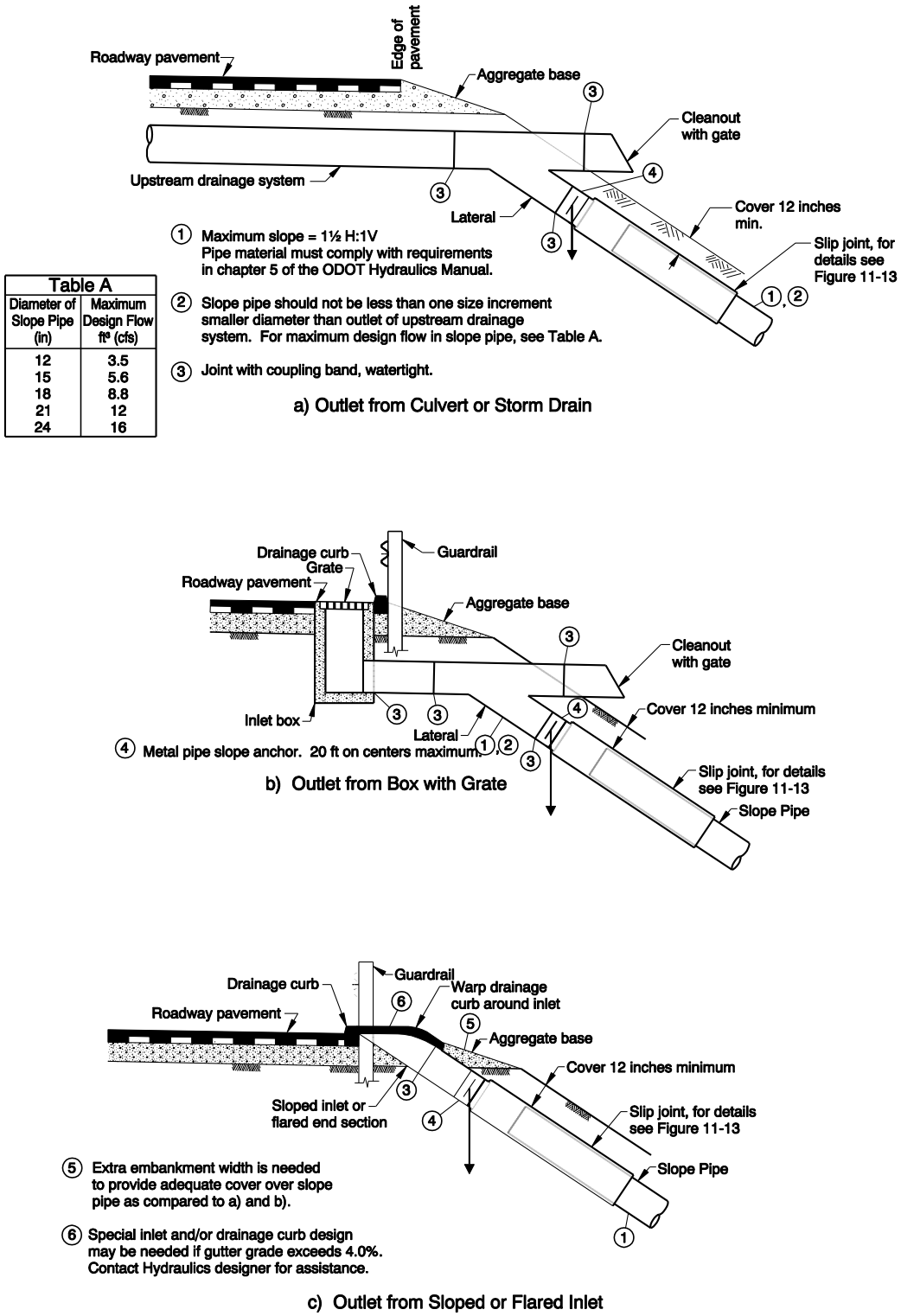



Table A

| Diameter of Slope Pipe (in) | Maximum Design Flow ft³ (cfs) |
|---|---|
| 12 | 3.5 |
| 15 | 5.6 |
| 18 | 8.8 |
| 21 | 12 |
| 24 | 16 |

**Figure 11-12 Upper Ends of Tee Dissipators**

this system are made from corrugated metal. Segmented concrete pipe and other smooth walled conduits are not recommended. The slope pipe is one of the components of this system that is most susceptible to invert erosion. Care should be taken to select materials that will provide an adequate design life, and pipe material selection is discussed in **Chapter 5**. Pipe joints on the slope pipe, elbow, tee, and all other components of the dissipator should be watertight.

**Slip Joint -** A slip joint is provided in some installations to prevent damage to the slope pipe and the dissipator if the embankment fill settles or shifts, as shown in Figure 11-12 and 11-13. The Geo-Environmental Unit or Region Geology staff can be contacted to provide guidance on whether or not a slip joint is needed. The pipe size should not be changed at the slip joint. Any reduction in pipe size should be at the lateral connection.

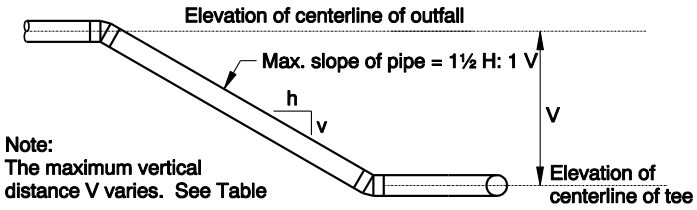
**Elbow and Tee -** The elbow and the tee perform the functions of splitting the flow and dissipating much of the flow energy as turbulence. The elbow and the stem of the tee should be the same diameter as the slope pipe, and the diameter of the end of the tee should be 6-inches larger than the stem, as shown in Figure 11-14. The minimum length of the stem of the tee is 10 feet. If there is sufficient room beyond the toe of the fill, it is recommended that a longer stem is used, and the stem should project at least 10 feet beyond the toe.

**Thrust Block -** The thrust block supports the tee and its dimensions are shown on Figure 11-14. The block should be placed on soil or rock with an ultimate bearing capacity of at least 2089 pounds per square foot. If a larger or longer tee than the one shown in the drawings is used, the size of the thrust block should be increased so that the tee is fully supported.

**Riprap -** The thrust block is surrounded on three sides by riprap, as shown in Figure 11-15. The riprap provides additional energy dissipation and it protects the thrust block from undermining. ODOT Class 50 loose riprap is adequate for the design flows listed in Figure 11-12.

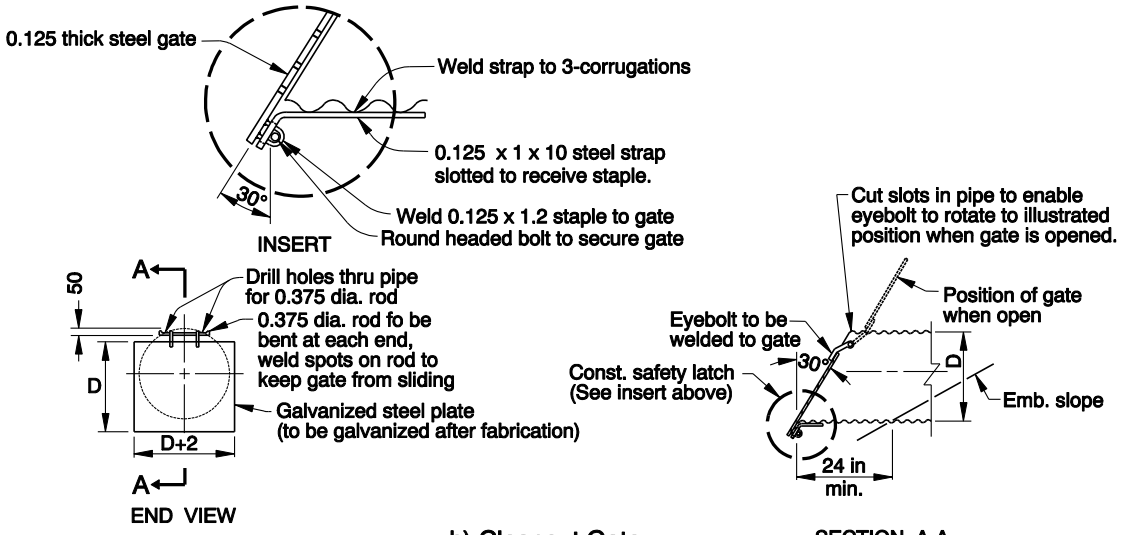
Vegetation can be planted on the highway embankment near the dissipator. The vegetation planted over and near the slope pipe should be sufficiently small so that the roots do not displace the buried conduit and the plants do not obstruct access to the cleanout gate. Generally small bushes and grasses are used and large bushes and trees are avoided. Vegetation planted on the embankment near the roadway should not grow up and obstruct the motorist's ability to see ahead. ODOT district maintenance personnel can provide information about these visibility requirements.

Vegetation can be planted around the tee dissipator. The plants should not obstruct the flow from the dissipator onto the riprap pad and they should not grow large enough to displace or damage the dissipator. In general, large trees such as pacific willows, red alders, western red cedars, etc should not be planted within 10 feet from the dissipator, and large bushes such as sitka willows, pacific dogwoods, serviceberries, etc should not be planted within 3 feet from the dissipator. A structural designer should be consulted if more detailed guidance is needed.

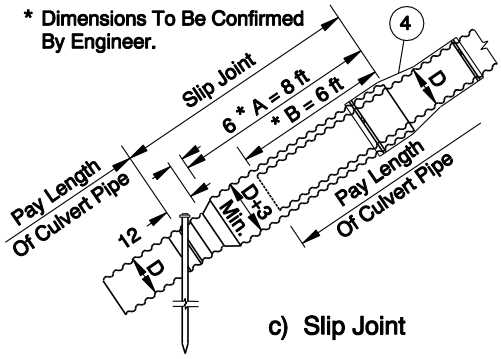



| Table | |
|---|---|
| Diameter of Slope Pipe, D (in) | Maximum Vertical Distance Between Centerlines of Outfall and Tee, V (ft) |
| 12 | 170 |
| 15 | 110 |
| 18 | 82 |
| 21 | 62 |
| 24 | 49 |

a) Maximum Elevations Between Outfalls and Tees




b) Cleanout Gate



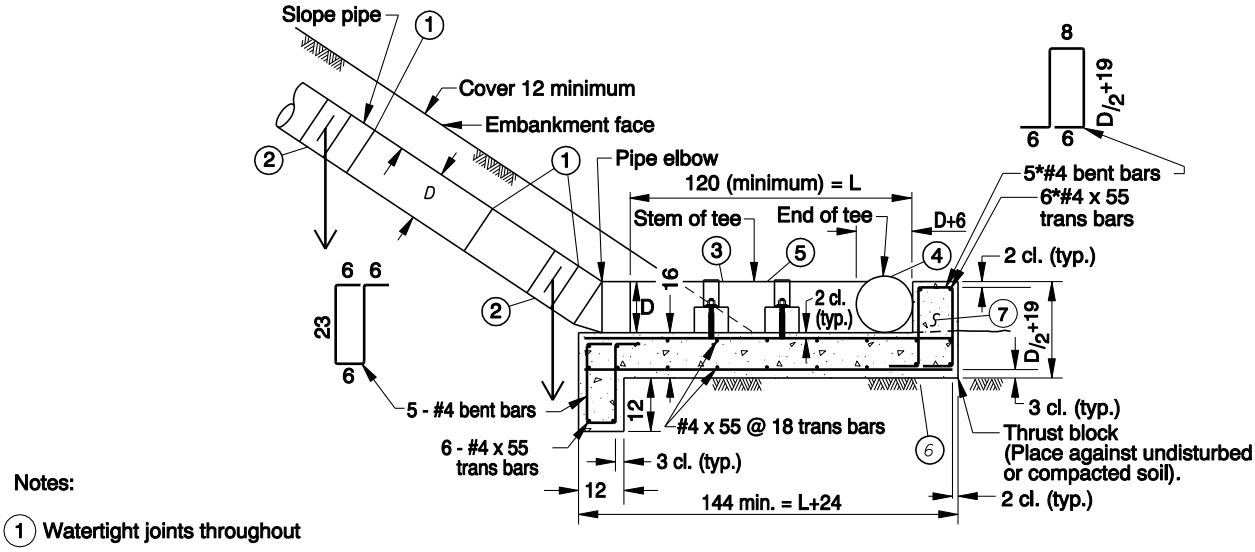

c) Slip Joint

General Notes:

1. The slip joint will normally be used only in areas where the design calls for a surcharge, counter-balance, or in known slide areas.
2. When the conditions in ① exist, contact the geotechnical engineer to determine the need for a slip joint if it has not been addressed by the region geologist or locating engineers' narrative.
3. The dimensions A & B are dependent upon the type of embankment to be placed. nominal measurements are A = 8 ft, B = 6 ft. This should be discussed when the need for the slip joint is being determined.
4. Wrap slip joint with geotextile (type??). Fasten geotextile to pipe with (metal bands ??) at each end. Attach extra fabric to allow for XXXmm of joint expansion.

"All dimensions are in inches unless otherwise noted."

**Figure 11-13 Slope Pipe, Cleanout Gate, and Slip Joint Details for Tee Dissipators**

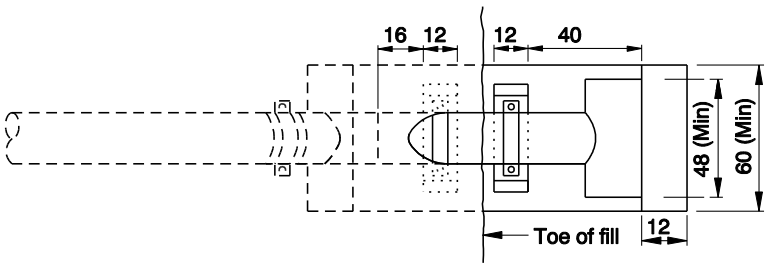



Notes:

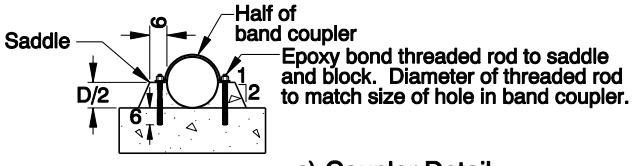
1. Watertight joints throughout
2. Metal pipe slope anchor. See RD330. 20 feet on centers, maximum.
3. Gradient of stem of tee = 1.0%
4. Gradient of end of tee = 0.0%
5. Minimum length of stem = 10 feet. If there is adequate space, it is recommended that the stem project at least 10 feet beyond the embankment toe
6. Required ultimate bearing capacity shall be 2089 lbs/ft$^2$.
7. Use 3.6 Ksi concrete

Note:
Epoxy coat all bars
D=Diameter of slope pipe, inches

a) Elevation View




b) Plan View




c) Coupler Detail

All dimensions in inches unless noted otherwise

**Figure 11-14 Lower End of Tee Dissipator**

Vegetation can be planted around the edges of the riprap. The vegetation should not obstruct the flow off of the riprap unless the effects of this obstruction are considered in the hydraulic design.

## 11.11 Design of Riprap Pads

The design of riprap pads is presented in this section. Guidance on the selection of these dissipators is provided in Section 11.5 and pads are shown in Figures 11-3 and 11-16. These dissipators protect the outlet of the conduit or channel from undermining and they also dissipate considerable flow energy. As a result, considerably more riprap is used in these pads than in the outlets into unlined pools that were discussed in the previous section.

The first step in designing a riprap pad is to determine the flow characteristics at the outlet of the upstream conduit or channel, such as the brink depth and the Froude number. The methods used to determine these characteristics are in Section 11.7. Actual brink depths and Froude numbers should be used for rectangular conduits and channels, and equivalent brink depths and equivalent Froude numbers should be used for non-rectangular conduits and channels.

The second step is to determine if a riprap pad, riprap lined basin, or a complex reinforced concrete dissipator is needed using the selection charts in Figures 11-17 through 11-20.

The third step is to determine the dimensions of the riprap pad using Figure 11-16.

Vegetation can be planted on the highway embankment as discussed in **Chapter 9**. Vegetation can also be planted around the outlet of the conduit or channel or along the edges of the pad. This vegetation should not obstruct the flow unless the effects of the obstruction are considered in the hydraulic study. Large vegetation should not be planted too close to any structure such as conduits, lined channels, or conduit and channel end treatments. The plants could grow large enough to damage the structure. As a general rule, large trees such as pacific willows, red alders, western red cedars, etc should not be planted within 10 feet of the structure, and large bushes such as sitka willows, pacific dogwoods, serviceberries, etc should not be planted within 3 feet from the structure. A structural designer should be consulted if more detailed guidance is needed.

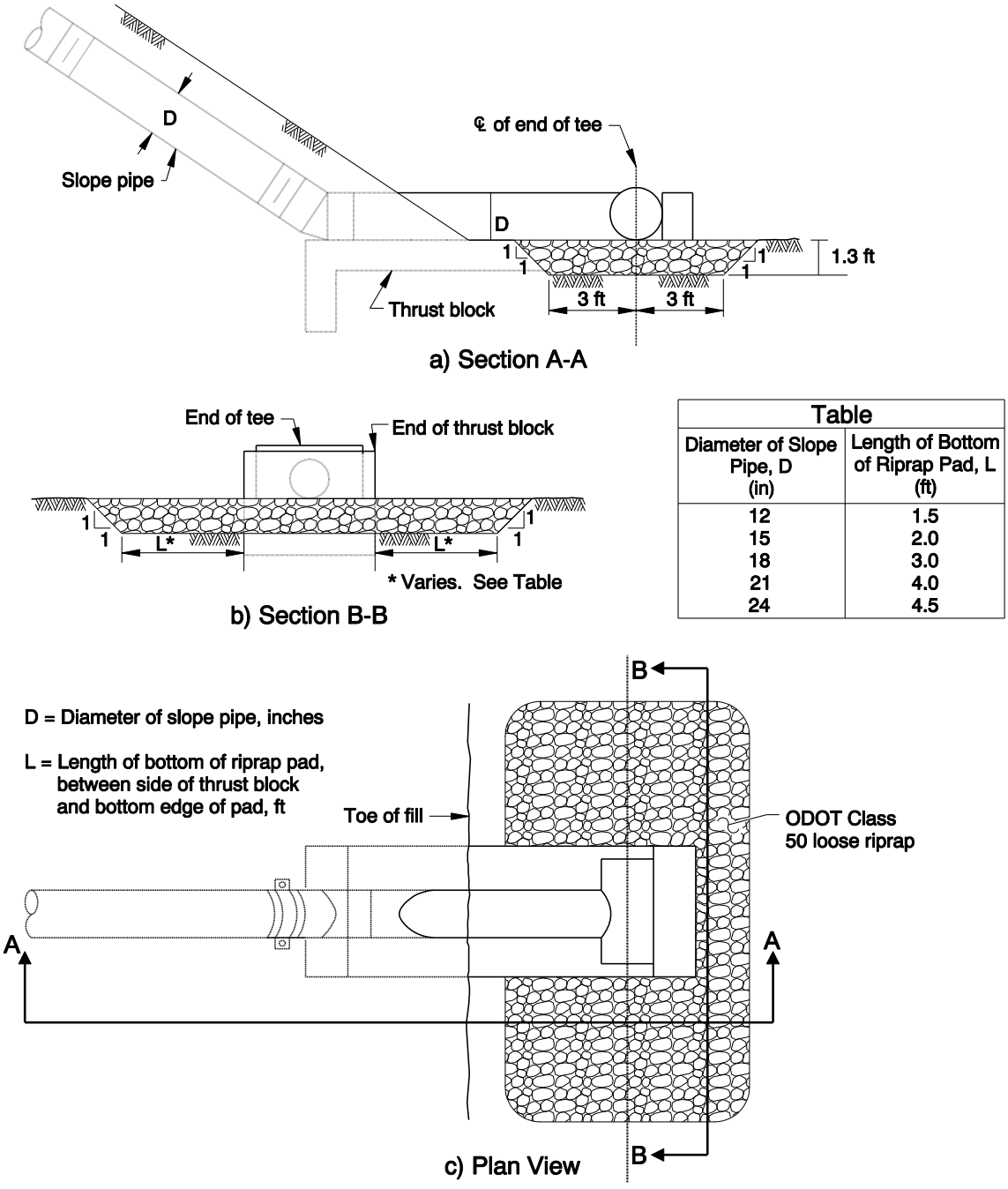
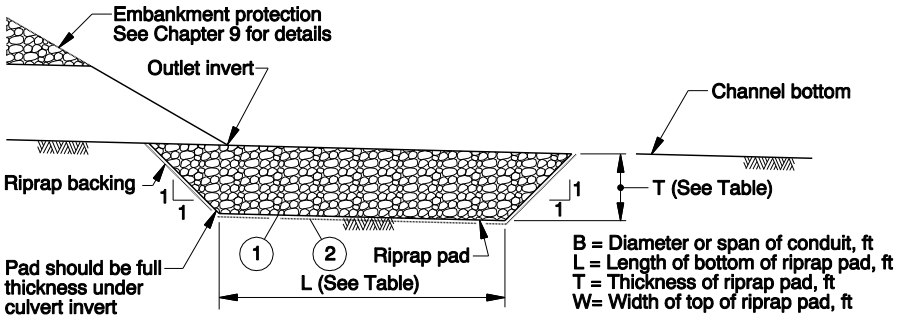
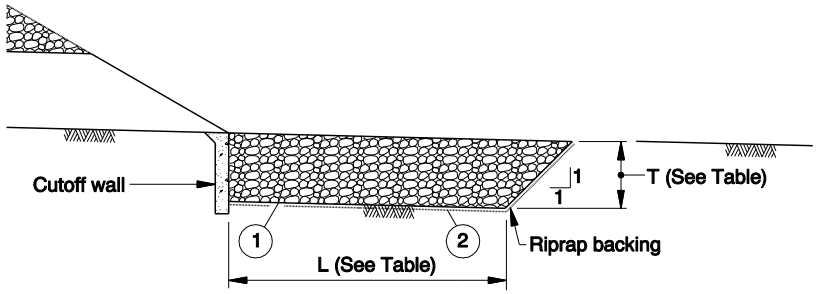


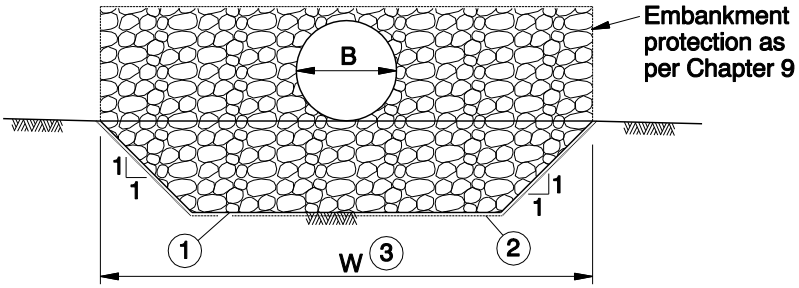

**Figure 11-15 Riprap Around Outlet of Tee Dissipator**




a) Riprap pad without cutoff wall, elevation view




b) Riprap pad with cutoff wall, elevation view




c) Riprap pad, end view

| TABLE | | |
|---|---|---|
| Riprap Class | L * (ft) | T (ft) |
| 50 | 4B or 1.3 | 2.3 |
| 100 | 4B or 1.6 | 3.3 |
| 200 | 4B or 2.0 | 4.3 |
| 700 | 4B or 3.3 | 5.6 |

* L is the greater of 4B or the listed dimension.

Notes:

1. Do not excavate non-erodible rock in order to place riprap.
2. Use riprap backing under Class 200 and Class 700 loose riprap.
3. Top width of the riprap pad is the larger of 5B or the width of the embankment protection (see Chapter 9).

**Figure 11-16 Dimensions of Riprap Pads**

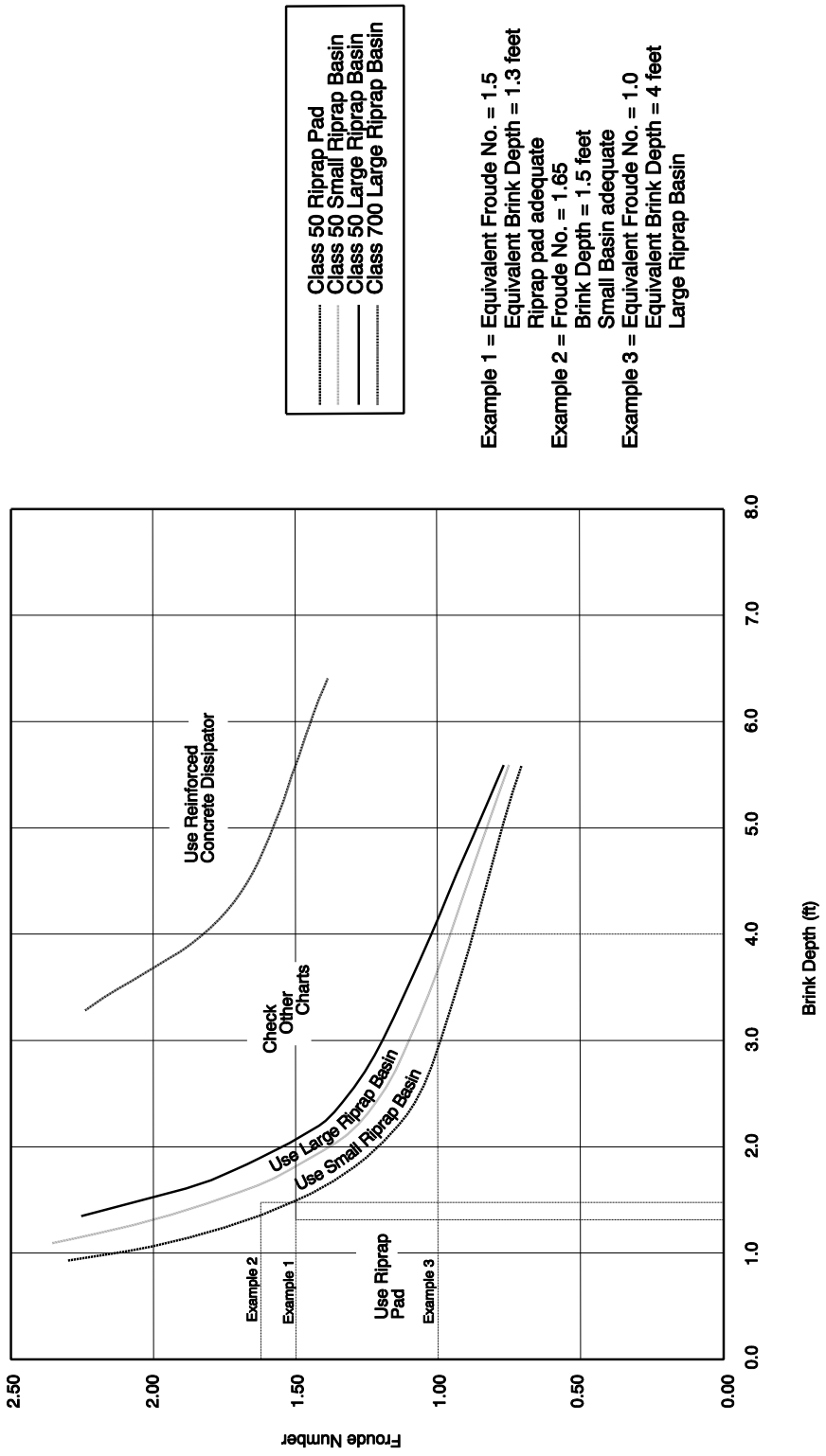



**Figure 11-17 ODOT Class 50 Riprap Energy Dissipator Selection Chart**

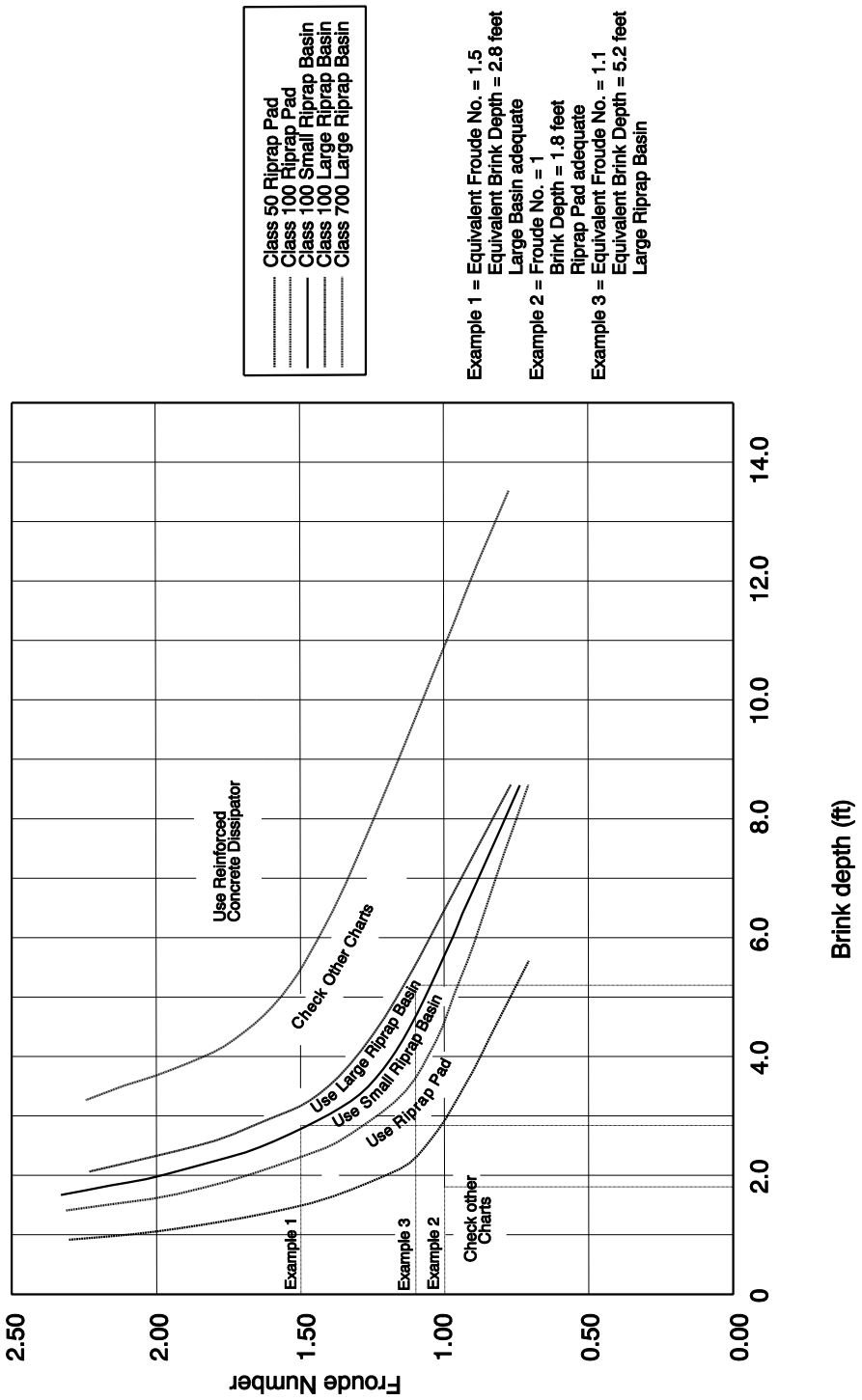



**Figure 11-18 ODOT Class 100 Riprap Energy Dissipator Selection Chart**

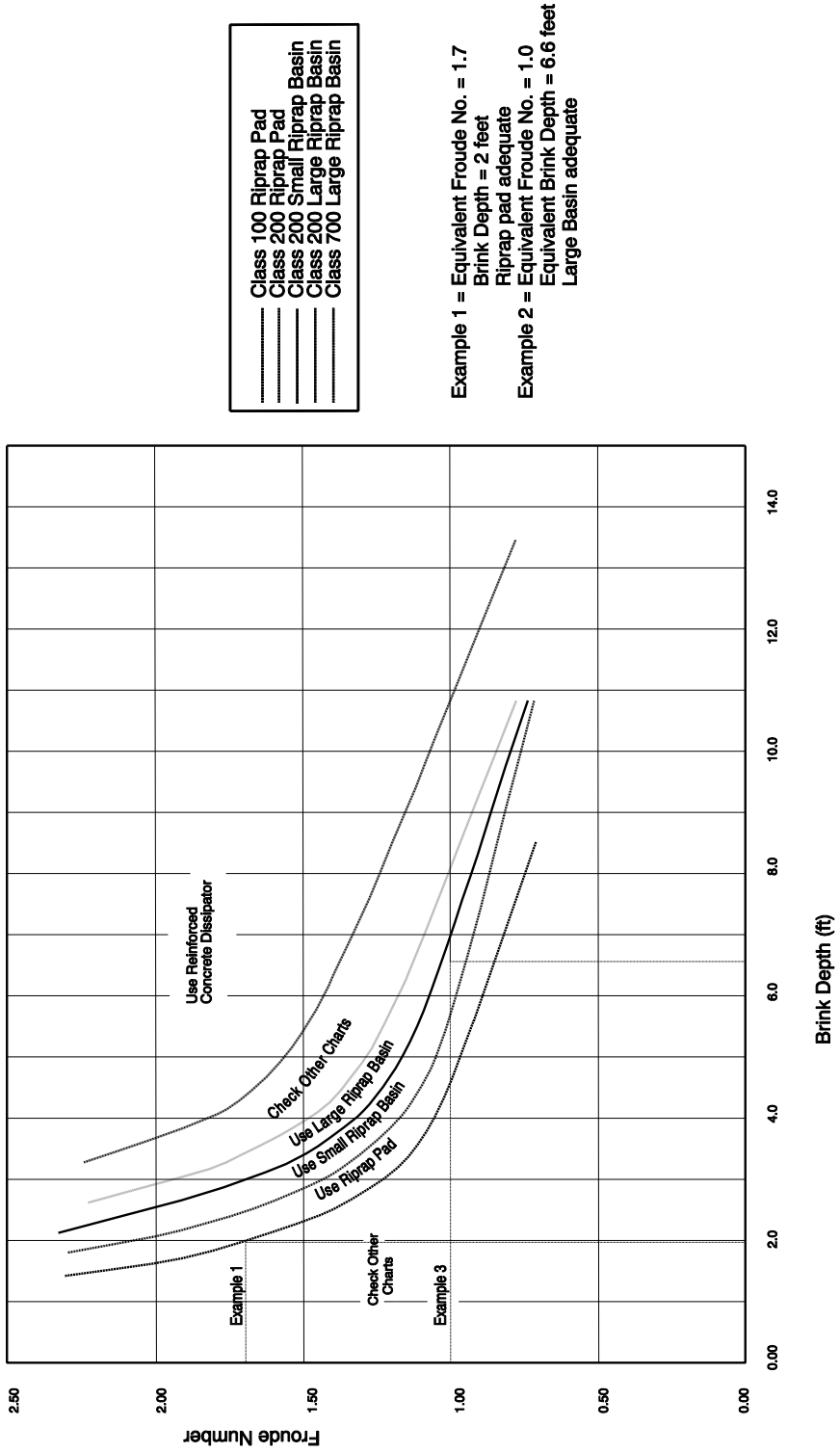



**Figure 11-19 ODOT Class 200 Riprap Energy Dissipator Selection Chart**

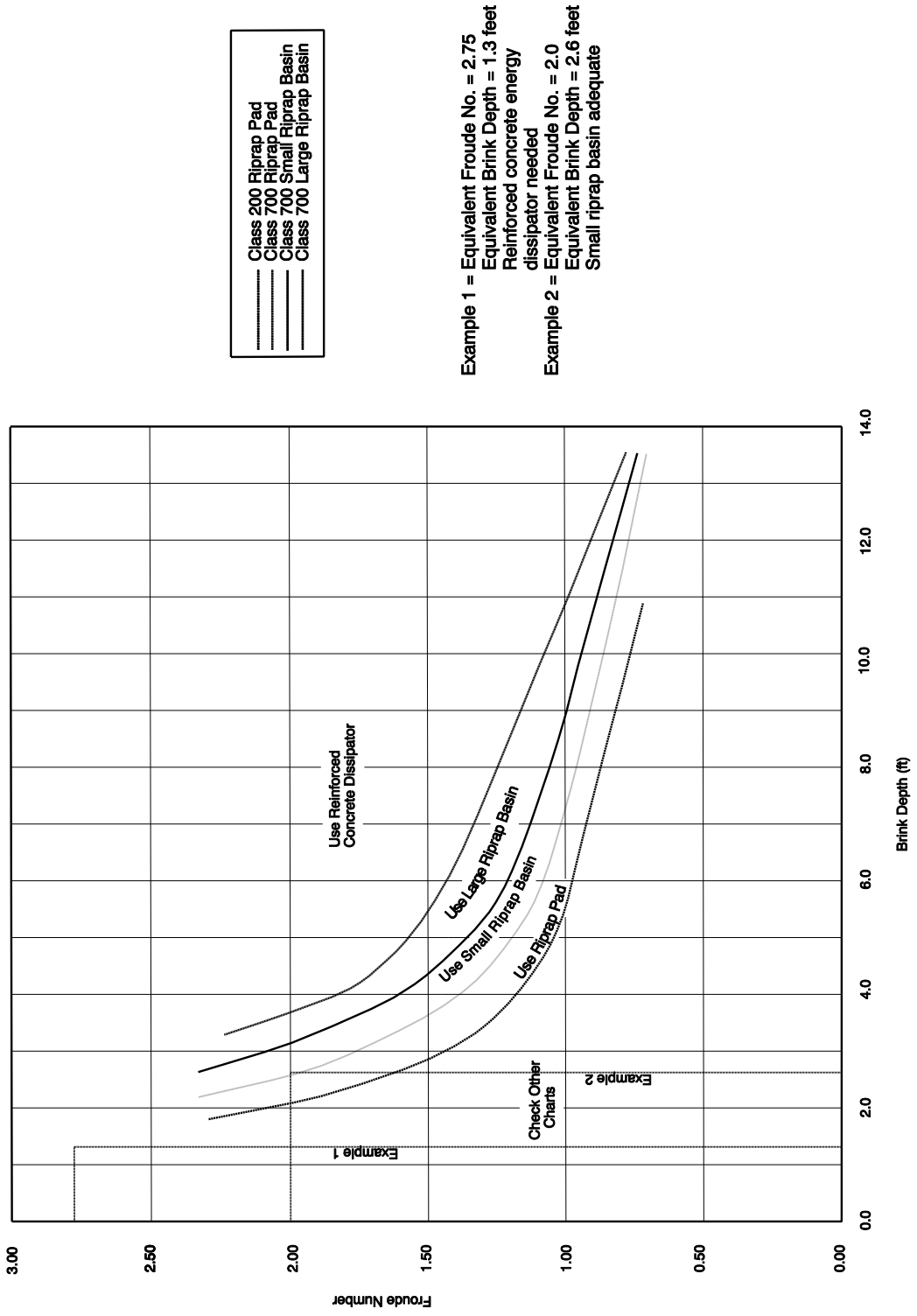



**Figure 11-20 ODOT Class 700 Riprap Energy Dissipator Selection Chart**

### 11.11.1 Example: Designing a Riprap Pad

The storm sewer outfall from the Bat Hills Subdivision has a design flow of 39 cubic feet per second out of a 36-inch diameter circular corrugated metal pipe on a 0.05 foot per foot slope. It is assumed the tailwater depth is negligible. A riprap pad needs to be designed to dissipate the flow energy and prevent a scour hole from developing.

*Note: The procedure in this example does not address erosion that could occur downstream from the pad. This topic is discussed in Subsection 11.12.1.*

**Step 1:** The equivalent brink depth and equivalent Froude number are needed. The calculation procedure is presented in detail in Section 11.7 and it is summarized for this example as follows:

Full flow without surcharge using Manning's equation for discharge, a roughness coefficient of 0.021, and an $AR^{2/3}$ value of 5.8:

$$Q_{FULL} = \left(\frac{1.49}{0.021}\right)(5.8)\,(0.05^{0.5}) = 92 \text{ cubic feet per second}$$

Depth of normal flow in the conduit using the hydraulic elements chart for circular pipe:

$$\frac{q}{Q_{FULL}} = \frac{39}{92} = 0.42 \quad \text{Using the chart, } \frac{d}{D_{FULL}} = 0.46 \quad d = (0.46)\,(3) = 1.38 \text{ feet}$$

Depth of critical flow using the critical depth chart for circular pipe:

$D_c$ = 2 feet

Since d is less than $D_c$ and there is negligible tailwater, there is Case 1 flow at the brink and the flow area is assumed to be the area of normal flow. Using the hydraulic elements chart for circular pipe:

$$A_{FULL} = 7.07 \text{ square feet} \quad \frac{q}{Q_{FULL}} = \frac{39}{92} = 0.42 \quad \text{Using the chart, } \frac{a}{A_{FULL}} = 0.45$$

$$a = (0.45)\,(7.07) = 3.2 \text{ square feet}$$

$$\text{The equivalent brink depth : } y_e = \left(\frac{3.2}{2}\right)^{0.5} = 1.3 \text{ feet}$$

$$v = \frac{Q}{a} = \frac{39}{3.2} = 12.2 \text{ feet per second}$$

The equivalent Froude number : $Fr_e = \frac{12.2}{[(32.2)\ (1.3)]^{0.5}} = 2$

**Step 2:** In this step the equivalent brink depth and equivalent Froude number are used with the charts in Figures 11-17 through 11-20 to determine if a riprap pad can be used and the class of the riprap in the pad. Based on the charts, a riprap basin will be needed if Class 50 riprap is used and riprap pads are adequate if Class 100, 200, and 700 riprap are used. A riprap pad using ODOT Class 100 loose riprap is selected for the design.

**Step 3:** The riprap pad dimensions are determined in this step using Figure 11-16. The length of the bottom of the pad, L, will be four times the diameter of the culvert:

L = 4B = (4) (3) = 12 feet

The thickness, T, of the pad is 3.3 feet (See table in figure 11-16).

The width of the embankment revetment is five times the diameter of the culvert, as shown in Figure 9-21 of **Chapter 9**. The total width of the riprap pad, W, will also be this width.

W = 5B = (5) (3) = 15 feet

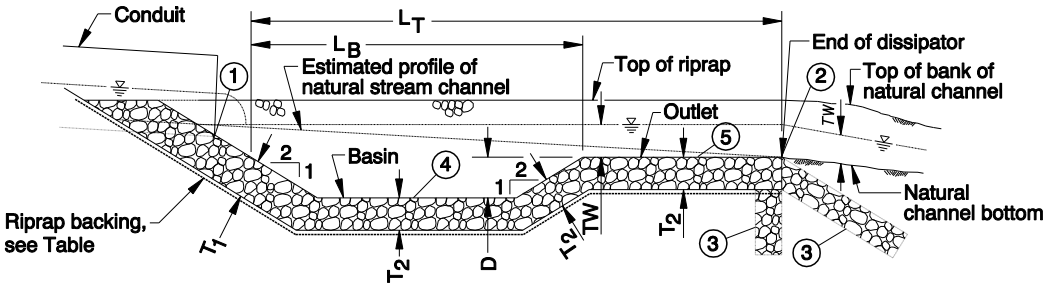
## 11.12 Design of Riprap Lined Basins

The design of a riprap lined basin is presented in this section. The selection of this type of dissipator is discussed in Section 11.5 and basins are shown in Figure 11-4 and Figures 11-21 through 24. These dissipators protect the outlet of the conduit or channel from undermining and they dissipate considerable flow energy.

The first step in riprap lined basin design is to determine the flow characteristics at the outlet of the upstream conduit or channel, such as the brink depth and the Froude number. The methods used to determine these flow characteristics are in Section 11.7. Actual brink depths and Froude numbers should be used for rectangular conduits and channels, and equivalent brink depths and equivalent Froude numbers should be used for non-rectangular conduits and channels.

The second step is to determine if a riprap pad, riprap lined basin, or a complex reinforced concrete dissipator is needed using the selection charts in Figures 11-17 through 11-20.

The third step is to determine the dimensions of the riprap lined basin using Figures 11-21 through 11-24. The dissipator outlet should be wide enough to match the existing channel. If




For dimensions, see Table

B = Diameter or span of conduit outlet, ft See Figure 11-23
D = Depth of basin below elevation of end of dissipator, ft
$L_T$ = Total length of dissipator, ft
$L_B$ = Length of basin, at elevation of end of dissipator, ft
$T_1$ = Thickness of revetment blanket on inlet side of basin, ft
$T_2$ = Thickness of blanket on remainder of basin, ft
TW = Tailwater depth during design flow, ft

Notes:

(1) Outlet invert is at elevation of natural stream bottom

(2) End of dissipator is at same elevation as bottom of natural channel

(3) Thickened or sloping toe optional - construct if degradation is expected in downstream channel. Extend toe downward to an elevation lower than expected degradation.

(4) Basin bottom is flat and at a depth "D" below the elevation of the end of the dissipator

(5) Outlet bottom is flat and at same elevation as the end of the dissipator

TABLE

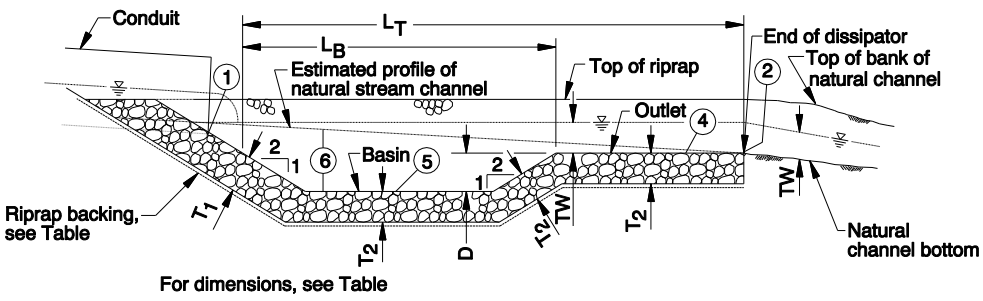
| ODOT Riprap Class | D in feet | | $L_T$ ** in feet | | $L_B$* in feet | | $T_1$ in feet | $T_2$ in feet |
|---|---|---|---|---|---|---|---|---|
| | Small Basin | Large Basin | Small Basin | Large Basin | Small Basin | Large Basin | Large and Small Basin | Large and Small Basin |
| 50 | 1.6 | 2.3 | 26 or 4B | 33 or 4B | 16 or 3B | 23 or 3B | 1.3 | 1.6 |
| 100 | 2.6 | 3.3 | 39 or 4B | 49 or 4B | 26 or 3B | 33 or 3B | 1.6 | 2.6 |
| 200 *** | 3.3 | 4.3 | 49 or 4B | 62 or 4B | 62 or 3B | 43 or 3B | 2.3 | 3.3 |
| 700 *** | 3.9 | 5.2 | 59 or 4B | 79 or 4B | 39 or 3B | 52 or 3B | 2.6 | 3.9 |

*$L_T$ is the greater of:
4B or the listed dimension

**$L_B$ is the greater of:
3B or the listed dimension

*** Riprap backing required

**Figure 11-21 Elevation View of Riprap Lined Basins - Fish Passage Not Required**




B = Diameter or span of conduit outlet, ft See Figure 11-23
D = Depth of basin below elevation of end of dissipator, ft
$L_T$ = Total length of dissipator, ft
$L_B$ = Length of basin, at elevation of end of dissipator, ft
$T_1$ = Thickness of revetment blanket on inlet side of basin, ft
$T_2$ = Thickness of blanket on remainder of basin, ft
TW = Tailwater depth during design flow, ft

Notes:

1 Outlet invert is set at elevation that produces jump pool with desired depth. See Appendix D, Chapter 9.

2 End of dissipator is at same elevation as bottom of natural stream channel if no degradation is expected in downstream channel. End of dissipator is at elevation of degraded channel if degradation is expected.

3 $L_B$ must not be less than the jump pool length needed for fish passage. See Appendix D, Chapter 9.

4 Outlet bottom is flat and at same elevation as end of dissipator.

5 Basin bottom is flat and at a depth "D" below the lower of: the outlet invert of the conduit, or the end of the dissipator.

6 The total jump pool depth should be checked at the fish passage design flow. The total jump pool depth must not be less than the minimum required for fish passage. See Appendix D of Chapter 9.

TABLE

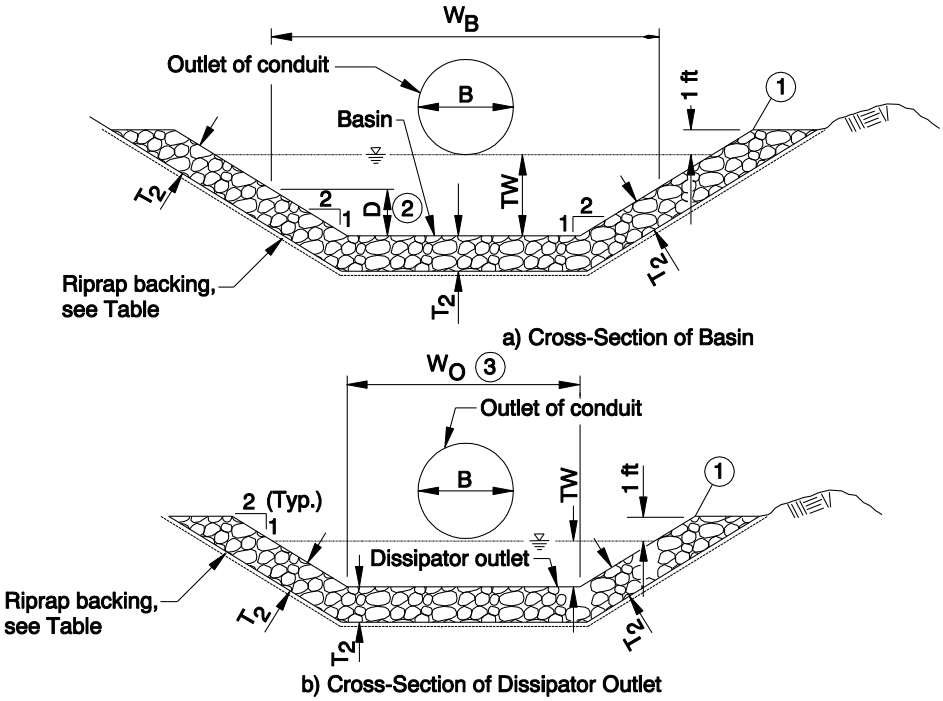
| ODOT Riprap Class | D in feet | | $L_T$ * in feet | | $L_B$** in feet | | $T_1$ in feet | $T_2$ in feet |
|---|---|---|---|---|---|---|---|---|
| | Small Basin | Large Basin | Small Basin | Large Basin | Small Basin | Large Basin | Large and Small Basin | Large and Small Basin |
| 50 | 1.6 | 2.3 | 26 or 4B | 33 or 4B | 16 or 3B | 23 or 3B | 1.3 | 1.6 |
| 100 | 2.6 | 3.3 | 39 or 4B | 49 or 4B | 26 or 3B | 33 or 3B | 1.6 | 2.6 |
| 200*** | 3.3 | 4.3 | 49 or 4B | 62 or 4B | 33 or 3B | 43 or 3B | 2.3 | 3.3 |
| 700*** | 3.9 | 5.2 | 59 or 4B | 79 or 4B | 39 or 3B | 52 or 3B | 2.6 | 3.9 |

*$L_T$ is the greater of:
4B or the listed dimension

**$L_B$ is the greater of:
3B or the listed dimension

*** Riprap backing required

**Figure 11-22 Elevation View of Riprap Lined Basin - Fish Passage Required**




B = Diameter or span of conduit outlet, ft
D = Depth of basin, ft See Figure 11-21 if fish passage not required. See Figure 11-22 if fish passage required.
$T_2$ = Thickness of revetment blanket, ft See Figure 11-21 or 11-22
TW = Tailwater depth during design flow, ft
$W_B$ = Basin width, at elevation of dissipator outlet, ft See Table
$W_O$ = Outlet width, at bottom of outlet, ft See Note (3)

Notes:

1 Extend riprap up side of basin to an elevation 1 foot higher than elevation of design tailwater

2 See Table, Figures 11-21 or 11-22

3 The width of the dissipator outlet:
- is the width of the downstream channel bottom if a channel exists, or
- is wide enough to create outlet flow velocities that do not erode the materials downstream from the outlet, or
- if fish passage is needed, the outlet is wide enough to create the desired jump pool depth

TABLE

| ODOT Riprap Class | $W_B$* in feet | |
|---|---|---|
| | Small Basin | Large Basin |
| 50 | 17 or 3B | 23 or 3B |
| 100 | 26 or 3B | 33 or 3B |
| 200** | 33 or 3B | 43 or 3B |
| 700** | 40 or 3B | 53 or 3B |

*$W_B$ is the greater of:
3B or the listed dimension
**Riprap backing required

**Figure 11-23 Cross-Sections of Riprap Lined Basin**

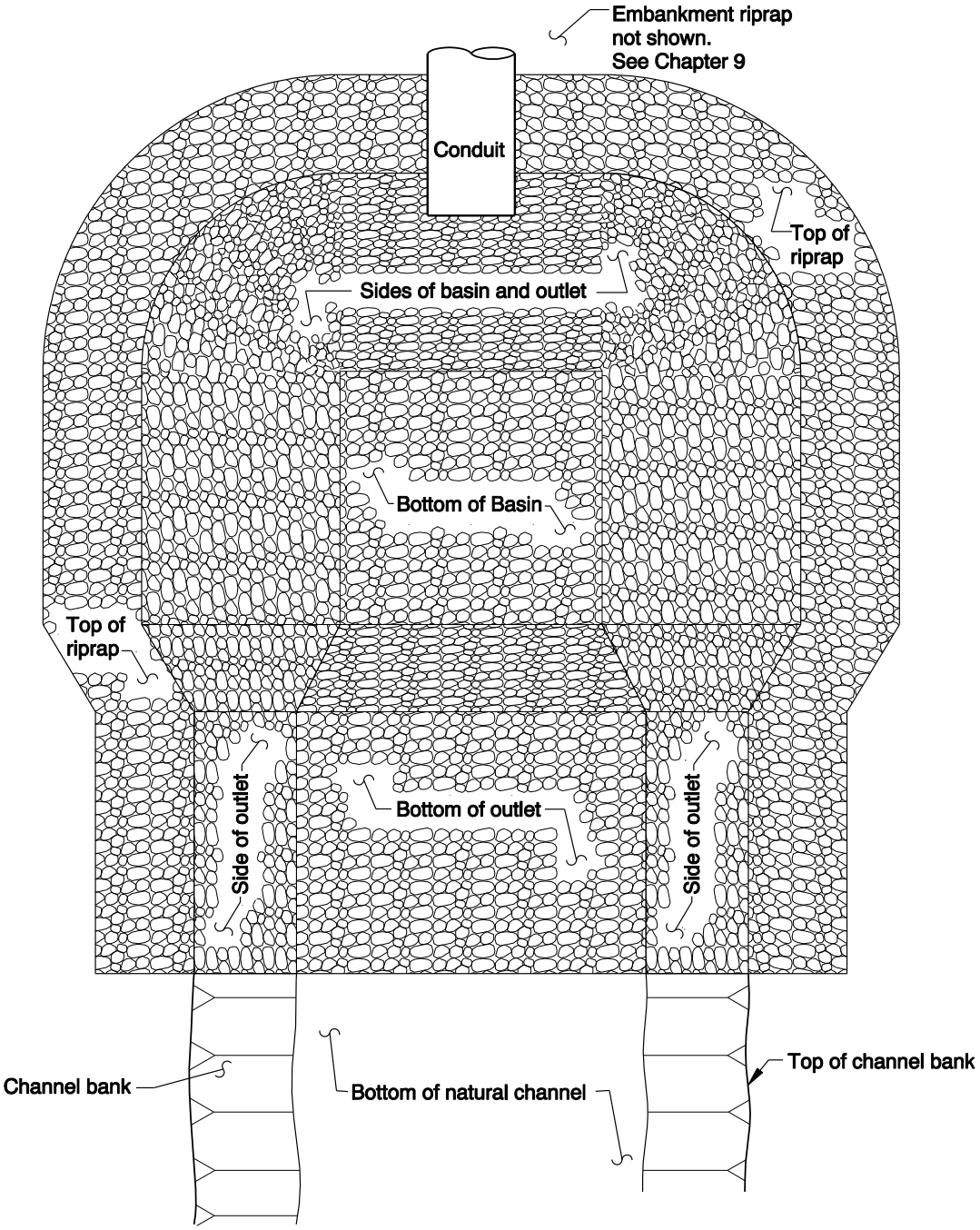



**Figure 11-24 Plan View of Riprap Lined Basin**

there is no existing channel, the need for a new outlet channel should be evaluated. The outlet from the dissipator should be sufficiently wide so there are non-erosive velocities at the upstream end of the channel, or the surface the water flows over if there is no channel. A method of determining this width is described in the following subsection.

The fourth step is to determine the depth of the water in the basin, and this depth typically depends on these factors: the discharge, the width of the outlet, and the depth of the tailwater above the bottom of the basin outlet. Three conditions usually occur:

1. The outlet is unsubmerged. This is the most common flow condition. It occurs when the dissipator outlet is less than 76 percent submerged by the tailwater. During this condition it is assumed the basin outlet is an unsubmerged broad-crested trapezoidal weir.

2. The outlet is partially submerged. This condition occurs when the dissipator outlet is not over 100 percent submerged or less than 76 percent submerged. During this condition it is assumed the outlet is a partially submerged broad-crested trapezoidal weir.

3. The outlet is fully submerged. This condition occurs when the outlet is over 100 percent submerged. During this condition it is assumed the basin outlet does not function as a weir. It is assumed the elevation of water in the dissipator basin is the same as the elevation of the tailwater.

The percent submergence of the dissipator outlet is calculated by the following equation:

$$\text{Percent Submergence} = 100\left(\frac{TW}{H}\right) \qquad \text{(Equation 11-11)}$$

Where:

TW = Depth of the tailwater above the bottom of the dissipator outlet, in feet
H = Difference in elevation between the surface of the water in the dissipator and the bottom of the outlet, in feet.

**Unsubmerged or Partially Submerged Outlet -** If it is assumed the dissipator outlet is unsubmerged or partially submerged, the elevation of the water surface in the dissipator is:

$$EL_{Water\ Surface} = EL_{Dissipator\ Outlet} + H \qquad \text{(Equation 11-12)}$$

Where:

$EL_{Water\ Surface}$ = The elevation of the water surface in the dissipator in feet
$EL_{Dissipator\ Outlet}$ = The elevation of the bottom of the dissipator outlet in feet
H = Difference in elevation between the water surface in the basin and the bottom of the dissipator outlet in feet. See Equation 11-13.

The distance H is calculated as follows:

$$H = \left[ \frac{Q}{(C_s C_f W_O)} \right]^{2/3} \qquad \text{(Equation 11-13)}$$

Where:

H = Difference in elevation between water surface in the basin and the bottom of the dissipator outlet in feet

Q = The dissipator design discharge, in cubic feet per second

$C_f$ = Weir coefficient. See Figure 11-25a.

$C_s$ = Submergence factor.

If percent submergence is less than 76, $C_s = 1.0$

If 76 is less than or equal to percent submergence is less than or equal to 100, $C_s$ varies. See Figure 11-25b.

If percent submergence is more than 100, assume the outlet is fully submerged.

$W_O$ = Width of the bottom of the dissipator outlet, in feet

The preceding equations are used in an iterative manner to determine the elevation of the water in the basin, as follows: $C_f$ and $C_s$ are assumed, H is calculated, and $C_f$ and $C_s$ are checked to see if the initial assumptions are correct. If the initial assumptions are not correct, $C_f$ and $C_s$ are reestimated and H is recalculated. The process is repeated until H, $C_f$, and $C_s$ are in agreement.

**Submerged Outlet -** If the dissipator outlet is submerged, the water surface elevation in the dissipator is:

$EL_{Water\ Surface}$ = Tailwater Elevation

Where:

$EL_{Water\ Surface}$ = The elevation of the water surface in the dissipator in feet

Once the water surface elevation in the dissipator has been established, the minimum elevation of the top of the revetment on the sides of the dissipator can be determined. The revetment should extend upward along the sides of the dissipator to an elevation at least 1 foot higher than the water surface elevation, as shown in Figure 11-23.

These basins can also be used to provide a stable jump pool for fish passage at the downstream end of a culvert.

Vegetation can be planted on the highway embankment as discussed in **Chapter 9**. Vegetation can also be planted at the edges of the basin and around the outlet of the conduit or channel. The vegetation around the outlet should not obstruct the flow unless these effects have been

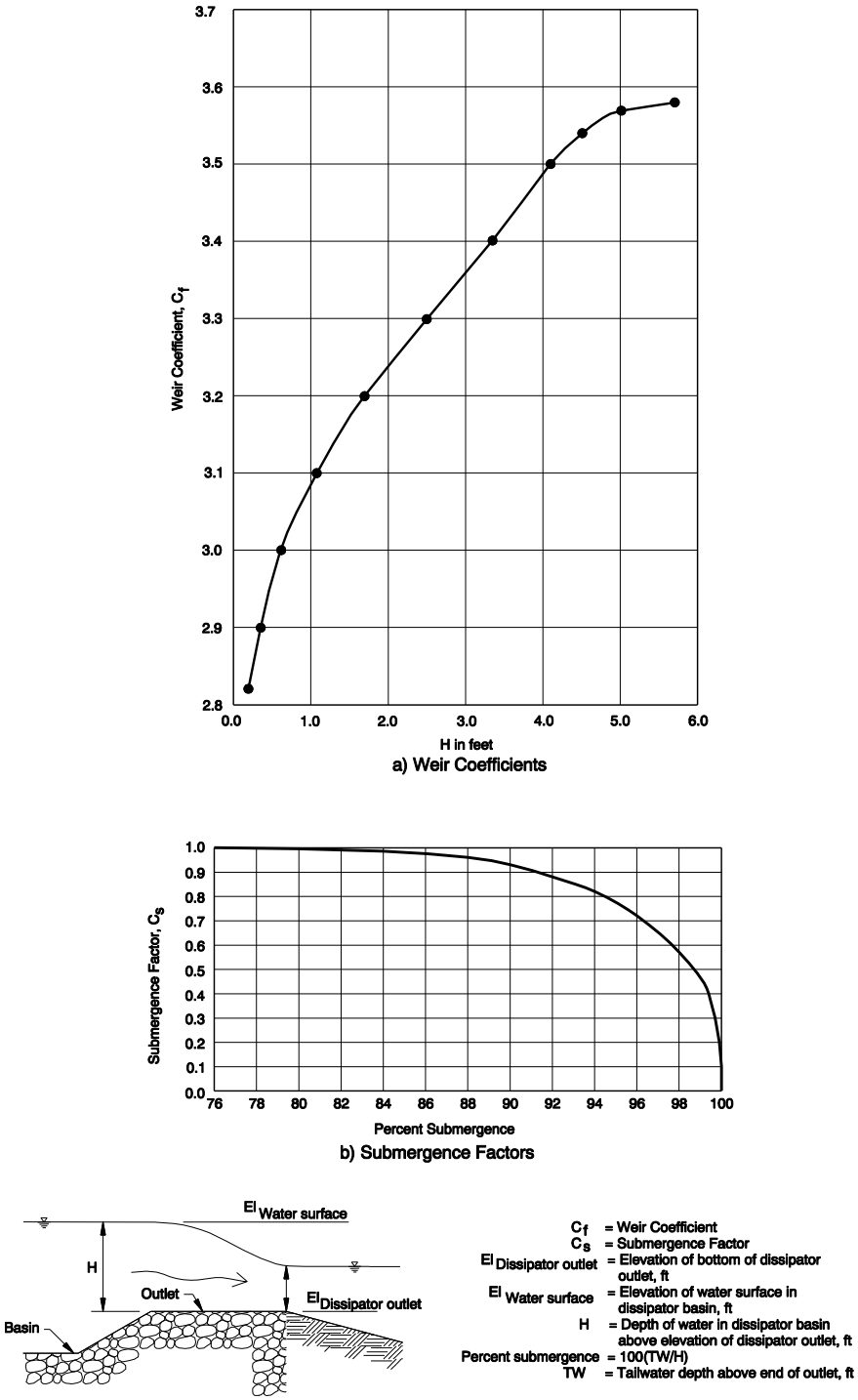



**Figure 11-25 Weir Coefficients and Submergence Factors for Unsubmerged and Partially Submerged Riprap Lined Basin Outlets**

considered in the hydraulic study. Large vegetation should not be planted too close to structures such as conduits, lined channels, or conduit and channel end treatments. The plants could grow large enough to damage the structure. As a general rule, large trees such as pacific willows, red alders, western red cedars, etc should not be planted within 10 feet of the structure, and large bushes such as sitka willows, pacific dogwoods, serviceberries, etc should not be planted within 3 feet from the structure. A structural designer should be consulted if more detailed guidance is needed.

### 11.12.1 Scour Protection Downstream From Basin Outlet

In most cases the dissipator discharges into a channel and the width of the bottom of the dissipator outlet is the same as the width of the bottom of the channel. In some instances, however, the dissipator discharges into an open area where there is no channel and the flow is not constrained laterally. This open area could be a field, an embankment, a grassy swale, etc. If this occurs, the dissipator outlet should be sufficiently wide so it does not concentrate the flow and cause erosion of the downstream open area.

The basic concept of preventing scour downstream from the dissipator outlet is to assure that the shear stress exerted by the water on the underlying material is less than the shear stress the material can withstand. Although this concept appears to be simple, a precise mathematical analysis is quite complex. This manual presents a simplified analysis method that is sufficiently accurate for most applications.

The designer should be aware that if preventing erosion downstream from the dissipator is especially critical, a more detailed analysis may be warranted. This analysis may require a more precise determination of the erosion or scour at the end of the dissipator as well as an analysis of the scour or erosion caused by the flow through the entire area of concern.

The simplified method is as follows:

**Step 1:** Determine the maximum permissible shear stress that the material downstream from the dissipator outlet can withstand, $\tau_p$, in pounds per square foot. The figures and tables in **Chapter 8** can be used for most materials. Shear stresses for temporary erosion control mats, grasses, and riprap are listed in **Chapter 8**. Shear stresses for non-cohesive and cohesive soils are shown in **Chapter 8**. These charts are applicable for flows up to 50 cubic feet per second. Allowable shear stresses for materials subject to higher flows are listed in most texts about river mechanics and sediment transport.

**Step 2:** Determine the maximum depth of flow that the underlying material downstream from the outlet can tolerate, as follows:

$$y = \frac{\tau_p}{\gamma(S)} = \frac{\tau_p}{(62.4)(S)}$$ (See Chapter 8)

Where:

y = Maximum depth of flow in feet

$\tau_p$ = Maximum permissible shear stress on underlying material in pounds per square foot

S = Slope of energy grade line in foot per foot. In most cases this slope can be assumed

**Step 3:** Determine the bottom width of a dissipator outlet that provides a flow depth equal to or less than the maximum depth determined in the previous step. It can be assumed the outlet has a trapezoidal shape and the flow is at normal depth. The depth can be determined by a trial-and-error solution using Manning's equation as presented in **Chapter 8**, or computer analysis. Regardless of the method used, the input variables are defined as follows:

y = Maximum depth of flow in feet

B = $W_o$ = Outlet bottom width at end of the dissipator, in feet

Q = Design discharge in cubic feet per second

N = Manning's roughness coefficient for the surface downstream from the dissipator outlet that the water flows over

S = Slope of energy grade line in foot per foot. This slope can often be assumed to be the slope of the surface downstream from the dissipator outlet that the water flows over.

The dissipator outlet can be quite wide if there is substantial discharge from the dissipator and the downstream surface is highly erodible. If this wide outlet is impractical, a more erosion resistant surface or channel may be needed downstream from the dissipator. This situation is discussed in the following example.

### 11.12.2 Example: Designing a Riprap Lined Basin

A riprap lined basin will be designed for the storm sewer outfall from the Bat Hills Subdivision. ODOT Class 50 riprap will be used. The design discharge is 39 cubic feet per second out of a 36-inch diameter circular corrugated metal pipe on a 0.05 foot per foot slope. The tailwater depth is assumed to be negligible. The discharge will leave the dissipator outlet and flow down the side of a grass lined swale. The swale is covered with a fair stand of mowed grass and it has a slope of 6 foot horizontal distance to 1 foot vertical distance. The grass is allowed to grow to a height of 1 foot before it is mowed.

**Basin Design Step 1:** The first step is to determine the flow characteristics at the brink of the storm sewer outlet. A riprap pad was designed for this outfall in the design example in Subsection 11.11.1. The equivalent flow depth, $y_e$, and equivalent Froude number, $Fr_e$, were calculated in the prior example, and they are:

$y_e$ = 1.3 feet, and $Fr_e$ = 2.0

**Basin Design Step 2:** In this step, Figure 11-17 is used to determine if a small or large basin is needed. Based on the Figure and the $y_e$ and $Fr_e$ values from Step 1, a large basin will be adequate.

**Basin Design Step 3:** In this step, Figures 11-21, 11-23, and 11-24 are used to determine the dimensions of the large basin. If a small basin were to be designed, its dimensions would be determined in a similar manner.

Based on Figure 11-21, the basin depth = D = 2.3 feet.

The total length of dissipator, $L_T$, is the greater of 33 feet or 4B. 4B = (4) (3 ft) = 12 feet
$L_T$= 33 feet

The length of basin, $L_B$, is the greater of 23 feet or 3B. 3B = (3) (3) = 9 feet $L_B$ = 23 feet

The thickness of the riprap blanket on the upstream end of the dissipator = $T_1$ = 1.3 feet

The thickness of the riprap blanket on the remainder of the dissipator = $T_2$ = 1.6 feet

Based on Figure 11-23, the basin width, $W_B$, is the greater of 23 feet or 3B.
3B = (3) (3) = 9 feet $W_B$= 23 feet

The outlet width, $W_O$, will be wide enough to have non-erosive velocities at the outlet to the channel. This width will be calculated using the method in Subsection 11.12.1 as follows:

**Outlet Design Step 1:** In this step the maximum shear stress the surface of the swale can withstand is calculated. The grass on the side of the swale has a retardance of C, based on Retardance Classes of Grass Channel Linings Table of **Chapter 8**. The maximum shear this covering can withstand is 1.0 pounds per square foot, based on Permissible Shear Stresses for Linings Table of **Chapter 8**.

**Outlet Design Step 2:** In this step the maximum depth of flow down the side of the swale is calculated. The slope of the energy grade line is assumed to be the same as the 1V: 6H

slope of the swale face. 1 / 6 = 0.17 S = 0.17 foot per foot Using Equation 11-10, the maximum allowable flow depth is:

$$y = \frac{1}{(62.4)(0.17)} = 0.09 \text{ feet}$$

**Outlet Design Step 3:** In this step the minimum width of the bottom of the dissipator outlet, $W_O$, is determined. It is assumed the deepest flow, and consequently, highest shear stress, occurs immediately beyond the end of the dissipator outlet. As a result, the flow depth at this location should not be deeper than the maximum depth determined in the previous step. The flow out of the end of the dissipator will be modeled by Manning's equation assuming normal flow occurs. The input into Manning's equation follows.

The maximum flow depth is 0.09 feet.

The slope of the swale face is the slope of the energy grade line, 0.17 foot per foot.

The shape of the outlet is a trapezoid with 1V: 2H side slopes.

The design discharge is 39 cubic feet per second.

A Manning's roughness coefficient of 0.15 is selected, based on the values in **Chapter 8**. This is the roughness value for sheet flow over a meadow or pasture. It is selected because the 1 foot grass height is considerably larger than the 0.09 foot flow depth, and the flow down the swale face will resemble overland sheet flow rather than channel flow.

Using Manning's equation, the minimum width of the bottom of the outlet is 492 feet. It is impractical to build a dissipator outlet this wide.

A channel with a lining that can withstand a shear larger than 1 pound per square foot will have a smaller bottom width than 492 feet. A shallow grass lined trapezoidal channel with a permanent flexible liner will be designed using the methods in **Chapter 8** to convey the flow down the slope to the bottom of the swale. Liners are available that can withstand a shear stress of 7.3 pounds per square foot, and a liner that can withstand this stress will be specified for this channel. The design is as follows:

**Outlet Redesign Step 1:** The maximum allowable shear stress on the liner is 7.3 pounds per square foot.

**Outlet Redesign Step 2:** The maximum allowable flow depth is:

$$y = \frac{7}{(62.4)(0.17)} = 0.69 \text{ feet}$$

**Outlet Redesign Step 3:** The .69 foot flow depth in the channel is too deep to be overland sheet flow. Instead, it is channel flow. The Manning's roughness coefficients for grass lined highway channels are listed in Appendix A, **Chapter 8**. A fair stand of grass with a height of 24-inches is assumed because the vegetation in the channel may be trimmed with a brush cutter rather than mowed. Listed roughness coefficients are 0.09 for a flow velocity of 6 feet per second, and 0.17 for a velocity of 2 feet per second.

Outlet bottom widths are calculated for a 0.69 foot flow depth using roughness values of 0.09 and 0.17, as listed in the following table. Channel velocities are also calculated, and Manning's equation is used as described earlier in this example. Using a roughness value of 0.09, the calculated flow velocity is 4.85 feet per second. The applicable velocity for this roughness is 6 feet per second based on Appendix A, **Chapter 8**. Likewise, at a roughness value of 0.17, the calculated flow velocity is different than the applicable flow velocity. The velocities are also calculated for roughness values of 0.14, 0.15, and 0.16. These velocities are compared to interpolated velocities based on the values listed in Appendix A, **Chapter 8**. At a roughness value of 0.15 the calculated and interpolated flow velocities are similar. The outlet width at this roughness is 17.4 feet. This will be the width of the dissipator outlet and the channel bottom.

| Manning's Roughness Coefficient | Calculated Velocity (feet/sec) | Interpolated Velocity (feet/sec) | Difference Between Velocities | Outlet Width (feet) |
|---|---|---|---|---|
| 0.09* | 4.85 | 5.91* | - 1.06 | 10.3 |
| 0.17* | 2.67 | 1.97* | 0.70 | |
| 0.14 | 3.21 | 3.45 | - 0.24 | |
| 0.16 | 2.83 | 2.46 | 0.37 | |
| 0.15 | 3.01 | 2.95 | 0.06 | 17.4 |

*Values listed in Appendix A, **Chapter 8**

In summary: the dissipator outlet width, $W_O$, is 17.4 feet. The minimum bottom width of the channel is also 17.4 feet. The channel must be deep enough to accommodate a flow depth of 0.69 feet. The maximum slope of the channel sides is 1V: 2H. Flatter side slopes can be used if desired. The channel liner must withstand shear forces up to 7.3 pounds per square foot. If grass is planted in the channel, it must not get taller than 24-inches.

**Basin Design Step 4 -** In this step, the elevation of the water in the basin is calculated and the minimum elevation of the revetment on the sides of the basin is determined. The 17.4 feet bottom width of the outlet, $W_O$, is the length of the weir. Earlier in this example it was assumed the tailwater depth is negligible because the water would spill out of the dissipator outlet onto the side of the swale. This is no longer a valid assumption because the discharge from the basin will be confined in a channel. Consequently, the tailwater depth, TW, is the 0.69 feet depth of flow in the downstream channel.

An iterative method is used to calculate H, as follows: H is estimated. The percent submergence is calculated using Equation 11-11. $C_s$ is read from Figure 11-25b. $C_f$ is read from Figure 11-25a. H is calculated using Equation 11-13. Assumed and calculated H values are compared. The process is repeated until the assumed and calculated H values are similar. This process is illustrated in the following table.

| Assumed H (feet) | Percent Submergence | $C_s$ for Assumed H | $C_f$ for Assumed H | Calculated H (feet) | Difference Between Assumed and Calculated H (feet) |
|---|---|---|---|---|---|
| 0.72 | 95 | 0.77 | 3.03 | 0.97 | -0.25 |
| 0.92 | 75 | 1.00 | 3.06 | 0.81 | 0.11 |
| 0.82 | 84 | 0.98 | 3.04 | 0.82 | 0.00 |

Based on the preceding table and Equation 11-12, the water surface elevation in the dissipator basin is 0.82 feet higher than the elevation of the bottom of the dissipator outlet. The minimum elevation of the top of the revetment on the sides of the basin is 1 foot higher than this, or 1.82 feet above the elevation of the dissipator outlet.

# CHAPTER 12

# STORAGE FACILITIES

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## 12.1 Introduction

The traditional design of a storm drainage system has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize, this unrestricted discharge of increased flows caused by development may result in major drainage and flooding problems downstream. The engineering community is now more conscious of the quality of the environment and the impacts that uncontrolled increases in runoff can have on downstream property owners and ecosystems, and means are often used to reduce or eliminate discharge increases.

The most common means of controlling runoff discharge rates or volumes are storage facilities that detain or retain the runoff. These detention or retention facilities can reduce or mitigate the increases in discharge due to urbanization or other development. In addition, under favorable conditions, the temporary storage of some of the storm runoff can decrease the downstream flow and often reduce the cost of the downstream conveyance system.

Storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs. This chapter provides general design criteria for detention/retention storage basins as well as procedures for performing preliminary and final sizing and storage routing calculations.

This chapter is primarily intended for:

- designers working on ODOT projects and the storage of runoff from these projects, and
- The storage of runoff to ODOT drainage facilities from external sources

## 12.2 Uses of Storage Facilities

The use of storage facilities for stormwater management has increased dramatically in recent years, and the facilities can vary considerably in size and complexity, depending on their use. They may be small in terms of storage capacities and dam heights when they serve single outfalls from watersheds of a few acres, or they may be larger facilities that serve as regional stormwater management controls.

The benefits of storage facilities can be divided into two major categories, control of water quality, and control of water quantity.

### 12.2.1 Quality

Control of stormwater quality using storage facilities offers the following potential benefits:

- decreased downstream channel erosion,
- controlled sediment deposition and
- improved water quality through
  - stormwater filtration, and
  - settlement of solids (this occurs after the capture of the first flush with detention of 24 hours or more).

Additional discussion on water quality treatment facilities is included in **Chapter 14**.

### 12.2.2 Quantity

Controlling the quantity of stormwater using storage facilities can provide the following potential benefits:

- prevention or reduction of peak runoff rate increases caused by urban development,
- mitigation of downstream drainage capacity problems,
- recharge of groundwater resources,
- reduction or elimination of the need for downstream outfall improvements, and
- maintenance of historic low flow rates by controlled discharges from storage.

## 12.3 Detention and Retention

---

Stormwater quantity control facilities can be classified by function as either detention or retention facilities. The primary function of detention is to store and gradually release or attenuate stormwater runoff by way of a control structure or other release mechanism. True retention facilities provide for storage of stormwater runoff and release via evaporation and infiltration only. Retention facilities that provide for slow release of storm water over an extended period of several days or more are referred to as extended detention facilities. Recharge basins are a special type of detention basin designed to drain into the groundwater table; these are not addressed in this manual. Since most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this chapter to include detention and retention facilities.

### 12.3.1 Detention

The detention concept is most often employed in highway and municipal stormwater management plans to limit the peak outflow discharge rate to that which existed from the same watershed before development for a specific range of flood frequencies. Detention storage may

be provided at one or more locations and may be either above ground or below ground. These locations may exist as impoundments, collection and conveyance facilities, underground tanks, and on-site facilities such as parking lots, pavements, and basins. Detention ponds are the most common type of storage facility used for controlling stormwater runoff peak discharges. The majority of these are dry ponds which release all the runoff temporarily detained during a storm.

For the purposes of this chapter, detention facilities are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are designed to completely drain after the design storm has passed. The detention facilities discussed in detail in this chapter are:

- Dry ponds. These are depressed storage areas that store runoff during wet weather and are dry the rest of the time. Usually they are earthen depressions.
- Tanks. These are underground storage facilities that are typically constructed from large diameter pipe.
- Vaults. These are enclosed underground storage facilities. They are typically constructed from reinforced concrete.

### 12.3.2 Retention

For a watershed without an adequate outfall, the total volume of runoff is critical and retention storage facilities are used to store the increases in volume and to control the discharge rates. In addition to stormwater storage, retention may be used for water supply, recreation, pollutant removal, aesthetics, and/or groundwater recharge.

Retention facilities are typically designed to provide the dual functions of stormwater quantity and quality control. These facilities may be provided at one or more locations and may be either above ground or below ground. These locations may exist as impoundments, collection and conveyance facilities, and on-site facilities. Retention facilities are designed to contain a permanent pool of water. Additional information and discussion about retention facilities and protection of groundwater is included in **Chapter 14**.

## 12.4 Design Objectives

The objectives of a storage facility can be to control the peak design release rates and/or volumes, or to improve water quality, as discussed previously in this chapter. The primary emphasis of this chapter is to address the control of peak design release rates through detention. The control of release rates and/or volumes through retention, or the improvement of water quality through detention or retention is discussed in detail in **Chapter 14**.

The objectives for managing stormwater release rates by storage facilities are typically based on limiting peak design release rates to match one or more of the following values:

- historic peak runoff rates for specific design conditions (i.e., post-construction peak equals pre-construction peak for a design storm with a particular frequency of occurrence. See Subsection 12.5.1.1)
- non-hazardous discharge capacity of the downstream drainage system, and
- a specified allowable release rate set by a regulatory jurisdiction.

The reduction in the peak design release rate due to storage can be illustrated by a hydrograph showing discharge rates in relation to time, as shown in Figure 12-1.

### 12.4.1 Location Considerations

It should be noted that the location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system which could decrease or increase flood peaks in different downstream locations. Thus it is an important design objective to have storage facilities as drainage structures that both control runoff from defined areas and interact with the other drainage structures within the drainage basin to provide effective flood control. As a result, effective stormwater management must be coordinated on a regional or basin-wide planning basis. The location of detention facilities is discussed in more detail in Section 12.7.

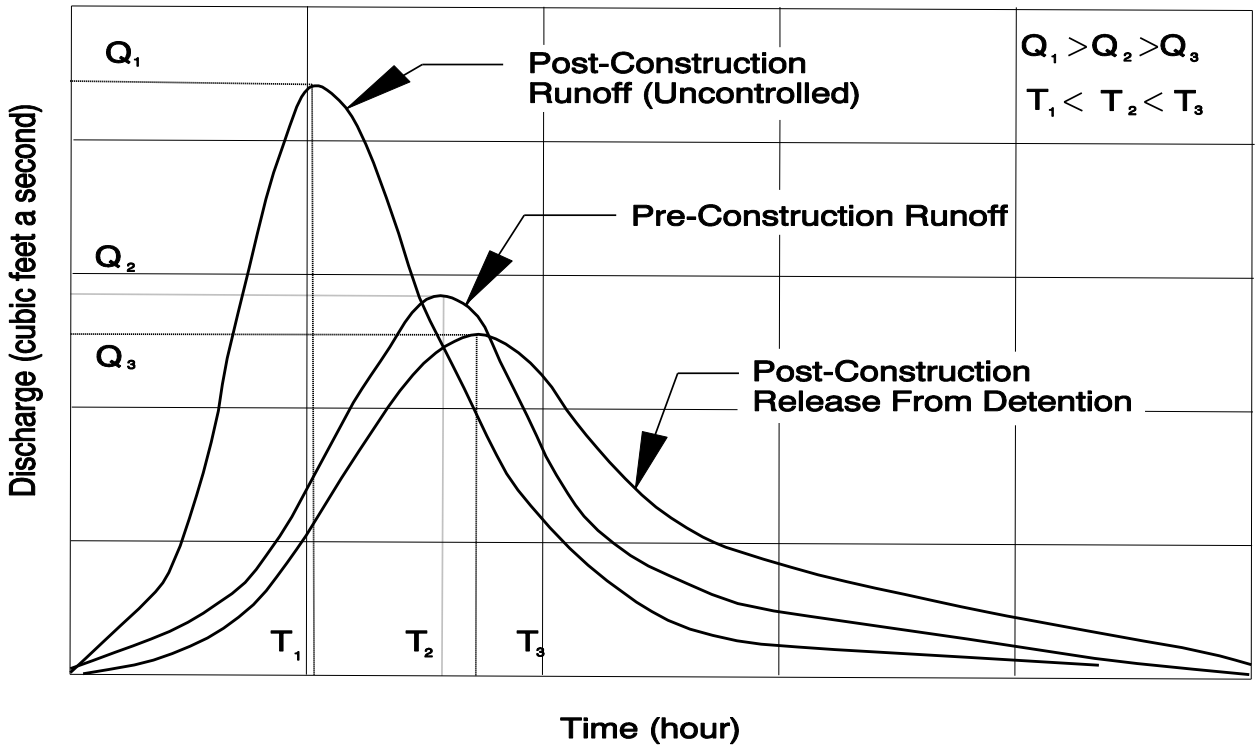



**Figure 12-1 Hydrograph Schematic**

## 12.5 Design Criteria

Storage may be concentrated in large basin-wide or regional facilities or distributed throughout an urban drainage system. Storage may be developed in depressed areas in parking lots, road embankments and freeway interchanges, parks and other recreation areas, and small lakes, ponds and depressions within urban developments. The utility of any storage facility depends on the amount of storage, its location within the system, and its operational characteristics.

Design criteria have been developed to help assure that storage facilities will accomplish the desired objectives. Storage facilities should be designed to meet these criteria. Exceptions to the criteria can be made in special circumstances, and these exceptions must be approved by the Geo-Environmental Section's Engineering and Asset Management Unit. The following criteria are for detention facilities. Design criteria for retention facilities are similar except that it may not be necessary to remove all runoff after each storm. More information about retention and water quality facilities is included in **Chapter 14**.

### 12.5.1 General Criteria

Storage facilities should be provided only where they are shown to be beneficial by hydrologic, hydraulic, and cost analysis. In addition, storage facilities should be provided where required by ordinances and should be constructed as deemed appropriate by the governing agency. The facility must satisfy Federal and State statutes, recognize local ordinances, and comply with the precedent established by Oregon Drainage Law. Some of these statutes are the Federal Water Pollution Control Act, Clean Water Act, and other federal, state, and local regulations.

The design of a storage facility should include a comparison of the peak design release rates to the peak pre-construction design runoff rates (i.e., 2-year through 50-year event, as required) at a point or points downstream of the proposed storage site with and without storage. The analysis should verify that the storage facility will limit the design release rates to the desired values. All detention system designs should also include an analysis of the check storm (i.e., 100-year or greater flood). The analysis should verify that the facility will safely store and release the check storm without damage to the facility.

Detention will likely be required when any one of the following criteria are met:

a. Detention is required by the local jurisdiction.

b. There is a history of drainage deficiencies in the area.

c. Discharge into an intermittent or perennial water body with an upstream drainage basin that is less than 100 $mi^2$ (measured upstream from the project's point of discharge into the receiving water)

d. The uncontrolled peak post-construction runoff rate during the design storm is 0.5 cubic feet per second or greater.

e. The total contributing area after the proposed development is 0.25 acre or greater.

Detention may not be needed when it is not required by the local jurisdiction and any one of the following criteria is met:

a. The uncontrolled peak post-construction runoff rate during the design storm is less than 0.5 cubic feet per second, and the total contributing area after the proposed development is less than 0.25 acre.

b. It is demonstrated the downstream ODOT drainage facilities are sufficiently sized. The analysis must evaluate the entire contributing basin upstream from the ODOT facilities with **fully developed** runoff coefficients based on current zoning.

c. It is demonstrated that the effects of the changed site conditions do not increase the peak runoff due to time lag and sub-basin location. A complete hydrograph analysis using multiple sub-basins is required for this method.

d. The changes in discharge comply with the precedent set by Oregon drainage laws.

e. The ODOT District, ODOT Region Technical Center, and appropriate regulatory agencies and watershed councils agree that detention is not in the best interest for the specific watershed at this location.

f. If the project discharges directly into a lake, reservoir, estuary, or the ocean, even if the drainage basin is less than 100 $mi^2$.

#### 12.5.1.1 Storms, Runoff Rates, Release Rates, and Storage Volumes

**Flood Flow Control Design Storm(s) -** An essential part of storage basin design is to analyze the design storm events. These events are often modeled as relationships of runoff versus time and described by hydrographs. In almost all cases, there are two design storm hydrographs, and they show the flow that occurs at the location of the inlet to the facility. One hydrograph describes the runoff versus time relationship for the upstream drainage basin before the proposed development. The other hydrograph describes the runoff versus time relationship at the same location after the proposed development.

*Note: In almost all cases, the post-construction hydrograph will show an increased peak discharge when compared to the pre-construction hydrograph. If this does not occur, the storm modeling should be carefully reviewed for errors.*

The design storm(s) recurrence interval(s) are the longer of the values listed below unless a more stringent recurrence interval is required by ODOT, federal, state, regional, or local agencies.

A. For detention facilities which serve 5 acres or less and discharge directly to and are physically connected to storm sewers or which discharge to ditches which do not lead directly to cross culverts or inlets:

- 10-year.

B. For detention facilities which serve 5 acres or less and do not discharge directly to storm sewers (This includes systems that utilize ditches and lead directly to cross culverts or inlets.) use one of the following:

- 25-year when the average daily traffic (ADT) volume is less than 750, or
- 50-year when the ADT is 750 or greater.

*Note: ADT values are listed annually in the ODOT Transportation Volume Tables.*

C. For detention facilities which serve an area of greater than 5 acres:

- 2-year, 10-year, and 50-year.

**Channel Processes Design Storms –** The channel processes design storm recurrence intervals are intended to avoid an increase in sediment transporting flows from pre-project to post-project (i.e., match the existing hydrology). The following design storm recurrence intervals applies to all projects where the uncontrolled peak post-construction runoff rate discharging to a receiving stream increases by 0.5 cubic feet per second or more than the peak pre-construction runoff rate for the 10-year storm event:

- Lower Discharge endpoint, by USGS flood frequency region, (see Figure 12-2)
  - Western Oregon = 42% of the 2-year, 24-hour event
  - Central and Eastern Oregon:
    - Southeast, Northeast, North Central Oregon = 48% of the 2-year, 24-hour event
    - East Cascade = 56% of the 2-year, 24-hour event
- Upper discharge endpoint:
  - For incised streams (those with an Entrenchment Ratio>=2.2)
    - The 10-year, 24-hour event
  - For unincised streams (those with an Entrenchment Ratio<2.2)
    - The 24 hour storm event with the return frequency equivalent to the bank overtoppoing event.

The entrenchment ratio is that defined by Rosgen (1996), and is the ratio between the width of the flood prone area and the channel width at bankfull discharge. Determination of the entrenchment ratio is not required: the 10-year 24-hour storm can be used as the default upper discharge point. The 2 year and 10 year 24 hour storm depths at specific sites are both available on the ODOT Precipitation Viewer. A simple spreadsheet program that can be used to calculate the flows from a project from specific sized storms is available from ODOT.

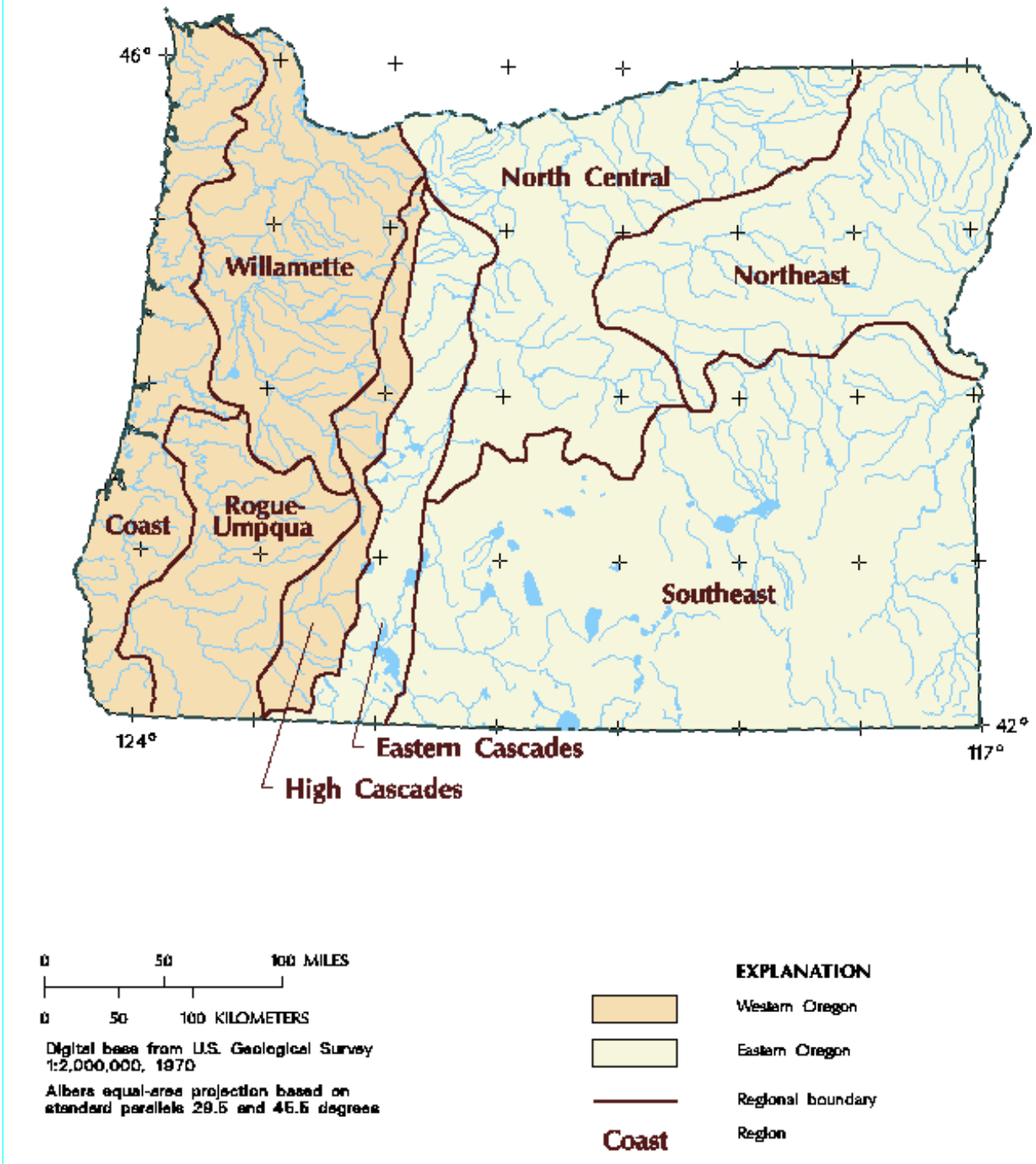



**Figure 12-2 USGS Flood-frequency region map for Oregon**

*Note: Project designs should comply with flood flow control and/or channel processes design storm(s) recurrence interval(s) noted above unless a more stringent recurrence interval(s) is required by ODOT, federal, state, regional, or local agencies.*

**Peak Pre-Construction Design Runoff Rate -** The hydrograph for the pre-construction conditions will show the maximum discharge rate from the design storms. This peak rate will also be shown by the hydrologic analysis of the existing conditions if a hydrograph is not used. This peak discharge rate is the "peak pre-construction design runoff rate." The analysis of larger storage facilities will require calculations of peak runoff rates for multiple design storms.

**Peak Design Release Rate and Storage Volume -** The hydrograph or table for the post-construction conditions is used with the routing procedure to show the maximum release rate from the facility during the design storm. This peak release rate is the "peak design release rate." The analysis of larger storage facilities will require the calculation of peak release rates for multiple design storms. These analyses will also show the maximum storage volume that is needed in the facility.

When calculating each peak design release rate, use the same recurrence interval as the corresponding peak pre-construction design runoff rate for the applicable design storm. For example, if a 10-year design storm is appropriate, the 10-year peak pre-construction design runoff rate and the 10-year peak post-construction design release rate are calculated and compared.

In most cases, the downstream entities will accept discharges up to the magnitude of the peak pre-construction design runoff rates. This should be verified before proceeding with the design. In areas with flood problems or outfalls with limited capacity, it may be requested, or a prudent design decision, to use a lower release rate than the pre-construction design runoff rate. The use of a reduced release rate may add considerable cost to the facility. The ODOT Region Technical Center staff should be consulted if this option is considered.

Some jurisdictions require that different design storms be used to calculate the runoff and release rates. For example, it may be required that the 10-year storm be used to calculate the peak pre-construction runoff, and detention be must be provided to limit the 25-year peak post-construction release rate to the value of this 10-year runoff. Detention can be provided to meet these stringent regulatory and statutory requirements without consultation with the ODOT Region Technical Center staff.

**Check Storm -** The check storm is based on post-construction land use. It is an event of greater magnitude than the design storm or storms. It is used in the design to insure that the facility will have the capacity and structural integrity to withstand large floods. The check storm is the larger of:

A. The 100-year storm.

B. A check storm justified by the potential threat to downstream life and property if the basin embankment were to fail. (for large storage facilities)

C. A storm specified by ODOT, federal, state, or local ordinances, or regulatory agencies.

**Peak Check Release Rate -** The hydrograph or table for the post-construction conditions is used with the routing procedures to show the maximum release rate from the facility during the check storm. This maximum release rate is the "peak check release rate."

### 12.5.1.2 Storage

Storage volume should be adequate to attenuate the post-construction peak inflow rates to the desired peak release rates. A simplified rational method is provided in Section 12.11 that can be used for preliminary storage estimates and for final design storage calculations for facilities serving areas 5 acres or smaller. When there are multiple design storms, the facility must provide adequate storage for the design storm which requires the greatest storage volume.

Calculations must be used to demonstrate that the storage volume is adequate for facilities serving areas larger than 5 acres and may be used for facilities of all sizes. This is best determined by routing the inflow hydrograph through the facility. Section 12.12 outlines techniques that can be used to estimate an initial storage volume and a discussion of storage routing techniques.

The system must be designed to release excess stormwater expeditiously to ensure that the entire storage volume is available for subsequent storms and to minimize hazards. For detention basins, all detention volume should be drained within 72 hours.

### 12.5.1.3 Site Selection

Site selection should consider both the natural topography of the area and the right of way boundaries. The planting and preservation of vegetation should be an integral part of the storage facility design. The auxiliary overflow should be located to ensure no property damage or injury will occur. The effects of the detained water on upstream drainage systems should be checked if the water surface level of the impounded water is higher than normal water surface elevations.

The site characteristics will often influence the choice of detention facilities. Good locations for open storage facilities are often:

- inside loop ramps, and
- open areas upstream from natural drainageways and wetlands.

*Note: Regulatory fish agencies may be concerned with open storage facilities being placed in natural channel floodplains due to the potential of trapping fish when flood waters recede. It is recommended to coordinate these designs early with project environmental staff. They will*

*discuss design with regulatory fish agencies. Regulatory concerns should be considered and modify design as necessary.*

Good locations for enclosed systems such as tanks or vaults are:

- in developed areas where space is at a premium,
- areas where a rise in nearby groundwater elevations cannot be tolerated, and
- sites where an open basin is not preferred due to safety, architectural, or aesthetic reasons.

Poor locations for detention facilities of all types are:

- steep hillsides and other geologically unstable areas, and
- areas with no access for maintenance.

#### 12.5.1.4 Outlet Works

Outlet works for storage facilities typically include a primary outlet to release the attenuated discharges from the design storms, and an auxiliary outlet to release discharge from the check storm or lesser storms if the primary outlet is clogged. The outlet works must be able to accomplish the design functions of the facility. Outlet works can take the form of combinations of weirs, spillways, drop inlets, risers, pipes, and orifices.

Most outlets use riser pipes. These risers can be designed to control different storms through the use of several orifices on the riser. A large flow can be controlled by stormwater flowing in through the top of the riser, using the entire riser diameter. Riser pipes should be designed to resist vortex flow during the check storm. A trash rack should be included in the design to protect the system from clogging. Slotted riser pipes are discouraged because of clogging problems, but curb openings may be used for parking lot storage. Slotted riser pipe outlet facilities should be limited to temporary structures.

Regardless of the outlet works configuration, the primary outlet is intended to convey the design storm without allowing flow to enter the auxiliary outlet. The sizing of a particular outlet works should be based on results of hydrologic routing calculations. When analyzing release rates, the effects of tailwater in the downstream system must be considered when determining the effective heads on each opening.

An auxiliary outlet must be provided to allow overflow that may result from excessive inflow or clogging of the primary outlet. This outlet should be positioned such that overflow will follow a predetermined route. Preferably, such outflows should discharge into open channels, swales, or other approved storage or conveyance features. The auxiliary outlet and downstream drainage system within the right-of-way should be designed to withstand scour and erosion from storms up to and including the check storm.

Anti-seepage collars must be placed on outflow pipes in berm embankments impounding water greater than 8 feet in depth (check storm water surface elevation to bottom of outlet). Refer to Subsection 12.9.5 for additional discussion on anti-seep collars.

#### 12.5.1.5 Construction and Maintenance Considerations

Stormwater management facilities must be properly maintained if they are to function as intended over a long period of time. To assure acceptable performance and function, storage facilities that require extensive maintenance are discouraged and proper design should focus on the elimination or reduction of maintenance requirements.

The recommendation is to consider design features that will minimize maintenance tasks. It is possible to minimize maintenance requirements during design while still achieving storage goals. A few ways to minimize and make maintenance easier or efficient include:

- Facility Selection
  - Use above ground facilities whenever possible and appropriate. Above ground facilities are easier to access and maintain than underground facilities.
- Site Selection
  - Site access must be adequate to allow for necessary maintenance vehicles and equipment to get to the facility. Avoid restrictive access conditions.
  - Do not place underground facilities in locations that would require closing a traffic lane to access
- Sediment control
  - Provide appropriate pretreatments as discussed in **Chapter 14**, or
  - Provide adequate dead storage (additional area below the inlet and outlet elevations) to allow for sediment accumulation between maintenance cycles.
- Inlet and Outlet structures
  - Screens or debris risers should be applied to orifices smaller than 6 inches because small diameter orifices can be susceptible to clogging from debris.
  - The appropriate inlet and outlet structures configuration and size should be selected to provide adequate access for maintenance or inspection.
  - Minimize depth of inlet and outlet structures to prevent the need for specialized equipment for maintenance or inspection.

#### 12.5.1.6 Operation and Maintenance

The proper operation, performance, structural integrity, and aesthetics of a stormwater storage facility are dependent on routine inspection and adequate maintenance. Facility inspection schedule and maintenance guidelines are summarized in an Operation and Maintenance Manual prepared to assist personnel who maintain the facility.

General requirements include:

- **Discuss proposed stormwater storage facilities with the responsible Maintenance District before selection and design. Maintenance input can help in selecting and developing BMPs that are maintainable.**
- All stormwater storage facilities require an Operation and Maintenance Manual. Prepare an operation and maintenance manual as outlined in **Chapter 4**.
- Distribute all prepared manuals to the appropriate district maintenance office and Geo-Environmental's Senior Hydraulics Engineer. An inventory of prepared manuals can be viewed at the following website:

  Operation & Maintenance Manuals Website

- All facilities need to be assigned a drainage facility identification number. Guidance on obtaining a drainage facility identification number is outlined in **Chapter 18**.
- All stormwater storage facility structures should be accessible by foot and vactor truck for inspection and maintenance.

#### 12.5.1.7 Field Marking

Field markers are used to locate and identify ODOT stormwater facilities or alert maintenance crews of the location of a stormwater facility's maintenance area. There are three stormwater markers. Two of these markers are used for marking above ground facilities and there is one marker applicable to underground stormwater facilities. ODOT's field marking process is outlined in **Chapter 17**.

#### 12.5.1.8 Drainage Facility Identification Number

A drainage facility identification number (DFI) is a unique identifier assigned to each stormwater treatment and storage facility. It is used to associate or link the stormwater facility to an Operation and Maintenance Manual. The number is assigned by contacting the Geo-Environmental Section's Senior Hydraulics Engineer to obtain a unique "DFI". The Geo-Environmental Section will maintain a database of assigned Drainage Facility IDs. Guidance on obtaining a drainage facility identification number is outlined in **Chapter 17**.

#### 12.5.1.9 Documentation

The following documents are required for most stormwater design projects. **Chapter 4** provides documentation guidance, outlines when the following reports are prepared, and what is to be included in each of these reports.

**ODOT Projects**

- Preliminary Stormwater Recommendations,
- Stormwater Design Report, and

- Operation and Maintenance Manual

Distribute a copy of the Operations and Maintenance Manual to the appropriate district maintenance office. Distribute a copy of the Stormwater Design Report and Operation & Maintenance Manual in Adobe Acrobat portable document format (pdf) and in word format, and the operational plan Microstation file to Geo-Environmental's Senior Hydraulics Engineer by completing the project report submittal form located at the following website:

http://www.oregon.gov/ODOT/HWY/GEOENVIRONMENTAL/hyd_data_resources.shtml

### 12.5.2 Design Criteria for Dry Ponds

Dry pond detention basins are depressed areas that store runoff during wet weather and are dry the rest of the time, as shown in Figure 12-3. They are very popular because of their comparatively low cost; few design limitations; ability to serve large as well as small watersheds; and their ability to be incorporated into other uses, such as recreational areas.

*Note: This subsection describes many features of dry ponds and the design criteria that apply specifically to these installations. In addition to the specific criteria, the facilities should also comply with the general criteria in Subsection 12.5.1.*

#### Site Selection

The site must be of sufficient size to accommodate the pond and also to provide adequate setback distances. The proper setback distances are important to ensure slope stability, maintenance access, and to minimize changes to ground water. These distances are shown in Figure 12-4 and listed in Table 12-1.

**Table 12-1 Setback Distances**

| Type | Distance |
|---|---|
| Check Flood High Water to Top Edge of Outer Embankment Slope | |
| Embankment Slopes more than 10 percent side slope | 200 feet minimum* |
| Embankment Slopes less than 10 percent | No minimum criteria |
| Check Flood High Water to Well | 100 feet minimum |
| Toe of Berm to Property Line | 1/2 Berm Height or 5 feet minimum |

* 200 foot minimum setback or provide geotechnical documentation supporting a lesser setback

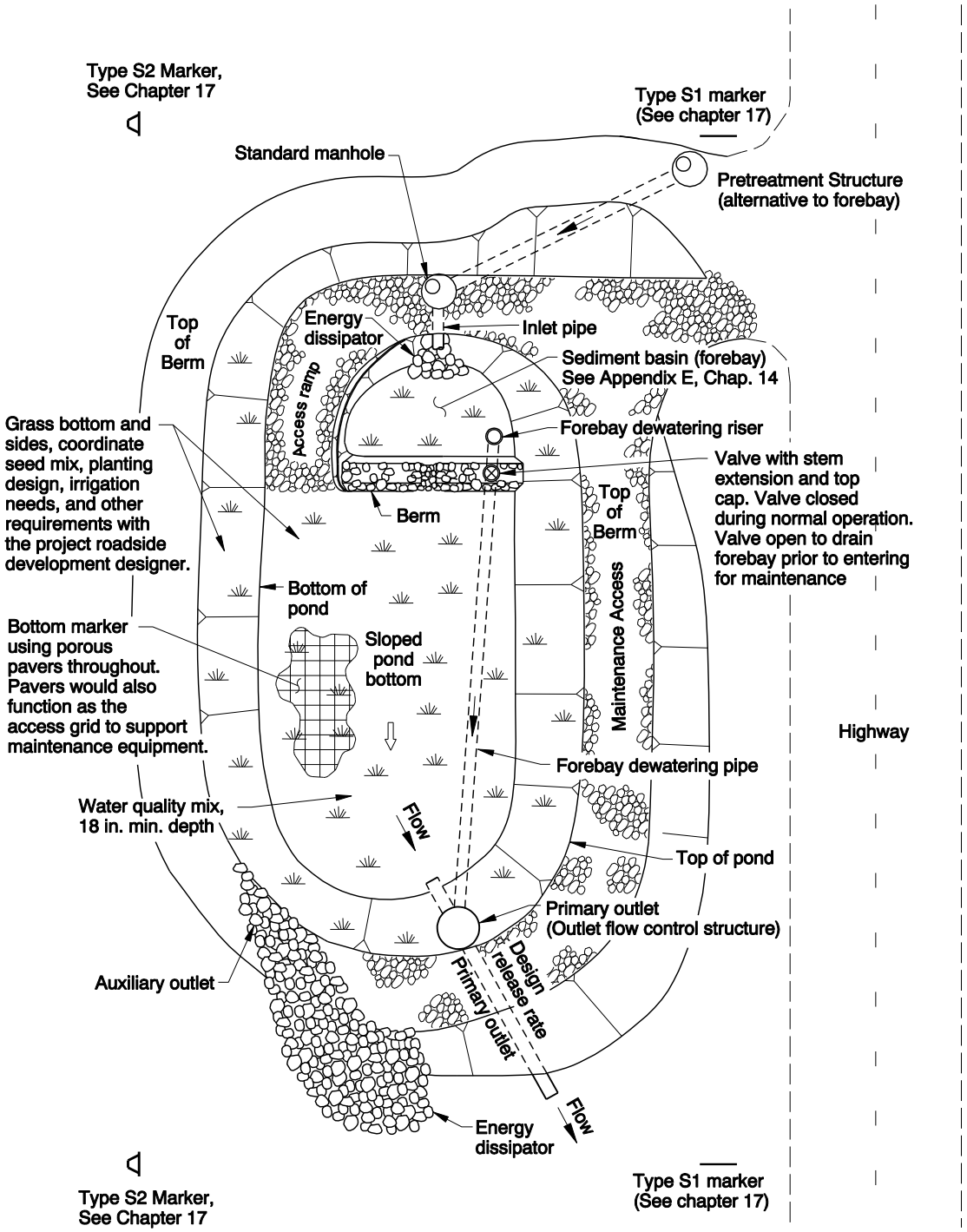



**Figure 12-3 Typical Detention Pond**

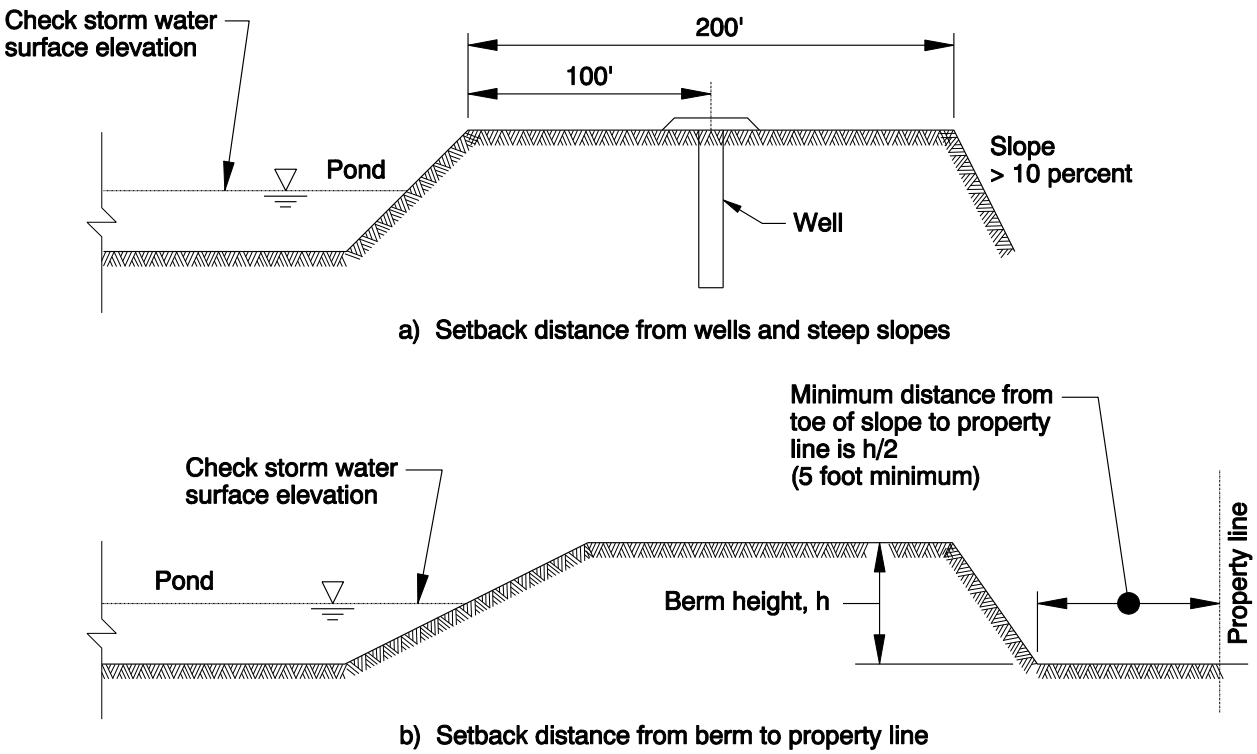



**Figure 12-4 Setback**

## Groundwater

1. Maintain a minimum distance of 2 feet from the bottom or invert of a facility to bedrock or seasonally high water table.

## Soil Suitability

1. Dry ponds are applicable in NRCS hydrologic soil groups B, C, and D.

## Pond Geometry

1. The **pond bottom area** must be sloped toward the outlet to prevent standing water conditions. A minimum bottom grade of 1 percent is recommended.

2. The pond must have adequate **depth** to provide the needed storage volume. Other considerations when setting depths include public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, shading, maintenance

requirements, and freeboard. Aesthetically pleasing features are also important in urbanized areas.

3. The **minimum bottom width** is 10 feet to provide the needed storage and allow for maintenance.

4. The **flow path** between the pond inlet and outlet must be maximized to ensure sufficient time is provided to allow for sedimentation of pollutants. A pond length-to-width ratio of 3:1 or greater is recommended.

5. The pond **storage volume** is designed to temporarily store the water volume discussed in Section 12.5.1.2. This is the volume between the bottom of the pond up to the start of the freeboard volume. Freeboard volume is in addition to the storage volume needed for detention.

6. A Pond with constructed embankments having a **storage capacity** of more than 50 acre-feet and embankment height greater than 6 feet are subject to requirements in the Safe Dams Act, as discussed in Section 12.6. Excavated ponds are not subject to these requirements.

7. The pond must have a **minimum dead storage depth** of 0.5 feet below the inlet and outlet to provide sediment storage.

8. **Interior side slopes** should not be steeper than 1V:4H.

9. **Pond walls** may be retaining walls designed in accordance with the ODOT Geotechnical Design Manual. Fence is typically provided along the top of the wall.

10. The **freeboard** criteria are outlined below. Freeboard is the vertical distance between the water surface and the rim of the auxiliary outlet or the top of the embankment, as shown in Figure 12-5.

    - **Design Storm** – 1 foot to 2 feet from design storm high water elevation to the auxiliary outlet rim elevation. The freeboard criteria apply to the highest intensity storm if there are multiple design storms. For very small impoundments (servicing less than 3 to 5 acres of surface area) an absolute minimum of 1 foot of freeboard may be acceptable.

    - **Check Storm** – 6 inches from the check storm high water elevation to the top of the embankment. The water surface elevation for the check storm freeboard calculations is based on the entire flow passing through the auxiliary outlet and no flow through the primary outlet.

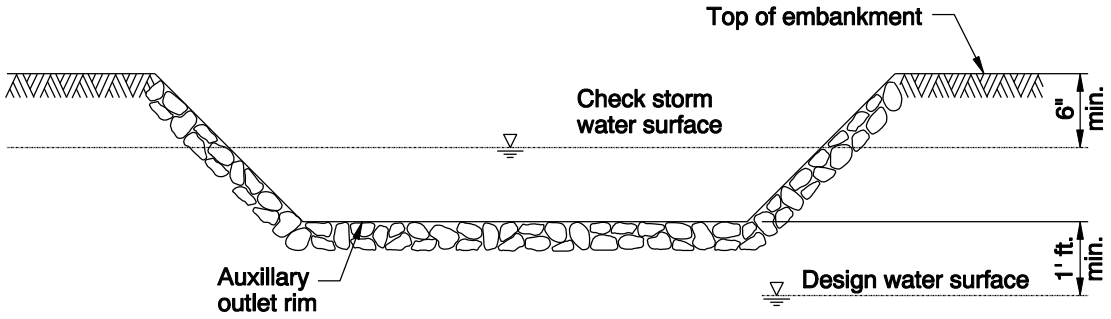



**Figure 12-5 Freeboard**

## Auxiliary Outlet

1. An **auxiliary outlet** such as a spillway or an outlet flow control structure with an auxiliary feature must be provided to convey the design high flow. The design high flow is the 100-year post construction peak flow.

2. A **spillway** used as an auxiliary outlet (emergency overflow) must be armored with riprap. Size riprap using the 100-year post construction peak flow. Armor the spillway embankment (pond side and downstream side) and downstream to where the spillway ties into the conveyance system with riprap.

## Bottom Marker

1. A **bottom marker** made of porous pavers must be installed along the swale bottom to indicate the bottom elevation. Select a porous paver from the Qualified Products List. The porous paver must provide a minimum 80 percent bottom area opening for grass growth. Spaced solid paver blocks are not allowed. Pavers can also function as the access grid to support maintenance equipment. See maintenance access section below.

2. Note the following in the facility's operation and maintenance manual:

   - the use of porous pavers to mark the bottom elevation of the pond
   - use sediment removal techniques that will not damage porous pavers

## Embankments

Pond berm embankments are often needed for dry ponds to obtain sufficient storage volume. Pond berm embankments must meet the following criteria:

1. Vegetated pond berm embankments must be less than 20 feet in height and have exterior side slopes no steeper than 1V:3H and interior side slopes no steeper than 1V:4H. Riprap-protected embankments should be no steeper than 1V:4H.

2. Pond berm embankments higher that 6 feet shall be **designed by a geotechnical enginee**r.

3. The **minimum top width** shall be 6 feet for pond berms 6 feet high or less, or as recommended by a geotechnical engineer.

## Maintenance Access

1. An **access road** shall be provided to the primary and auxiliary outlet control structures. The proposed access road must be able to support heavy equipment such as a vactor truck, dump truck, track how, or large mower.

2. Access road must be 16 feet in **width**.

3. The access road **maximum longitudinal slope** must be:

   a) 2 percent (edge of pavement to a longitudinal distance of 20 feet)
   b) 10 percent (20 feet from edge of pavement to end of access road)

4. The access road **maximum cross slope** is 4 percent.

5. An access ramp is required for mowing, repairs, and sediment removal. The ramp must extend to the pond bottom.

6. An **access grid** made of porous pavers must be installed along the pond bottom for maintenance vehicle and mowing equipment access. Select a porous paver from the Qualified Products List. The porous paver must provide a minimum 80 percent bottom area opening for grass growth. Spaced solid paver blocks are not allowed. Pavers would also function as the bottom marker.

7. **Maximum grade** of an access road or ramp shall be 10 percent

8. Implement with Maintenance District Concurrence: **Manhole lids** located in non-traffic areas outside or beyond the clear zone such as grassed areas or behind guardrail must be set 1 foot above finish ground so that manhole location is visible for locating and for maintenance. Lid elevations must match proposed finish grade in traffic areas.

**Protective Treatment**

1. Protective treatment may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences are recommended for detention areas where one or more of the following conditions exist:

   - areas where small children are present,
   - areas where rapid water level increases would make escape practically impossible,
   - water depths either exceed 3 feet for more than 24 hours or are permanently wet and have side slopes steeper than 1V:3H, or
   - side slopes are equal to or steeper than 1V:1.5H.

**Planting Requirements**

1. Grass shall be established along the sides and bottom of pond prior to facility operation. Grass seeding is not required, but should be considered for Oregon climate zones 5,6,7,8, and 9 (see **Chapter 14**, Appendix B, Figure 3 for climate zone boundaries). Coordinate herbaceous plants and shrubs planting plan with the project roadside development designer or landscape architect when grass is not an appropriate option.

2. Permanent seeding is best performed as follows:

   - West of the Cascades – March 1 through May 15 and September 1 through October 31 if grass areas are watered regularly during the establishment period.
   - East of the Cascades – October 1 through February 1 or March 1 through October 1 if grass areas are watered regularly during the establishment period.

**Field Markers**

1. Field Markers are required to be installed at the start and end of a facility's maintenance area. Marking guidance is provided in **Chapter 17**.

### 12.5.3 Design Criteria for Tanks

Detention tanks are underground storage facilities that are typically fabricated and constructed with large diameter pipes. They are commonly used in areas where topography or high property values make ponds less cost effective. This subsection describes many features of detention tanks and the design criteria that apply specifically to these installations. In addition to the specific criteria, the facilities should also comply with the general criteria in Subsection 12.5.1.
Detention tank systems are of two types, "flow through" and "back-up." The flow through tank has an inlet on one end and the outlet on the other, as shown in Figure 12-6a, and the backup tank has both the inlet and outlet at the same end, as shown in Figure 12-6b.

*Note: This subsection provides general minimum criteria for tank design. Local, state, and federal design codes may provide more stringent requirements, and they should be followed where applicable.*

**Buoyancy**

1. In moderately pervious soils where seasonal groundwater may induce flotation, buoyancy tendencies must be balanced either by ballasting with backfill or concrete backfill, providing concrete anchors, increasing the total weight, or providing subsurface drains to permanently lower the groundwater table.

**Geological and Geotechnical Considerations**

1. Tanks may float up through the soil cover if there are sufficient uplifting buoyant forces, as discussed in the previous subsection. This most often occurs when the tank is empty or partially full and the water table is sufficiently high. As a result, geological information on the seasonal elevations of the water table is desired before a tank design is commenced. Replacement of soft subsoils and compaction of soils around tanks are of critical importance. A geotechnical report analyzing stability and constructability is required for tanks in fills.

**Tank Geometry**

1. The **minimum tank diameter** allowed is 36 inches.

2. Tanks **larger than 36 inches in diameter** may be connected to each adjoining structure with a short section (24 inches maximum length) of 36 inch (minimum) diameter pipe.

3. The detention **tank bottom** should be flat or slightly sloping and at least 6 inches below the outlet invert elevation to provide dead storage for sediment, as shown in Figure 12-6c.

4. The tank(s) should provide the **required detention volume** in the space above the elevation of the outlet invert and below the elevation of the design flow water surface. If a sloping tank is used, the lost volume due to the higher invert at the upstream end should be considered in the volume calculations.

5. A pretreatment facility or water quality facility is required to be installed upstream of the proposed tank. Design a pretreatment or water quality facility according to guidance provided in **Chapter 14**.

6. The **freeboard** criteria are outlined below. Freeboard is the vertical distance between the water surface and the rim of the auxiliary outlet or the top of the tank as shown in Figure 12-6c.

- **Design Storm –** 6 inches from design storm high water elevation to the auxiliary outlet rim elevation. The freeboard criteria apply to the highest intensity storm if there are multiple design storms.

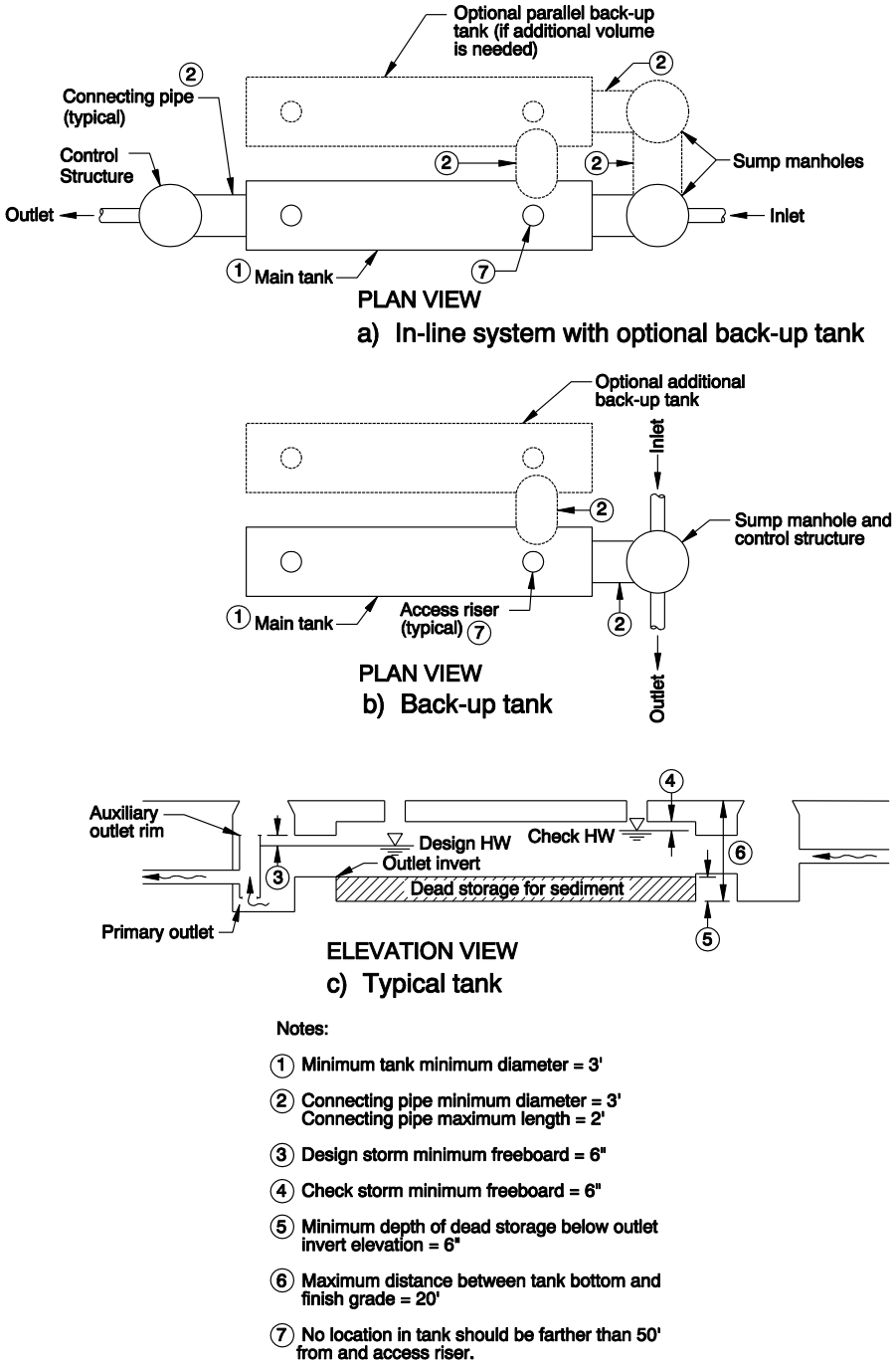
- **Check Storm –** 6 inches from the check storm high water elevation to the top of the tank. The water surface elevation for the check storm freeboard calculations is based on the entire flow passing through the auxiliary outlet and no flow through the primary outlet.

7. The tank should have adequate venting to prevent pressure or vacuum as the water surface level rises or falls within the tank. This can be accomplished by having vented access cover over the tank or by connecting the top of the tank to a ventilated area with a pipe having a minimum diameter of 2 inches.

**Tank Access Requirements**

1. The **maximum depth** from finish grade to the tank invert must be 20 feet due to the limitations of vactor trucks.
2. **Access openings** must be positioned a maximum of 50 feet from any location within the tank. These openings are provided for inspection and maintenance.

3. All **tank access openings** must be readily accessible by maintenance vehicles.




Notes:

1. Minimum tank minimum diameter = 3'
2. Connecting pipe minimum diameter = 3'
   Connecting pipe maximum length = 2'
3. Design storm minimum freeboard = 6"
4. Check storm minimum freeboard = 6"
5. Minimum depth of dead storage below outlet invert elevation = 6"
6. Maximum distance between tank bottom and finish grade = 20'
7. No location in tank should be farther than 50' from and access riser.

**Figure 12-6 Tank**

**Maintenance Access**

1. An **access road** shall be provided to the primary and auxiliary outlet control structures. The proposed access road must be able to support heavy equipment such as a vactor truck or dump truck.

2. Access road must be 16 feet in **width**.

3. The access road **maximum longitudinal slope** must be:

   a) 2 percent (edge of pavement to a longitudinal distance of 20 feet)
   b) 10 percent (20 feet from edge of pavement to end of access road)

4. The access road **maximum cross slope** is 4 percent.

5. **Maximum grade** of an access road shall be 10 percent

6. **Manhole lids** located in non-traffic areas such as grassed areas or behind guardrail must be set 1 foot above finish ground so that manhole location is visible for locating and for maintenance. Lid elevations must match proposed finish grade in traffic areas.

**Field Marker**

1. A field marker is required to be installed onto the top of an access opening cover. Marking guidance is provided in **Chapter 17**.

### 12.5.4 Design Criteria for Vaults

Detention vaults are underground storage facilities typically constructed with reinforced concrete, as shown in Figure 12-7. They are commonly used in areas where topography or high property values make ponds less cost effective and heavy traffic loads are anticipated. This subsection describes many features of detention vaults and the design criteria that apply specifically to these installations. In addition to the specific criteria, the facilities should also comply with the general criteria in Subsection 12.5.1.

*Note: This subsection provides general minimum criteria for vault design. Local, state, and federal design codes may provide more stringent requirements, and they should be followed where applicable.*

**Geotechnical Considerations**

1. Replacement of soft subsoils and compaction of soils around vaults are of critical importance. A geotechnical report analyzing stability and constructability is required for tanks in fills.

## Ventilation

1. Ventilation pipes (minimum 1 foot diameter or equivalent) must be provided in all four corners of vaults to allow for ventilation for maintenance personnel. This is not required if removable panels are provided over the entire vault. Ventilation should also be provided to assure that pressure or vacuum does not occur within the vault due to fluctuations in the water surface elevation. Often this ventilation is provided by the holes in a standard manhole cover or the openings in a grate cover.

## Vault Geometry

1. The vault typically has an area reserved for the collection of sediment called the "dead storage area." This area should be separated from the remainder of the vault by a partition with a transfer pipe or orifice. The invert elevation of the transfer pipe or orifice should be at least 6 inches above the bottom of the dead storage area to provide room for accumulated sediment. At least 12 inches of clearance should be provided between the top of the partition and the check storm water surface elevation. This open area will allow the partition to overtop if the transfer pipe or orifice plugs. The opening area of the transfer pipe or orifice should not be less than twice, nor greater than five times, the opening area of the primary outlet orifice. The transfer pipe or orifice does not require screening as per the requirements in Subsection 12.5.1.5.
2. A sediment collection sump should be provided under the outlet structure. This sump is not a substitute for the dead storage area. The purpose of this sump is to collect the small amounts of sediment that may pass through the dead storage area. The sump should have at least the minimum dimensions shown in Figure 12-7.

3. The vault should provide the required detention volume in the space above the elevation of the dead storage area and below the elevation of the design flow water surface. The calculated volume should be the net volume after subtracting the volume occupied by the partition, outlet works, etc.

4. A pretreatment facility or water quality facility is required to be installed upstream of the proposed tank. Design a pretreatment or water quality facility according to guidance provided in **Chapter 14**.

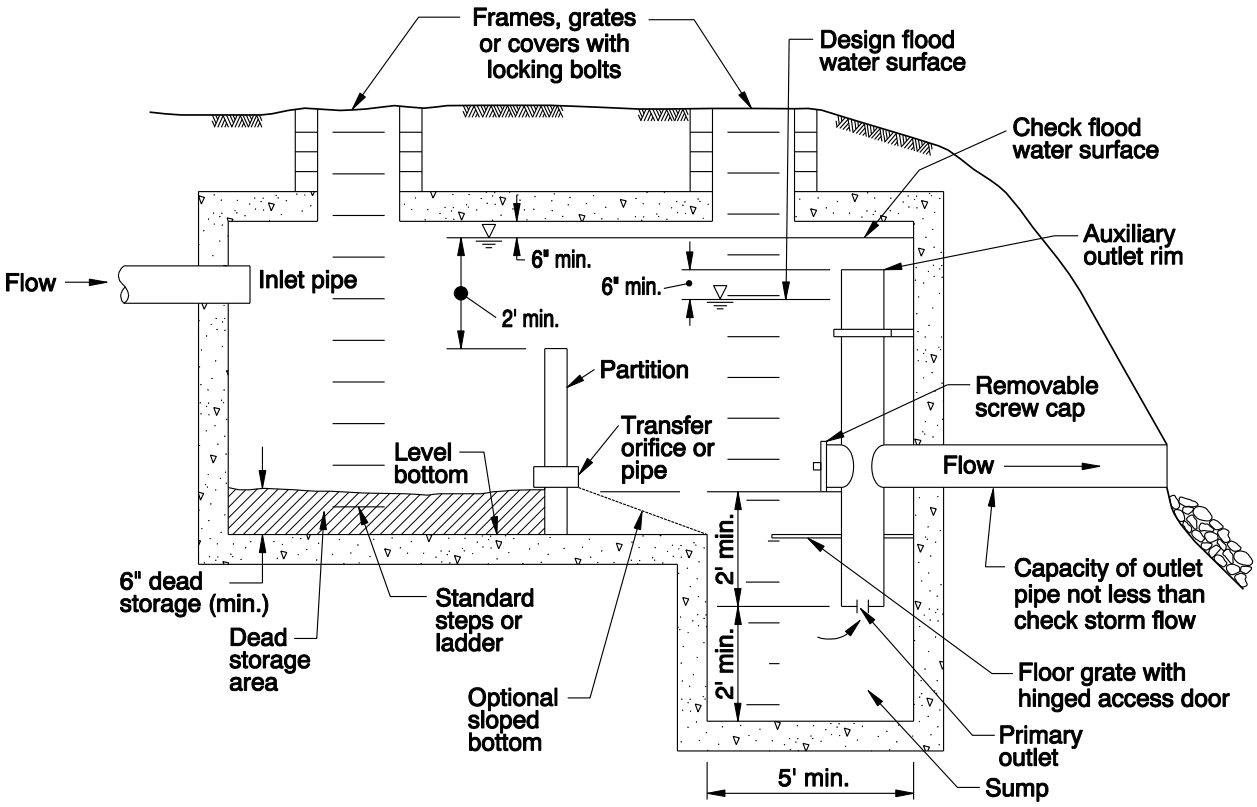



**Figure 12-7 Vault**

5. The freeboard criteria are outlined below. Freeboard is the vertical distance between the water surface and the rim of the auxiliary outlet or the ceiling of the vault as shown in Figure 12-7.

   - **Design Storm –** 6 inches from design storm high water elevation to the auxiliary outlet elevation. The freeboard criteria apply to the highest intensity storm if there are multiple design storms.
   - **Check Storm –** 6 inches from the check storm high water elevation to the ceiling of the tank. The water surface elevation for the check storm freeboard calculations is based on the entire flow passing through the auxiliary outlet and no flow through the primary outlet.

**Vault Access Requirements**

1. The **maximum depth** from finish grade to the vault invert must be 20 feet due to the limitations of vactor trucks.

2. **Access openings** must be positioned a maximum of 50 feet from any location within the vault. The access openings should be large enough to allow people and equipment to reach all areas of the vault for inspection, maintenance and repair. For vaults with greater than 4,000 square feet of floor area, a 5-foot by 10-foot or larger removable panel should be provided over the inlet pipe (instead of a standard frame, grate and cover). Alternatively, a separate access vault may be provided. For vaults under roadways, the removable panel must be located outside of the travel lanes. Alternatively, multiple standard locking manhole covers may be provided. The spacing of manhole covers should not be greater than 12 feet, measured on center, to facilitate the removal of sediment. Ladders and hand-holds need only be provided at the outlet pipe and inlet pipe and as needed to meet OSHA confined space requirements. Vaults providing manhole access at 12 feet spacing or less need not provide corner ventilation pipes.

3. All vault access openings must be readily accessible by maintenance vehicles.

**Maintenance Access**

1. An **access road** shall be provided to the primary and auxiliary outlet control structures. The proposed access road must be able to support heavy equipment such as a vactor truck or dump truck.

2. Access road must be 16 feet in **width**.

3. The access road **maximum longitudinal slope** must be:

   a) 2 percent (edge of pavement to a longitudinal distance of 20 feet)
   b) 10 percent (20 feet from edge of pavement to end of access road)

4. The access road **maximum cross slope** is 4 percent.

5. **Maximum grade** of an access road shall be 10 percent

6. **Manhole lids** located in non-traffic areas such as grassed areas or behind guardrail must be set 1 foot above finish ground so that manhole location is visible for locating and for maintenance. Lid elevations must match proposed finish grade in traffic areas.

**Field Marker**

1. A field marker is required to be installed onto the top of an access opening cover. Marking guidance is provided in **Chapter 17**.

## 12.6 Safe Dams Act

National responsibility for the promotion and coordination of dam safety lies with the Corps of Engineers in partnership with states, territories, federal agencies, and The Association of State Dam Safety Officials. However, individual States are responsible for administration of non-federal projects within their respective boundaries. Rules and regulations relating to dam safety in Oregon are promulgated by the Oregon Department of Water Resources.

According to the federal definitions a dam is an artificial barrier that does, or may, impound water having a measured height and potential storage that is equal to or greater than 25 feet, and greater than 15 acre-feet storage, or greater than 6 feet and equal to or greater than 50 acre-feet storage. In addition, Oregon dam safety laws enumerated in ORS 540.400 provide specific requirements for any hydraulic structure that is 10 feet or more in height or retains 9.2 acre-feet or more of storage. Any above-ground storage of water in Oregon, with few exceptions, requires a reservoir permit from the Oregon Department of Water Resources regardless of the size of the impoundment.

### 12.6.1 Classification

Dams are classified according to their physical dimensions and by the potential consequences to downstream life and property in the event the dam should suddenly fail and release the contents of its reservoir. Potential consequences of a failure are categorized according to the following three-part listing:

**Category 1 -** Dam is located where its failure and sudden release of reservoir contents would most likely result in direct loss of human life, excessive property damage, and/or loss of essential services and lifelines downstream of the dam.

**Category 2 -** Dam is located where its failure and sudden release of reservoir contents would probably damage downstream property, but would not result in direct loss of human life. Public inconvenience due to loss of roads, utilities, and other infrastructure would be minor and of limited duration.

**Category 3 -** Dam is located where its failure and sudden release of reservoir contents may cause some damage to downstream property, but such damage would be confined largely to the dam owner's property. No loss of human life would be expected.

### 12.6.2 New Dams, Reservoirs, and Storage Structures

In Oregon, detailed engineering plans and specifications for construction of new dams and reservoirs greater than or equal to 10 feet in height and having 9.2 acre-feet or more storage must be prepared by a licensed professional engineer. The design must be submitted to the Water

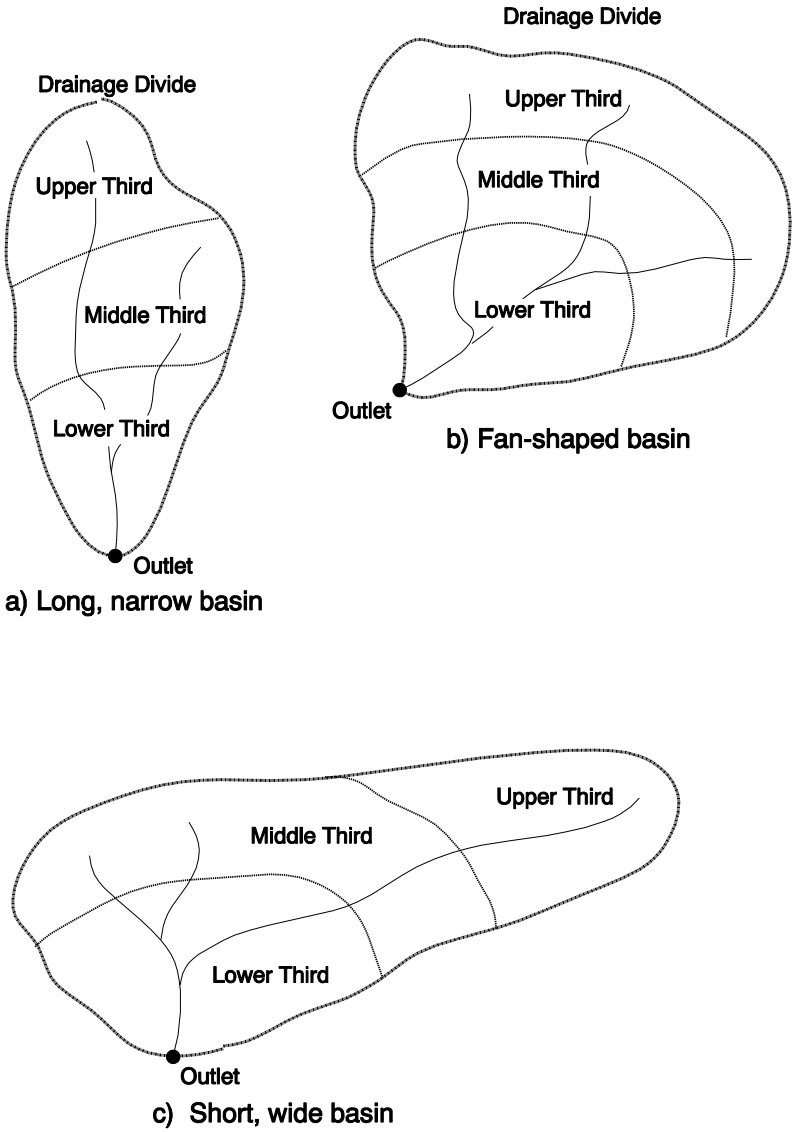
Resources Dam Safety program for review and approval prior to construction of the project works. The Water Resources Department should be consulted for further permitting, details and engineering requirements when preparing the design.

The ODOT Region Technical Center staff should be contacted if the detention system is in Category 1, 2, or 3 **and** the structure is considered a dam by the previously listed federal or state requirements. The Region Technical Center staff should also be contacted if a smaller impoundment would cause a Category 1 level of damage after a failure.

## 12.7 Location

In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. This occurs because the detention facility releases the discharge over a prolonged period. If several storage facilities are located within a basin, it is important to determine what effects a specific facility may have on combined hydrographs in downstream locations. It may be possible that the use of detention will actually increase peak flows at some locations in the basin. This is shown in the following two examples, and the critical location where the discharges are compared is the outlet of the total drainage basin.

The discharge hydrographs of a hypothetical drainage basin before development are shown in Figure 12-9. The hydrographs shows the discharge versus time relationships at the outlet of the basin due to flow from the upper, middle, and lower thirds of the basin, as well as the total combined discharge from the entire basin. The division of the basin into thirds is illustrated in Figure 12-8.




Schematic of typical drainage basin shapes and subdivision into basin thirds. The basin thirds should have approximately equal areas. The lengths of the various branches of the stream channel within each third should be approximately equal. The total length of stream channel within a third does not have to be the same as the channel length within any other third.

**Figure 12-8 Basin Subdivision**

The curves in Figure 12-9 graphically illustrate the timing of the peak flows from the various basin thirds. As expected, the peak flow from the lower third arrives at the outlet first, and it is

followed successively by peak flows from the middle and upper thirds. The combined discharge for the total basin peaks at the same time as the peak flow arrives from the middle third. At this instant the entire basin drainage area is contributing discharge to the outlet.

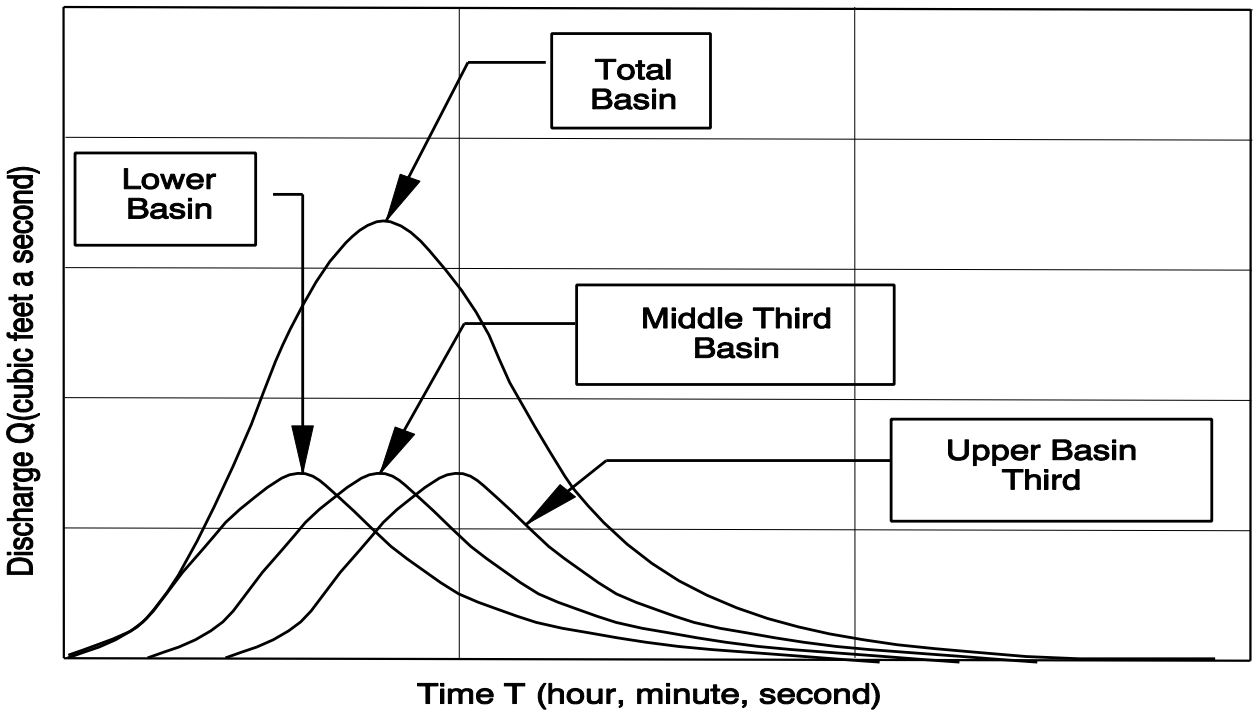
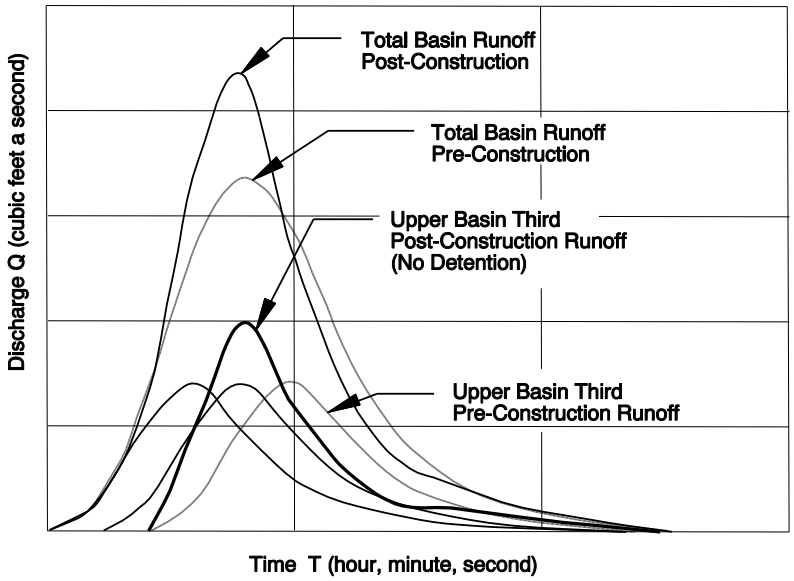
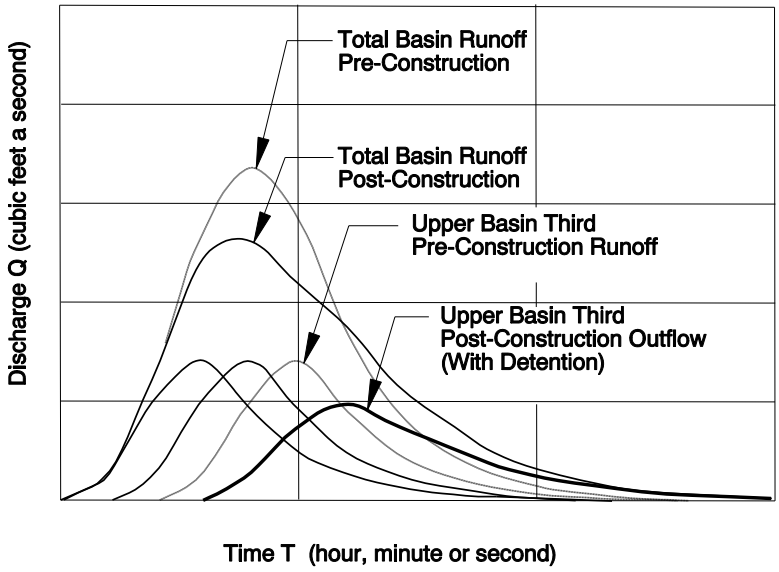



**Figure 12-9 Runoff Hydrographs of Undeveloped Basin**

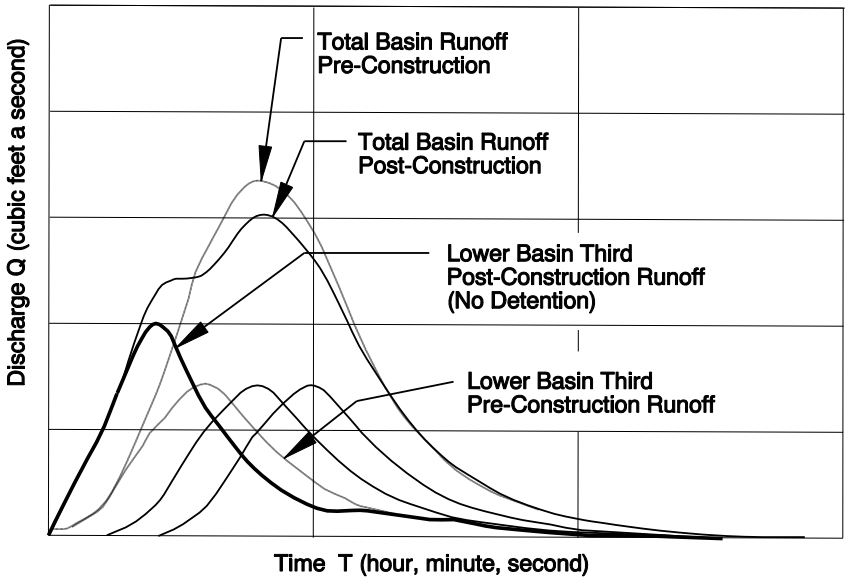



**a) Development in upper third without detention**

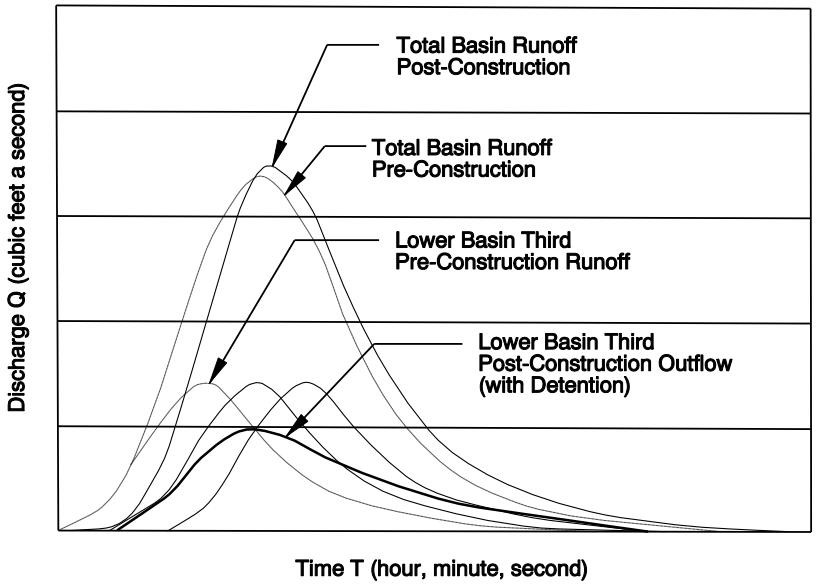



**b) Development in upper third with detention**

**Figure 12-10 Good Location in Basin for Detention**




**a) Development in lower third without detention**




**b) Development in lower third with detention**

**Figure 12-11 Poor Location in Basin for Detention**

In the first example, development without runoff detention is proposed for the upper basin third, and the effects of this change are shown in Figure 12-10a. It can be seen that the development increases the peak discharge from the upper basin third, and the peak arrives at the outlet sooner. The effects of this change in peak discharge magnitude and timing affect the total basin discharge-time relationship. The total peak discharge has been significantly increased.

Detention is now considered in the upper third of the basin in order to attenuate the peak discharge after development. The storage decreases the peak discharge from the upper third, and the peak arrives at the outlet later than it did in the undeveloped basin, as shown in Figure 12-10b. This detention achieves its objective. It reduces the post-construction peak discharge from the basin to a value equal to or lower than the pre-construction peak discharge.

In the second example, development without detention is considered in lower basin third rather than the upper third. The effects of this change are shown in Figure 12-11a, and it is seen that the development increases the peak flow from the lower third and the peak arrives at the outlet sooner. The effects of this change are shown on the hydrograph of the discharge from the total basin, and the cumulative peak discharge is reduced. This occurs because the discharge from the lower third now arrives much sooner than the discharges from the other thirds.

Detention is now considered for the development in the lower third. As expected, the storage decreases the peak runoff and the peak arrives at the outlet later, as shown in Figure 12-11b. The peak discharge from the lower third arrives at the outlet at the same time as the peak from the middle third, and the total discharge at the outlet is increased. This detention does not achieve its objective because it increases the magnitude of the discharge at a critical location - the outlet of the entire drainage basin.

The preceding examples are of a generic basin and they may not apply to all basins because the individual discharge-time relationships may be considerably different. The examples do, however, illustrate the need to verify if detention actually achieves the desired objectives, both at the outlet of the developed subbasin and at critical locations downstream.

## 12.8 Preliminary Data

The storage-routing calculations for storage facilities are mathematically complex and they are performed late in the design process. In order to do the storage-routing procedure preliminary data is developed, as follows,

- inflow hydrographs for all selected design storms and the check storm ($Q_i$ versus T),
- the stage-storage curve for the proposed facility (D versus $V_s$),
- the stage-discharge curves for all outlet control structures (D versus $Q_o$),
- the storage-discharge curves for the proposed facility ($Q_o$ versus $V_s$), and

- the storage-indication curve ($Q_o$ versus $V_s + ½Q_o\Delta T$).

The inflow hydrographs and stage-storage curve are used in both design methods presented in this chapter, the simplified rational method and the hydrograph and routing method. All of the listed curves are used in the hydrograph and routing method.

### 12.8.1 Inflow Hydrograph

The inflow hydrographs show the relationship of discharge versus time at the inlet to the storage facility. Several methods of developing the inflow hydrographs are presented in **Chapter 7**, and the topic is discussed in more detail in the discussion of design methods in the remainder of this chapter. The choice of hydrograph depends on the analysis method that will be used.

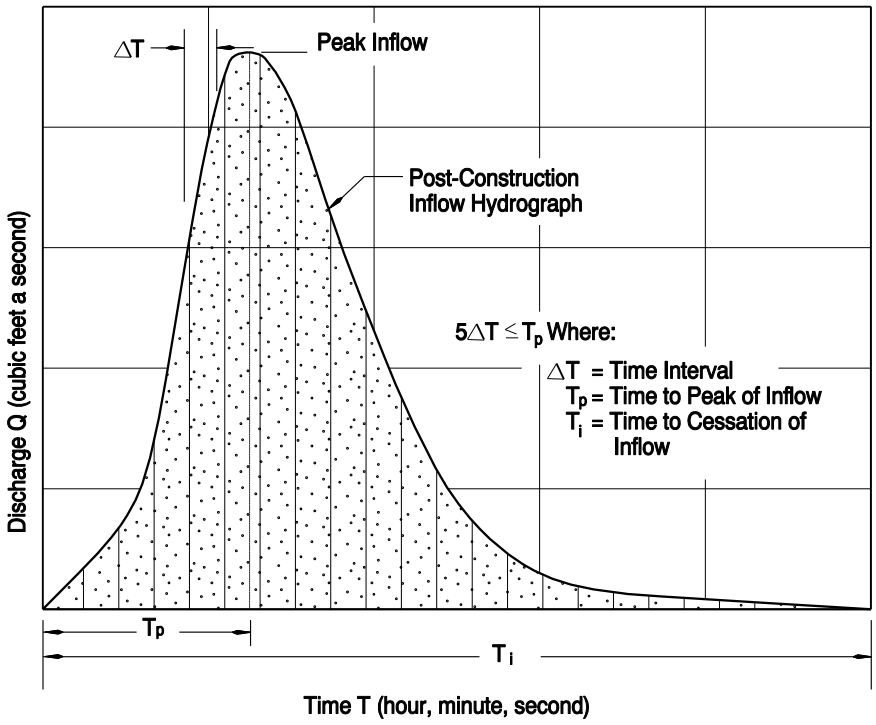



**Figure 12-12 Time Intervals and Period for Routing Analysis**

The time interval ($\Delta T$) and period for the analysis ($T_i$) are chosen in this step if the hydrograph-routing method is to be used. $\Delta T$ should be chosen so there are at least five intervals on the rising limb of the inflow hydrograph for the post-construction conditions, as shown in Figure 12-12. $T_i$ should extend at least as long as the inflow during the post-construction inflow conditions. The post-construction inflow can be considered to have ceased when the inflow rate has reduced to 2 percent of the peak post-construction inflow.

### 12.8.2 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water or the elevation of the water surface (often called the "stage"), and the storage volume in a reservoir, as shown in Figure 12-13. The data for this type of curve are usually developed using one of the formulas described in the following subsection. The storage volume is usually expressed in units of acre-feet for large storage volumes such as reservoirs and cubic feet for smaller impoundments such as the typical detention system.

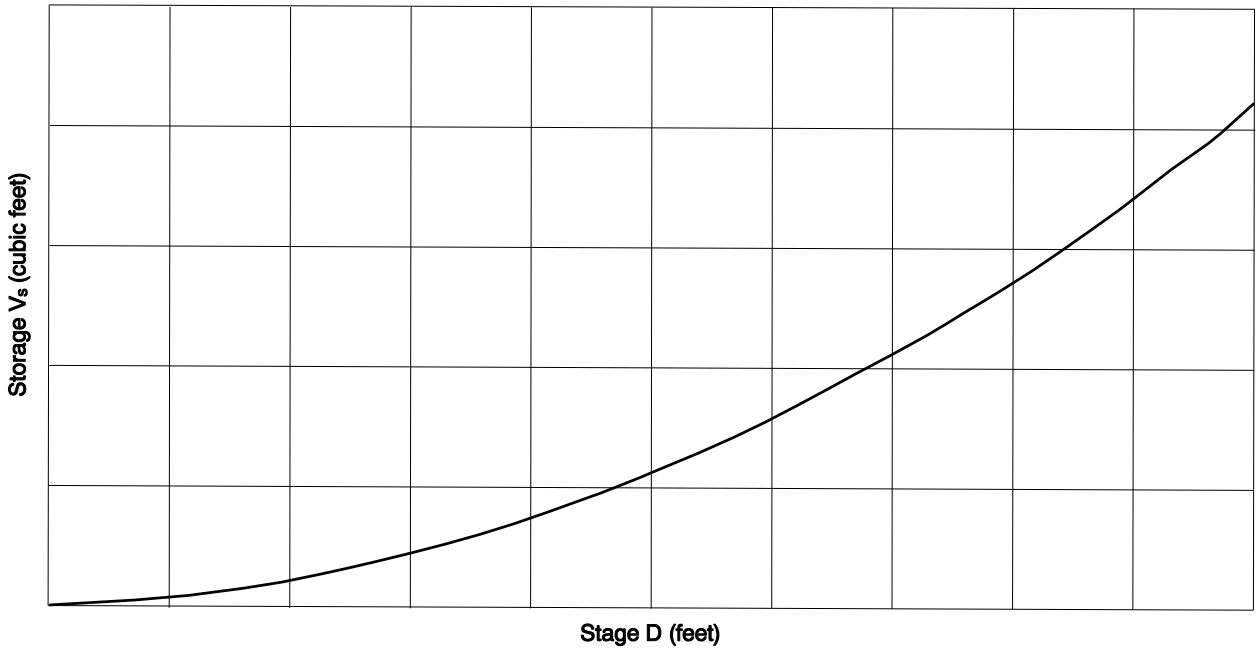



**Figure 12-13 Stage-Storage Curve**

#### 12.8.2.1 Storage Volume of Rectangular Basin

Underground storage tanks are often rectangular. The volume of a rectangular basin with a sloping bottom can be computed by dividing the volume into rectangular box and wedge shapes and using Equation 12-1 with the variables illustrated in Figure 12-14.

Box: $V = LWD$ Wedge: $V = \frac{D^2W}{2S}$ (Equation 12-1)

Where:

V = Volume at the depth of ponding in feet
D = Depth of ponding in the basin in feet
W = Width of basin at base in feet in feet
L = Length of basin at base in feet
S = Slope of basin bottom, in feet vertical distance per feet horizontal distance in foot per foot

*Note: If the volume of the basin is desired instead of the volume of the ponding, the basin depth can be used as input variable "D." If the basin bottom is not sloped, then the geometry will consist only of a rectangular shaped box.*

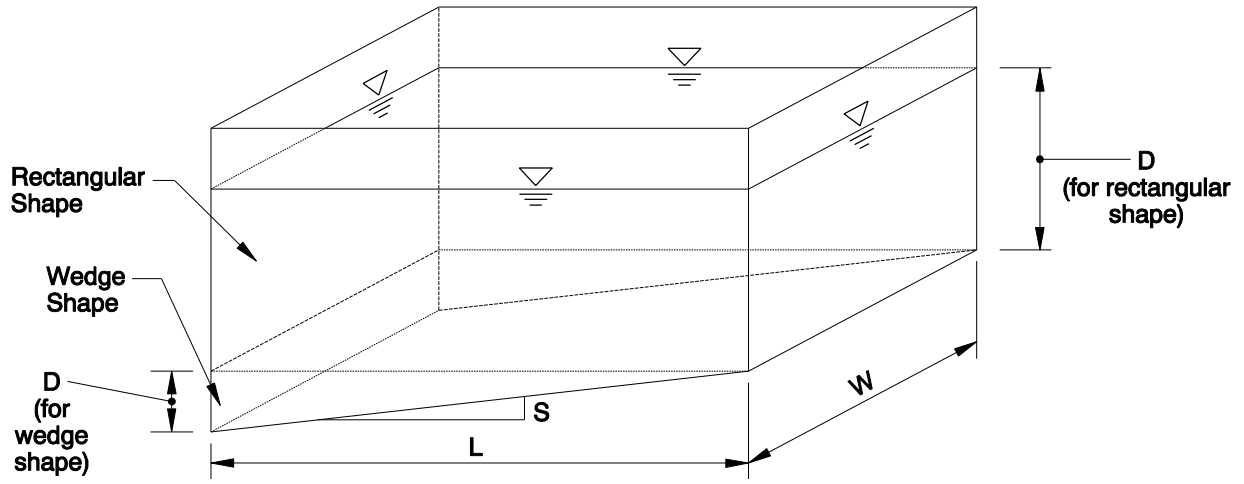



**Figure 12-14 Rectangular Basin**

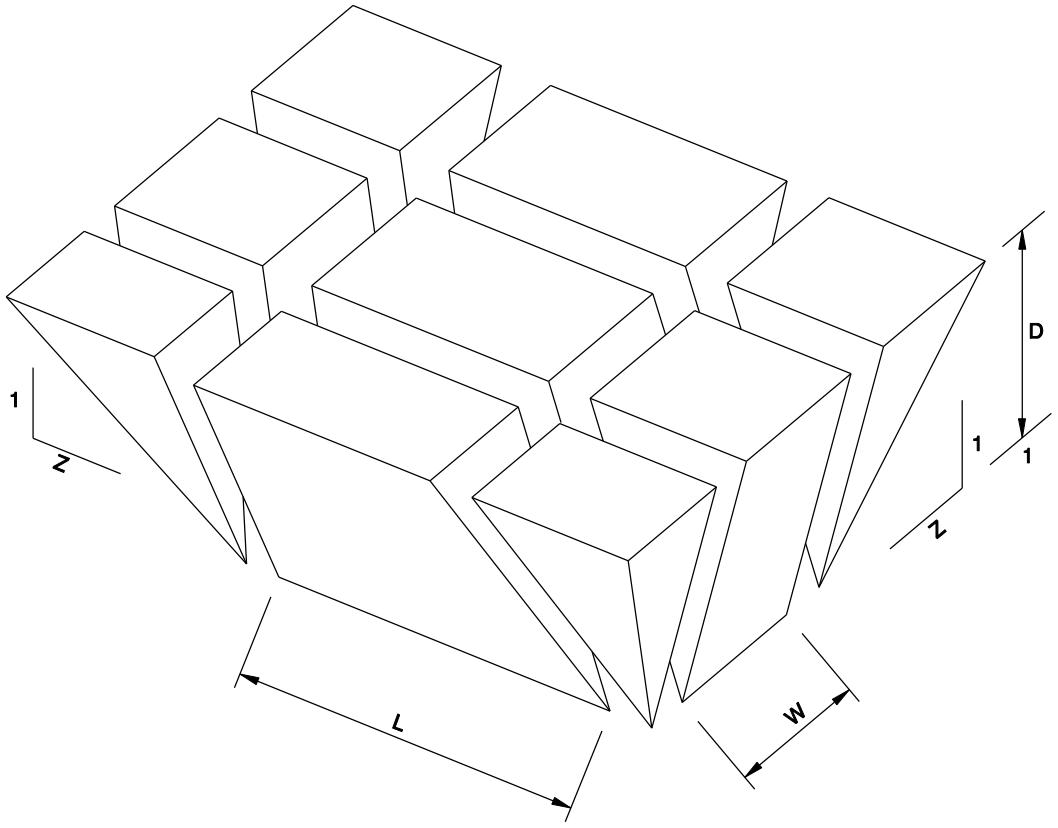



**Figure 12-15 Trapezoidal Basin**

### 12.8.2.2 Storage Volume of Trapezoidal Basin

The volume of a trapezoidal basin can be calculated in a manner similar to that of a rectangular basin by dividing the volume into triangular and rectangular shaped components and applying Equation 12-2 using the variables illustrated in Figure 12-15. "Z" in this equation is the inverse of the basin side slope. It is the ratio of the horizontal to vertical components of the side slope. For example, if the side slope is 1 vertical unit to 2 horizontal units, "Z" will be equal to 2.

$$V = [LWD] + [Z\,(L+W)\,(D^2)\,] + [4/3\,(Z^2)\,(D^3)] \qquad \text{(Equation 12-2)}$$

Where:

V = Volume at depth of ponding in cubic feet
D = Depth of ponding in the basin in feet
L = Length of basin at the base in feet
W = Width of basin at the base in feet
Z = Side slope factor (ratio of horizontal to vertical components of the side slope)

*Note: If the volume of the basin is desired instead of the volume of the pond, the basin depth can be used as input variable "D."*

Equation 12-2 is intended for a trapezoidal basin with a flat and level bottom. In many instances a trapezoidal basin is designed with slightly sloping bottom. In preliminary estimates, the volume is typically calculated using Equation 12-2 and the added volume due to the slope is ignored. In final estimates, the added volume due to the sloping bottom is often considered. One method that is often used is to divide the basin into sections as shown in Figure 12-15, calculate the volume of each section, and to add the volumes together to get the total volume. Another method is to use the procedures included in many automated drafting packages.

The dimensions of a trapezoidal basin with a level and flat bottom can be estimated for a given storage volume by rearranging Equation 12-2 into Equation 12-3. The length of a trapezoidal basin for a given volume, width, depth, and side slope is:

$$L = \frac{\left[-ZD(r+1)\right] + \left\{\left[(ZD)^2 (r+1)^2\right] - \left[5.33 (ZD)^2 r\right] + \left[\frac{4rV}{D}\right]\right\}^{0.5}}{2r} \qquad \text{(Equation 12 - 3)}$$

Where:

r = Ratio of width to length of basin at the base W/L
(Remaining variables are the same as Equation 12-2.)

### 12.8.2.3 Storage Volume of Pipe and Conduit

If pipes or other storm drain conduits are used for storage, positive slope should be provided to transport sediment. This complicates storage calculations. The calculation of pipe storage volume involves these steps, as needed:

1. If part of the pipe is full, calculate the volume of the full section using the full pipe formula.

2. If part of the pipe is partially full, calculate the volume of the partially full section using the prismoidal or ungula of a cone formulae.
3. If part of the pipe will be used to store sediment, use the prismoidal or ungula formulae to determine the sediment storage volume.

4. The storage volume is calculated by adding the volumes determined in Steps 1 and 2 to get the total volume, and subtracting the sediment storage volume determined in Step 3.

**Full Pipe Formula -** The volume of the section of the pipe or tank that is full can be calculated by multiplying the cross-sectional area of the pipe or tank by the length of the full section.

$$V = A\,L \qquad \text{(Equation 12-4)}$$

Where:

V = Volume of storage in cubic feet
L = Length of pipe or tank section in feet
A = Cross-sectional area of full pipe or tank in square feet
($A = \pi D^2 / 4$ when a cylindrical volume is calculated for a circular pipe or tank)

**Prismoidal Formula -** The prismoidal formula presented in Equation 12-5 can be used to determine the volume in partially full sections of sloping storm drain pipes. It is most applicable for non-cylindrical pipes such as boxes, pipe-arches, and elliptical pipes. Figure 12-16 provides a definition sketch for the terms in Equation 12-5.

$$V = \left(\frac{L}{6}\right)\left(A_1 + 4M + A_2\right) \qquad \text{(Equation 12-5)}$$

Where:

V = Volume of storage in cubic feet
L = Length of pipe or tank section in feet
$A_1$ = Cross-sectional area of flow at downstream end in square feet
$A_2$ = Cross-sectional area of flow at upstream end in square feet
M = Cross-sectional area of flow at midsection in square feet

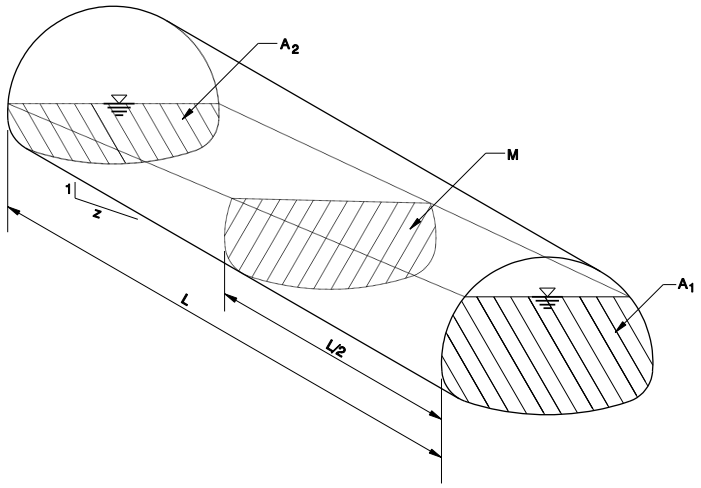



**Figure 12-16 Prismoidal Shape**

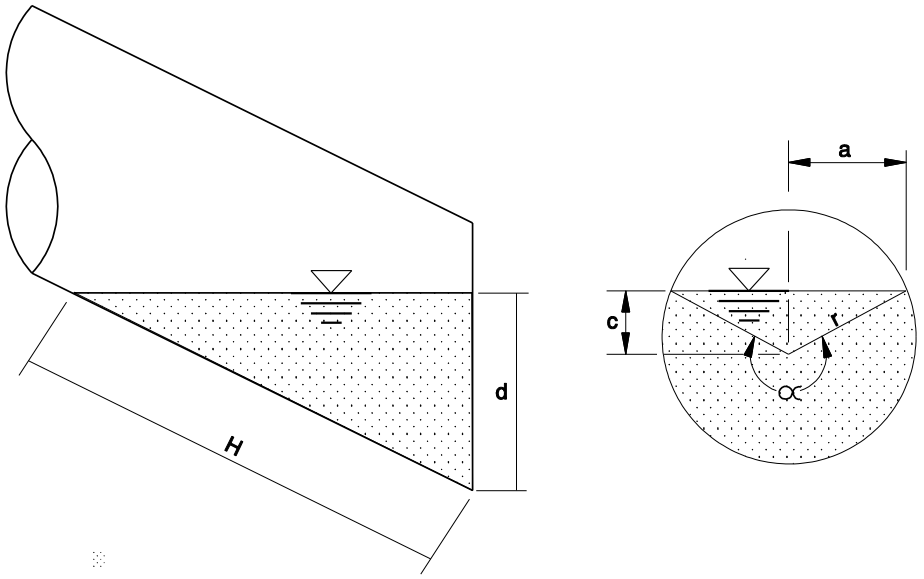



**Figure 12-17 Ungula of a Cone Shape**

**Ungula of a Cone Formula -** Calculations will be simplified if circular pipes are used, since the full pipe volume formula based on a cylinder can be used for the full section, and the ungula of a cone formula can be used for the partially full section. The ungula of a cone is shown in Figure 12-17. If the pipe is full at the base of the ungula end view, the following equation can be used:

$$V = \frac{\left[H\pi\ (r^2)\right]}{2} \qquad \text{(Equation 12 - 6)}$$

Where:

V = Volume of ungula when depth of water equals diameter of pipe at base of ungula in cubic feet

H = Wetted pipe length in feet (This is length of partially full section. The pipe can be full at the end of the ungula, only. It cannot be full within the ungula.)

r = Pipe radius in feet

The volume of the ungula when the pipe is between full and half full at the end of the ungula can be determined by the following equation:

$$V = \frac{H\left[\left(\frac{2}{3}a^3\right) + (c\,B)\right]}{(r + c)} \qquad \text{(Equation 12 - 7)}$$

Where:

V = Volume of ungula when depth of water at the base of the ungula is less than diameter of pipe, but half full or more in cubic feet

A = "a" as defined in Equation 12-8, as follows:

$$a = ((2r - D)\ D)^{0.5} \qquad \text{(Equation 12-8)}$$

Where:

a = One half of the free surface width at end of ungula in feet. See Figure 12-17.

D = Depth of water at end of ungula in feet

c = "c" as defined in Equation 12-9, as follows:

$$c = D - r \qquad \text{(Equation 12-9)}$$

Where:

c = Vertical distance between water surface and center of pipe at end of ungula, in feet See Figure 12-16.

B = "B" as defined in Equation 12-10, as follows:

$$B = \pi\,(r^2) - \left[(\alpha - \sin\alpha)\left(\frac{r^2}{2}\right)\right] \qquad \text{(Equation 12-10)}$$

Where:

B = Cross-sectional area of flow at end of ungula in square feet

α = "α " as defined in Equation 12-11 as follows:

$$\alpha = 2\sin^{-1}\left(\frac{a}{r}\right) \qquad \text{(Equation 12-11)}$$

Where:

α = Angle in <u>radians</u> as shown in Figure 12-17.

The volume of the ungula when the pipe is less than half full at the end of the ungula can be determined by the following equation:

$$V = \frac{H\left[\left(\frac{2}{3}a^3\right) - (c\ B)\right]}{(r - c)} \qquad \text{(Equation 12-12)}$$

Where:

V = Volume of ungula when depth of water at the base of the ungula is less than half the diameter of pipe in cubic feet

c = "c" as defined in Equation 12-13, as follows:

$$c = r - D \qquad \text{(Equation 12-13)}$$

B = "B" as defined in Equation 12-14, as follows:

$$B = (\alpha - \sin\alpha)\left(\frac{r^2}{2}\right) \qquad \text{(Equation 12-14)}$$

An example of the calculation of an ungula stage-storage curve, using both hand calculations and a computer solution, is included in the Federal Highway Administration Publication FHWA-SA-96-078 "Hydraulic Engineering Circular No. 22: Urban Drainage Design Manual," November 1996. Alternatively, various texts such as Brater, E.F. and H.W. King, Handbook of Hydraulics, 6th ed., (McGraw Hill Book Company: New York, NY, 1976) contain tables and charts which can be used to determine the depths and areas described in the above equations.

### 12.8.2.4 Storage Volume of Natural Basin

The storage volumes for natural basins in irregular terrain are usually developed using a topographic map and the double-end area or the frustrum of a pyramid formulae. The process has three steps. The first step is to plot the contours of the natural basin using a contour interval small enough to accurately model the terrain. Contour intervals of 1 or 2 feet are often used, and the contours are the elevation increments for the formulae. In the second step, the basin volumes are calculated for each elevation increment. In the third step, the incremental volumes are added to get the total basin volume.

**Double-End Area Formula -** The double-end area formula is expressed as:

$$V_{1,2} = \left[\frac{(A_1 + A_2)}{2}\right] d \qquad \text{(Equation 12-15)}$$

Where:

$V_{1,2}$ = Storage volume between Elevation Increments 1 and 2 in cubic feet

$A_1$ = Surface area at Elevation Increment 1 in square feet

$A_2$ = Surface area at Elevation Increment 2 in square feet

d = Change in elevation between increments 1 and 2 in feet

**Frustrum of a Pyramid Formula -** The frustum of a pyramid is shown in Figure 12-18 and is expressed as:

$$V = \frac{d}{3}\left[A_1 + (A_1 A_2)^{0.5} + A_2\right] \qquad \text{(Equation 12 - 16)}$$

Where:

V = Volume of frustum of a pyramid between elevation increments in cubic feet
$A_1$ = Surface area at Elevation Increment 1 in square feet
$A_2$ = Surface area at Elevation Increment 2 in square feet
d = Change in elevation between elevation increments 1 and 2 in feet

### 12.8.3 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water or the elevation of water (often called the "stage") in the basin and the discharge or outflow from a storage facility. The stage-discharge curves are usually developed using one or more of the formulae described in Section 12.9. The designer should consider as applicable, the head losses due to the outlet structure, the head losses that occur as the flow passes through the drainage system downstream from the outlet, and the tailwater elevations when developing stage-discharge curves.

The stage-discharge curve for a single outlet device is shown in Figure 12-19. The stage (D) is shown on the horizontal axis. The stage is a measure of the water surface elevation or depth in the storage facility. It can be a water surface elevation above a datum, or it can be the depth of water in the facility above a reference, such as the bottom of the pond or tank, or the bottom of the spillway. The rate of discharge from the outlet device ($Q_o$) is shown on the vertical axis. The relationship between stage and discharge is shown by the line on the figure. As is typical, the rate of discharge increases as the stage increases.

There can be single, multiple, or composite stage-discharge curves. The single stage-discharge curve is used for the following applications:

- designing the auxiliary outlet (it is often assumed that the primary outlet is plugged and all flow passes through the auxiliary outlet during the check storm),
- designing the primary outlet when it has a single opening (it is often assumed that all flow goes out of the primary outlet during the design storm), or
- designing the outlet at the lowest elevation in a facility with the primary outlet works consisting of multiple outlets at different elevations.

Stage-discharge curves for single outlets cannot be used when multiple outlets are operating. This often occurs when designing the higher outlets in a facility with multiple outlets at different elevations. Multiple and composite stage-discharge curves are shown in Figure 12-20.

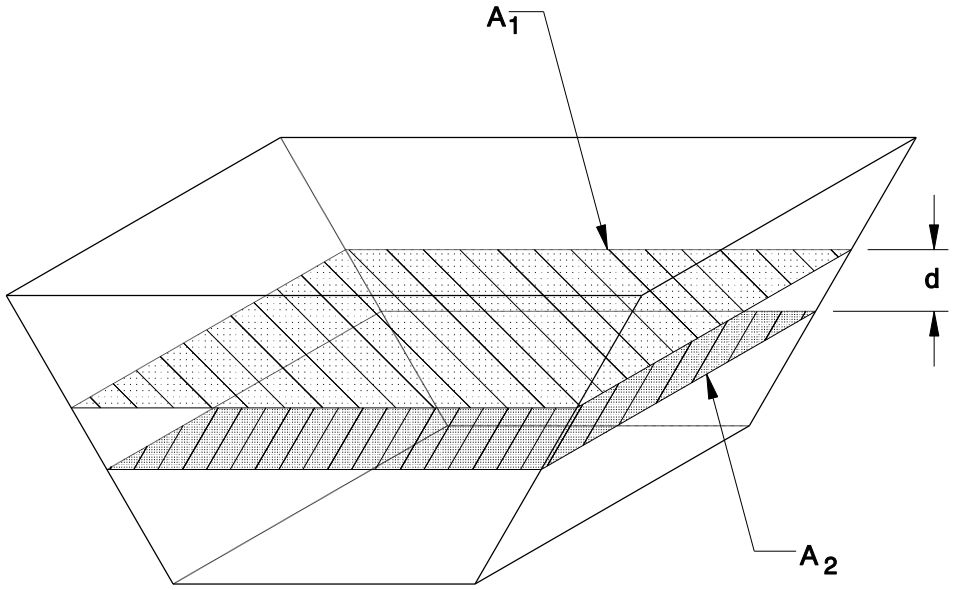



**Figure 12-18 Frustrum of a Pyramid**

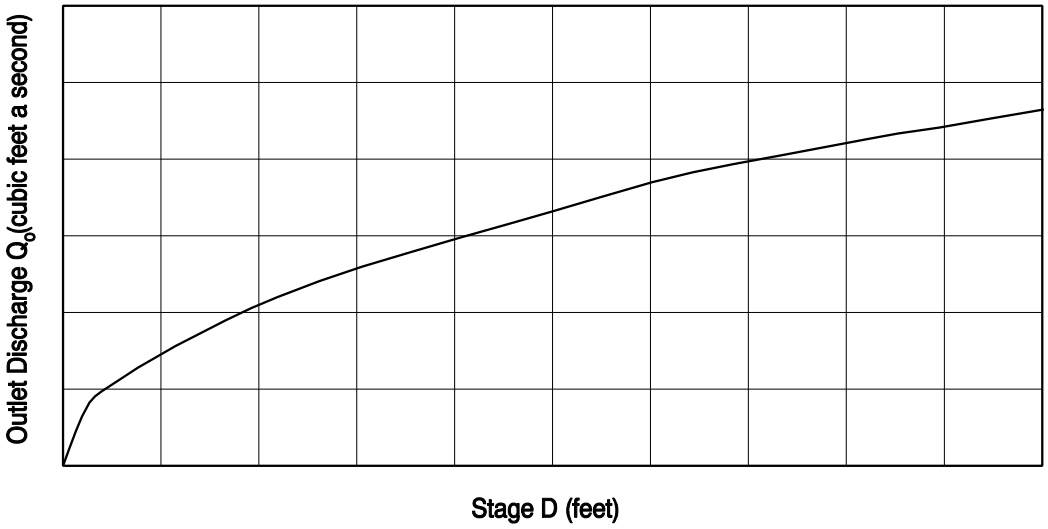



**Figure 12-19 Stage-Discharge Curve**

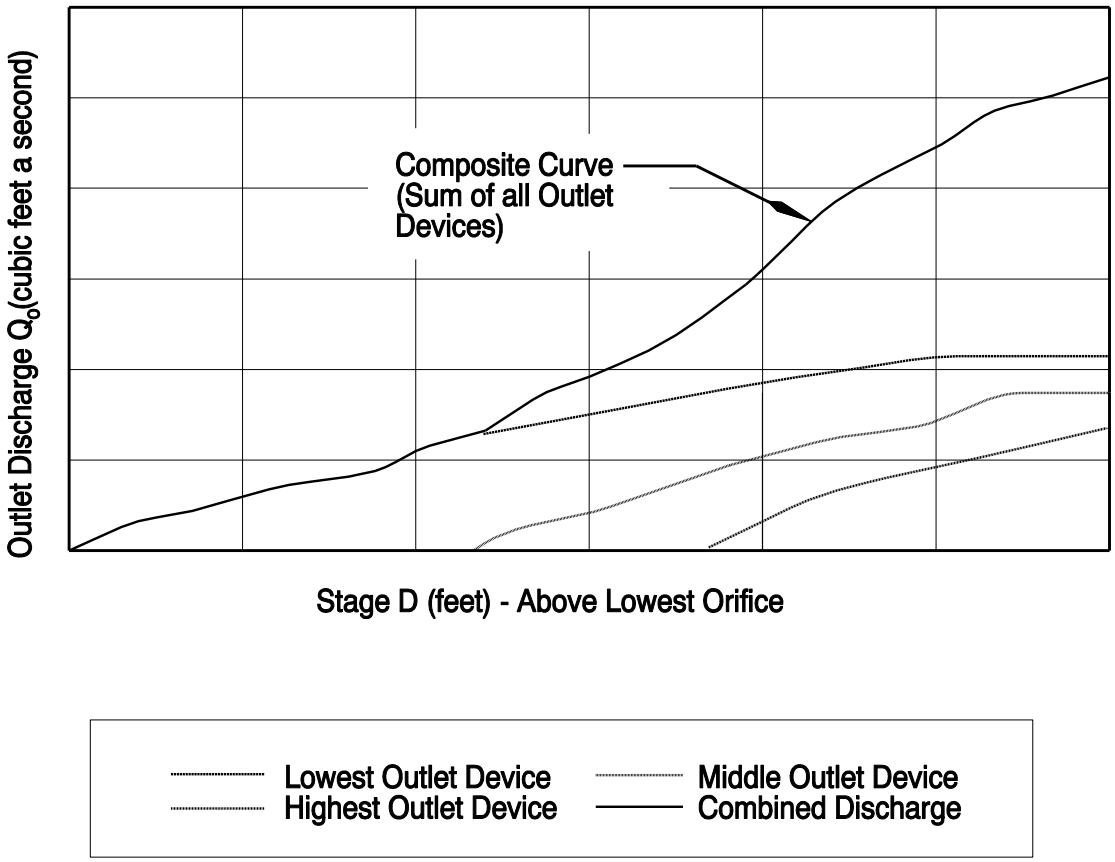



**Figure 12-20 Composite Stage-Discharge Curve**

### 12.8.4 Storage-Discharge Curve

The storage-discharge curve represents the relationship between storage volume ($V_s$) in the facility and the discharge rate from the outlet ($Q_o$), as shown in Figure 12-21. It is used to develop the storage-indication curve. The storage-discharge curve is developed in two steps, as follows:

**Step 1 -** Plot the range of discharge rates on the x-axis. The range should include the peak post-construction inflow rate. Discharges higher than this rate are not needed.

**Step 2 -** Select a discharge value on the x-axis. Determine the stage for this discharge using the stage-discharge curve (see Subsection 12.8.3). Determine the storage for this stage using the stage-storage curve (see Subsection 12.8.2). Plot the corresponding storage on the y-axis. Repeat this step as needed to well define the storage-discharge curve.

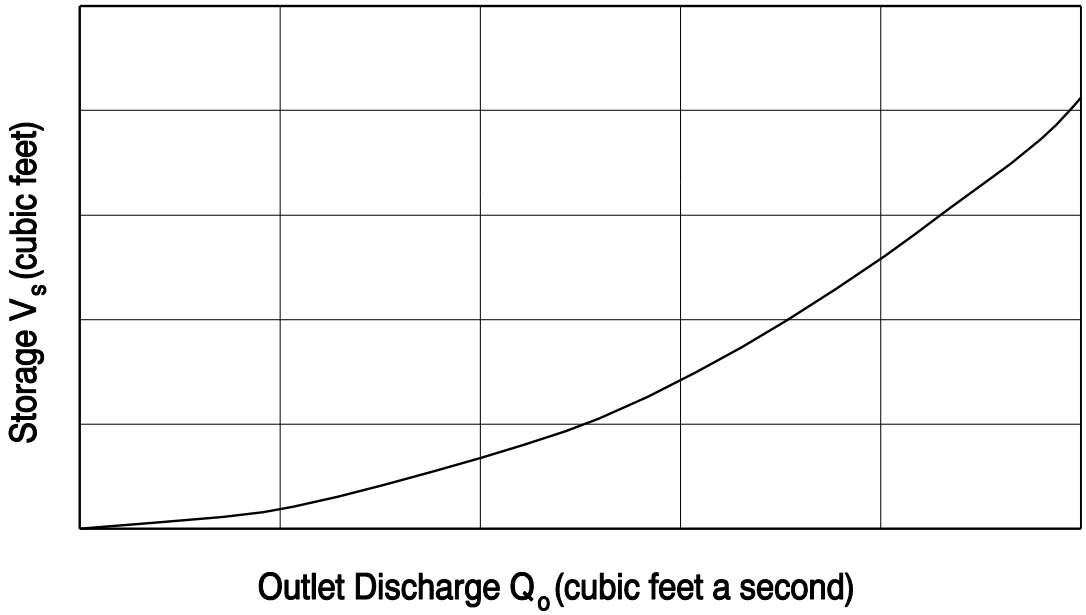



**Figure 12-21 Storage-Discharge Curve**

### 12.8.5 Storage Indication Curve

The storage indication curve is an abstract relationship that is used in the storage routing procedure. Unlike the other curves, it does not represent a physical phenomenon. The values of $(V_s + 1/2Q_o\Delta T)$ are shown on this curve, and they are determined as follows:

**Step 1 -** Select a value of outlet discharge, $Q_o$.

**Step 2 -** Determine the corresponding storage volume $V_s$ from the storage-discharge curve.

**Step 3 -** Use the values of $Q_o$ and $V_s$ determined in the previous two steps to compute $(V_s + 1/2Q_o\Delta T)$.

Where:

$\Delta T$ = Time interval used to develop the inflow hydrograph in seconds
$Q_o$ = Discharge rate from the outlet in cubic feet per second
$V_s$ = Storage volume in the facility in cubic feet

**Step 4 -** Plot a point on the storage indication curve to correspond to the values $Q_o$ determined Step 1 and $(V_s + 1/2Q_o\Delta T)$ determined in Step 3.

Repeat these four steps for a sufficient number of values of $V_s$ to well define the storage indication curve. An example of a curve is shown in Figure 12-22.

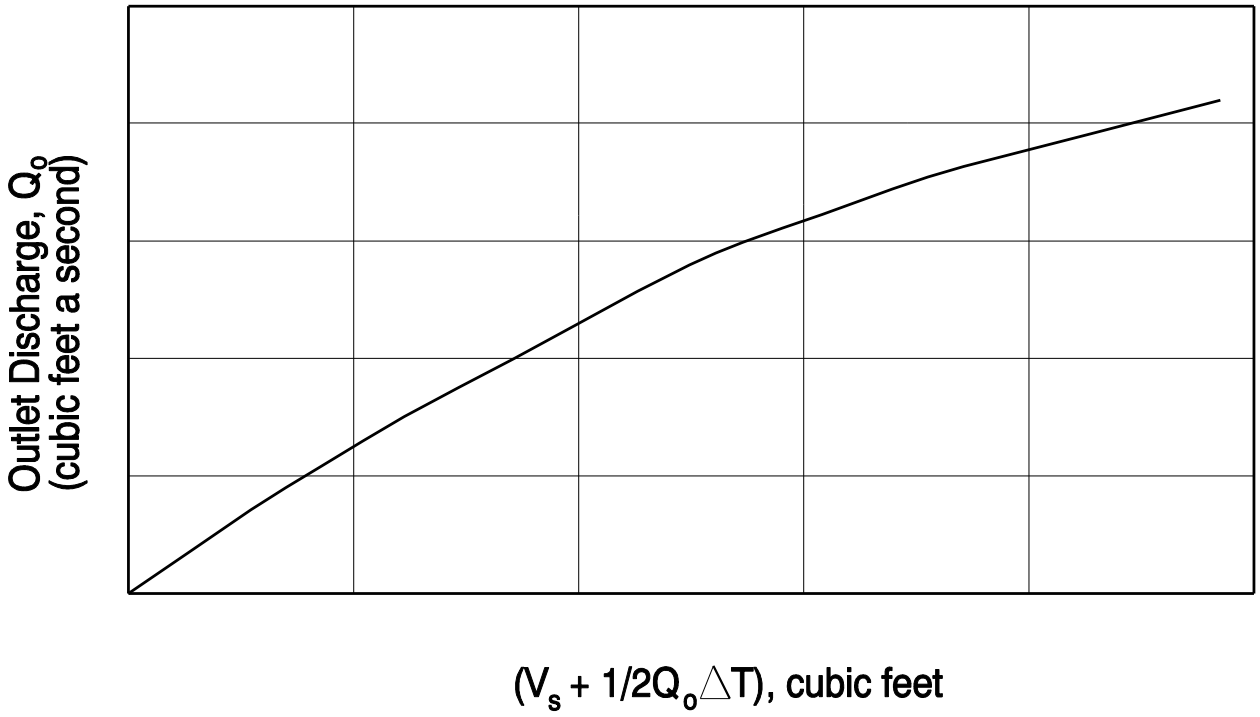



**Figure 12-22 Storage Indication Curve**

## 12.9 Outlet Flow Control Structures

An outlet flow control structure is needed to control flow leaving the stormwater storage facility. In most cases the outlet structure consists of a catch basin or vault or manhole, and a primary and/or auxiliary outlet. Its primary functions are:

- a primary outlet to release the attenuated discharges from the design storms,
- and an auxiliary outlet to release discharge from the check storm or lesser storms if the primary outlet is clogged.

The following design guidance and criteria is for the two preferred types of outlet flow control structures. Other flow control structures may be proposed but must perform the primary functions noted above.

*Note: The hydraulic characteristics of outlets are fairly complex. These references are cited in this subsection, and they contain additional information:*

- *American Society of Civil Engineers (ASCE) and Water Pollution Control Federation (WPCF) Joint Committee, Design and Construction of Sanitary and Storm Sewers, ASCE Manuals and Reports on Engineering Practice No. 37 and WPCF Manual of Practice No. 9, 1969),*
- *Brater, E. F. and H. W. King, Handbook of Hydraulics, 6th Edition (McGraw Hill: New York, N.Y., 1976),*
- *Chow, Ven Te, Open-Channel Hydraulics, (McGraw-Hill: New York, N.Y., 1959),*
- *Sandvik, A., Proportional Weirs for Stormwater Pond Outlets, "Civil Engineering" (American Society of Civil Engineers: New York, N.Y. 1985), and*
- *U.S. Department of Agriculture - Soil Conservation Service (SCS), Standards and Specifications for Soil Erosion and Sediment Control in Urbanizing Areas, (SCS: College Park, MD, 1969).*

### 12.9.1 Outlet Flow Control Structure I

Outlet flow control structure I is most commonly used with dry ponds, tanks, and vaults. It is illustrated in Figures 12-23 and 12-24. A riser pipe and orifice(s) are used to limit the rate of runoff draining from a stormwater storage facility. One orifice would be placed at the bottom of the flow control riser pipe and additional orifices are mounted along the side of the flow control riser pipe as needed to limit the rate of runoff. The riser pipe is also used to convey or bypass high flows if the flow control orifices are plugged by debris or trash. High flows drain using the pipe opening located at the top (auxiliary outlet) of the flow control riser pipe.

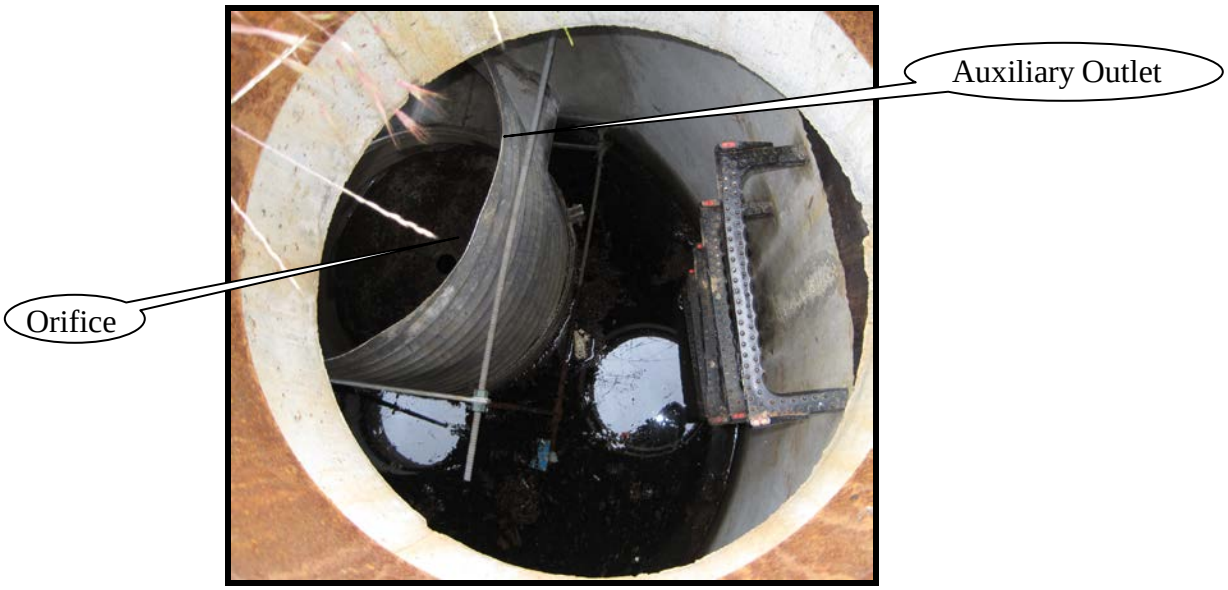



**Figure 12-23 Outlet Flow Control Structure I (Pipe Riser) Photo**

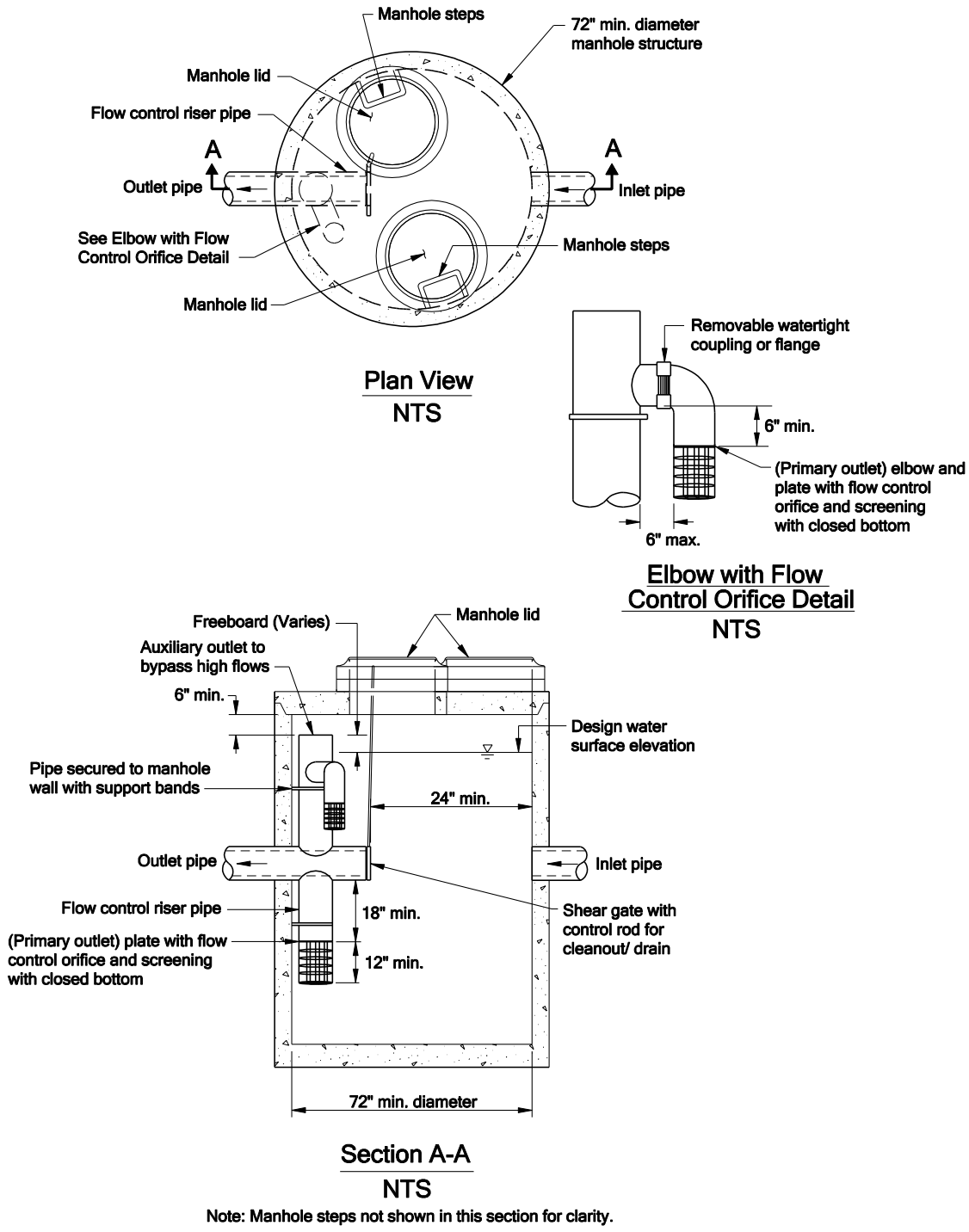



**Figure 12-24 Outlet Flow Control Structure I (Pipe Riser)**

### 12.9.1.1 Design Criteria for Outlet Flow Control Structure I

This section describes the features of an outlet flow control structure I and the design criteria that apply specifically to these installations.

**Outlet Flow Control Structure I Manhole Dimensions**

1. The minimum manhole diameter is 72 inches.

2. Avoid structure **depths** greater than 20 feet due to the limitations of vactor trucks. Adjacent access to the structure is needed for the vactor to operate to this maximum depth. Therefore, verify access is appropriate by coordinating the design with the Maintenance District.

**Outlet Flow Control Structure I Features**

1. The **flow control orifice(s)** must be sized according to the design storm requirements discussed in Section 12.5.1.1.

2. Provide **orifice screening** to protect orifice(s) from plugging for all orifices 6 inches or less. Orifice screen must contain multiple openings that are equal to or less than the orifice diameter.

3. The **auxiliary outlet** shall be designed to convey the design high flow. The design high flow is the 100-year post construction peak flow. The auxiliary outlet must be designed so it operates as a weir rather than an orifice. The minimum riser pipe size is 12 inches.

**Sediment Control**

1. Provide a sump with a minimum depth of 2 feet.

**Maintenance and Inspection Access**

1. The facility access road must extend to the outlet flow control structure for maintenance and inspection. Access road requirements are discussed in sections 12.5.2 (dry ponds), 12.5.3 (tanks), and 12.5.4 (vaults).

### 12.9.2 Outlet Flow Control Structure II

Outlet flow control structure II is most commonly used with dry ponds, tanks, and vaults. It is illustrated in Figure 12-25. A barrier with a flow control weir is placed across the center of the manhole creating two compartments. The upstream compartment contains the upstream conveyance system inlet pipe. All flows exit the upstream compartment by way of the flow control weir and into the downstream compartment that contains the outlet pipe. The weir is

designed to limit the rate of runoff draining from a stormwater storage facility. It also is used to convey high flows.

#### 12.9.2.1 Design Criteria for Outlet Flow Control Structure II

This section describes the features of an outlet flow control structure II and the design criteria that apply specifically to these installations.

**Outlet Flow Control Structure II Manhole Dimensions**

1. The minimum manhole diameter is 72 inches.

2. Avoid structure **depths** greater than 20 feet due to the limitations of vactor trucks. Adjacent access to the inlet structure is needed for the vactor to operate to this maximum depth. Therefore, verify access is appropriate by coordinating the design with the Maintenance District.

**Outlet Flow Control Structure II Features**

1. The **flow control weir** must be centered within the manhole. The top of barrier and weir is set equal to the stormwater storage facility design water surface elevation. Design storm requirements are discussed in Section 12.5.1.1. The weir must also convey the design high flow. The design high flow is the 100-year post construction peak flow.

**Sediment Control**

1. Provide a sump with a minimum depth of 2 feet.

**Maintenance and Inspection Access**

1. The facility access road must extend to the outlet flow control structure for maintenance and inspection. Access road requirements are discussed in sections 12.5.2 (dry ponds), 12.5.3 (tanks), and 12.5.4 (vaults).

2. Provide manhole steps on each side of the barrier and weir for maintenance and inspection.

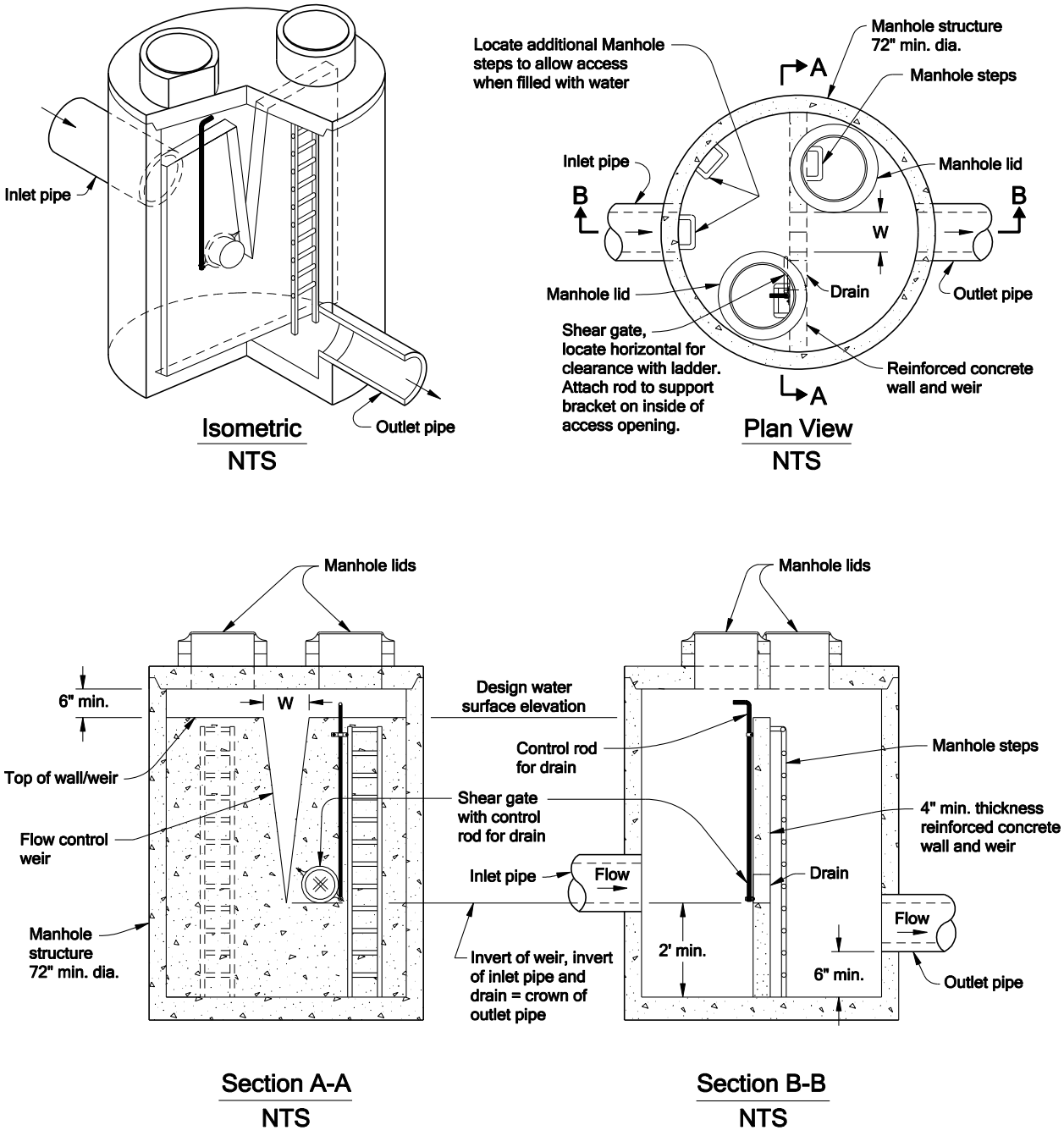



**Figure 12-25 Outlet Flow Control Structure II (V-Notch Weir)**

### 12.9.3 Methods of Analysis (Primary Outlets)

The hydraulic characteristics of primary outlets are discussed in this subsection. The primary outlet is usually designed with a capacity sufficient to convey the design flood within the allowed freeboard criteria. See Subsections 12.5.2 (ponds), 12.5.3 (tanks), or 12.5.4 (vaults). A sharp or broad-crested weir, orifice, pipe or other appropriate outlet can be used. The primary outlet can convey discharge to a pipe, riser, spillway, or other suitable receiving facility.

#### 12.9.3.1 Weirs

**Sharp-Crested Weirs -** Sharp-crested weirs are flow control devices that are usually made from flat plates or wood planks, as shown in Figure 12-26. The nappe is not supported by the weir, as shown in Figure 12-26c. This is a distinguishing feature of a sharp-crested weir.

**Sharp-Crested Weir with No End Contractions (Unsubmerged) -** A sharp-crested weir with no end contractions is illustrated in Figure 12-26a, and the unsubmerged flow condition is illustrated in Figure 12-26c. The discharge equation for this configuration, assuming that there is still water upstream from the weir, is (Chow, 1959),

$$Q = C_{scw} L (H_1^{1.5}) \qquad \text{(Equation 12-17)}$$

Where:

Q = Discharge in cubic feet per second
$C_{scw} = [3.27 + 0.4(H_1/H_c)]$
L = Horizontal weir length in feet
$H_1$ = Upstream head above weir crest excluding velocity head in feet
$H_c$ = Height of weir in feet

As indicated above, the value of the coefficient $C_{scw}$ is known to vary with the ratio $H/H_c$. For values of the ratio $H/H_c$ less than 0.3, a constant $C_{scw}$ of 3.33 is often used.

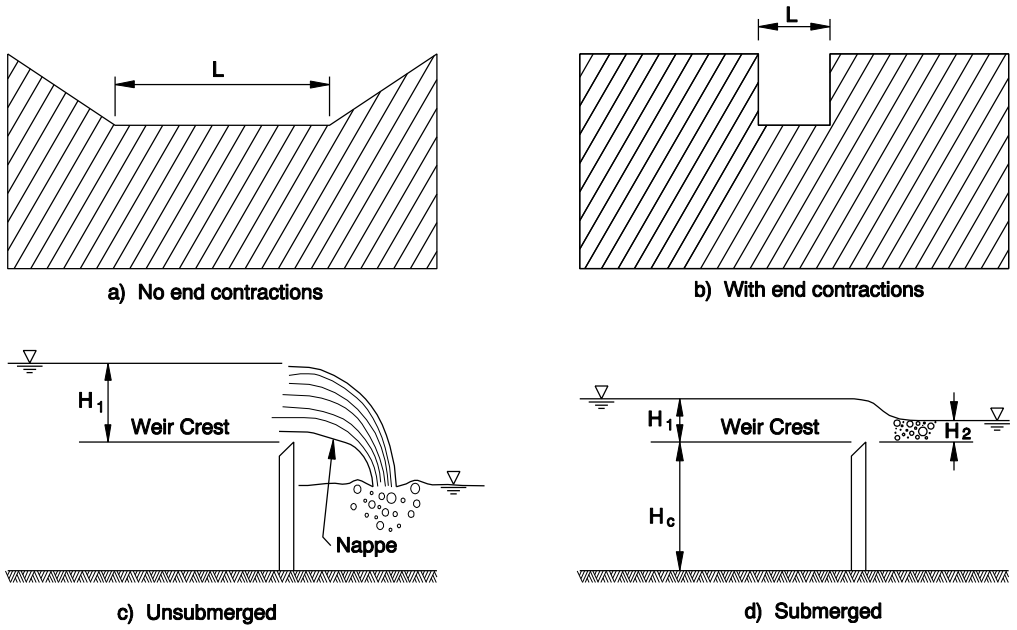



**Figure 12-26 Sharp-Crested Weirs**

**Sharp-Crested Weir with End Contractions (Unsubmerged)** - A sharp-crested weir with two end contractions is illustrated in Figure 12-26b. A notch weir, illustrated in Figure 12-27, is another configuration of the sharp-crested weir with two end contractions. The discharge equation for this configuration is (Chow, 1959):

$$Q = C_{scw}(L - 0.2H_1)(H_1^{1.5}) \qquad \text{(Equation 12-18)}$$

Where: Variables are the same as Equation 12-17.

**Sharp-Crested Weir with or Without End Contractions (Submerged)** - A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation, as illustrated in Figure 12-26d. The result will be that the discharge over the weir will be reduced.

The discharge equation for a submerged sharp-crested weir with or without end contractions is (Brater and King, 1976):

$$Q_s = Q_r\left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385} \qquad \text{(Equation 12-19)}$$

Where:

$Q_s$ = Submergence flow in cubic feet per second

$Q_r$ = Unsubmerged weir flow from Equations 12-17 or 12-18 in cubic feet per second

$H_1$ = Upstream head above crest in feet
$H_2$= Downstream head above crest in feet

**Sharp-Crested Side Overflow Weir *(Unsubmerged)* -** A sharp-crested weir operating as a side overflow is illustrated in Figure 12-28. The discharge equation for this configuration is (ASCE 1969):

$$Q = C_{scw}(L - 0.2H_1)\ (H_1^{1.67}) \qquad \text{(Equation 12-20)}$$

Where:

Q = Discharge through weir in cubic feet per second
$H_1$ = Head above weir crest on the downstream end excluding velocity head in feet.
Other variables are the same as Equation 12-17.

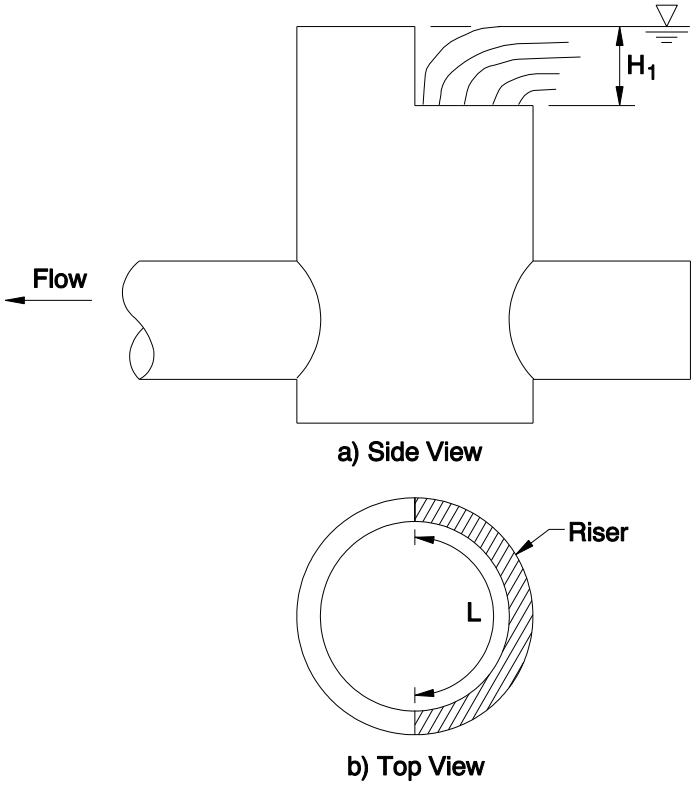



**Figure 12-27 Notch Weir**

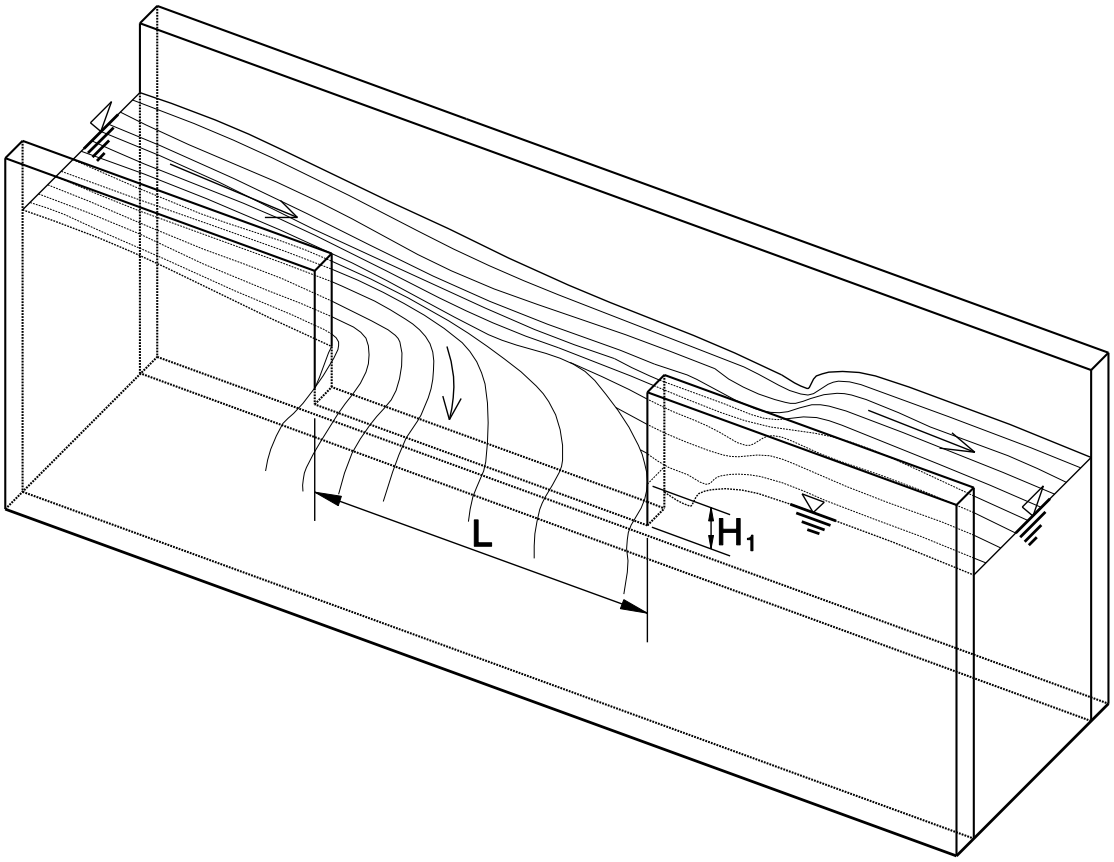



**Figure 12-28 Side Overflow Weir**

**Sharp-Crested V-Notch Weirs (Unsubmerged) -** A v-notch weir is a form of sharp-crested weir, as shown in Figure 12-29. The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976).

$$Q = 1.38 \tan\left(\frac{\theta}{2}\right)(H_1^{2.5}) \qquad \text{(Equation 12 - 21)}$$

Where:

Q = Discharge in cubic feet per second
Θ = Angle of v-notch in degrees
$H_1$ = Head depth above apex of notch in feet

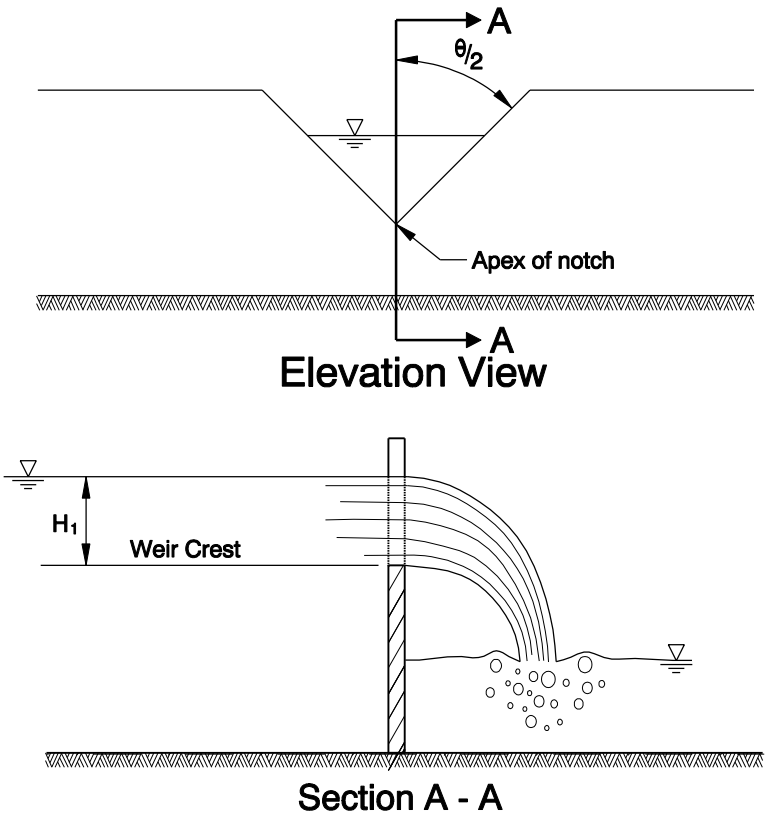



**Figure 12-29 V-Notch Weir**

**Sharp-Crested Proportional Weirs (Unsubmerged) -** Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is a sharp-crested weir, and it is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to

vary nonlinearly in comparison to the head depth. A proportional weir is shown in Figure 12-30 and the design equations are (Sandvik, 1985):

$$Q = 2.74\,(a^{0.5})\,b\left(H_1 - \frac{a}{3}\right) \qquad \text{(Equation 12 - 22)}$$

and

$$\frac{x}{b} = 1 - (0.315)\left[\arctan\left(\frac{y}{a}\right)^{0.5}\right] \qquad \text{(Equation 12 - 23)}$$

Where:

Q = Discharge in cubic feet per second
$H_1$ = Head above horizontal sill, as shown in Figure 12-30 in feet
A,Dimensions a, b, x, and y as shown in Figure 12-30 in feet

**Broad-Crested Weirs (*Unsubmerged*) -** Broad-crested weirs are usually made from reinforced concrete or timber. Unlike the sharp-crested weir, the broad crested weir supports the nappe, as shown in Figure 12-31. These weirs can be used as either primary outlets or auxiliary outlets. The equation generally used for an unsubmerged broad-crested weir is (Brater and King, 1976):

$$Q = CL(H_1^{1.5}) \qquad \text{(Equation 12-24)}$$

Where:

Q = Discharge in cubic feet a second
C = Broad-crested weir coefficient (2.34 to 3.32)
L = Broad-crested weir length in feet
$H_1$ = Head above weir crest in feet
b = Breadth of crest of weir in feet (This dimension is used in Table 12-2 for selection of a weir coefficient.)
$H_c$ = Height of weir in feet ($H_1$ occurs at a distance of 2.5 $H_c$ upstream from the weir face, as shown in Figure 12-31.)

The weir coefficient is influenced by the shape of the weir as shown in Table 12-2. If the upstream edge of a broad-crested weir is so rounded or inclined as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.07. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. The weir coefficient is also influenced by the depth of the head upstream from the weir and the breadth of the weir. Table 12-2 shows this relationship, and the weir used to develop this table is shown in Figure 12-31. For more discussion about coefficients for broad crested weirs with inclined or rounded upstream edges refer to Brater and King.

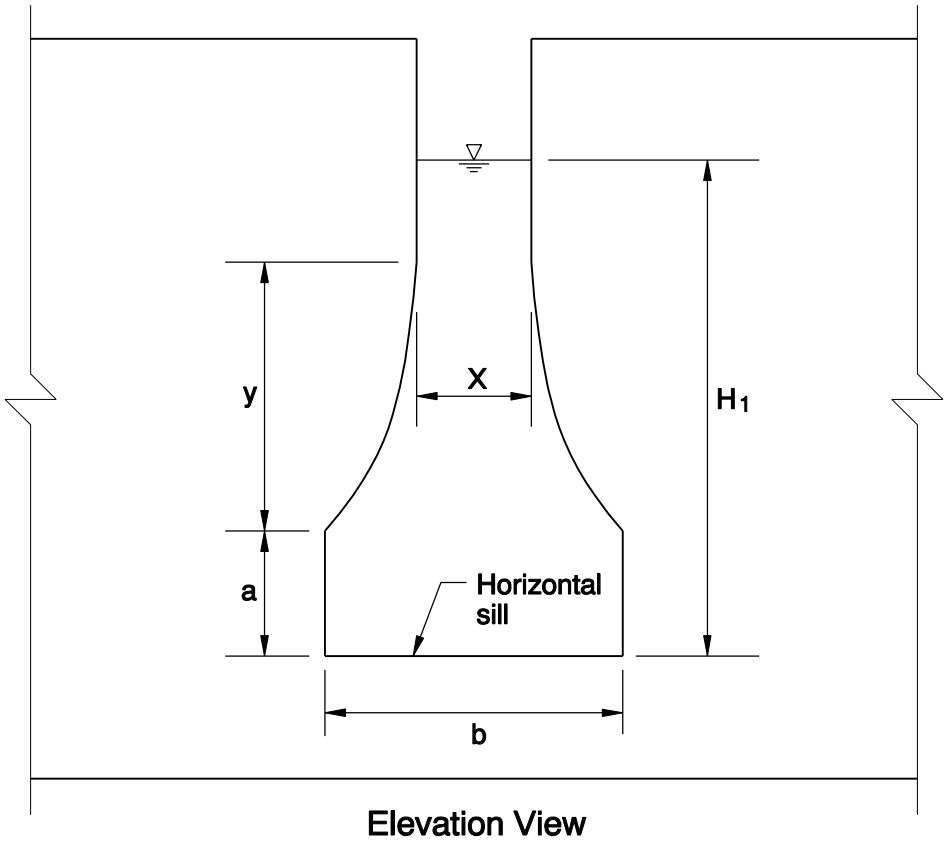
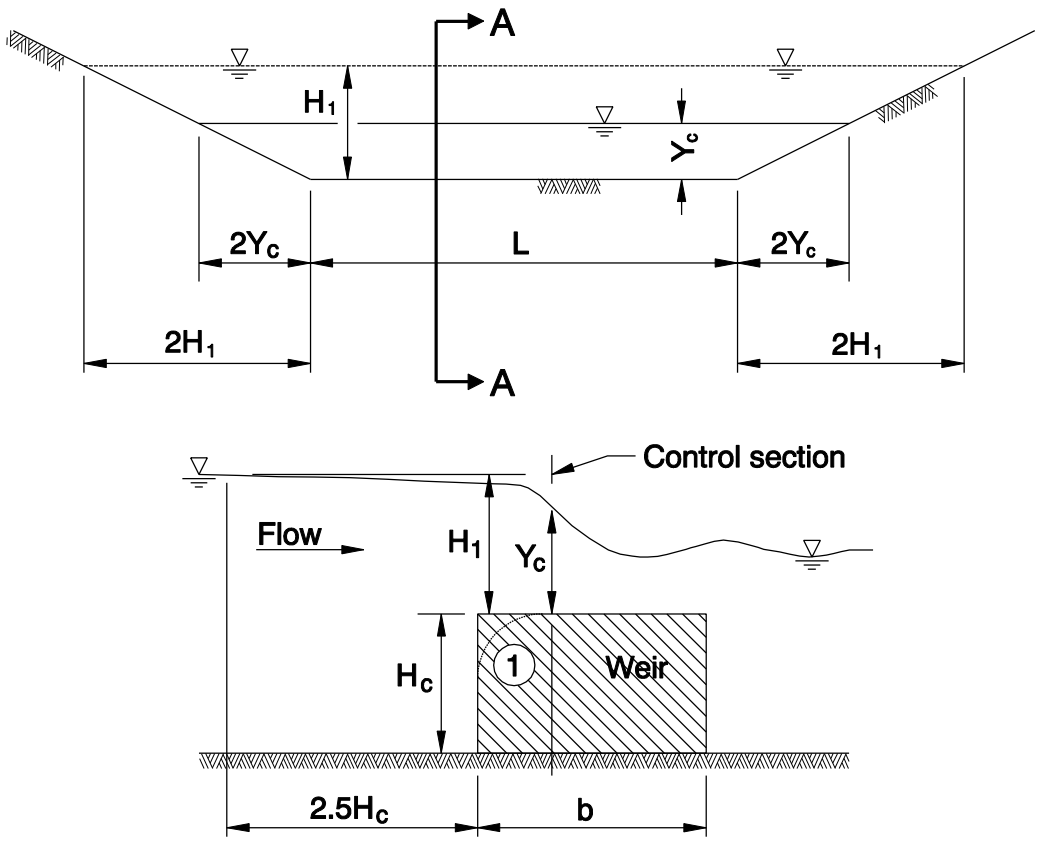



**Figure 12-30 Proportional Weir**




Cross-Section A-A

(1) Optional inclined or rounded crest

$Y_c$ = Critical depth at control section
$H_1$ = Effective head upstream from spillway measured at a distance 2.5 $H_c$ upstream from weir face.

**Figure 12-31 Broad-Crested Weir**

### 12.9.3.2 Orifices

The typical detention control orifice is a square edged circular hole in a metal plate, as shown in Figures 12-32a and b. Orifices can be vertical, as shown in Figure 12-32c, or they can be horizontal, as shown in Figure 12-32d. The flow through a single square edged circular orifice can be determined using the following equation:

$$Q = CA(2gH_o)^{0.5} \quad \text{(Equation 12-25)}$$

Where:

Q = Discharge in cubic feet per second
C = Discharge coefficient (0.40 – 0.60)
A = Cross-sectional area of orifice opening in square feet
g = Acceleration due to gravity, 32.2 square feet per second
$H_o$ = Effective head on the orifice, measured from the centroid of the orifice opening in feet (See note.)

*Note: Effective head is measured as follows:*

- if the orifice discharges as a free outfall, then the effective head is measured from the centerline of the orifice to the upstream water surface elevation, as shown in Figure 12-33a,
- if the tailwater elevation is higher than the center of the opening, the effective head is calculated as the difference in water surface elevations, as shown in Figure 12-33b,
- if the orifice is in a group, orifices at the same elevation can be analyzed as a single orifice and the flow rate multiplied by the number of orifices at the same elevation, as shown in Figure 12-33c,
- if the orifice is in a pipe with a down-turned elbow, the effective head is the difference in elevation between the water surface in the pipe and the free water surface, as shown in Figure 12-33d,
- if the orifice is in the bottom of a riser, the effective head is the difference in elevation between the water surface in the outlet pipe and the free water surface, as shown in Figure 12-33e.

For square-edged, smooth finished, uniform orifice entrance conditions, a discharge coefficient of 0.6 should be used. For ragged edged orifices, such as those resulting from the use of an acetylene torch to cut orifice openings, a value of 0.4 should be used.

For circular orifices with C set equal to 0.60, the following equation results:

$$Q = 3.78\, D^2\, H_o^{0.50} \quad \text{(Equation 12-26)}$$

**Table 12-2 Broad-Crested Weir Coefficient C Values as a Function of Weir Crest Breadth and Head (feet)**

| Measured Head, $H_1$ | Breadth of the Crest of Weir, b (feet) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| (ft) | 0.50 | 0.75 | 1.00 | 1.5 | 2.0 | 2.50 | 3.00 | 4.00 | 5.00 | 10.00 | 15.00 |
| 0.2 | 2.80 | 2.75 | 2.69 | 2.62 | 2.54 | 2.48 | 2.44 | 2.38 | 2.34 | 2.49 | 2.68 |
| 0.4 | 2.92 | 2.80 | 2.72 | 2.64 | 2.61 | 2.60 | 2.58 | 2.54 | 2.50 | 2.56 | 2.70 |
| 0.6 | 3.08 | 2.89 | 2.75 | 2.64 | 2.61 | 2.60 | 2.68 | 2.69 | 2.70 | 2.70 | 2.70 |
| 0.8 | 3.30 | 3.04 | 2.85 | 2.68 | 2.60 | 2.60 | 2.67 | 2.68 | 2.68 | 2.69 | 2.64 |
| 1.0 | 3.32 | 3.14 | 2.98 | 2.75 | 2.66 | 2.64 | 2.65 | 2.67 | 2.68 | 2.68 | 2.63 |
| 1.2 | 3.32 | 3.20 | 3.08 | 2.86 | 2.70 | 2.65 | 2.64 | 2.67 | 2.66 | 2.69 | 2.64 |
| 1.4 | 3.32 | 3.26 | 3.20 | 2.92 | 2.77 | 2.68 | 2.64 | 2.65 | 2.65 | 2.67 | 2.64 |
| 1.6 | 3.32 | 3.29 | 3.28 | 3.07 | 2.89 | 2.75 | 2.68 | 2.66 | 2.65 | 2.64 | 2.63 |
| 1.8 | 3.32 | 3.31 | 3.30 | 3.03 | 2.85 | 2.76 | 2.72 | 2.68 | 2.65 | 2.64 | 2.63 |
| 2.0 | 3.32 | 3.31 | 3.30 | 3.03 | 2.85 | 2.76 | 2.72 | 2.68 | 2.65 | 2.64 | 2.63 |
| 2.5 | 3.32 | 3.32 | 3.31 | 3.28 | 3.07 | 2.89 | 2.81 | 2.72 | 2.67 | 2.64 | 2.63 |
| 3.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.20 | 3.05 | 2.92 | 2.73 | 2.66 | 2.64 | 2.63 |
| 3.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.19 | 2.97 | 2.76 | 2.68 | 2.64 | 2.63 |
| 4.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.07 | 2.79 | 2.70 | 2.64 | 2.63 |
| 4.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 2.88 | 2.74 | 2.64 | 2.63 |
| 5.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.07 | 2.79 | 2.64 | 2.63 |
| 5.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 2.88 | 2.64 | 2.63 |

Notes: 1) Head depth is measured at least $2.5H_1$ upstream from the weir.
2) If $H_1$ is more than or equal to 2b, then weir operates as sharp crested.
Brater and King (1976).

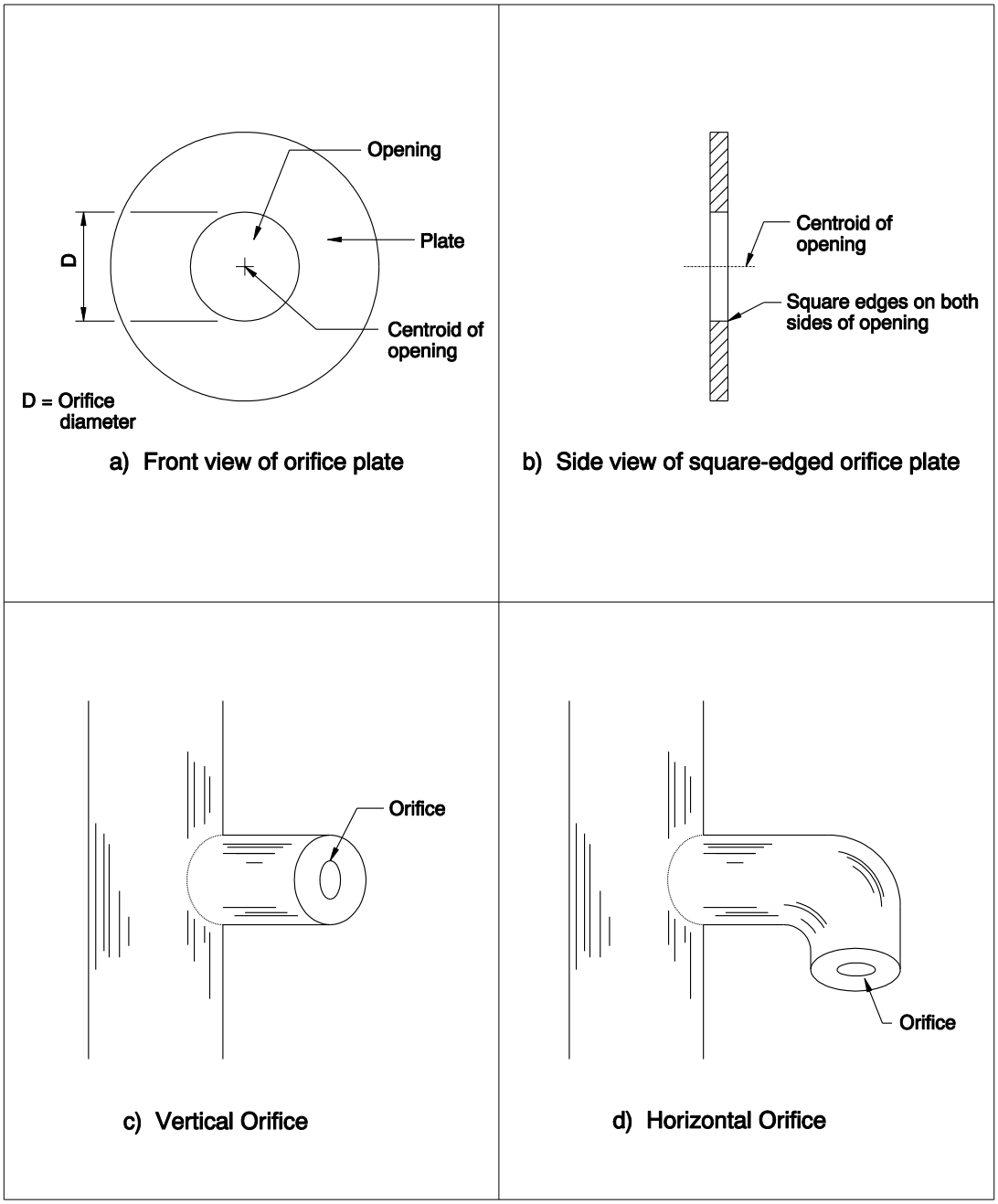



**Figure 12-32 Orifices**

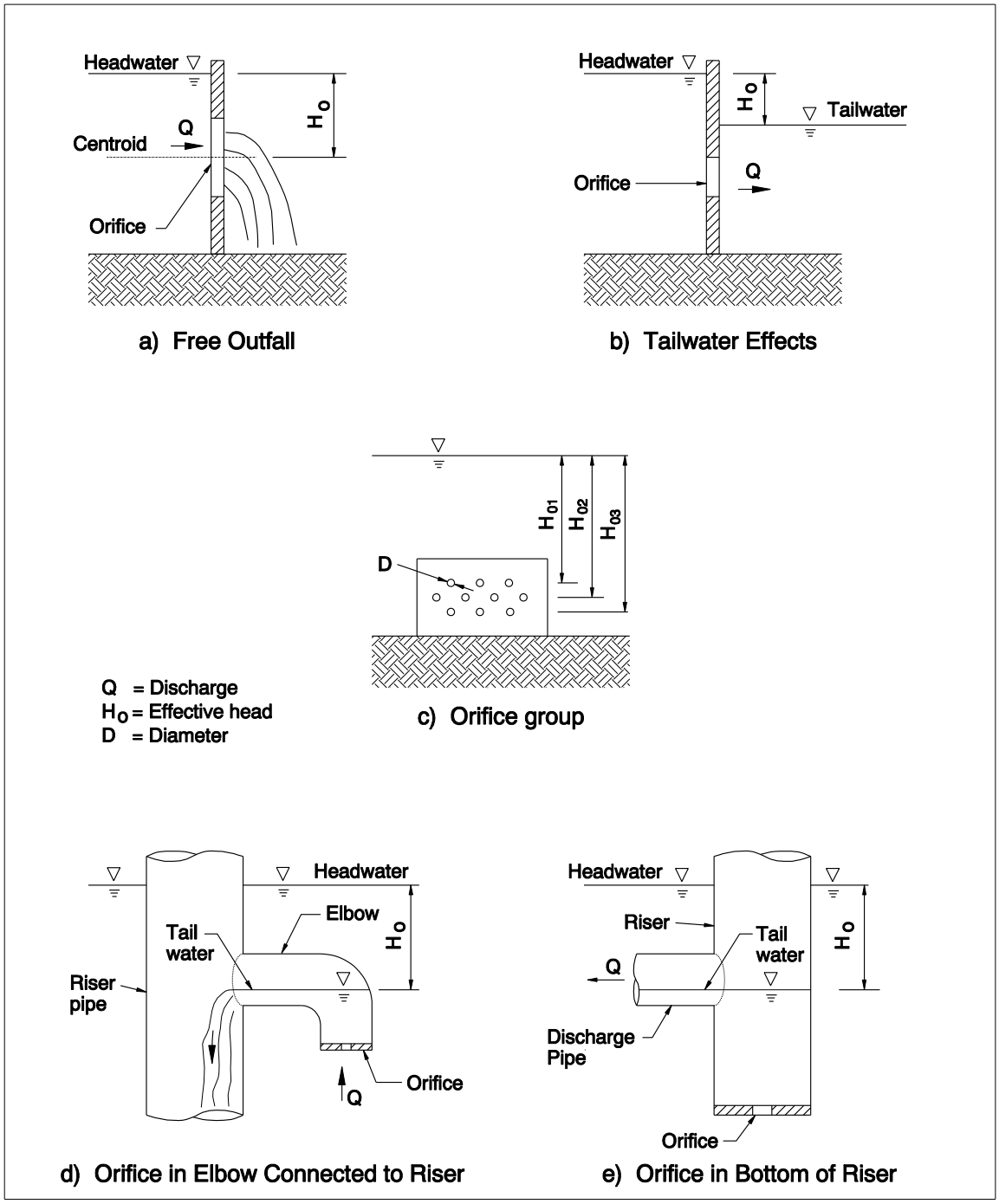



**Figure 12-33 Effective Heads for Orifices**

Sometimes it is useful to calculate the orifice diameter for a known head depth and discharge. The following form of the orifice equation can be used:

$$D = \left[ \frac{Q}{\left(3.78\, H_o^{0.5}\right)} \right]^{0.5} \qquad \text{(Equation 12 - 27)}$$

Where:

D = Diameter of orifice in feet

Other variables are the same as Equation 12-25.

**Pipes -** An outlet pipe smaller than 12 inches in diameter may be analyzed as a submerged orifice if the ratio of headwater depth to pipe diameter, H/D, is greater than 1.5. Larger outlet pipes or pipes with lower head depths should be analyzed using the procedures in **Chapter 9**.

### 12.9.4 Method of Analysis (Auxiliary Outlets)

The purpose of an auxiliary outlet is to provide a controlled overflow relief for storm flow in excess of the design discharge for the storage facility. The auxiliary outlet is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the outlet. The auxiliary outlet should be designed taking into account the potential threat to downstream life and property if the storage facility were to fail. Suitable auxiliary outlets for detention storage facilities on highway applications include:

- broad-crested overflow weirs and channels cut through the original ground if the detention facilities are excavated cavities,
- engineered broad-crested weirs and spillways if the detention facilities are contained within constructed embankments,
- riser overflow pipes with any style of basin, or
- large outlet pipe structures if the detention facilities are contained systems such as vaults or tanks.

#### 12.9.4.1 Broad-Crested Spillway Weirs

Broad-crested spillway weirs are used as auxiliary outlets on many detention basins to convey discharge safely out of the pond and around or down the embankment slope, as shown in Figure 12-34. The figure shows a spillway weir auxiliary outlet excavated in the natural ground that carries discharge around a constructed embankment. A spillway weir across the top of a constructed embankment would be similar, with the exception of the approach channel. The upstream face of the embankment would function as the approach channel. The transverse cross-section of the weir cut is typically trapezoidal in shape for both stability and ease of construction.

The hydraulic characteristics of spillway weirs are similar to the broad crested weirs described in Equation 12-24. One difference is the discharge coefficient. The coefficients for spillway weir applications are presented in this subsection, and they include adjustments for higher roughness

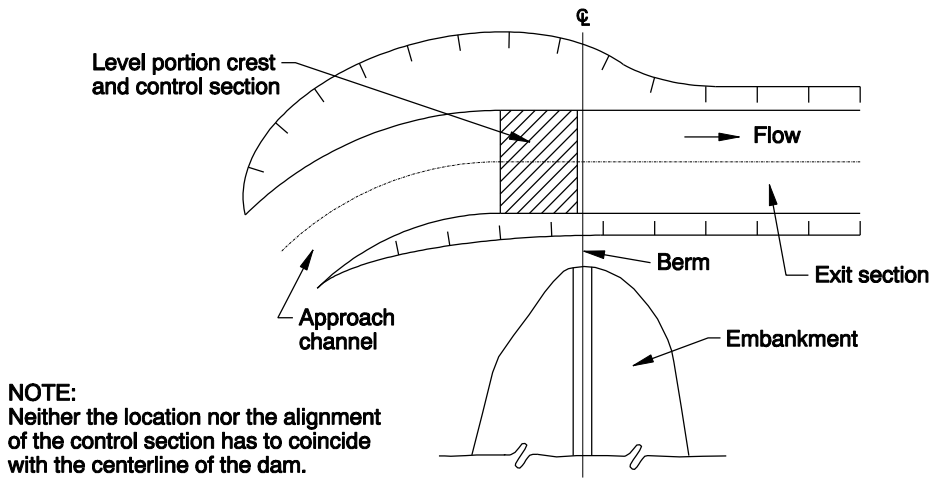
coefficients across the top of the weir. Another difference is the modification to the formula to include the side slopes of a trapezoidal section with significant roughness.

The relationship between discharge and head elevation for a broad-crested spillway weir can be calculated by Equation 12-28. The dimensional terms in the equation are illustrated in Figure 12-31.

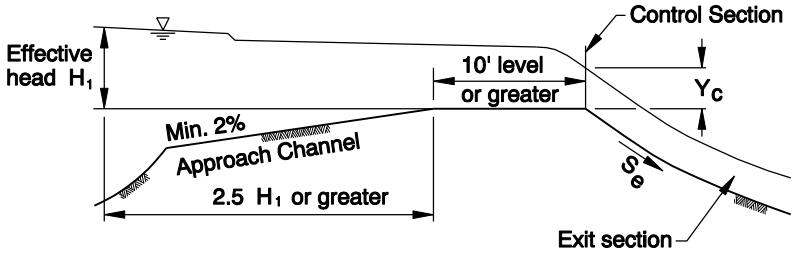
$$Q = C_{sp}\,(2g)^{1.5}\left[\frac{2}{3}LH_1^{1.5} + \frac{8}{15}(\text{Tan}\,\theta)\,H_1^{2.5}\right] \qquad \text{(Equation 12 - 28)}$$

Where:

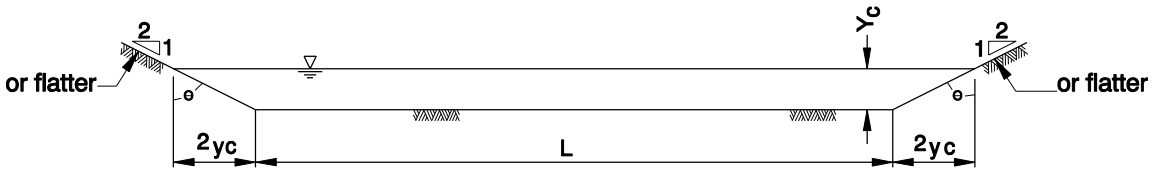
| | | |
|---|---|---|
| Q | = | Spillway discharge in cubic feet per second |
| $C_{sp}$ | = | Discharge coefficient |
| g | = | gravitational acceleration, 32.2 in square feet per second |
| L | = | Width of the spillway in feet |
| $H_1$ | = | Effective head on the spillway in feet |
| Tan θ | = | Angle of side slope for trapezoidal section, |
| | | For 1V:2H side slope, Tan θ = 1.73 |
| | | 1V:3H side slope, Tan θ = 2.83 |
| | | 1V:4H side slope, Tan θ = 3.87 |




a. Plan View of Excavated Auxiliary Outlet Spillway




b. Profile Along Centerline




$Y_c$ = Critical depth at control section

c. Cross-Section at Control Section

**Figure 12-34 Auxiliary Spillway Design Schematic**

The discharge coefficient, $C_{sp}$, varies as a function of spillway roughness. For auxiliary spillways with established grass turf or rip rap protection use $C_{sp} = 0.4$. Paved spillways can use $C_{sp} = 0.6$.

Another useful form of the auxiliary spillway equation is,

$$L = \frac{\left\{\left(\frac{3}{16}\right)\left(\frac{Q}{C}\right) - \left[0.8\,(\text{Tan}\,\theta)\,H^{2.5}\right]\right\}}{H^{1.5}} \qquad \text{(Equation 12 - 29)}$$

Where the variables are the same as Equation 12-28.

Flow in a channel at or near critical depth is unstable and subject to hydraulic jumps and drops. The resulting turbulence can cause excessive shear stresses on the spillway lining. Also, the shear stress through auxiliary spillways should be checked to assure adequate protection from erosion is provided. As a result, it is recommended that the exit section be designed with a slope one percent flatter or steeper than the critical slope. The hydraulics of open-channels such as exit sections is discussed in more detail in **Chapter 8**.

### 12.9.4.2 Overflow Risers

Most outlets use riser pipes of concrete or corrugated metal. The riser and the pipes downstream from the riser must be adequately sized to convey the entire check storm overflow. This discharge would occur during the check storm if the outlet was clogged. Risers can often be designed to control the runoff from different design storms through the use of several orifices on the riser, and the larger check storm discharge flows into the top of the riser.

The analysis of flow into a riser is fairly complex because several types of flow can occur. When the water in the basin first overtops the riser pipe, sharp-crested weir flow can occur over the entire perimeter of the riser. As the flow depth above the top of the riser becomes deeper, the flow regime changes to orifice flow. Figure 12-35 shows the relationships between head depth and discharge into the riser and the point where the flow regime changes.

Sharp-crested weir flow into risers can be calculated by several methods, and two procedures are presented in this chapter. One method is to use the chart in Figure 12-35. The other method is to determine the weir length for the riser using Table 12-3 and to calculate the flow into the riser using Equation 12-17.

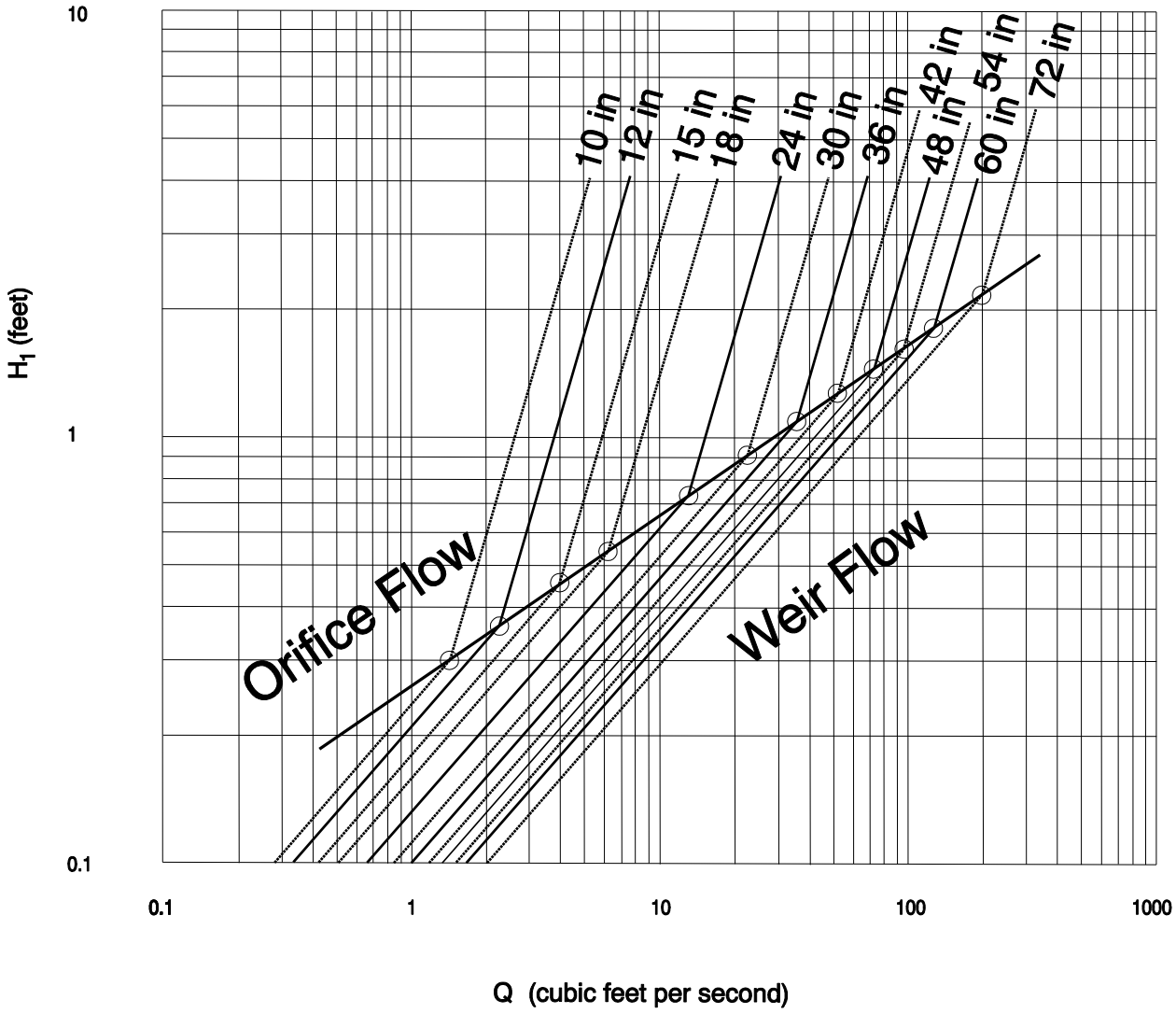



**Figure 12-35 Riser Overflow Hydraulics**

**Table 12-3 Weir Lengths for Pipe Risers**

| Pipe Diameter, D (inches) | Weir Length, $L = \pi D$ (feet) |
|---|---|
| 10 | 2.62 |
| 12 | 3.14 |
| 18 | 4.71 |
| 24 | 6.28 |
| 30 | 7.85 |
| 36 | 9.42 |
| 42 | 11.00 |
| 48 | 12.57 |
| 54 | 14.14 |
| 60 | 15.71 |
| 72 | 18.85 |

Risers operating in orifice flow may be subject to a phenomenon called vortex flow. This is a circular spiraling of flow immediately above the submerged riser and it can reduce the flow through an orifice by as much as 75 percent. Vortex flow can be prevented by the addition of an anti-vortex plate, as shown in Figure 12-36. Another method of preventing vortex flow is to increase the size of the riser to ensure that weir flow is predominant during the check storm and orifice flow does not occur.

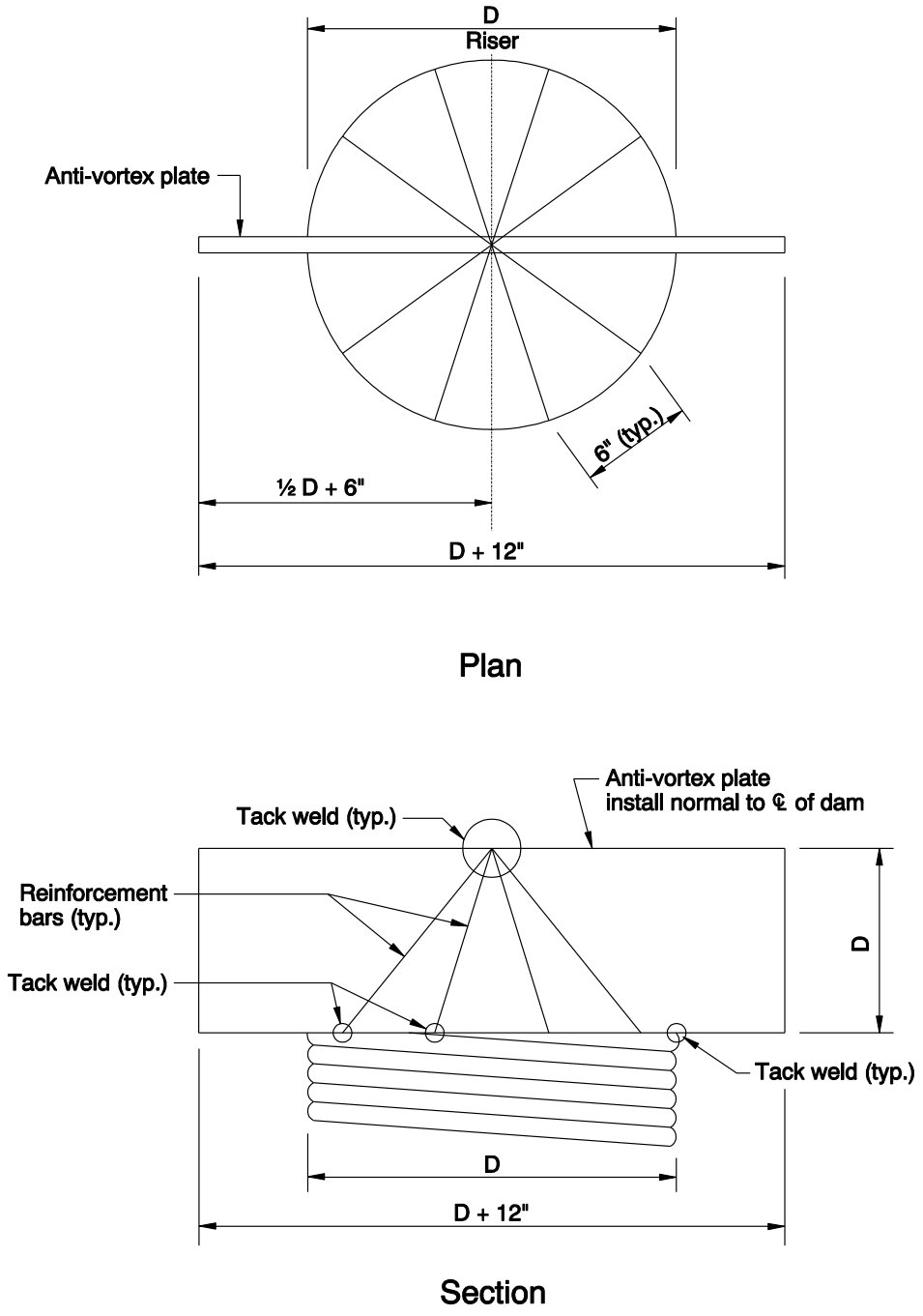



**Figure 12-36 Anti-Vortex Device and Trash Rack**

### 12.9.5 Anti-Seep Collars

Embankments designed to impound large amounts of water under high head are sometimes susceptible to seepage through the embankment fill. Seepage may occur along the outside of a pipe (also known as piping) which will lead to the washing out of the material surrounding the pipe, and this can lead to a catastrophic failure of the embankment fill. Anti-seep collars can be used as a countermeasure to reduce or eliminate seepage alongside the pipe.

Anti-seep collars should be considered:

- at impoundments where the safety of the dam is especially critical,
- at installations where the saturated length of the pipe without collars ($L_s$) is greater than 75 percent of the embedded length ($L_e$) of the pipe (see Figure 12-37), or
- sites where it may be difficult to make a waterproof seal between the backfill around the pipe and the outside of the conduit.

This chapter presents a simplified method of designing anti-seep collars and it is intended to augment, but not replace, the geotechnical aspects of dam design necessary to produce a safe and serviceable structure. More sophisticated methods of designing collars are recommended for critical structures.

The earth embankment surrounding a water filled impoundment is composed of both saturated and unsaturated soils. The division between saturated and unsaturated soil is often represented on the embankment cross-section by the phreatic line. This line is often curvilinear and it is determined from a flow net analysis of the passage of water through the soil. In the procedure described in this section, a conservative and simplified approximation is made by extending a straight phreatic line through the embankment at a 1V:4H slope from the point of intersection between the check storm water surface and the inside face of the embankment. This assumed phreatic line is shown in Figure 12-37 and soils above and below this line are assumed to be unsaturated and saturated, respectively.

*Note: An assumed phreatic line extending through the outside face of the embankment indicates that water may flow through the embankment. This should be avoided and several means are commonly used, such as increasing the width of the embankment or placing a waterproof barrier within the embankment*

The length of the pipe that is surrounded by soil and within the saturated zone is $L_s$, as shown in the figure. This distance is also the flow path distance alongside the pipe in the saturated zone if there are no anti-seep collars. The objective of anti-seep collars is to increase the flow path distance by a factor of 1.2, and this increase will make it more difficult for water to travel through the embankment alongside the pipe. This increased flow path length is $L_{s(Revised)}$. The flow path increase can be done by a small number of large collars or an increased number of smaller collars. The following equations can be used. Equation 12-30a provides $L_{s(Revised)}$ if the number of collars and collar dimensions are known, and Equation 12-30b provides the needed number of collars if $L_{s(Revised)}$ and the collar dimensions are known.

$$a)\ L_{S\,(Revised)} = L_s + 2nV \qquad or \qquad b)\ n \geq \frac{(0.05\,L_S)}{V} \qquad \text{(Equation 12 - 30)}$$

Where:

$L_{s(Revised)}$ = Revised flow path length alongside conduit with collars in feet (This distance must be equal to or greater than $1.2L_s$.)

$L_s$ = Length of pipe that is surrounded by soil and within the saturated zone in feet

n = Number of anti-seep collars.

V = Minimum vertical projection of collar measured perpendicular to the pipe in feet

The collar shape can be round or square, as shown in Figure 12-37. The "V" distances are the same for square and round collars with same widths and diameters, respectively. Although the square collar typically uses more metal, it is often provided due to ease of fabrication. The total outside width of the collar (W) can be determined as follows:

$$W = D + 2T + 2V \qquad \text{(Equation 12-31)}$$

Where:

W = Total width or diameter of collar in feet

D = Pipe inside diameter in feet

T = Pipe wall thickness in feet

V = Vertical projection as defined for Equation 12-30

The anti-seep collar and its pipe connection must be watertight. The anti-seep collars should be equally spaced and located on the section of pipe that is within the saturated zone of the embankment. The maximum spacing between the collars is 14 times the "V" distance. There must be sufficient distance between the collars for adequate placement and compaction of the bedding material. Collars should not be located within 2 feet of any pipe joint.

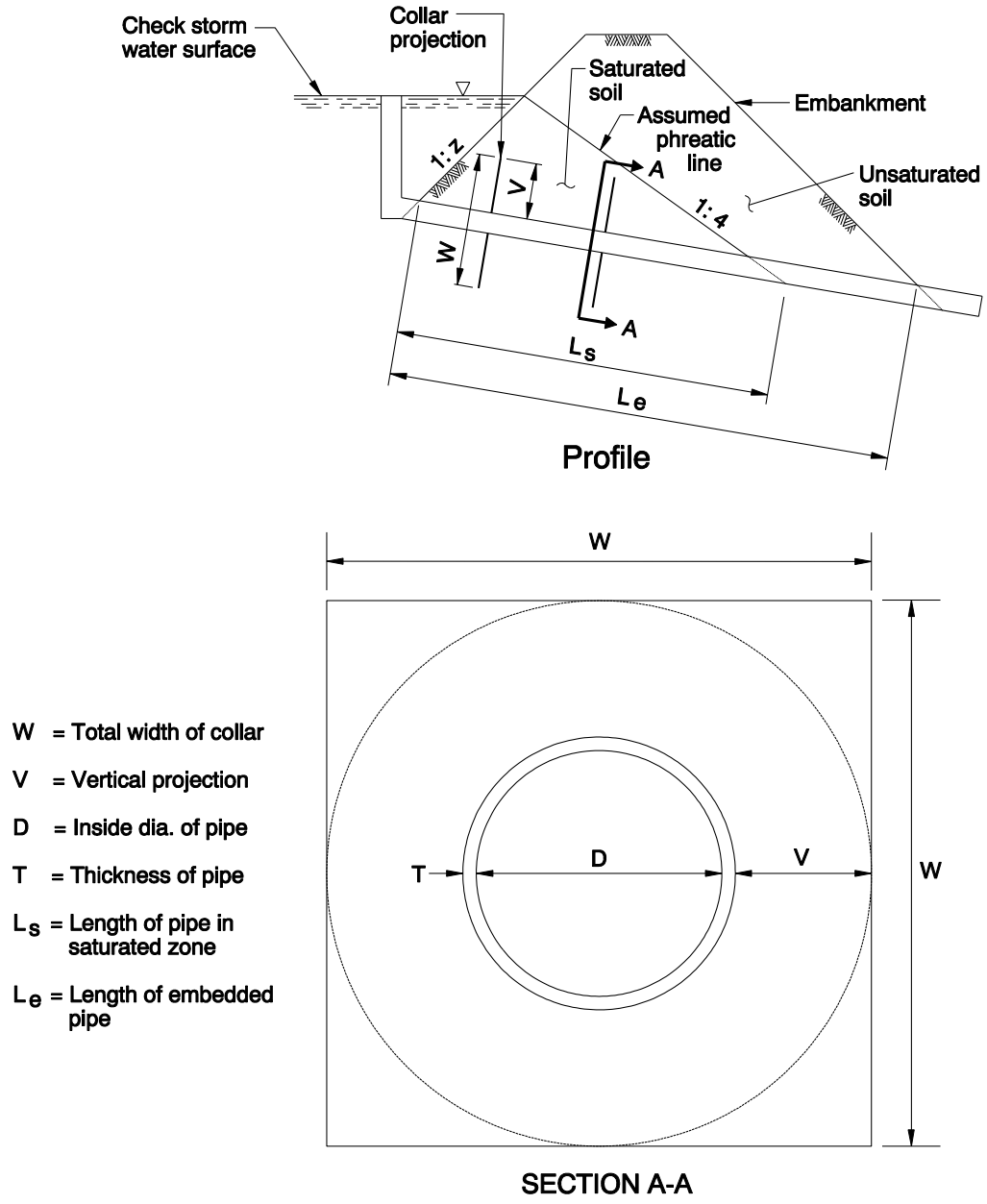



**Figure 12-37 Anti-Seep Collars**

## 12.10 Method Selection

The intent of these guidelines is to aid in the selection of methods for sizing detention storage systems to minimize the drainage impacts produced by changes in land use. The guidance is primarily intended to address the attenuation of increased peak flows due to changes in land use on agency right-of-way. The methods can also be used in many cases to analyze the attenuation of peak flows draining from adjacent property onto the agency right-of-way or into agency drainage systems.

There are many methods currently in use to design detention facilities. They range in complexity from simplified methods that can be done by hand or spreadsheet applications, such as the simplified rational method to detailed iterative approaches that are best modeled by computer programs, such as the unit hydrograph and routing procedure. When selecting a method, one must be aware that the complexity of the calculations or modeling does not guarantee a more accurate representation of required storage volumes. The sensitivity of several variables, such as time-of-concentration and runoff coefficients, can significantly skew results. It is recommended practice to calculate storage volumes and release rates using the appropriate method with input variables at the upper and lower limits of the probable range of input values, and to compare the results of these analyses prior to the final design. As an example, the time of concentration might not be precisely known, but it may be possible to determine the shortest and longest probable times. In this case, detention volumes would be calculated for shortest and longest probable times, and the detention system would be designed to provide adequate storage for the worst case. When selecting which method(s) to use the following issues should be considered:

- use (preliminary estimate or final design),
- size of drainage area,
- surrounding land use, and
- local regulations.

Projects which are small or are surrounded by land uses which would not be adversely impacted by periods of shallow flooding (such as forests or farmland) require different levels of calculation precision than projects which are large or are adjacent to populated areas. Local regulations may require a specific method be used for final computations. The applications for the two methods presented in this chapter are listed in Table 12-4.

### 12.10.1 Computer Programs

The routing calculations needed to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations there are many reservoir routing computer programs. These programs include, but are not limited to, the U.S. Corps of Engineers

HEC-1 and HEC-HMS, the U.S. Natural Resources Conservation Service TR-55, and the Federal Highway Administration HYDRA module in the HYDRAIN software package.

## 12.11 Simplified Rational Method

The simplified rational method uses the assumptions for the rational method described in **Chapter 7**, Appendix F. The primary assumptions used by the rational method include:

- the duration of the rainfall event is equal to the time-of-concentration, and
- the rainfall intensity and runoff rate are constant during the entire rainfall event.

**Table 12-4 Storage Methodology Selection**

| Methodology | Calculation Method | Complexity | Applicability |
|---|---|---|---|
| Simplified Rational | Hand or Spreadsheet | Low | Drainage area less than or equal to 5 acres or preliminary estimate. |
| Unit Hydrograph with Routing | Computer Model | High | Areas of all sizes. Preliminary estimates or final design. (Check software documentation.) |

The peak design release rate $Q_o$ ($Q_o$ less than or equal to peak pre-construction runoff rate) and peak post-construction inflow rate $Q_i$ are determined using the Rational Equation.

$$Q = C_f\, C\, i\, A$$ (**Chapter 7**, Appendix F, Equation 1)

By using the assumptions for the rational method and plotting the peak runoff (Q) and a time interval (T), square hydrographs can be developed as shown in Figure 12-38. The required storage is a function of the depletion (release) rate ($Q_o$), inflow (runoff) rate ($Q_i$), and the time of detention (T). The general equation is:

$$V_S = T\,(Q_i - Q_o)$$ (Equation 12-32)

Where:

$V_S$ = Storage volume estimate in cubic feet

$Q_i$ = Post-construction design inflow rate when maximum storage volume is required in cubic feet per second

$Q_o$ = Peak design release rate in cubic feet per second
T = Time duration to design inflow rate $Q_i$, s

The storage volume is calculated for as many time intervals necessary to find the peak volume required. For any time interval the calculated storage should not exceed the storage capacity of the detention basin. The tabular form in Table 12-5 can be used with the following procedure to determine the required storage volume:

**Step 1 -** Determine the Outflow Rate ($Q_o$). This is the Peak Design Release Rate based on criteria in Subsection 12.5.1.1. **Chapter 7**, Appendix F, Equation 1 can be used to calculate $Q_o$. Enter $Q_o$ into all rows of Column 6.

**Step 2 -** Determine the impervious area CA. Use the post-construction runoff coefficient. See **Chapter 7**, Appendix F for guidance. Enter in all rows of Column 2.

**Step 3 -** Select the Time (T). Usually 5-minute intervals are used. Enter T in rows sequentially in Column 1. There should be enough intervals to cover a duration of time approximately equal to the time-of-concentration of the pre-construction basin. It is seldom necessary to analyze intervals beyond this duration.

**Step 4 -** Determine Rainfall Intensity (i). Intensities are shown in **Chapter 7**, Appendix A. Enter in Column 3 opposite the corresponding Time Interval in Column 1.

**Step 5 -** Calculate the Inflow Rate ($Q_i$). $Q_i$ = ($C_f$) (Column 2) (Column 3). $C_f$ is the runoff coefficient adjustment factor from **Chapter 7**, Appendix F, Table 2. Enter in Column 4 for each time interval.

**Step 6 -** Calculate the Inflow Volume ($V_i$). $V_i$ = (60) (Column 1) (Column 4). Enter in Column 5 for each time interval.

**Step 7 -** Calculate the Outflow Volume ($V_o$). $V_o$ = (60) (Column 1) (Column 6). Enter in Column 7 for each time interval.

**Step 8 -** Calculate the Required Storage ($V_s$). $V_s$ = Column 5 - Column 7. Enter in Column 8 for each time interval.

**Step 9 -** Determine the maximum Required Storage Volume among the values in Column 8. Evaluate the storage requirements at subsequent time intervals until the required storage volume begins to drop. If the maximum Required Storage Volume occurs at the last time interval, additional time intervals should be analyzed.

**Step 10 -** Calculate the time of concentration for the post-construction drainage area ($T_{c'}$). In this method, it is assumed that part of the drainage area is contributing discharge at

time intervals less than $T_{c'}$, and the full drainage area is contributing flow at time intervals equal to or longer than $T_{c'}$. If the time interval where peak storage occurs, T, is less than $T_{c'}$, the CA value in the storage calculations should be adjusted as follows:

$$CA_2 = CA_1\left(\frac{T}{T_{c'}}\right) \qquad \text{(Equation 12 - 33)}$$

for $T < T_{c'}$:

Where:

$T_{c'}$ = Time of concentration in the post-construction condition, in minutes
T = Time interval where peak storage occurs, in minutes
$CA_1$ = CA for post-construction condition using entire contributing area in basin $A_1$, acres
$CA_2$ = Adjusted CA for post-construction condition using reduced contributing area $A_2$ corresponding to T, acres

If T determined in Step 9 is less than $T_{c'}$, it is recommended that the storage requirements be checked with adjusted CA values for additional time increments before and after T, in order to verify that the correct maximum required storage value has been selected.

**Step 11 -** Determine the dimensions of the storage facility that will retain the maximum required storage. The methods of determining the dimensions of various basin, tank, or vault shapes are presented in Subsection 12.8.2.

**Step 12 -** Select a primary outlet device and determine the effective head that will force the water through the outlet during the design storm at the stage when peak storage occurs. Size the primary outlet to release water at the peak design release rate when the water is at this stage. The peak release rate must meet the criteria in Subsection 12.5.1.1. Designing outlet flow control structures (outlet works) is discussed in Section 12.9.

**Step 13 -** Calculate the peak inflow rate from the check storm for post-construction conditions. Many of the parameters used in steps 2-5 for the design flood can be used to determine the check flow.

**Step 14 -** Assume that the outlet is plugged and the entire check storm discharge passes through the auxiliary outlet, and do the following:

- design the auxiliary outlet,
- design, as needed, scour and erosion protection downstream from the auxiliary outlet, and

- establish the minimum top of embankment elevation that will provide the necessary freeboard.

**Step 15 -** Summarize the basin and outlet dimensions determined in the prior analysis. See example.

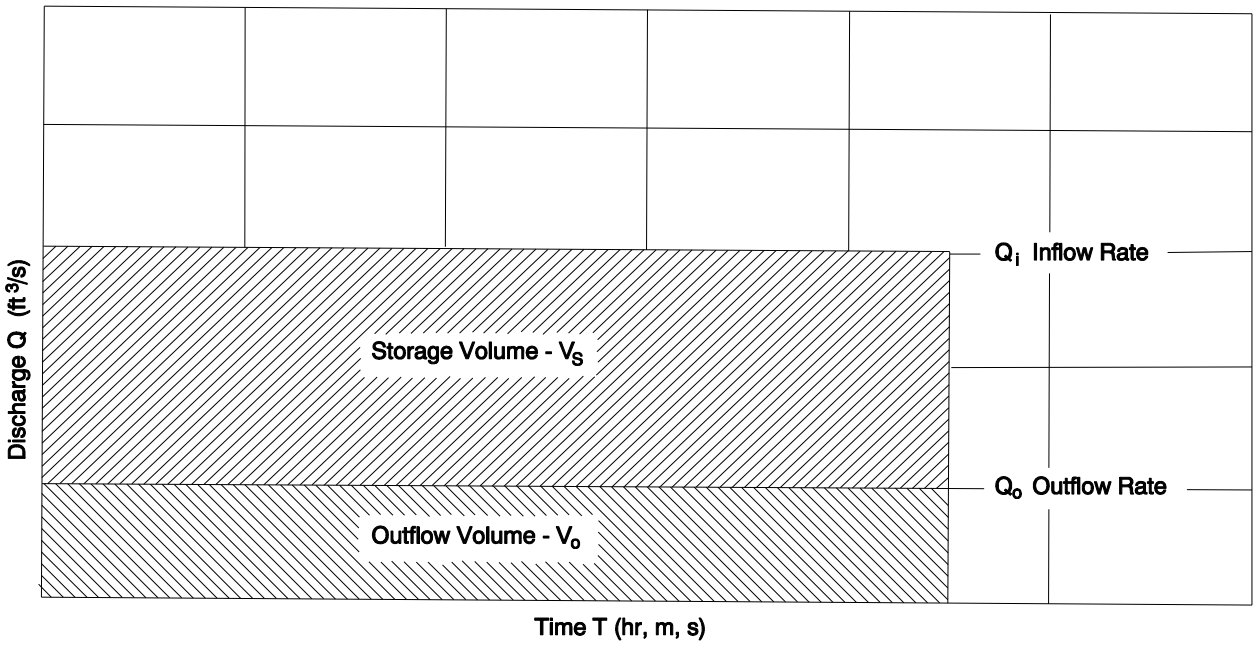



**Figure 12-38 Simplified Rational Method Storage Volume**

**Table 12-5 Tabular Form for Simplified Rational Method**

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|
| **Time (T)** | **CA*** | **Rainfall Intensity (i)** | **Inflow Rate ($Q_i$)** | **Inflow Volume ($V_i$)** | **Outflow Rate ($Q_o$)** | **Outflow Volume ($V_o$)** | **Req'd Storage ($V_S$)** |
| Minute | Acre | Inches per hour | Cubic feet per second | Cubic feet | Cubic feet per second | Cubic feet per second | Cubic feet per second |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

*CA adjusted by $C_f$ as needed.

### 12.11.1 Example: Simplified Rational Method

Drainage from a naturally landscaped basin shown in **Chapter 7**, Appendix F, Figure 3 flows to a highway culvert crossing. Peak flow is needed for a 50-year design storm using the Rational Method, $Q = C_fCiA$. The project is in Bend. The 10.9 acre site is proposed to be completely redeveloped into a flat dense residential neighborhood. These calculations are shown in tabular form in Table 12-6.

**Step 1 -** The Outflow Rate ($Q_o$) is determined in this step. It is the same as the Peak Design Release Rate based on existing conditions. A runoff coefficient adjustment factor ($C_f$) is needed because peak flow is to be determined for a 50-year storm. From **Chapter 7**, Appendix F, Table 2, the adjustment factor is: $C_f = 1.2$

A composite runoff coefficient (C) is needed because the drainage basin contains subareas with different C values. Subarea C values are from **Chapter 7**, Appendix F, Table 1, and the composite C value is calculated as follows:

| Description | "C" Value | Area (acres) | Impervious Area $C_iA_i$ (acres) |
|---|---|---|---|
| Rolling Forest | 0.15 | 3.2 | 0.5 |
| Flat Light Residential | 0.35 | 3.0 | 1.1 |
| Flat Pasture | 0.25 | 4.7 | 1.2 |
| Total | | 10.9 | 2.8 |

The composite runoff coefficient is calculated using **Chapter 7**, Appendix F, Equation 2 as follows:

$$C = \frac{(0.55 + 1.1 + 1.2)}{10.9} = \frac{2.8}{10.9} = 0.26$$

The total drainage area (A) is also calculated during this step: A = 10.9 acres

The rainfall intensity (i) is needed. The Bend area is in Zone 10 according to the I-D-R Curve Index in **Chapter 7**, Appendix A. The 50-year curve will be used. To obtain the rainfall intensity, the time of concentration, $T_c$, must first be estimated. For this example, the drainage path used to determine the time of concentration is composed of two segments. The first segment is 300 feet long and it is assumed to be overland sheet flow. The remaining 900 feet long segment is assumed to be shallow concentrated flow.

**Overland Sheet Flow Segment -** The travel time for the overland sheet flow segment is calculated as follows:

From **Chapter 7**, Appendix F, Table 3: n = 0.40 (woodland and forest)

L = 300 feet , n = 0.4, S = 5 percent .

From Chapter 7, Appendix F, Equation 4 : $T_{osf} = \frac{0.93\,(300^{0.6}\ 0.4^{0.6})}{(i^{0.4}\ 0.05^{0.3})} = \frac{40.4}{i^{0.4}}$

| Tc (assumed) | i (I-D-R Chart) | Tc (Formula) | |
|---|---|---|---|
| 30 | 1.5 | 34.5 | N.G. |
| 40 | 1.2 | 37.6 | N.G. |
| **36** | **1.3** | **36.4** | **O.K.** |

From the 50-year, Zone 10, I-D-R curve, and using a trial and error solution: $T_{osf}$ = 36 minutes

**Shallow Concentrated Flow Segment -** The travel time for the shallow concentrated flow segment is calculated as follows:

From the location data, 160 feet of the drainage path is over forested land with a 5 percent slope: L = 160 feet and S = 5 percent

From **Chapter 7**, Appendix F, Figure 1 V = 0.575 feet per second

From **Chapter 7**, Appendix F, Equation 5, the travel time for shallow concentrated flow over forested land is:

$$T_{scf}\ (\text{forested land}) = \frac{160}{(60)\,(0.575)} = 5\ \text{minutes}$$

From the location data, the last 740 feet of the drainage path is a grassed waterway at a 1 percent slope: L = 740 feet, and S = 1 percent

From **Chapter 7**, Appendix F, Figure 1: V = 1.5 feet per second

The travel time for shallow concentrated flow down the grassed waterway is:

$$T_{scf}\ (\text{grassed waterway}) = \frac{740}{(60)\,(1.5)} = 8\ \text{minutes}$$

From **Chapter 7**, Appendix F, Equation 3, the time of concentration is:
$T_c$ = 36 + 5 + 8 = 49 minutes

From the 50-year Zone 10 I-D-R curve using a rainfall duration that corresponds to the 49-minute $T_c$: i = 1.07 inches per hour

From **Chapter 7**, Appendix F, Equation 1, the peak flow is calculated as follows:

$Q_{50}$ = (1.2) (0.26) (1.07) (10.9) = 3.6 cubic feet per second
Enter in Column 6 in all rows.

**Step 2 -** The impervious area (CA) is determined in this step for the post-construction conditions. The 10.9 acre site is proposed to be completely redeveloped into a flat dense residential neighborhood. From **Chapter 7**, Appendix F, Table 1, the runoff coefficient (C) for a flat dense residential area is 0.70.

CA = (0.70) (10.9) = 7.63 acres
This value is entered in Column 2 in all rows.

**Step 3 -** The time intervals (T) are selected in this step. A time interval of 10 minutes is chosen. Time intervals up to 60 minutes are analyzed, and this total length of time encompasses the 49-minute time of concentration for the undeveloped watershed. It is expected that the peak storage will be needed during this 60-minute interval. Additional rows in the table will be left blank between the 10-minute intervals. These intermediate time increments will be analyzed as needed The time intervals are entered in Column 1.

**Step 4 -** The rainfall intensities (i) are determined during this step. The intensities are obtained from **Chapter 7**, Appendix A for each time interval, and these values are entered in Column 3 opposite their corresponding times in Column 1.

**Step 5 -** The Inflow Rate ($Q_i$) is calculated during this step. The runoff coefficient adjustment factor ($C_f$) of 1.2 is from **Chapter 7**, Appendix F, Table 2, and it is included in the calculations.

($Q_i$) = $C_f$ (Column 2) (Column 3)

For T = 10 minutes: $Q_i$ = (1.2) (7.6) (2.45) = 22.4 cubic feet per second

The inflow rate is calculated and entered in Column 4 for each time interval.

**Step 6 -** Inflow Volume ($V_i$) is calculated during this step.

($V_i$) = (60 sec/min) (Column 1) (Column 4)

For T = 10 minutes: $V_i$ = (60) (10) (22.4) = 13,459 cubic feet

The inflow volume is calculated and entered in Column 5 for each time interval.

**Step 7 -** Outflow Volume ($V_o$) is calculated during this step.

$(V_o)$ = (60 sec/min) (Column 1) (Column 6)

For T = 10 minutes: $V_o$ = (60) (10) (3.6) = 2,183 cubic feet

The outflow volume is calculated and entered in Column 7 for each time interval.

**Step 8 -** Required Storage $(V_s)$ is calculated during this step. $(V_S)$ = Column 5 - Column 7

For T = 10 minutes: $V_S$ = 13,459 – 2,183 = 11,276 cubic feet

The required storage is calculated and entered in Column 8 for each time interval.

**Step 9 -** The maximum Required Storage Volume is determined during this step. The largest storage volume requirement occurs between 40 and 50 minutes, according to the values in Column 8. An additional storage volume calculation was made at the 45-minute time interval, and the maximum storage of 18,110 cubic feet is needed at this interval.

**Step 10 -** The time of concentration for the post-construction conditions $(T_{c'})$ is calculated during this step. The drainage path used to determine the time of concentration is composed of two segments. The first segment is 160 feet long and it is assumed to be overland sheet flow. The remaining 600 foot long segment is assumed to be pipe flow.

**Overland Sheet Flow Segment -** The travel time for the overland sheet flow segment is calculated as follows:

From **Chapter 7**, Appendix F, Table 3: n = 0.08 (Urban Residential Areas)

L = 160 feet, n = 0.08, S = 5 percent.

From Chapter 7, Appendix F, Equation 4: $T_{osf} = \frac{0.93\,(160^{0.6}\ 0.08^{0.6})}{(i^{0.4}\ 0.05^{0.3})} = \frac{10.5}{i^{0.4}}$

| Tc (Assumed) | i (I-D-R Chart) | Tc (Formula) | |
|---|---|---|---|
| 5 | 3.22 | 6.6 | N.G. |
| 10 | 2.45 | 7.4 | N.G. |
| 8 | 2.68 | 7.1 | N.G. |
| **7** | **2.8** | **7.0** | **O.K.** |

From the 50-year, Zone 10, I-D-R curve, and using a trial and error solution: $T_{osf}$ = 7 minutes

**Pipe Flow Segment -** The travel time for the pipe flow segment is calculated as follows:

From the location data, 150 feet of the drainage path is through a 12 inch diameter storm drain system with a 5 percent slope: L = 150 feet, and S = 5 percent. Also from the location data, the last 450 feet of the storm drain system varies in diameter from 12 inches to 24 inches, at a one-half percent slope: L = 450 feet, and S = 0.5 percent.

Begin by assuming the velocity in the pipe is equivalent to the velocity when the pipe is flowing full. More detailed storm drain system hydraulic parameters may be used, when available, which show actual velocities in each section of pipe. From **Chapter 8** and **Chapter 13**:

- 12 inch diameter concrete pipe at 5 percent slope: $V_{Full}$ = 10.2 feet per second
- 12 inch diameter concrete pipe at 0.5 percent slope: $V_{Full}$ = 3.2 feet per second
- 24 inch diameter concrete pipe at 0.5 percent slope: $V_{Full}$ = 5.1 feet per second

The travel time for pipe flow is:

$$T_{pf} = \frac{150}{(60)(10.2)} + \frac{450}{(60)\left[\frac{3.2+5.1}{2}\right]} = 2 \text{ minutes}$$

From **Chapter 7**, Appendix F, Equation 3, the time of concentration is:

$T_{c'}$ = 7 (overland flow) + 2 (pipe flow) = 9 minutes

$T_{c'}$ is compared to the time interval, T, that corresponds to the time when peak storage volume is needed. As shown in the previous step, T = 49 minutes. This is greater than the post-construction $T_{c'}$ of 9 minutes. This indicates that drainage from the entire contributing basin enters the storage facility, so the maximum storage volume required is 18,110 cubic feet, as calculated in Step 8.

*Note: It is interesting to note that the location within the rainfall event where the peak runoff occurs does not correspond to the time when the peak storage volume requirement occurs.*

**Step 11 -** In this step, the basin dimensions are determined. The basin must retain 18,110 cubic feet during the design storm. A square trapezoidal basin (ratio of length to width = 1) having 1V:2H side slopes will be used. The basin depression must fit within an 85-foot by 200-foot area. A maintenance access road will be located just outside this footprint area.

Using a depth of 3 feet and Equation 12.3:

V = 18,110 cubic feet, D = 3 feet, Z = 2, and W/L = r = 1

$$L = \frac{\left[(-2 \times 3)(1+1)\right] + \left\{\left[(2 \times 3)^2 (1+1)^2\right] - \left[5.33 (2 \times 3)^2 \times 1\right] + \left[\frac{(4 \times 1 \times 18{,}110)}{3}\right]\right\}^{0.5}}{(2 \times 1)}$$

L = 71.6 feet.

There will be 1.5 feet of freeboard between the design storm water surface elevation and the auxiliary outlet rim elevation. It is assumed the water surface elevation during the check storm will be 1 foot higher than the auxiliary outlet rim elevation. And there will be another 6 inches of freeboard between the check storm elevation and the top if the embankment. This makes the estimated total depth of the pond:

$D_{Total}$ = 3 + 1.5 + 1 + 0.5 = 6 feet

The total pond top width is calculated using the formula $W_{Total}$ = ZD + L + ZD

$W_{Total}$ = [( 2) (6)] + 71.6 + [(2) (6)] = 95.6 feet

**Table 12-6 Storage Volume Tabulation for Simplified Rational Method Example**

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|
| **Time (T)** | **CA** | **Rainfall Intensity (i)** | **Inflow Rate ($Q_i$)** | **Inflow Volume ($V_i$)** | **Outflow Rate ($Q_o$)** | **Outflow Volume ($V_o$)** | **Req'd Storage ($V_S$)** |
| minute | acre | Inches per hour | Cubic feet Per second | Cubic feet | Cubic feet Per second | Cubic feet | Cubic feet |
| 10 | 7.6 | 2.45 | 22.4 | 13,459 | 3.6 | 2,183 | 11,276 |
| | | | | | | | |
| 20 | 7.6 | 1.87 | 17.1 | 20,546 | 3.6 | 4,367 | 16,179 |
| | | | | | | | |
| 30 | 7.6 | 1.47 | 13.5 | 24,227 | 3.6 | 6,550 | 17,677 |
| | | | | | | | |
| 40 | 7.6 | 1.22 | 11.2 | 26,809 | 3.6 | 8,733 | 18,076 |
| 45 | 7.6 | 1.13 | 10.3 | 27,935 | 3.6 | 9,825 | **18,110** |
| 50 | 7.6 | 1.05 | 9.6 | 28,841 | 3.6 | 10,917 | 17,925 |
| | | | | | | | |
| 60 | 7.6 | 0.92 | 8.4 | 30,325 | 3.6 | 13,100 | 17,225 |

This basin is too big because it will be wider than 85 feet. Try using a length-to-width ratio of 2 and side slopes 1V:3H: r=2, Z=3, D=3 feet.

$$L = \frac{[(-3 \times 3)(2+1)] + \left\{ \left[ (3 \times 3)^2 (2+1)^2 \right] - \left[ 5.33 (3 \times 3)^2 \times 2 \right] + \left[ \frac{(4 \times 2 \times 18{,}110)}{3} \right] \right\}^{0.5}}{(2 \times 2)}$$

L = 48.1 feet

$D_{Total}$ = 3 + 1.5 + 1 + 0.5 = 6 feet

$W_{Total}$ = [(3) (6)] + 48.1 + [(3) (6)] = 84.1 feet
R = 2, so the top width dimension in the other direction is obtained by 84.1 x 2 = 168.2 feet

The 168 feet x 84 feet footprint fits within the available 85 feet x 200 feet area and can be used for the project.

**Step 12 -** In this step the primary outlet control structure will be designed. An orifice will be sized to release the 3.6 cubic feet per second peak design release rate determined in Step 1. The head forcing the water through the orifice will be the 3 foot water depth calculated in Step 11. Using Equation 12-27, the diameter of the orifice will be:

For Q = 3.6 cubic feet per second, and $H_o$ = 3 feet

$$D = \left( \frac{3.6}{(3.78)(3^{0.5})} \right)^{0.5} = 0.74 \text{ feet} = 8.9 \text{ inches} = 8 - 7/8\text{ths inch diameter.}$$

This outlet has a diameter greater than 6 inches, and no trash screen is required, as per the criteria in Subsection 12.5.1.5.

**Step 13 -** In this step the post-construction check storm discharge is determined. The 100-year storm is the check storm, and it is calculated as follows:

The post-construction time-of-concentration ($T_{c'}$) is estimated to be 9 minutes using the procedures outlined in Step 10. These calculations are not shown in this example. From the 100-year Zone 10 I-D-R curve using a rainfall duration that corresponds to the 9 minute $T_{c'}$:

i = 3 inches per hour

From **Chapter 7**, Appendix F, Table 2, the runoff coefficient adjustment factor is:

$C_f$ =1.25

From **Chapter 7**, Appendix F, Equation 1, the peak flow is calculated as follows:

$Q_{100}$ = (1.25) (0.70) (3) (10.9) = 28.6 cubic feet per second

**Step 14(a) -** In this step the auxiliary outlet using a riser pipe will be sized and the minimum top of embankment elevation established. The check storm discharge is 28.6 cubic feet per second and it is assumed that all flow is passing through the auxiliary outlet. The

riser pipe will be sized so it operates as a weir, rather than an orifice, during the check flood. Based on Figure 12-35, risers with diameters from 36 inch or larger will provide weir flow. A 48-inch diameter riser will be used. Referring to Table 12-3, the sharp crested weir length, L = 12.57 feet. Equation 12-17 can be modified to solve for H as follows:

$$H = \left( \frac{Q}{3.33\,L} \right)^{2/3} = \left[ \frac{28.6}{(3.33)\,(12.57)} \right]^{2/3} = 0.78 \text{ feet}$$

This riser operates under 0.78 feet of head during the check storm.

*Note: Since a riser pipe was used for passing the check storm, all downstream pipes and channels must be sized for the check storm.*

The minimum top of embankment elevation will be calculated next. This elevation is a total of the following (see Figure 12-39):

- the elevation of the centroid of the orifice or the lip of the control structure; whichever is higher (in this example the lip of the control structure is used and this elevation will be assumed to be 0.00 feet),
- the height of the design storm water surface elevation above the centroid of the orifice or the lip of the control structure; whichever is higher (in this example this height is 3 feet),
- the freeboard between the design storm water surface elevation and the rim of the auxiliary outlet (in this example this freeboard is 1.5 feet),
- the head above the rim of the auxiliary outlet assuming the entire check flood passes through the outlet (in this example this height is calculated to be 0.78 feet), and
- the freeboard between the check flood water surface elevation and the minimum elevation of the top of the bank ( in this example this distance is assumed to be 0.50 feet).

The minimum top of bank elevation is shown in Figure 12-39, and it is calculated as follows:

0.00 + 3 + 1.5 + 0.78 + 0.5 = 5.8 feet less than 6 feet (Step 11) O.K.

**Step 14(b) -** In this step the auxiliary outlet using a broad-crested spillway weir will be sized instead of using the riser pipe. The check storm discharge is 28.6 cubic feet per second and it is assumed that all flow is passing through the auxiliary outlet. If it is assumed a rock lined spillway weir with 1V:3H side slopes and a flow depth of 1 foot, then based on Equation 12-29,

$$L = \frac{\left\{\frac{\left(\frac{3}{16}\right)28.6}{(0.4 - [(0.8)(2.83)(1)^{2.5}]}\right\}}{(1)^{1.5}} = 11.2 \text{ foot wide bottom width of weir is required}$$

The minimum top of bank elevation is calculated as follows:
0.00 + 3 + 1.5 + 1 + 0.5 = 6 feet less than or equal to 6 feet (Step 11) O.K.

**Step 15 -** The design information is summarized as follows:

**Hydrology -**

Peak pre-construction design runoff rate (50-year) = 3.6 cubic feet per second
Peak design release rate (50-year) = 3.6 cubic feet per second
Peak check release rate (100-year w/o detention) = 28.6 cubic feet per second

**Primary Outlet -**

Type: Orifice: round square-edged hole in plate
Diameter: 8-7/8$^{th}$ inches

Screen protection needed? No.

**Basin -**

Shape: Trapezoidal
Volume: 18,110 cubic feet between elevations of the lip of the control structure and maximum design storm water surface
Size: Bottom width at elevation of the lip of the control structure = 84.1 feet by 168.2 feet
Side slopes 1V:3H
Depth from top of embankment to the lip of the control structure = 6 feet (minimum)
Freeboard: Maximum design storm water surface to rim of riser: 1.5
Maximum check storm water surface elevation to minimum top of embankment elevation = 0.5 feet

**Auxiliary Outlet -**

Type: Riser pipe: 48 inch diameter.
Height: Rim elevation is 4.5 feet above elevation of the lip of the control structure.
Flow Depth: Check storm flow depth is 0.78 feet above riser rim.
Vortex plate needed? No.

Type: Trapezoidal Spillway Weir:
Bottom Width: 11.2 feet.
Side Slope: 1V:3H

Height: Flowline elevation is 4.5 feet above elevation of the lip of the control structure.
Flow Depth: Check storm flow depth is 1 foot above weir flowline.

## 12.12 Hydrograph and Routing Method

This method, although fairly complicated, can be used to design storage facilities for almost all situations encountered in highway design. Unlike the simplified rational method, this procedure can model rainfall events that vary in intensity throughout the duration of the storm. Key features of this procedure are the hydrographs that show in detail the relationship between discharge and time. The steps in using this method are as follows:

**Step 1 -** Calculate the inflow hydrographs. Curvilinear hydrographs based on unit hydrographs and local storm and rainfall data are preferred, such as the NRCS TR-55 method presented in **Chapter 7**. The pre-construction runoff and post-construction inflow hydrographs are needed for each of the design storms. Only the post-construction hydrograph is needed for the check storm. Examples of these hydrographs are shown in Figure 12-40.

**Step 2 -** The peak design release rate is determined in this step. Determine the peak design release rates for each of the design storms. In most cases, the peak design release rate for the most intense design storm is the maximum discharge that will be allowed from the detention facility. Usually it is equal to or less than the peak pre-construction design runoff rate, as discussed in Subsection 12.5.1.1. These discharges are shown on Figure 12-40.

**Step 3 -** Estimates of storage volumes are calculated for each of the design storms in this step using a type of triangular hydrograph that has been developed for this purpose, as follows:

$$V_s = 0.5\ T_i(Q_i - Q_o) \qquad \text{(Equation 12-34)}$$

Where:

$V_s$ = Estimated storage volume in cubic feet
$T_i$ = Time period between beginning and cessation of inflow (as described in Subsection 12.8.1) for post-construction hydrograph, sec
$Q_i$ = Peak inflow rate for post-construction conditions in cubic feet per second
$Q_o$ = Peak design release rate in cubic feet per second

The procedure can also be done graphically, as shown in Figure 12-41. To do this, plot a triangular hydrograph between the origin (the point where inflow begins), the peak post-construction inflow, and the point where the post-construction inflow ceases (as described in Subsection 12.8.1). Next, plot a straight line between the

origin and a point on the trailing limb of the triangular hydrograph that corresponds to the magnitude of the peak design release rate. The area under the hydrograph triangle and above the straight line represents the estimated storage volume.

**Step 4 -** The dimensions of the storage facility are estimated during this step. Estimate the physical dimensions of the storage facility necessary to hold the maximum storage volume for the most intense design storm determined in Step 3, including freeboard. In the initial stage of the process, the outlet or auxiliary outlet structures have not been designed. Calculate the freeboard distances or elevations using estimates of the head depths upstream from the outlet and auxiliary outlet structures. Develop a stage-storage curve for the facility. Show the stages where the maximum storage is needed for each of the design storms. For more information, see the example for the simplified rational method.

**Step 5 -** In this step the outlets are sized. Start with the outlet for the least intense design storm. This outlet operates under the head that occurs during the time of peak storage for the storm. The estimated storage volume from the previous step can be used, and the head depth can be determined from the stage-storage curve developed in Step 4. The outlet can be sized using the equations in the Section 12.9. For more information, see Step 12 of the example for the simplified rational method.

Size the outlet for the next most intense design storm, using the stage elevation that occurs during maximum storage for this storm. The outlet design should consider that discharge is passing through the outlet for this design storm and also through the outlets for the lesser design storm(s). Repeat this process as needed to size all of the design storm outlets.

**Step 6 -** In this step storage routing calculations are performed to check the preliminary design that was performed in the previous steps. Routing is discussed in detail in the next Subsection. Use the inflow hydrographs from Step 1. Return to Step 4 if the routed post-construction maximum design release rate or stage elevation from the most intense design storm exceeds the peak design release rate or stage elevation. Also return to Step 4 if the release rates or stage elevations for the lesser design floods are significantly different than the values used in the preliminary steps.

**Step 7 -** In this step the auxiliary outlet is sized. Size this outlet to convey the entire discharge from the check storm. It is assumed that the primary outlets are plugged. Check the basin dimensions determined in Step 4 and verify that there is adequate freeboard between the lowest part of the embankment and the water surface during this flood. Adjust the minimum top of bank elevation, if needed.

**Step 8 -** Evaluate the downstream effects of the detention outflows during the design storms to ensure that the discharges and release timing represented by the routed hydrographs do not increase downstream flooding problems.

**Step 9 -** Determine the peak outlet control structure discharge velocity during the most intense design storm with all discharge from the primary outlets. Determine the peak auxiliary outlet discharge velocity during the check storm with all flow out of the auxiliary outlet. Provide channel and bank stabilization if the velocities will cause erosion problems downstream.

*Note: The procedure in the preceding steps can involve a significant number of iterations to obtain the desired results.*

**Step 10 -** Summarize the basin and outlet dimensions determined in the prior analysis. See simplified rational method for an example of the procedure.

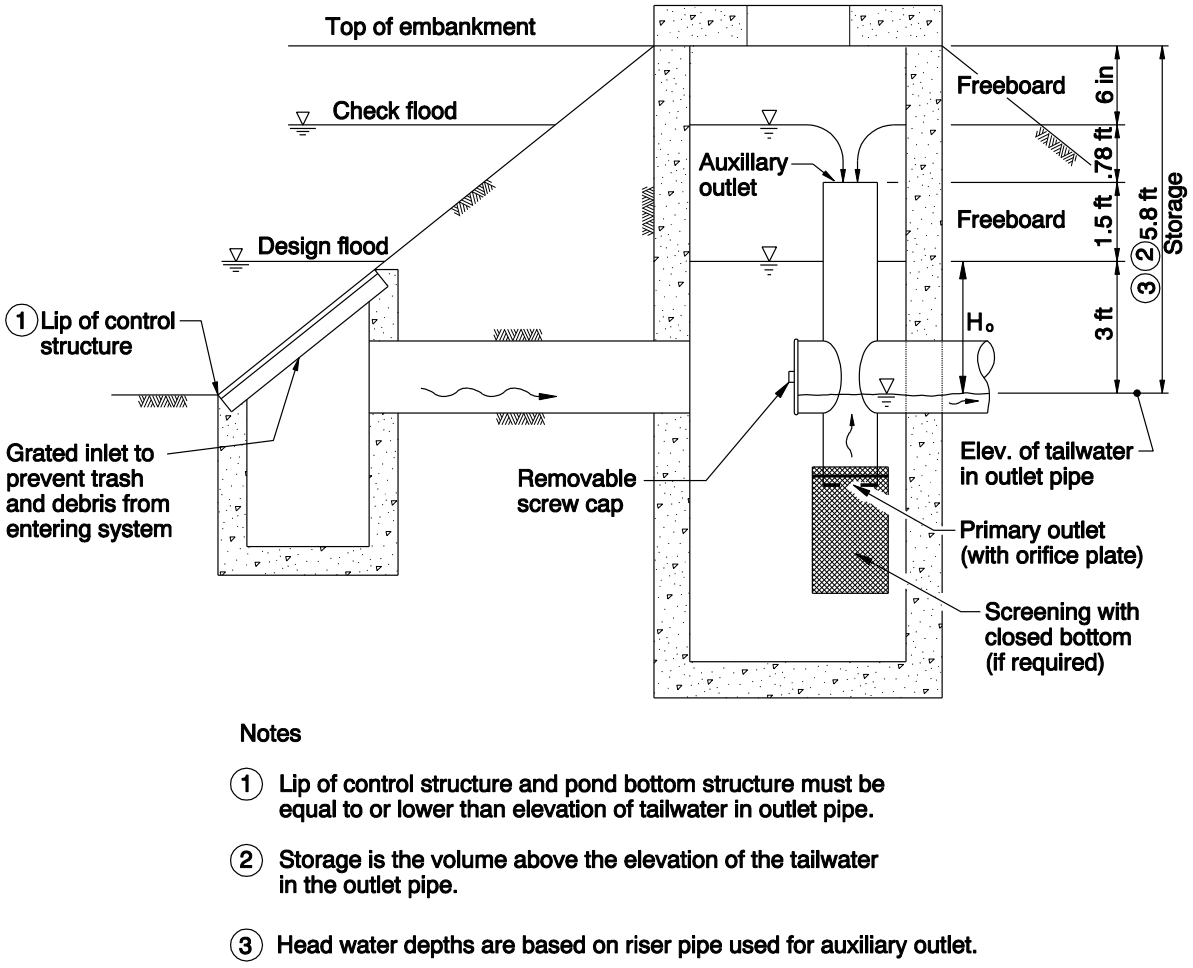



**Figure 12-39 Detention Outlet Structure for Example**

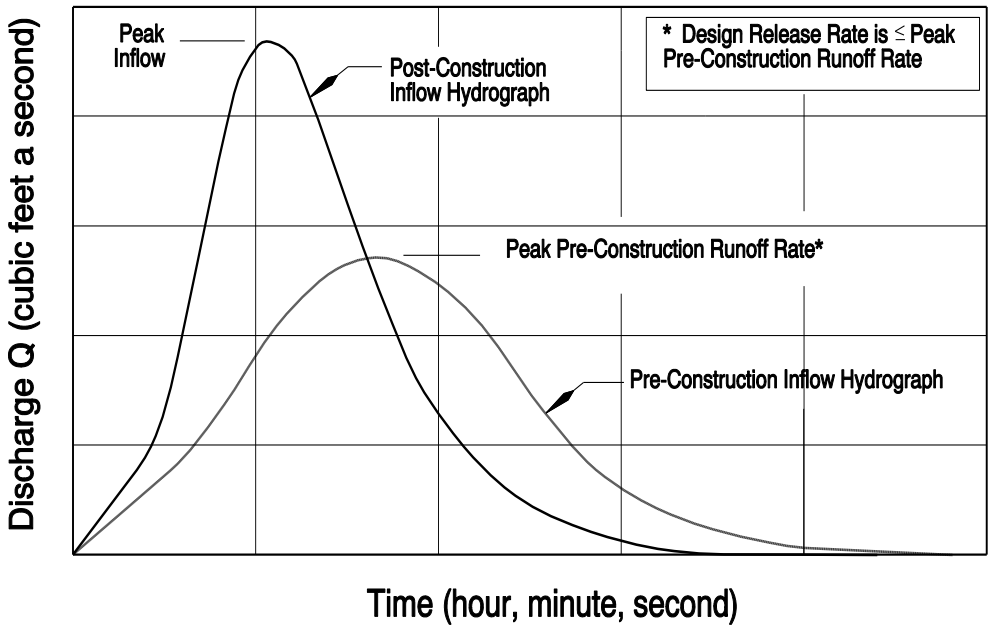



**Figure 12-40 Runoff and Inflow Hydrographs**

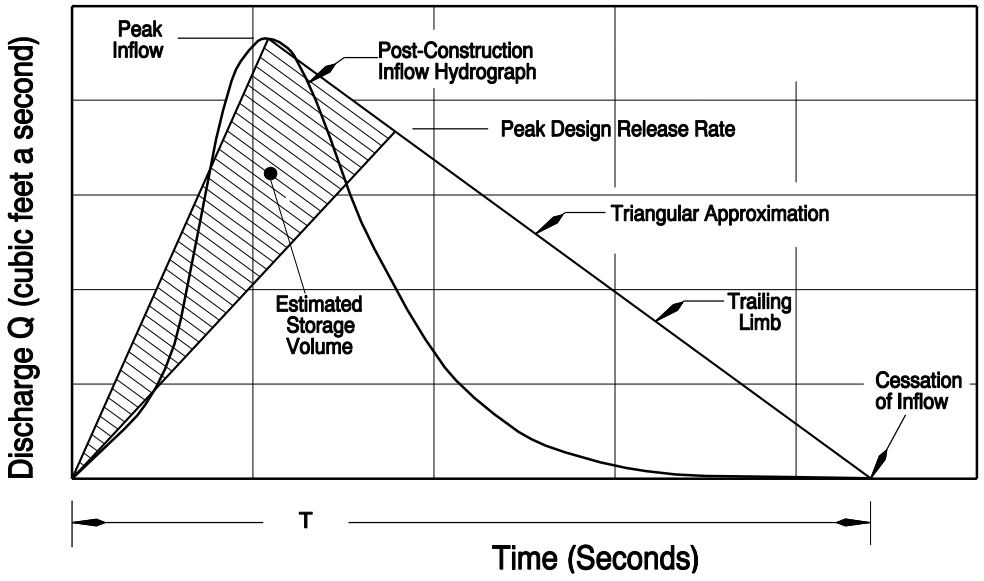



**Figure 12-41 Estimated Storage Volume**

### 12.12.1 Storage Routing

Storage routing is a more precise method and will provide better results than the simplified rational method. The procedures described in this manual provide the guidance on how to conduct the analysis by tabulated and graphical hand calculations. These procedures are time consuming, but there are many computer programs available that provide fast and accurate storage routing results. There are several methods of performing storage routing calculations. One procedure, the Storage Indication Method, is presented in this chapter. This method is presented in more detail in FHWA Hydraulic Design Series No. 2, Highway Hydrology (FHWA: Washington D.C., 1996). This procedure is used at Step 6 of the storage facility design procedure described in Section 12.12, as follows:

**Step 6a -** In this step, tables or curves are obtained for four relationships defined in Section 12.8, as follows:

- the routing time interval ($\Delta T$),
- the inflow hydrograph for the post-construction conditions ($Q_i$ versus T),
- the storage-discharge relationships for the proposed facility ($Q_o$ versus $V_s$), and
- the storage indication relationships for the proposed facility ($Q_o$ versus $V_s + 1/2Q_o\Delta T$).

The routing procedure can be done manually by the tabular procedure shown in Table 12-7 on the form shown in Table 12-8. The time increment values $\Delta T$ are entered into Column 1, the elapsed time since the start of the runoff (T) is listed in Column 2, and the inflow values $Q_i$ are listed in Column 3.

**Step 6b -** In this step, the average inflow volume ($\overline{V}$) is determined for each of the routing analysis time intervals. $\overline{V}$ is calculated as follows:

$$\overline{V} = 1/2\ (Q_{i1} + Q_{i2})\ \Delta T \qquad \text{(Equation 12-35)}$$

Where:

$\overline{V}$ = Average inflow volume for time interval "$\Delta T$" in cubic feet
$Q_{i1}$ = Inflow rate at beginning of $\Delta T$ in cubic feet per second
$Q_{i2}$ = Inflow rate at end of $\Delta T$ in cubic feet per second
$\Delta T$ = Time interval for the routing analysis in seconds

*Note:* *$\overline{V}$ is entered into Column 4 of the table.*
*$Q_{i1}$ and $Q_{i2}$ are from Column 3.*
*$Q_{i1}$ is zero (0) for the first time interval.*

**Step 6c -** In this step the ($V_{s1} - 1/2Q_{o1}\Delta T$) values are calculated for each time increment, as follows:

$$V_{s1} - 1/2Q_{o1}\Delta T$$

Where:

$V_{s1}$ = Storage volume at beginning of time increment "ΔT," in cubic feet,
$Q_{o1}$ = Outflow rate at beginning of ΔT in cubic feet per second

*Note: ($V_{s1} - 1/2Q_{o1}\Delta T$) is entered into Column 5.*
*$V_{s1}$ is $V_{s2}$ from Column 8 for the previous time increment.*
*$Q_{o1}$ is $Q_{o2}$ from Column 7 of the previous time increment.*

**Step 6d -** In this step the ($V_{s2} + 1/2Q_{o2}\Delta T$) values are calculated for the time increment using the following equation:

$$V_{s2} + 1/2Q_{o2}\Delta T = (\overline{V}) + (V_{s1} - 1/2Q_{o1}\Delta T) \qquad \text{(Equation 12-36)}$$

Where:

$\overline{V}$ = Average inflow volume for time interval "ΔT" calculated in Step 6b in cubic feet
$V_{s1} - 1/2Q_{o1}\Delta T$ value calculated in Step 6c in cubic feet

*Note: ($V_{s2} + 1/2Q_{o2}\Delta T$) is entered into Column 6.*
*($\overline{V}$) is from Column 4.*
*($V_{s1} - 1/2Q_{o1}\Delta T$) is from Column 5.*
*$V_{s2} + 1/2Q_{o2}\Delta T = \overline{V}$ for first time increment.*

**Step 6e -** In this step, the $V_{s2} + 1/2Q_{o2}\Delta T$ value determined in Step 6d is used with the storage indication curve ($Q_o$ versus $V_s + 1/2Q_o\Delta T$) to determine $Q_{o2}$ at the end of the time increment "ΔT." To do this, select the appropriate $V_s + 1/2Q_o\Delta T$ value on the x-axis, go up to the curve, and read the corresponding $Q_o$ value on the y-axis. $Q_{o2}$ is entered in Column 7.

**Step 6f -** In this step, the $Q_{o2}$ value determined in Step 6e is used with the storage-discharge curve ($Q_o$ versus $V_s$) to determine $V_{s2}$ at the end of time increment "ΔT." To do this, select the appropriate $Q_o$ value on the x-axis, go up to the curve, and read the corresponding $V_s$ value on the y-axis. $V_{s2}$ is entered in Column 8.

$V_{s1}$, $Q_{o1}$, and ($V_{s1} - 1/2Q_{o1}\Delta T$) are zero (0) for the first time increment.

**Step 6g -** Repeat the preceding steps for the next time increment using $Q_{i2}$, $Q_{o2}$, and $V_{s2}$ as the new values of $Q_{i1}$, $Q_{o1}$, and $V_{s1}$, respectively. The results of the routing analysis are

usually listed for each time interval in tabular form, as shown the example in the following subsection.

**Table 12-7 Tabular Procedure for Hydrograph and Routing Method**

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|
| **Time Increment ($\Delta T$)** | **Time (T)** | **Inflow Rate ($Q_i$)** | **Average Inflow Volume ($\bar{V}$)** | **($V_{s1} - {}^1/_2 Q_{o1} \Delta T$)**** | **($V_{s2} + {}^1/_2 Q_{o2} \Delta T$)***** | **Outflow Rate ($Q_o$)** | **Storage Volume ($V_S$)** |
| second | minute | cubic feet per second | cubic feet | cubic feet | cubic feet | cubic feet per second | cubic feet |
| | $T_1$ | $Q_{i1}$ | | | | $Q_{o1}$ | $V_{s1}$ |
| $\Delta T$ | | | $\bar{V}$ | ** | *** | | |
| **Step 6a** Difference of values in Column 2 | $T_2$ | $Q_{i2}$ | **Step 6b** Average of values in Column 3 Multiplied by Column 1 | **Step 6c** Calculate using values from first row in Columns 7 & 8 | **Step 6d** Add values from Columns 4 & 5 | $Q_{o2}$ **Step 6e** Use value from Column 6 and apply to storage-indication | $V_{s2}$ **Step 6f** Use value from Column 7 and apply to storage-discharge |
| | **Step 1** Tabulate Values From Inflow Hydrograph | | | | | | |

**Table 12-8 Tabular Form for Hydrograph and Routing Method**

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|
| **Time Increment ($\Delta T$)** | **Time (T)** | **Inflow Rate ($Q_i$)** | **Average Inflow Volume ($\bar{V}$)** | **($V_{s1} - {}^1/_2 Q_{o1} \Delta T$)** | **($V_{s2} + {}^1/_2 Q_{o2} \Delta T$)** | **Outflow Rate ($Q_o$)** | **Storage Volume ($V_S$)** |
| second | minute | cubic feet per second | cubic feet | cubic feet | cubic feet | cubic feet per second | cubic feet |
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### 12.12.2 Example: Hydrograph and Routing Method

A detention facility will be built to retain the increased peak flow from a newly developed property for the 10-year design storm. The check storm will not be included in this example.

**Step 1 -** The runoff hydrograph for the pre-construction conditions and the inflow hydrograph for the post-construction conditions are calculated in this step using the NRCS TR-55 method described in **Chapter 7**. The hydrograph data is listed in Table 12-9.

The time interval ($\Delta T$) is determined during this step using the guidance in Subsection 12.8.1. The time duration between the beginning of inflow and the peak post-construction discharge is 0.51 Hrs (30.6 minutes). A $\Delta T$ of 206 seconds (3.4 minutes) will be chosen. Using this interval, there will be 9 intervals between the beginning of inflow and the peak post-construction flow. This is adequate because it exceeds the 5-interval minimum.

The time period ($T_i$) is also determined during this step based on guidance in Subsection 12.8.1. The time periods between the beginning and end of inflow with post-construction conditions is approximately 1.43 hours (85 minutes) during the 10-year storm. The time periods and intervals are shown on Figure 12-42.

The inflows for each increment of the time interval are not always shown on the printout from the hydrograph analysis. Often a hydrograph curve is plotted and the inflow values are taken from the curve. The inflows at each time interval are listed on the design worksheets in Table 12-10, Column 3.

**Step 2 -** The maximum allowable peak pre-construction runoff rate is determined in this step. This discharge is 19.4 cubic feet per second for the 10-year flood as shown in Column 3 of Table 12-9.

**Step 3 -** The preliminary estimate of the storage volume is calculated in this step using Equation 12-34 as follows:

$V_s$ (10-year) = (0.5) (85 min)(60 sec/min) (31.2 cfs – 19.4 cfs) = 30,090 cubic feet

The estimated storage volumes are shown graphically on Figure 12-43.

**Step 4 -** The dimensions of the storage facility and the stage-storage curve are determined in this step. The development of the basin shape and dimensions are not shown. (See simplified rational method, Step 11, for an example of the procedure.) The stage-storage curve is developed as discussed in Subsection 12.8.2, and it is shown in Figure 12-44 and Columns I and II of Table 12-11. Also shown in the Figure is the

stage elevation of 3.93 feet corresponding to the estimated storage volume from Step 3 for the 10-year storm.

**Step 5 -** The primary outlets are sized during this step. The outlet sizing procedure is not shown. (See simplified rational method, Step 12, for an example of the procedure.) The primary outlet is sized to provide peak design release rates that are equal to or lower than the maximum allowable peak design release rate. For example, the primary outlet discharges 18.2 cubic feet per second during the 10-year storage elevation of 4 feet, which is less than the allowable maximum of 19.4 cubic feet per second. The stage-discharge curve is shown in Figure 12-45 and Columns I and III of Table 12-11.

**Step 6 -** The storage-routing procedures are performed in this step.

**Step 6a -** The preliminary data is gathered or compiled. The routing time interval and inflow rates were determined during Step 1, the stage-storage curve was developed in Step 4, and the stage-discharge curve was developed in Step 5. The storage-discharge curve is developed in this step using the values from the stage-discharge curves and the stage-storage curve as described in Subsection 12.8.4. The curve is shown in Figure

12-45 and Columns II and III of Table 12-11. The remaining piece of preliminary data, the storage indication curve, was compiled using procedures in Subsection 12.8.5. It is shown in Figure 12-47 and Columns III and IV of Table 12-11.

**Step 6b -** The average inflow volumes for all time increments are calculated in this step using Equation 12-35, and they are entered into Column 4 of Table 12-10.

**Step 6c -** In this step ($V_{s1}$ - 1/2$Q_{o1}$□Δ$T$) is calculated for a time increment, entered into Column 5, and the calculations proceed to the next step.

**Step 6d -** In this step ($V_{s2}$ + 1/2$Q_{o2}$□Δ$T$) is calculated for a time increment using Equation 12-36, entered into Column 6, and the calculations proceed to the next step.

**Step 6e -** In this step $Q_{o2}$ is determined graphically for a time increment using the ($V_{s2}$ + 1/2$Q_{o2}$□Δ$T$) value from step 6d and the storage-indications curve in Figure 12-47. $Q_{o2}$ is entered in Column 7, and the calculations proceed to the next step.

**Step 6f -** In this step $V_{s2}$ is determined graphically for a time increment using the $Q_{o2}$ value and the storage-discharge curve in Figure 12-46. $V_{s2}$ is entered in Column 8, and the calculations proceed to the next step.

Steps 6c through 6f are repeated consecutively for each time increment. The peak discharge during the 10-year design flood is the maximum value in Column 7 of 17.1

in cubic feet per second, and the maximum volume of water to be stored is the greatest value in Column 8 of 26,700 cubic feet.

The estimated storage volume from Step 3 was 30,090 cubic feet. After calculating the routing effects, the required storage volume is only 26,700 cubic feet. Additional iterations may be conducted to maximize the efficiency of the design and minimizing the required storage volume and foot print of the storage facility.

A routing analysis considers the outflow rates during the rising leg of the hydrograph and this will frequently demonstrate less storage volume being required compared to the storage volume estimate from Step 3 or the Simplified Rational Method.

**Step 7 through Step 10**- These steps are not shown. They are similar to the procedure described in the simplified rational method example.

*Note: The procedure shown in the previous example used graphically displayed curves for many procedures. This is an acceptable method and it aids the explanation of the process. Many of the curves, especially those shown in Figures 12-44, 12-45, 12-46, and 12-47, can be replaced by tabular calculations for each time interval. Hand or spreadsheet methods can be used. This tabulation can greatly increase both the speed and accuracy of this method.*

**Table 12-9 Runoff and Inflow Hydrograph Table for Example**

| 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|
| **Time** | | **Pre-Construction Runoff 10-year** | **Time** | | **Post-Construction Inflow 10-year** |
| **(T)** | **(T)** | **(Q)** | **(T)** | **(T)** | **($Q_i$)** |
| Hour | Minutes | Cubic feet a second | Hour | Minutes | Cubic feet a second |
| 0.00 | 0 | 0.0 | 0.00 | 0 | 0.0 |
| 0.09 | 5.3 | 0.8 | 0.06 | 3.4 | 1.2 |
| 0.18 | 10.6 | 1.6 | 0.11 | 6.8 | 2.5 |
| 0.27 | 16.0 | 2.7 | 0.17 | 10.2 | 3.1 |
| 0.36 | 21.4 | 4.1 | 0.23 | 13.6 | 6.5 |
| 0.45 | 26.8 | 7.2 | 0.29 | 17.0 | 11.5 |
| 0.53 | 32.1 | 10.8 | 0.34 | 20.4 | 17.5 |
| 0.62 | 37.4 | 14.7 | 0.40 | 23.8 | 23.7 |
| 0.71 | 42.8 | 17.8 | 0.46 | 27.2 | 28.7 |
| 0.80 | 48.2 | **19.4** | 0.51 | 30.6 | **31.2** |
| 0.89 | 53.5 | 19.0 | 0.57 | 34.0 | 30.0 |
| 0.98 | 58.9 | 17.5 | 0.63 | 37.4 | 28.1 |
| 1.07 | 64.2 | 15.1 | 0.68 | 40.8 | 24.3 |
| 1.16 | 69.6 | 12.6 | 0.74 | 44.2 | 20.3 |
| 1.25 | 74.9 | 10.5 | 0.80 | 47.6 | 16.8 |
| 1.34 | 80.3 | 8.5 | 0.86 | 51.0 | 13.7 |
| 1.42 | 85.6 | 7.0 | 0.91 | 54.4 | 11.2 |
| 1.51 | 91.0 | 5.8 | 0.97 | 57.8 | 9.4 |
| 1.60 | 96.3 | 4.9 | 1.03 | 61.2 | 7.8 |
| 1.69 | 101.7 | 4.1 | 1.08 | 64.6 | 6.6 |
| 1.78 | 107.0 | 3.3 | 1.14 | 68.0 | 5.3 |
| 1.87 | 112.4 | 2.5 | 1.20 | 71.4 | 4.1 |
| 1.96 | 117.7 | 1.9 | 1.25 | 74.8 | 3.1 |
| 2.05 | 123.1 | 1.2 | 1.31 | 78.2 | 1.9 |
| 2.14 | 128.4 | 0.6 | 1.37 | 81.6 | 0.9 |
| 2.23 | 133.8 | 0.0 | 1.43 | 85.0 | 0.0 |

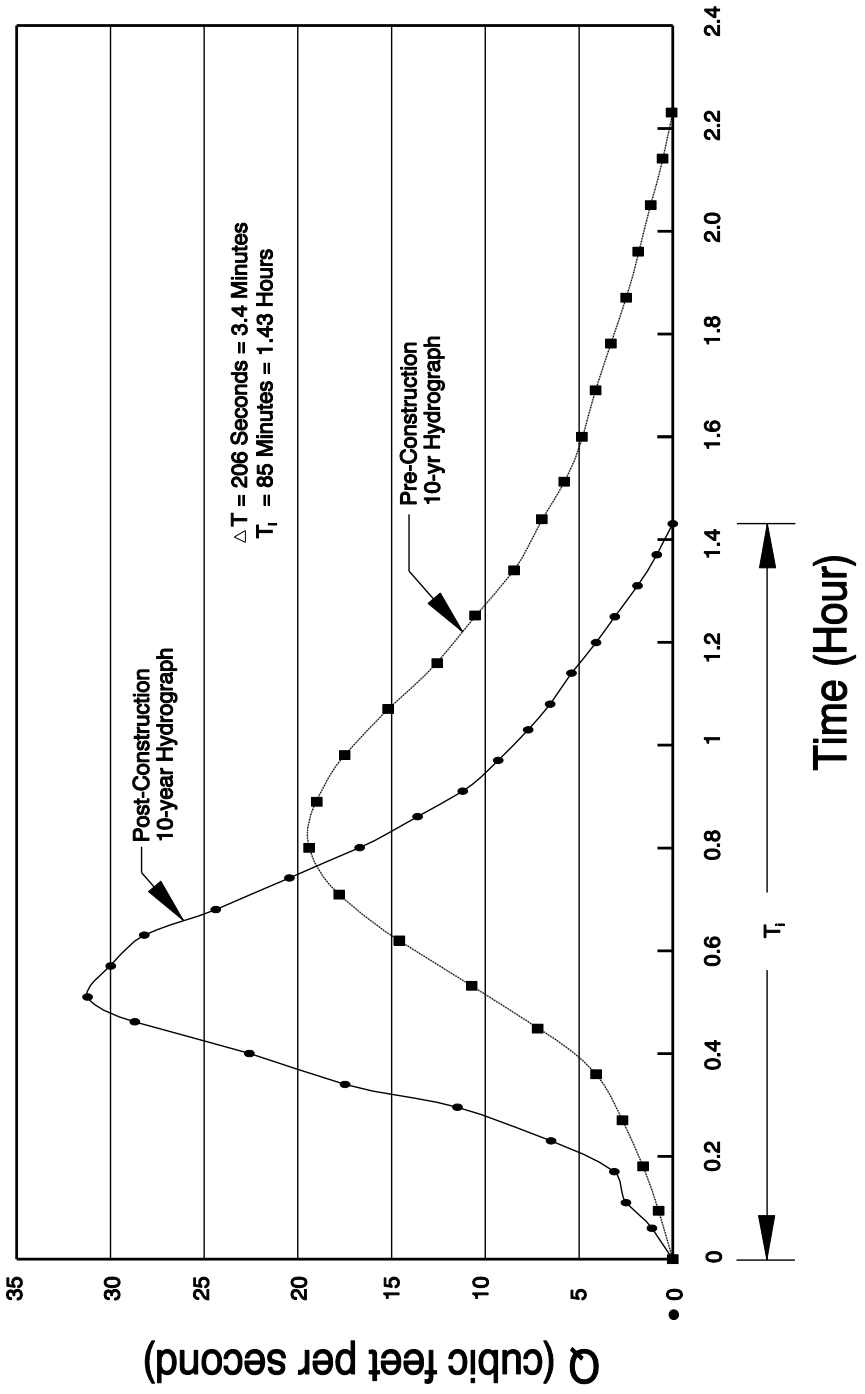


**Figure 12-42 Time Periods, Time Intervals, and Inflow Rates for Example**

**Table 12-10 Tabular Form for Hydrograph and Routing Example**
**10-Year Event**

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|
| **Time Increment ($\Delta T$)** | **Time (T)** | **Inflow Rate ($Q_i$)** | **Average Inflow Volume ($\bar{V}$)** | **($V_{s1} - \frac{1}{2} Q_{o1} \Delta T$)** | **($V_{s2} + \frac{1}{2} Q_{o2} \Delta T$)** | **Outflow Rate ($Q_o$)** | **Storage Volume ($V_S$)** |
| sec | min | $ft^3/s$ | $ft^3$ | $ft^3$ | $ft^3$ | $ft^3/s$ | $ft^3$ |
| | 0 | 0 | | | | 0 | 0 |
| 206 | | | 123.6 | 0 | 124 | | |
| | 3.4 | 1.2 | | | | 0.2 | 107 |
| 206 | | | 381.1 | 86 | 468 | | |
| | 6.8 | 2.5 | | | | 0.6 | 404 |
| 206 | | | 576.8 | 342 | 919 | | |
| | 10.2 | 3.1 | | | | 1.2 | 795 |
| 206 | | | 988.8 | 671 | 1,660 | | |
| | 13.6 | 6.5 | | | | 2.2 | 1,435 |
| 206 | | | 1,854.0 | 1,210 | 3,064 | | |
| | 17.0 | 11.5 | | | | 3.2 | 2,700 |
| 206 | | | 2,987.0 | 2,370 | 5,357 | | |
| | 20.4 | 17.5 | | | | 4.1 | 5,000 |
| 206 | | | 4,243.6 | 4,589 | 8,832 | | |
| | 23.8 | 23.7 | | | | 4.95 | 8,200 |
| 206 | | | 5,397.2 | 7,690 | 13,087 | | |
| | 27.2 | 28.7 | | | | 5.7 | 12,400 |
| 206 | | | 6,169.7 | 11,813 | 17,983 | | |
| | 30.6 | 31.2 | | | | 9.9 | 17,000 |
| 206 | | | 6,303.6 | 15,980 | 22,284 | | |
| | 34.0 | 30.0 | | | | 15.5 | 20,600 |
| 206 | | | 5,984.3 | 19,004 | 24,988 | | |
| | 37.4 | 28.1 | | | | 16.2 | 23,300 |
| 206 | | | 5,397.2 | 21,631 | 27,029 | | |
| | 40.8 | 24.3 | | | | 16.8 | 25,300 |
| 206 | | | 4,593.8 | 23,570 | 28,163 | | |
| | 44.2 | 20.3 | | | | 17.1 | 26,400 |
| 206 | | | 3,821.3 | 24,639 | 28,460 | | |
| | 47.6 | 16.8 | | | | 17.1 | 26,700 |
| 206 | | | 3,141.5 | 24,939 | 28,080 | | |
| | 51.0 | 13.7 | | | | 17.0 | 26,300 |

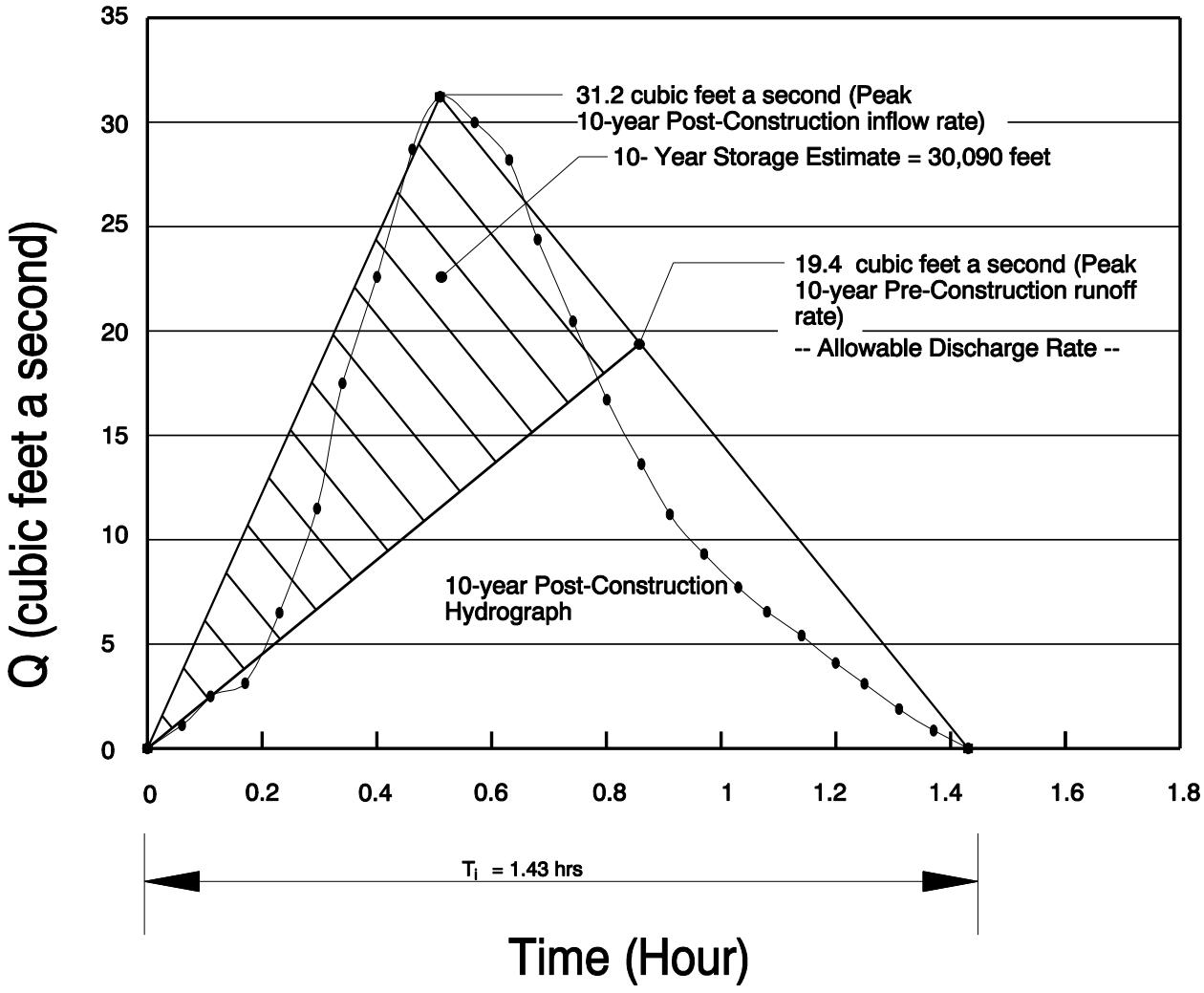


**Figure 12-43 Storage Volume Estimates for Example**

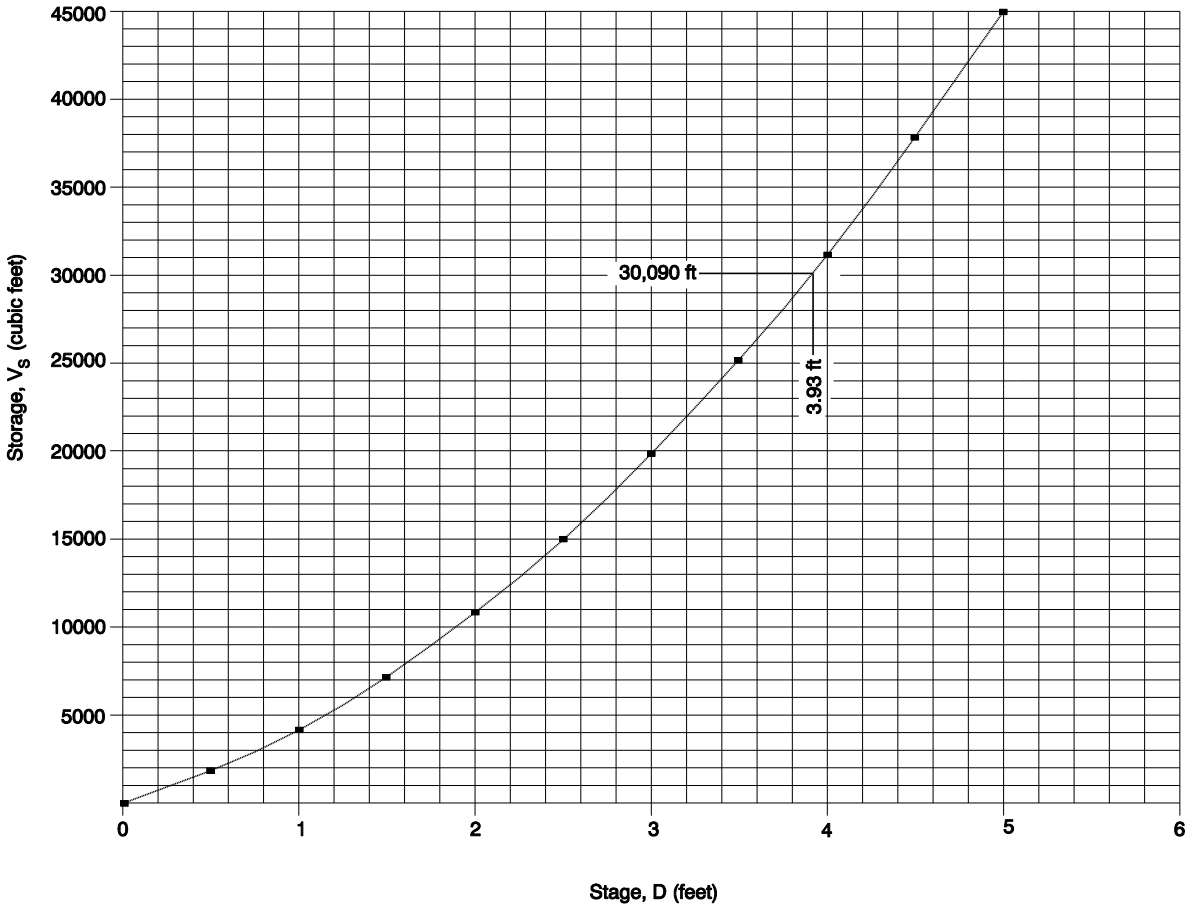


**Figure 12-44 Stage-Storage Curve for Example**

**Table 12-11 Stage-Storage-Discharge-Storage Indication Values for Example**

| I | II | III | IV |
|---|---|---|---|
| Stage (D) | Storage Volume ($V_s$) | Discharge ($Q_o$) | $V_{s1} + \left(\frac{Q_i}{2}\right)\Delta T$ |
| ft | $ft^3$ | $ft^3/s$ | $ft^3$ |
| 0.0 | 0 | 0.00 | 0 |
| 0.5 | 1,800 | 2.74 | 2,082 |
| 1.0 | 4,200 | 3.87 | 4,599 |
| 1.5 | 7,200 | 4.74 | 7,688 |
| 2.0 | 10,800 | 5.48 | 11,364 |
| 2.5 | 15,000 | 6.12 | 15,630 |
| 3.0 | 19,800 | 15.23 | 21,369 |
| 3.5 | 25,200 | 16.78 | 26,928 |
| 4.0 | 31,200 | 18.20 | 33,075 |
| 4.5 | 37,800 | 19.51 | 39,810 |
| 5.0 | 45,000 | 20.74 | 47,136 |
| The data in the columns above were developed during these steps: | | | |
| Step 4 | Step 4 | Step 5 | Step 6a |

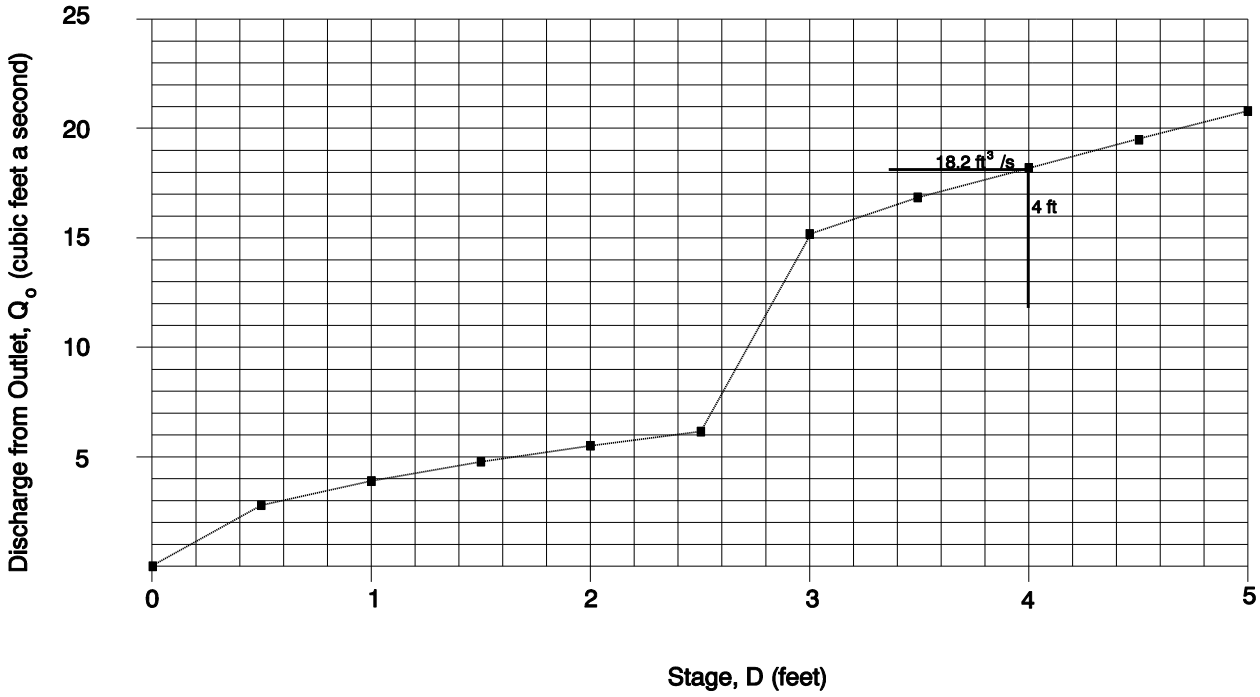


**Figure 12-45 Stage-Discharge Curve for Example**

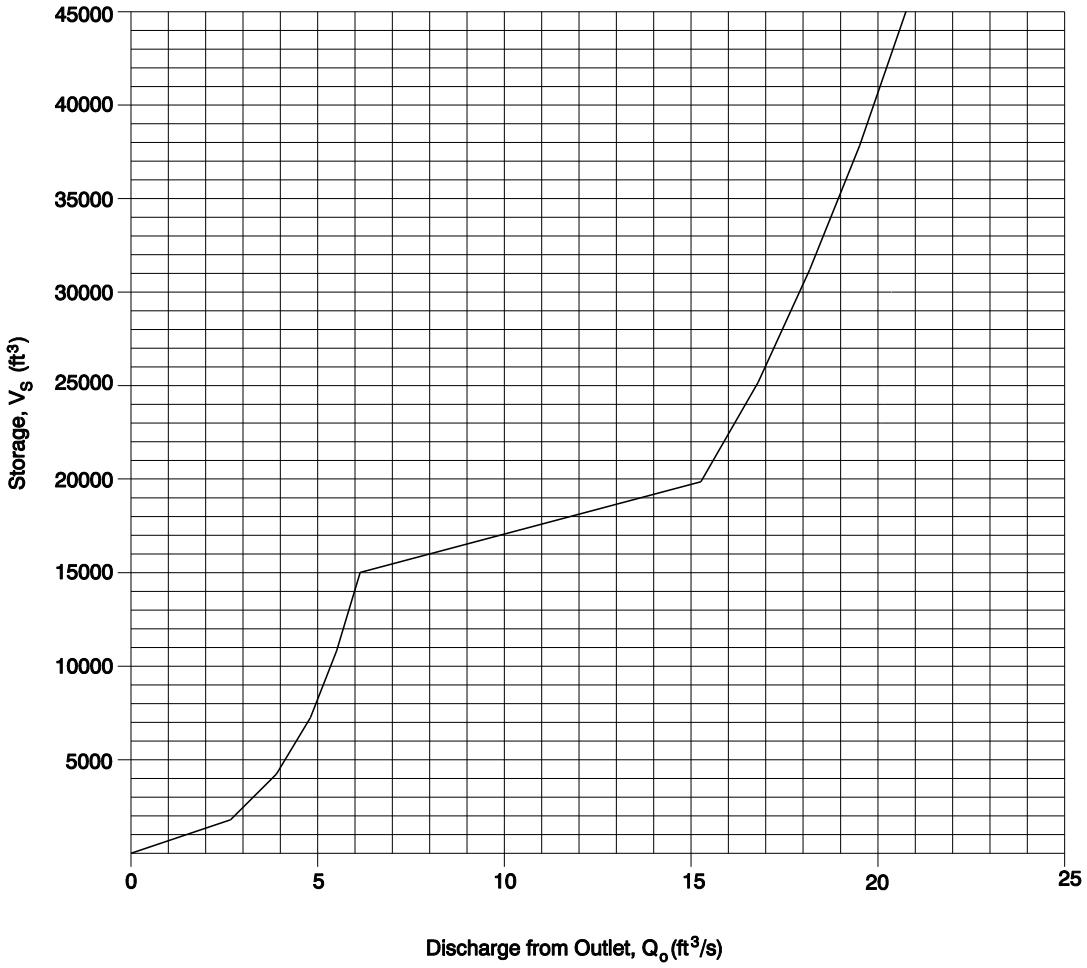


**Figure 12-46 Storage-Discharge Curve for Example**

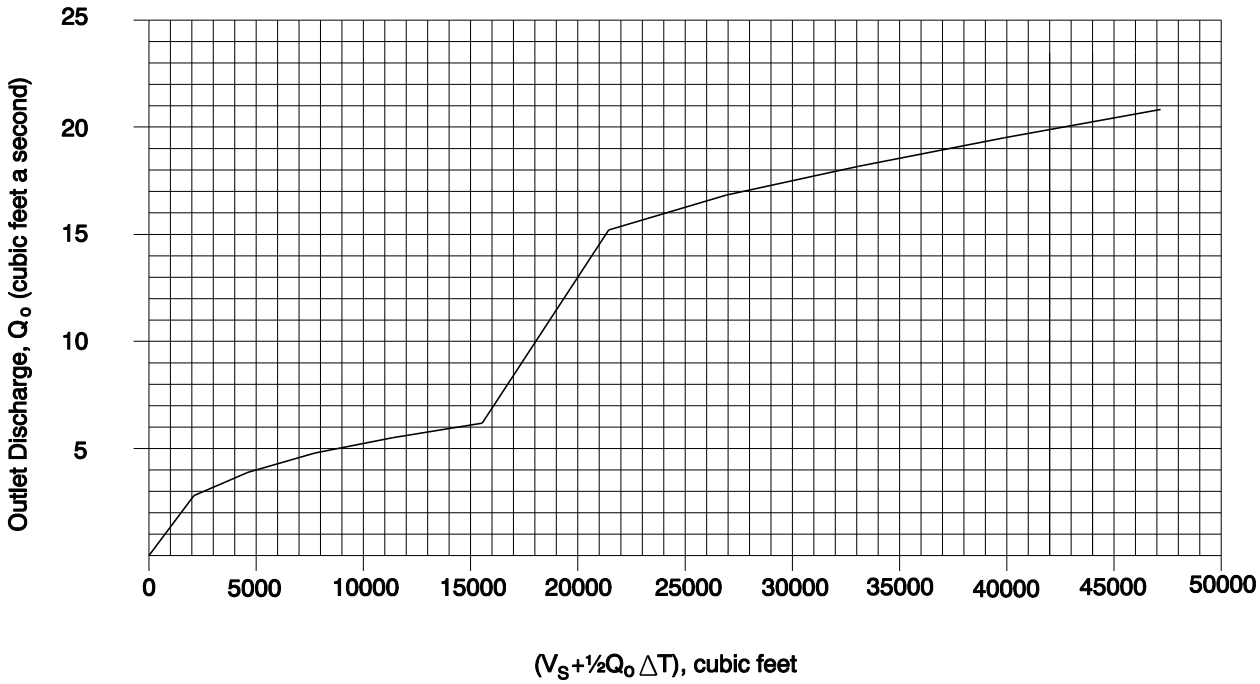


**Figure 12-47 Storage Indication Curve for Example**

## 12.13 Downstream Effects

An estimate of the potential downstream effects (i.e., increased peak flow rate and recession time) of detention storage facilities may be obtained by comparing hydrograph recession limbs from the pre-construction and routed post-construction runoff hydrographs.

Potential effects on downstream facilities should be minor when the maximum difference between the recession limbs of the pre-developed and routed outflow hydrographs is less than about 20 percent, as shown in Figure 12-48. The results of the previous example are well within this 20 percent range, and downstream effects can thus be considered negligible.
Downstream effects may need to be considered and prevented if the routed discharges are greater than 20 percent higher than the pre-construction discharges. Damage prevention could includedesigning a larger upstream detention facility, protection of the downstream channel by revetment, etc.

Regardless of the detention facility's ability to limit the magnitude of the peak discharge, it is important to be aware that development of the watershed almost always increases the volume of the runoff. This increased volume of water, released slowly over a longer period of time, may contribute to bed and bank decay in the receiving channel.

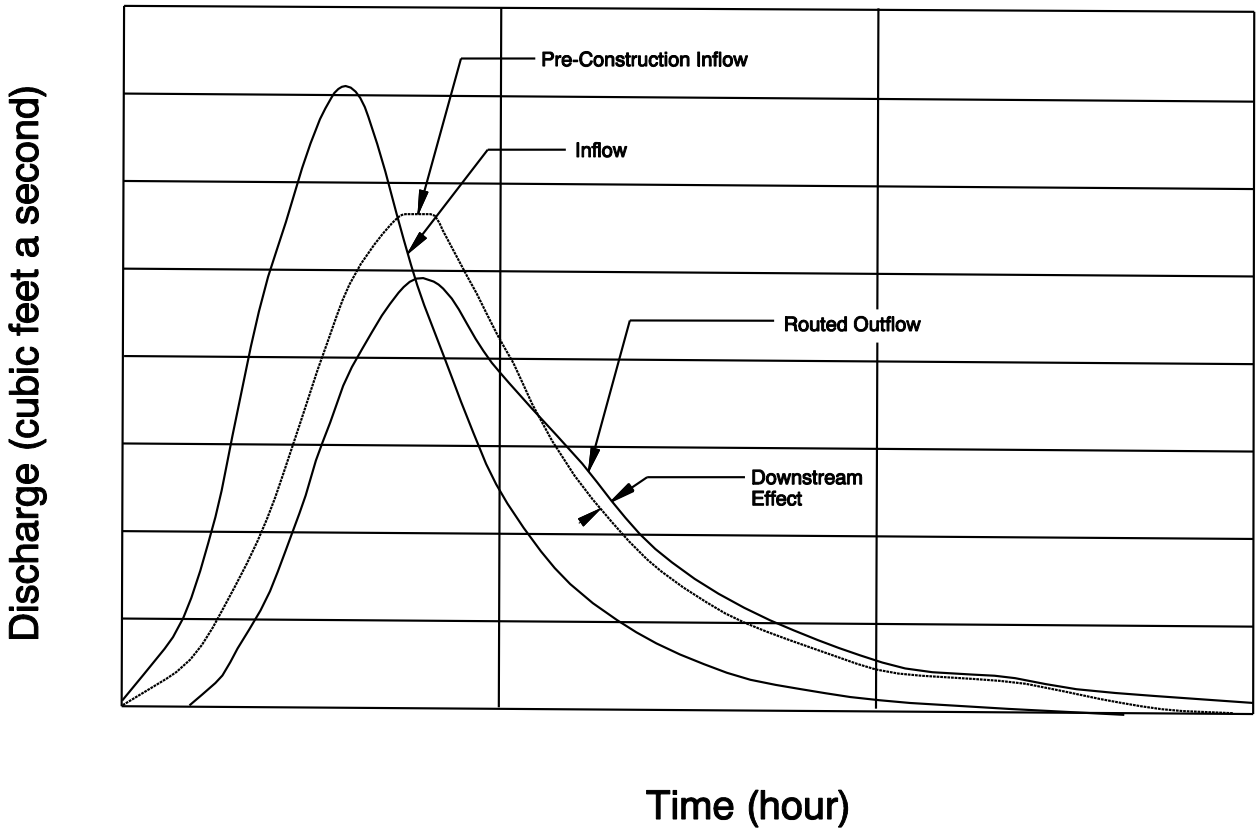


**Figure 12-48 Downstream Effects**

# CHAPTER 13

# STORM DRAINAGE

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Appendix E Access Structures
Appendix F Storm Drains
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Appendix H Design Charts

## 13. 1 Introduction

This Chapter provides information for the planning and design of storm drainage. The methodology is intended for those with an understanding of basic hydrologic and hydraulic methods and some experience in the design of hydraulic structures.

Much of the information in this chapter was originally presented in the FHWA's "Hydraulics Engineering Circular No. 22 – Urban Drainage Design Manual" (HEC-22) and "American Association of State Highway and Transportation Officials Model Drainage Manual".

## 13. 2 Design Objectives

The placement and hydraulic capacity of storm drainage systems should be evaluated early in project development to take into consideration damage to adjacent property, the design traffic service requirements, available funds, and to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road.

The design of a drainage system must address the needs of the traveling public as well as those of the local community through which it passes. The drainage system for a roadway traversing an urbanized region is more complex than for roadways traversing sparsely settled rural areas.

This is often due to:

- the wide roadway sections, flat grades, both in longitudinal and transverse directions, shallow water courses, absence of side channels;
- the more costly property damages which may occur from ponding of water or from flow of water through built-up areas;
- the fact that the roadway section must carry traffic, but also act as a channel to convey the water to a disposal point. Unless proper precautions are taken, this flow of water along the roadway will interfere with or possibly halt the passage of highway traffic.

## 13. 3 Definitions

Definitions of terms which will be important in a storm drainage analysis and design are provided in this section. These and other terms defined in the manual glossary will be used throughout the remainder of this chapter in dealing with different aspects of storm drainage analysis.

Crown - The crown, sometimes known as soffit, is the top inside of a pipe.

Grate Perimeter - The sum of the lengths of all sides of a grate, except that any side adjacent to a curb is not considered a part of the perimeter in weir flow computations.

Inlet Efficiency - The ratio of flow intercepted by an inlet to total flow in the gutter.

Lateral Line - Sometimes referred to as a lead, has inlets connected to it but has no other storm drains connected. It is usually 15 inches or less in diameter and is tributary to the trunk line.

Sag Point - A low point in a vertical curve.

Sag Point, Local - A localized low point in a warped shoulder or road profile of limited length. Ponding water has an opportunity to reach other downstream inlets if inlets are clogged.

Sag Point, Main Line – A low point in the roadway centerline vertical curve. Ponding water has no other outlet than from inlets in this location.

Sag Point, Major - A major sag point refers to a low point that can overflow only if water can pond to a depth of 1.5 feet or more.

Scupper – A vertical hole through a bridge deck for the purpose of deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.

Side-Flow Interception - Flow which is intercepted along the side of a grate inlet, as opposed to frontal interception.

Trunk Line - A trunk line is the main storm drain line. Lateral lines may be connected in at inlet or access structures. A trunk line is sometimes referred to as a "main".

## 13. 4 Design Guidelines

Storm drainage systems can be classified into two major categories based on design: (1) pavement drainage (2) storm drainage. Pavement drainage design consists of gutter flow analysis, and inlet capacity and spacing analysis. Storm drainage design consists of access structure analysis, pipe sizing, and hydraulic grade line analysis.

The goal is to design a storm drainage system to remove surface water from highway pavement and/or prevent runoff from reaching the roadway. For designs to meet this purpose requires balancing the risk of future damages from runoff events against the initial construction cost. Therefore, aspects of pavement drainage and storm drainage guidelines that address this design goal are included in this chapter or in their respective chapters.

The design of a storm drainage system is generally a process that evolves as a project develops. The components to this process are listed below in a general order by which they may be carried out. The components to this process include data collection, coordinating the design with other agencies, hydrology calculations, designing/analyzing pavement and storm drainage features (gutters and ditches, inlets, access structures, storm drains, drywells, outfall energy dissipators), preparing contract plans, specifications and/or special provisions.

Guidelines for the following topics can be located in their respective chapters:

- Data Collection – **Chapter 6**
- Coordinating the design with other agencies – this is a very important element to the design process of storm drainage. Coordinating ODOT work with cities and counties would:
    - inform the designer of local drainage ordinances that ODOT projects must satisfy.
    - benefit ODOT because it allows for local agencies demonstrating a need to participate in defining the scope of the project and presents an opportunity for cost sharing.
- Hydrology – **Chapter 7**
- Designing/analyzing drainage features:
    - Gutters and ditches – **Chapter 8**
    - Inlets (capacity and spacing) – Chapter 13, Appendix C and Appendix D
    - Access Structures – **Chapter 13**, Appendix E
    - Storm drains – **Chapter 13**, Appendix F
    - Hydraulic Grade line – **Chapter 13**, Appendix G
    - Drywells – **Chapter 14**
    - Energy dissipators – **Chapter 11**
- Prepare contract plans – ODOT Contract Plans and Development Guide
- Prepare specifications and/or special provisions – ODOT Specification Manual

## 13. 5 Documentation

**Chapter 4** provides documentation guidelines for stormwater drainage systems. The documentation guidelines provided in **Chapter 4** for standard stormwater designs and significant facilities are of interest for designers within the agency and consultants.

Standard stormwater designs include roadway inlets, small storm drains and small channels or ditches. This information is part of the drainage design provided to the roadway designer who incorporates the drainage features into the roadway design. It also may be part of the work done by the roadway designer if the drainage and roadway designs are done concurrently. As discussed in **Chapter 4**, in most cases the package of design documentation (e.g. calculations, assumptions, drainage map, and narrative) should be bound together with a cover letter sealed by the engineer.

Significant facilities design includes storm drain systems with pipes larger than 24 inches in diameter. As discussed in **Chapter 4**, a storm drainage report is typically prepared documenting facility design information such as type, size, location, critical dimensions, and features.

*Note: Documentation for a stormwater design should provide all information needed to review the drainage design and verify that design objectives and standards have been met.*

## 13. 6 Design Recurrence Interval

The design recurrence interval is a significant variable in the design of highway storm drainage. Selecting the recurrence interval involves decisions regarding acceptable risks of accidents, traffic delays, existing and future conveyance capacity needs, and system costs. The selected recurrence interval that provides reasonable safety for traffic and pedestrians is used to evaluate and design all components of highway drainage such as gutter flow, inlet capacity, drainage piping, open channels and energy dissipators. Storm drainage design recurrence intervals are summarized in **Chapter 3** and in the following appendices.

## 13. 7 Hydrologic Methods

### 13.7.1 Rational Method

The design discharge used in sizing storm drainage systems is determined by the rational method.

The rational method is the most common and most simple method in use for the design of storm drains when the momentary peak flow rate is desired. Its use should be limited to systems with drainage areas of 200 acres or less. Additional rational method limitations and assumptions are presented in **Chapter 7**, Appendix F.

### 13.7.2 Other Hydrologic Methods

Other methods are often used to calculate peak discharge rates and runoff hydrographs from larger or more hydraulically complex basins. Refer to **Chapter 7** for additional discussion of these methods and example applications.

# APPENDIX A
# PAVEMENT DRAINAGE

## 1.0 Introduction

The objective of an effective drainage design is to provide for safe passage of vehicles by reducing:

- damage to surrounding or adjacent property, resulting from water overflowing the roadway curbs and entering such property,
- risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway,
- weakening of base and subgrade due to saturation from frequent ponding of long duration, and
- the potential for hydroplaning.

Pavement drainage design is greatly influenced by the roadway geometric design features and guidelines for effective removal of water from roadway surfaces are presented in the following sections.

## 2.0 Hydroplaning

To provide appropriate drainage of highway pavement requires the designer to evaluate the factors that influence the depth of water on pavement. These factors include length of flow path, surface texture, surface slope and rainfall intensity. If water is allowed to reach undesirable depths, the driver could experience what is known as vehicular hydroplaning.

Hydroplaning occurs when the drainage capacity of the tire tread pattern and the pavement surface is exceeded and the water begins to build up in front of the tire. As the water builds up, a water wedge is created and this wedge produces a hydrodynamic force which can lift the tire off the pavement surface. This is considered as full dynamic hydroplaning and, since water offers little shear resistance, the tire loses its tractive ability and the driver has a loss of control of the vehicle.

Designers do not have control over all of the factors involved in hydroplaning. However, many remedial measures can be included in development of a project to reduce hydroplaning potential. The following is provided as guidance for the designer as practical measures to consider:

Pavement Sheet Flow

- maximize cross slope (Section 4 - Appendix A)
- maximize pavement roughness
- limit draining of multi-lane highways to no more than three lanes in one direction.

Gutter Flow

- limit width of flow spread on highway (reference Appendix D)
- maximize interception of gutter flow above superelevation transitions.
- limit concentrations of sheet flow to no more than 0.1 cubic feet per second. Particular attention should be given to reversal points of superelevation where shoulder and gutter slopes may direct flow across the roadway and gore areas.

Sag Areas

- limit pond duration and depth
- minimize length of sag vertical curve (Section 3 – Appendix A)

Overtopping

- limit depth and duration of overtopping flow

In the event that suitable measures cannot be implemented to address an area of high potential for hydroplaning, or an identified existing problem area, consideration should be given to installing advance warning signs.

## 3.0 Longitudinal Slope

Longitudinal slope is a roadway feature considered during gutter, inlet, and pavement drainage calculations. The following general guidelines are presented:

1. A minimum longitudinal gradient is more important for a curbed pavement than for an uncurbed pavement since the water is constrained by the curb. However, flat gradients on uncurbed pavements can lead to a spread problem if vegetation or other debris is allowed to build up along the pavement edge.

2. Desirable gutter grades should not be less than 0.5 percent for curbed pavements with an absolute minimum of 0.3 percent. Minimum grades can be maintained in very flat terrain by use of a rolling profile, or by warping the cross slope to achieve rolling gutter profiles.

3. To provide adequate drainage in sag vertical curves, a minimum slope of 0.3 percent should be maintained within 50 feet of the low point of the curve. This is accomplished where the length of the curve in feet divided by the algebraic difference in grades in percent (K) is equal to or greater than 167. This is represented as:

$$K = \frac{L}{(G_2 - G_1)} \qquad \text{(Equation 1)}$$

Where:

K = vertical curve constant in feet per percent
L = horizontal length of curve in feet
$G_i$ = grade of roadway in percent

Superelevation and/or widening transitions can create a gutter profile far different from the centerline profile. The designer must carefully examine the geometric profile early in the design of the gutter to eliminate the formation of sumps or "ponds" created by these transitions. These areas should be identified and eliminated.

## 4.0 Cross Slope

The design of pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. It has been widely published that a cross slope of 2 percent has little effect on driver effort in steering or on friction demand for vehicle stability. It is not desirable to use a cross slope steeper than 2 percent on pavements with a central crown line. A steeper cross slope (2.5 percent) may be used to facilitate drainage in areas of intense rainfall.

On multi-lane highways where three (3) lanes or more are sloped in the same direction, it is desirable to counter the resulting increase in flow depth by increasing the cross slope of the outermost lanes. The two (2) lanes adjacent to the crown line should be pitched at 2 percent, and successive lane pairs, or portions thereof outward, should be increased to 2.5 percent.

Additional guidelines related to cross slope are:

1. Although not widely encouraged, inside lanes can be sloped toward the median if conditions warrant.

2. Median areas should not be drained across travel lanes.

3. The number and length of flat pavement sections in cross slope transition areas should be minimized. Consideration should be given to increasing cross slopes in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades.

4. Shoulders should be sloped to drain away from the pavement, except with raised, narrow medians and superelevations.

## 5.0 Curb and Gutter

Curbs are normally used at the outside edge of pavements for low-speed, highway facilities, and in some instances adjacent to shoulders on moderate to high-speed facilities. They serve the following purposes:

- contain the surface runoff within the roadway and away from adjacent properties,
- prevent erosion on fill slopes,
- provide pavement delineation, and
- enable the orderly development of property adjacent to the roadway.

The designer is referred to **Chapter 8** for additional information on ODOT curb types and dimensions.

A pavement gutter is defined as the section of pavement next to the curb which conveys water during a storm runoff event. Gutter cross-sections usually have a triangular shape with the curb forming the near vertical leg of the triangle. The gutter may have a straight cross slope or a composite cross slope where the gutter slope varies from the pavement cross slope. Shallow swale gutters typically have V-shaped or circular sections and are often used in paved median areas or roadways with inverted crowns. The designer is referred to **Chapter 8** for additional information on ODOT gutter types and dimensions.

A curb and gutter combination forms a triangular channel that can convey runoff equal to or less than the design flow without interruption of the traffic. When a design flow occurs, there is a spread or widening of the conveyed water surface. The water spreads to include not only the gutter width, but also parking lanes or shoulders, and portions of the traveled surface. The distance of the spread (T) is measured perpendicular to the curb face to the extent of the water on the roadway.

Limiting spread to gutters and shoulder widths on highways is essential to the service level and traffic safety. The designer is referred to Appendix B for additional discussion and guidelines.

Effective drainage of highway pavement is essential to the service level and traffic safety. To provide these conditions requires the evaluation of stormwater spread on pavement surfaces. Stormwater spread should be considered early in the design while the roadway geometry is being developed because changes to roadway or structure geometrics are less problematic early in the design phase. Spread is dependent on the selected recurrence interval and roadway geometry as presented in Appendices B, C, and D.

Where practical, runoff from cut slopes and other areas draining toward the roadway should be intercepted before it reaches the highway. By doing so, the deposition of sediment and other debris on the roadway as well as the amount of water which must be carried in the gutter section

will be minimized. Where curbs are not needed for traffic control, shallow ditch sections at the edge of the roadway pavement or shoulder offer advantages over curbed sections by providing less of a hazard to traffic than a near-vertical curb and by providing hydraulic capacity that is not dependent on spread on the pavement. These ditch sections are particularly appropriate where curbs have historically been used to prevent water from eroding fill slopes.

## 6.0 Roadside and Median Channels

Roadside channels would typically be used with uncurbed roadway sections to collect and convey highway pavement runoff. They also would be used to intercept runoff from areas that drain toward the highway. Within urbanized areas, roadside channels would only be effectively used in cut sections, depressed sections, and locations with adequate right-of-way.

It is preferable to slope median areas and inside shoulders to a median channel to prevent drainage from the median area from running across the pavement. This is recommended for high-speed facilities, and for facilities with more than two lanes of traffic in each direction.

Roadside and median channels should be designed to convey the predicted peak flow. They should be located and shaped in a manner that does not present a traffic hazard. All channels should have a vegetative lining where permitted by the design velocities. When vegetative linings could not control erosion, the designer is referred to **Chapter 8** for additional discussion on stability of channel linings.

## 7.0 Bridge Decks

The main requirement of the drainage system on a bridge deck is that it removes rainfall-generated runoff from the bridge deck before it collects and spreads in the gutter to encroach onto the travel roadway to the limit of a design spread as described in Appendix D. In accomplishing this purpose, the drainage system must meet other design criteria, as presented below.

### 7.1 Differences with Pavement Components

While drainage of bridge decks is accomplished in the same manner as drainage of other curbed roadway sections, bridge decks are often less efficient, because cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets or scuppers have a higher potential for clogging by debris. Bridge inlets collect flow into relatively small ductile-iron or welded-steel chambers. By contrast, pavement systems have pre-cast, cast-in-place, or masonry structures that are much larger. Such weight and size is incompatible with bridge structures. Bridge drains

are typically steel tubes that must withstand vibrations and deflections better than the storm drains associated with pavement drainage.

## 7.2 Deck, Gutters, and Grades

Bridge deck drainage should be evaluated early in project development and designed to take into consideration:

- Avoid zero gradients, sag vertical curves and superelevation transitions with flat pavement sections on bridges.
- The minimum longitudinal slope for a bridge deck should be 0.5 percent or the designer should design gutter grades with high points that create acceptable longitudinal slopes between inlets.
- Spread on bridge decks should be evaluated to determine if deck inlets are required. Bridge deck inlets would not be required when adequate cross slope, longitudinal slope, and shoulder width is available to meet spread guidelines noted in Appendix D.

## 7.3 Structural Considerations

The two main structural considerations in drainage system design on bridge decks are:

1. inlet sizing and placement must be compatible with the structural reinforcement and components of a bridge; and

2. the drainage system should be designed to deter flow (and associated corrosives) from contacting vulnerable structural members or eroding embankments.

Structural and hydraulic designers should work together to design a system that has the necessary hydraulic capacity and is compatible with structural elements. To avoid corrosion and erosion, the design must:

- Include proper placement of outfalls,
- prevention of flow from splashing or being blown back onto support members
- provide protection from erosive velocities
- Prevent water from running down a crack at the paving notch joint, between pavement and bridge, and undermining an abutment or wingwall.

## 7.4 Maintenance Considerations

Maintenance requirements must be considered in the system design because the drainage system will not function properly if it becomes clogged with debris. In particular, the design should avoid the following common maintenance problems:

- Clogging on inlets or outlet pipes because of flat grades, points where debris is trapped, poor location, or lack of self-cleansing velocities at low-flow conditions.
- Lack of room for maintenance on the bridge deck and access beneath the bridge.
- Unsafe working areas for maintenance personnel that could result in infrequent maintenance.
- No provisions for cleanouts of the outlet pipe system or poorly placed cleanouts.

### 7.5 Inlet Locations

Gutter flow from roadways should be intercepted before it reaches a bridge because of the difficulties in providing and maintaining adequate deck drainage systems. If bridge deck inlets are required, they should be spaced according to procedures described in Appendix D.

Scuppers are another method of deck drainage. Scuppers have a low initial cost and are relatively easy to maintain. However, the use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over:

- embankments
- slope pavement
- slope protection
- aquatic habitat
- driving lanes or
- railroad tracks.

Runoff collected and transported to the end of the bridge should generally be collected by inlets and down drains although sod flumes may be used for extremely minor flows in some areas. Inlets located on the downslope bridge end should be sized assuming inlets on the bridge are clogged.

*Note: Bridge runoff should also be handled in compliance with applicable storm water quality regulations.*

## 8.0 Median/Median Barriers

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Medians are commonly used to separate opposing lanes of traffic on divided highways. It is preferable to slope median areas and inside shoulders to a center depression to prevent drainage from the median area from running across the traveled pavement. Where median barriers are used and, particularly on horizontal curves with associated super-elevations, it is necessary to provide inlets and connecting storm drains to collect the water which accumulates against the barrier. Slotted drains adjacent to the median barrier and in some cases scuppers in the barrier can also be used for this purpose.

## 9.0 Impact Attenuators

The location of impact attenuator systems should be reviewed to determine the need for drainage structures in these areas. With impact attenuator systems such as G.R.E.A.T. or C.A.T. systems, it is necessary to have a clear or unobstructed opening as traffic approaches the point of impact to allow a vehicle to impact the system head on. If the impact attenuator is placed in an area where superelevation or other grade separation occurs, grate inlets and/or slotted drains may need to be placed to prevent water from running through the clear opening and crossing the highway lanes or ramp lanes. Curb, curb-type structures or swales cannot be used to direct water across this clear opening as vehicle vaulting could occur when the impact attenuator system is utilized.

## 10.0 Pavement Texture

The pavement texture is an important consideration for roadway surface drainage. Although the hydraulic designer will have little control over the selection of the pavement type or its texture, it is important to know that pavement texture does have an impact on the buildup of water depth on the pavement during rain storms. A good macrotexture provides a channel for water to escape from the tire-pavement interface and thus reduces the potential for hydroplaning.

A high level of macrotexture may be achieved by tinning new portland cement concrete pavements while it is still in the plastic state. Re-texturing of an existing portland cement concrete surface can be accomplished through pavement grooving and cold milling. Both longitudinal and transverse grooving is very effective in achieving macrotexture in concrete pavement. Transverse grooving aids in surface runoff resulting in less wet pavement time. Combinations of longitudinal and transverse grooving provide the most adequate drainage for high speed conditions.

# APPENDIX B
# GUTTER FLOW

## 1.0 Introduction

Gutter flow hydraulics is an essential part of storm drainage system design because limiting spread to gutters, shoulders and sometimes portions of the traveled lane is essential to the service level and traffic safety. Several methods used in calculating gutter flow hydraulics are noted in Section 3.

*Note: Spread is the width of stormwater flow in the gutter measured laterally from the roadway curb.*

## 2.0 Gutter Types

The three most common types of gutters encountered in pavement drainage design are defined in **Chapter 8**.

## 3.0 Gutter Flow Calculations

Gutter capacities, average velocities, flow depths and flow widths can be determined using the Manning equation in modified form. The Manning equation has to be modified because the hydraulic radius in the conventional equation does not adequately describe a gutter cross-section.

Equations, nomographs and computer programs used in gutter flow analysis are presented in **Chapter 8**.

Methods to estimate pavement and off-site runoff are discussed in **Chapter 7**.

## 4.0 Design Objectives

- Gutters should be designed using the design discharge recurrence intervals listed in **Chapter 3**.

- When a design flow occurs, there is a spread or widening of the conveyed water surface. The water spreads to include not only the gutter width, but also parking lanes or shoulders, and portions of the traveled surface. Limiting spread to gutters, shoulders, and sometimes portions of the traveled lane is essential to the service level and traffic safety. The designer is referred to Appendix D for allowable spread of stormwater runoff flow.

- As gutter flow approaches the low point in a sag vertical curve the flow can exceed the allowable design spread values as a result of the continually decreasing gutter slope. The spread in these areas should be checked to insure it remains within allowable limits. If the computed spread exceeds design values, additional inlets should be provided to reduce the flow as it approaches the low point. Sag vertical curves and measures for reducing spread are discussed further in Appendix D.

# APPENDIX C
# INLETS

## 1.0 Introduction

The application and types of storm drainage inlets are presented in detail in this Appendix.

## 2.0 Inlet Locations

Inlets are required at locations needed to collect runoff within the design controls specified in the design frequency and spread section in Appendix D. In addition, there are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or bypass. Examples of such locations are as follows:

- Sag points in the gutter grade.
- Upstream of median breaks, entrance/exit ramp gores, cross walks and street intersections.
- Immediately upstream and downstream of bridges.
- Immediately upstream of cross slope reversals.
- Upstream side of crosswalk or side streets at intersections.
- Placed on side streets upstream of intersections.
- At the end of channels in cut sections.
- Behind curbs, shoulders, or sidewalks to drain low areas.
- Where necessary to collect snow melt.

Inlets should not be located in the path where pedestrians are likely to walk.

## 3.0 Types

Storm drain inlets are used to collect surface runoff and discharge it to an underground storm drainage system. Inlets are typically located in gutter sections, paved medians, roadside ditches and median channels. Inlets used for the drainage of highway surfaces can be divided into five classes and are discussed in the following sections.

### 3.1 Grate Inlets

These inlets consist of an opening in the gutter covered by one or more grates. They are best suited for use on continuous grades. Since they are susceptible to clogging with debris, the use of standard grate inlets at sag points should be limited to minor sag point locations without debris potential. Special design (oversize) grate inlets can be utilized at major sag points if sufficient capacity is provided for clogging. In this case, flanking inlets are definitely recommended. Grates should be bicycle safe when bike traffic is possible and structurally designed to handle the appropriate loads when subject to traffic.

### 3.2 Curb-Opening Inlets

These inlets are vertical openings in the curb covered by a top slab. They are best suited for use at sag points since they can convey large quantities of water and debris. They are a viable alternative to grates in many locations where grates would be hazardous for pedestrians or bicyclists. They are generally not recommended for use on steep continuous grades.

### 3.3 Slotted Drain Inlets

These inlets consist of a slotted opening with bars perpendicular to the opening. Slotted inlets function as weirs with flow entering from the side. They can be used to intercept sheet flow, collect gutter flow with or without curbs, modify existing systems to accommodate roadway widening or increased runoff, and reduce ponding depth and spread at grate inlets. The two types of slotted inlets in general use are the vertical riser type and the vane type.

*Note: Standard drawing RD328 is an example of a vertical riser slotted inlet. Reference manufacturer literature for information on vane type slotted inlets.*

Slotted drains have been used with varying degrees of success. In fact, the situations which warrant the use of slotted drain inlets can actually hinder their performance. Slotted drain inlets are usually placed in areas of minimal horizontal slope and superelevation. Since the invert of the drain is parallel to the pavement, siltation can occur due to flow velocities. Slotted drain inlets are most efficient when constructed on pavement slopes that are sufficient to produce velocities for clean-out or are provided with a tapered slot that allows an adequate slope on the pipe.

### 3.4 Combination Inlets

Various types of combination inlets are in use. Curb opening and grate combinations are common, some with the curb opening upstream of the grate, and some with the curb opening adjacent to the grate. Slotted inlets are also used in combination with grates, located either longitudinally upstream of the grate, or transversely adjacent to the grate. Engineering judgment is necessary to determine if the total capacity of the inlet is the sum of the individual components

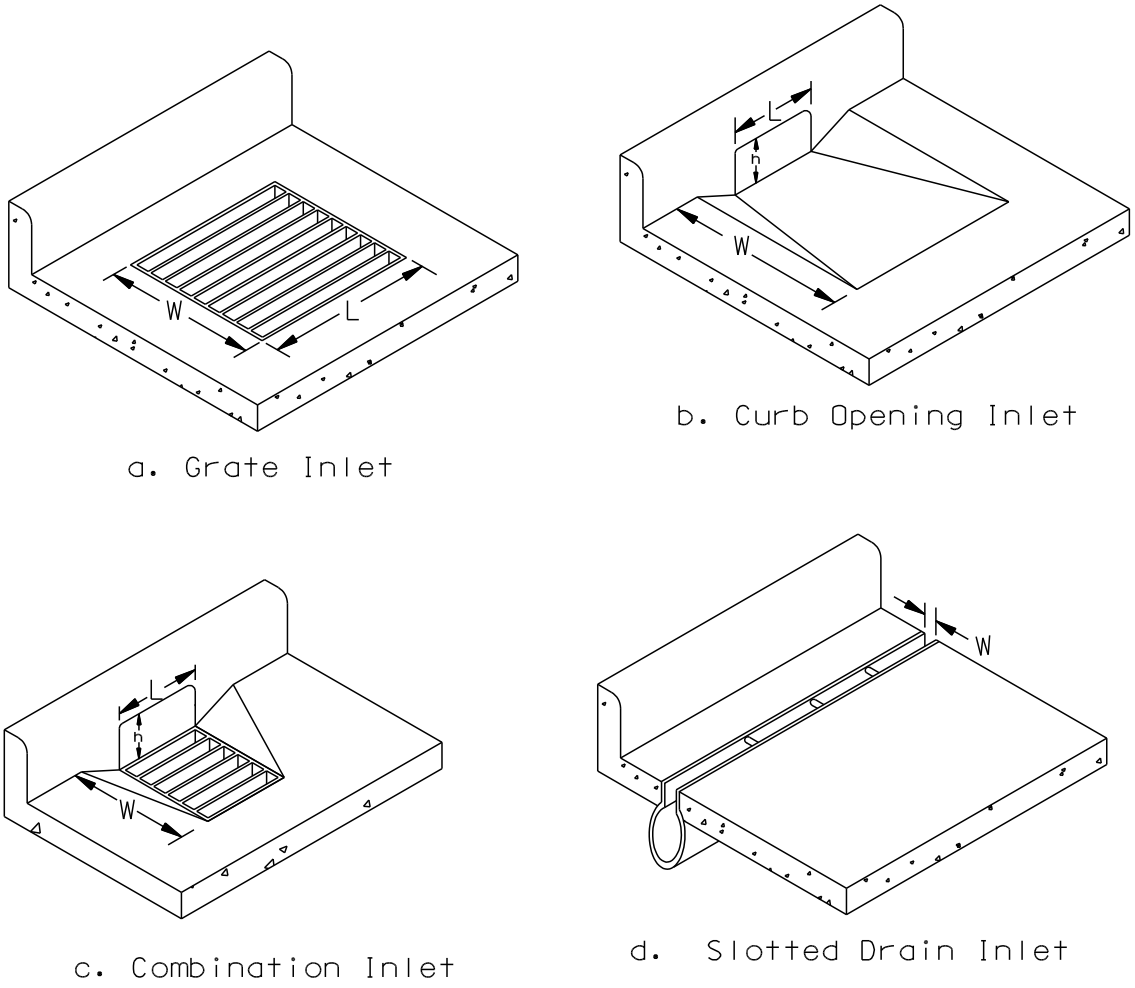


**Figure 1 - Typical Storm Drain Inlets**

or a portion of each. The gutter grade, cross slope and proximity of the inlets to each other will be deciding factors. Combination inlets may be desirable in sags because they can provide additional capacity in the event of plugging.

### 3.5 Trench Drain Systems

Trench Drain systems are a modular slotted drain system with a sloping invert and removable grates. The interior of the system has a radius smooth finish to improve flow and minimize siltation. These systems with their continuous grates are best suited for intercepting flow on very flat surfaces with little or no longitudinal grade. A limiting factor of trench drains is they are incapable of handling flow over 0.5 cubic feet per second over a 300 foot length because drain depths graduate from about 5-inches to 12-inches. The majority of other considerations such as constructability and maintenance are similar between the trench drain system and the traditional catch basin system. Where trench drains may be a little more expensive than concrete pipe, large equipment is not needed to install them so the costs balance out.

### 3.6 ODOT Inlets

Several Standard inlets are used in ODOT drainage systems and are listed in Table A.

The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. To maximize hydraulic capacity, it is recommended to use the following ODOT inlets with the optimal gutter grade ranges listed in the table below.

| Gutter Grade | Inlet Recommendations |
|---|---|
| less than 8 percent | CG-2, G-2 are both acceptable |
| less than 1 percent | CG-3 more efficient than CG-1 and G-1 |
| greater than 1 percent | CG-3 not recommended |

## 4.0 Maximum/Minimum Depth

Minimum inlet depth is controlled by the minimum cover requirements for the type of pipe used. Refer to **Chapter 5** and Standard Drawings RD380, 382, 384, 386 for guidance.

Maximum inlet depth shall be 6 feet. Deeper depths are permissible if maintenance practices will allow. The designer should check with the appropriate District Maintenance Supervisor before specifying inlets deeper than 6 feet.

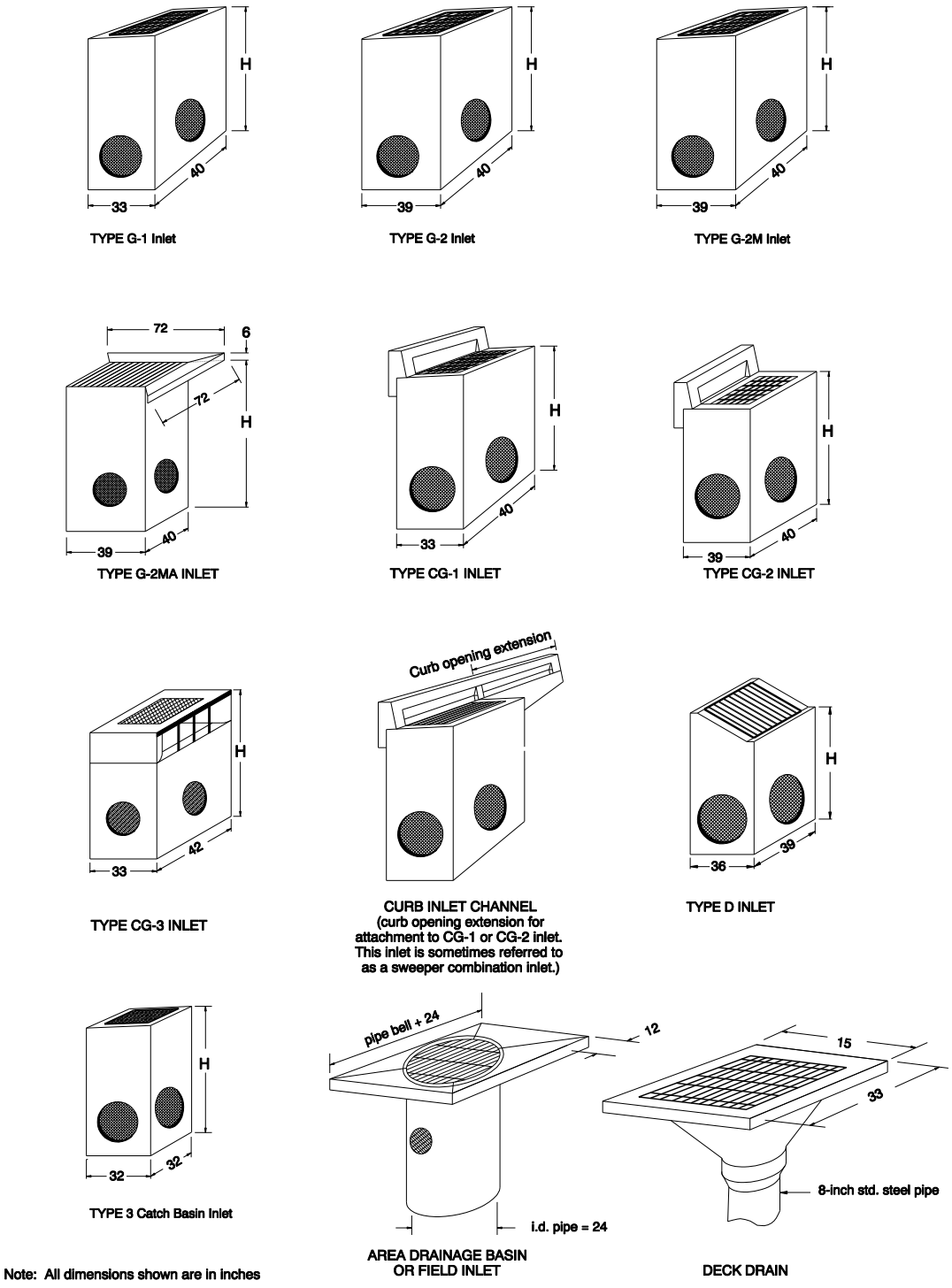


**Figure 2 - ODOT Inlets**

**Table A**
**ODOT Inlets**

| I.D. | Configuration | Where Used | Standard Drawing No. |
|---|---|---|---|
| G-1 | Single Grate* | Small areas with no debris; recommended for two-lane highways. | RD364 |
| G-2 | Double Grate* | Where considerable water or debris is anticipated; recommended for multiple-lane highways | RD364 |
| G-2M | Single Grate | Areas without traffic, bicycles or pedestrians such as medians and ditches. | RD364 |
| G-2MA | Single Grate with concrete apron | Areas without traffic, bicycles or pedestrians such as medians and ditches having variable 1V: 10H-1V: 20H side slope. | RD364 |
| CG-1 | Single Grate* plus curb opening | Where debris or trash is anticipated and in sags; recommended for two-lane highways. | RD366 |
| CG-2 | Double Grate* plus curb opening | Where considerable water or debris is anticipated and in sags; recommended for multiple-lane highways | RD366 |
| CG-3 | Curb Opening only | Where longitudinal grade is relatively flat and bicycle traffic is anticipated; good debris passage capabilities. | RD372 |
| Curb Inlet Channel | Curb Opening Extension for attachment to CG-1 or CG-2 inlets. | Where longitudinal grade is relatively flat and bicycle traffic is anticipated; good debris passage capabilities. | RD366 |
| D | Single Grate on inclined slope | Side drainages and at end of ditches where debris is anticipated; functions as a trash rack. | RD370 |
| Type 3 C.B. | Single Grate* | Where minimal debris or trash is anticipated and where pedestrians may be present. | RD378 |
| Area Drainage Basin or Field Inlet | Single Round Grate* with concrete apron . | Where No Traffic is anticipated. Medians, ditches, flat vegetated areas. | RD374 |
| Slotted Drain | C.M.P. Slotted Drain | Areas with minimal debris ; not recommended for sags. | RD328 |
| Deck Drain Type A | Single Grate* deck drain. | Bridge deck drain | BR120 |

* Bicycle safe grate(s)

**Table B**
**ODOT Inlet Standard Design Parameters**

| I.D. | Grate, curb opening, or slot Width (feet) | Grate, curb opening or slot Length (feet) | Grate Perimeter (feet) | Depression (inches) | Clear Opening Area (square feet) | Clear Opening Ratio (percent) | Equiv. Grate Type |
|---|---|---|---|---|---|---|---|
| G-1 | 1.75 | 2.67 | 6.16 | 0 | 3.48 | 75 | P-1-7/8-4 |
| G-2 | 2.25 | 2.67 | 7.17 | 0 | 4.29 | 72 | P-1-7/8-4 |
| G-2M | 2.25 | 2.67 | 9.83 | 0 | 4.79 | 80 | P-1-7/8 |
| G-2MA | 2.25 | 2.67 | 9.83 | 1.5 | 4.79 | 80 | P-1-7/8 |
| CG-1 | 1.75 | 2.67 | 6.16 | 1.5 | 3.48 | 75 | P-1-7/8-4 |
| CG-2 | 2.25 | 2.67 | 7.17 | 1.5 | 4.29 | 72 | P-1-7/8-4 |
| CG-3 | h = 0.36 | 2.5 | N/A | 2 | 0.90 | N/A | N/A |
| Curb Inlet Channel | h = 0.33 | 6 | N/A | 1.5 | 2 | N/A | N/A |
| D | 2.25 | h = 1.38 | N/A | 0 | 4.79 | 80 | P-1-7/8 |
| Type 3 Catch Basin | 2.29 | 2.29 | 9.16 | 1.5 | 2.04 | 40 | |
| | | | | | On-Grade | | Reticuline |
| | | | | | Sump | | 30° Tiltbar |
| Area Drainage Basin or Field Inlet | 2 Round | 2 Round | 6.28 | 0.24 | 2.08 | 68 | P-1-7/8-4 |
| Slotted Drain | 0.15 | specified on plans | N/A | 0 | N/A | N/A | N/A |
| Deck Drain Type A | 1.12 | 2.67 | 4.91 | 0 | 1.66 | 56 | P-1-1/8 |

## 5.0 Maximum Diameter Pipe Connections

Figure 3 illustrates the maximum diameter that may be connected to standard inlet types. The designer should check skewed connections to ensure they will fit into the inlet box.

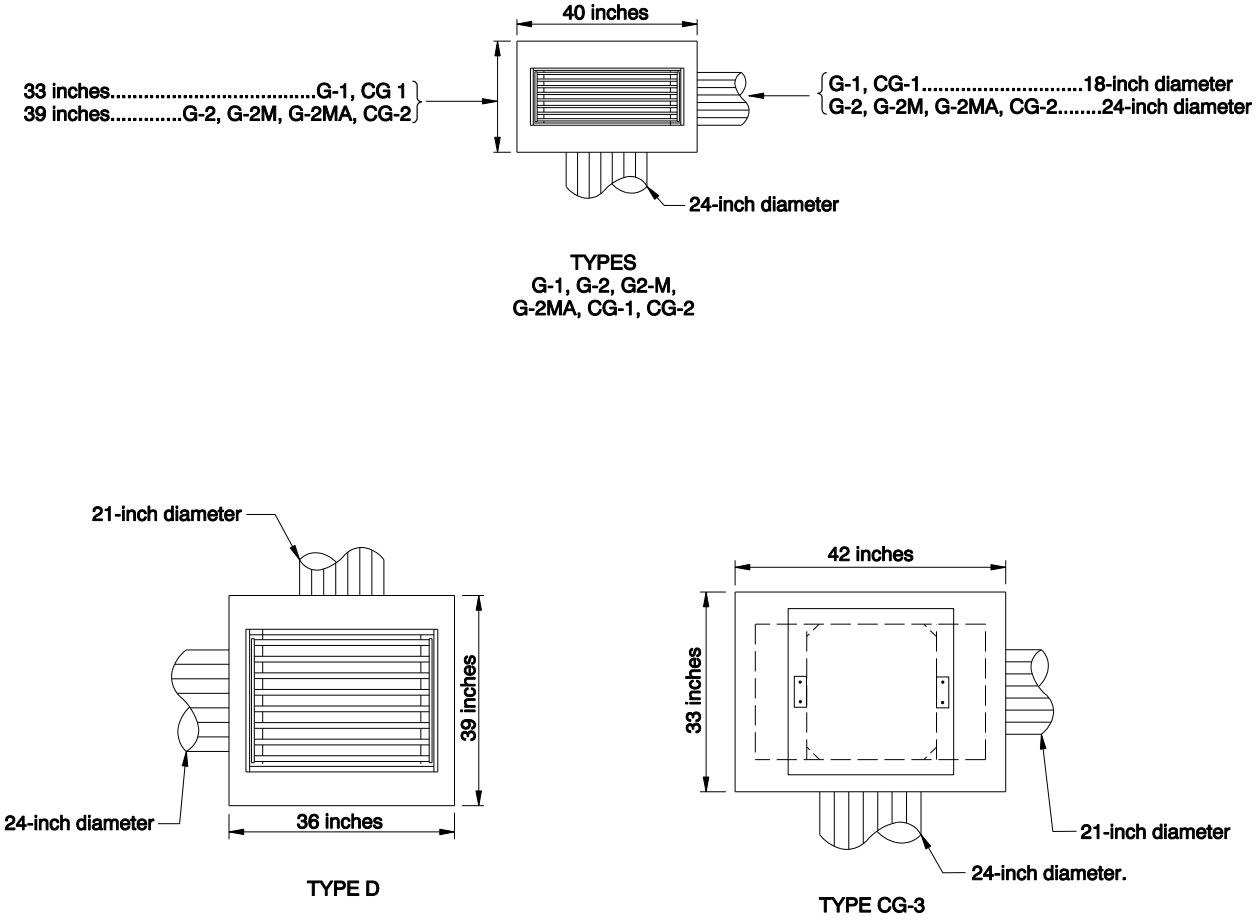


**Figure 3 - Maximum Diameter pipe connections to inlets**

# APPENDIX D
# INLET CAPACITY AND SPACING

## 1.0 Introduction

The capacity and spacing design of storm drainage inlets are presented in detail in this Appendix.

## 2.0 Design Recurrence Interval and Spread

The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Inadequate inlet capacity or poor inlet location may cause flooding on the roadway resulting in a hazard to the traveling public.

The following table provides guidance of the appropriate design recurrence interval, allowable spread, and clogging factors based on road classification. These parameters should be used to design and/or analyze gutter flow and inlet capacity.

**Table A - Pavement Drainage Design Parameters**

| Road Classification | | | Recurrence Interval (years) | Spread | Clogging (percent) |
|---|---|---|---|---|---|
| Highways, Freeways and Collectors | Less than 45 miles per hour | On-grade | 10 | Shoulder + 2 feet | 30* |
| | | Local sag point | 25 | Shoulder + 2 feet | 50 |
| | | Main line sag point | 50 | Shoulder + 2 feet | 50 |
| | Greater than or equal to 45 miles per hour | On-grade | 10 | Shoulder | 30* |
| | | Local sag point | 25 | Shoulder | 50 |
| | | Main line sag point | 50 | Shoulder | 50 |
| Local Streets and Frontage Roads | | On-grade | 10 | 1/2 Driving Lane or local standard | 30* |
| | | Local sag point | 25 | 1/2 Driving Lane or local standard | 50 |
| | | Main line sag point | 50 | 1/2 Driving Lane or local standard | 50 |

*Note: Reference Section 13.3 for definitions of local and main line sag points.*
** Always use a clogging factor of 50 percent for slotted drains, trench drains, and scuppers.*

Inlets on-grade shall meet the following criteria:

- Inlets on-grade shall be spaced for a 5-minute, 10-year rainfall intensity. Design on-grade inlets using Method A or Method B.
- The maximum gutter flow width shall be limited to the spread described in Table A.
- Spacing of first inlet from the crest shall be as calculated to meet spread widths.
- The maximum inlet spacing for subsequent inlets shall be limited to 400 feet.
- Section 5 provides additional design guidelines for inlets on-grade.

Inlets in sags shall meet the following criteria:

- Inlets in a sag shall be designed for a 5-minute, 25-year or 50-year rainfall intensity (reference Table A). Design inlets in a sag using Method A or Method B.
- The design discharge shall be the 25-year or 50-year bypass flow (reference Table A) from the first upstream inlet on each side of the sag plus the runoff from any additional areas draining to the sag.
- One inlet shall be placed at the lowest point in the sag. Flanking inlets shall be included in sag locations. One flanking inlet shall be placed on each side of the low point inlet. Additional flanking inlet design guidelines are discussed in Section 6.6.
- Section 6.0 provides additional design guidelines for inlets in sags.

*Note: There are two methods for designing stormwater inlets. These methods are discussed in more detail below. The key difference between the two methods is that Method A assumes a grate operates in an unclogged condition and no more than 30 percent of the flow would bypass during the design event identified in Table A. Method B has no established minimum inlet efficiency, however, the inlet is assumed 30 to 50 percent clogged as noted in Table A.*

**METHOD A:**

- The bypass flow for inlets on grade shall be limited to a maximum of 30 percent.
- No clogging factors need be used (assume zero percent clogging).
- Low point inlets shall be designed using the allowable spread width from Table A.

**METHOD B:**

- The bypass flow for on-grade inlets may be maximized to use the entire allowable spread width from Table A.
- Clogging factors as described in Table A (as a minimum) and Section 4.0 shall be used.

## 3.0 Inlet Spacing and Computation Sheets

Inlet spacing design consists of locating the inlet, calculating the peak runoff (hydrology), determining the spread, and evaluating inlet efficiency. This design process can be viewed as a circular process that does not have a defined starting point and order of the process can vary as described in this section.

The design starting point depends on site drainage characteristics. For example, some inlets are placed to collect runoff at locations with little regard for contributing area such as immediately up grade from pedestrian cross walks, entrance/exit ramp gores, sags, etc. Based on diagram A-A, the design process would begin at *Locate Inlet*, proceed to *Calculate Hydrology*, leading to spread calculations and end with calculating inlet efficiency. Additional detail on this design process is presented in this section.

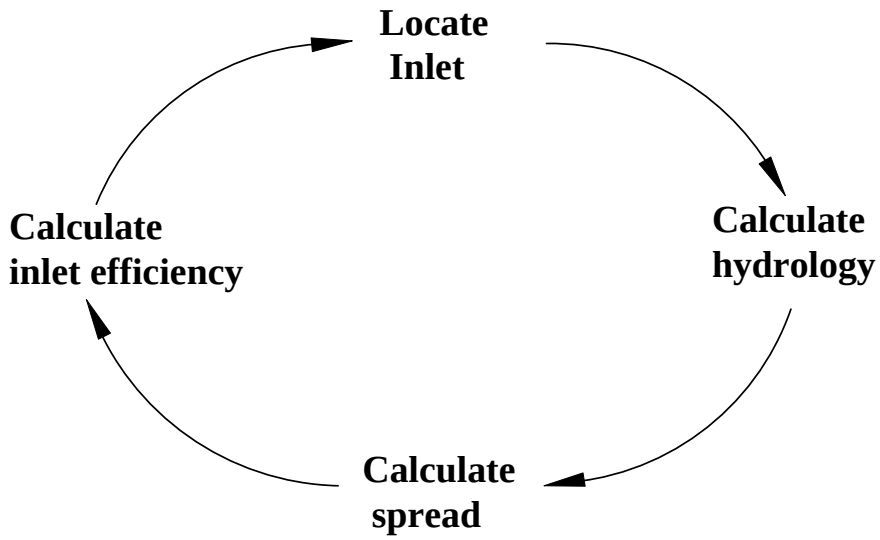


**Diagram A - A**

*Note: Figure 1 is a computation summary sheet that is recommended to document the analysis process noted in Diagram A-A.*

Inlet spacing design could also begin by the designer assuming a spread width. This is a common approach to evaluating inlets placed on continuous grades. Based on diagram B-B, the design process would begin at *assume spread*, then compute hydrology, inlet efficiency, and end with calculating inlet spacing.

*Note: Figure 2 is a computation summary sheet that is recommended to document the analysis process noted in Diagram B-B.*

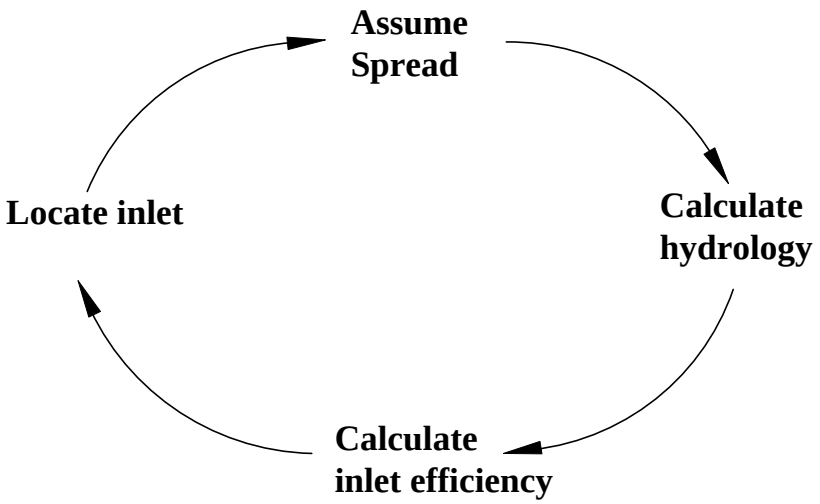


**Diagram B - B**

Geometric information is needed when designing storm drainage inlets. The type of information needed for design includes highway longitudinal slope, cross slope, gutter characteristics, location of crest and sag points, drainage patterns, and delineated drainage areas. This information can be obtained from highway profiles, typical cross sections, superelevation diagrams, and surveyed maps with contours.

A number of inlets are required to collect runoff at locations with little regard for contributing drainage area. Locate and note these inlets on the design plan. After locating these inlets, obtain and use the roadway geometric information to assure that the design criteria such as minimum inlet efficiency and maximum gutter flow widths are not exceeded. If the design criteria are not satisfied, adjustments to the inlet system (e.g. type of inlet, inlet spacing, etc) may be required. This information should be included on the computation sheet (Figure 1) provided to document the analysis.

The spacing of inlets on continuous grades is based on the maximum spread allowed per Table A. Two methods (A or B) are available for designing these types of inlets and are defined in Section 2. The first inlet and subsequent inlets are placed downstream of the crest point to meet the requirements of methods A or B. These inlets can be equally spaced when the drainage areas consists of pavement only, uniform runoff characteristics, and are rectangular in shape. It is also assumed that the time of concentration would be the same for all inlets.

The following equation can be used to locate the first on-grade inlet on continuous grade: (subsequent inlets have a 400 foot maximum spacing)

$$L = 43560 \frac{Q_t}{CC_f iW_p} E \qquad \text{(Equation 1)}$$

Where:

- $L$ = distance from the crest in feet
- $W_p$ = width of drainage area in feet
- $C$ = runoff coefficient
- $C_f$ = runoff coefficient adjustment factor to account for reduction of infiltration and other losses during high intensity storms (See **Chapter 7**, Appendix F; for 10-year storm $C_f = 1$)
- $i$ = rainfall intensity in inches per hour
- $Q_t$ = maximum allowable flow in cubic feet per second (calculated as noted in Section 13.7 and **Chapter 7**).
- $E$ = This constant is equal to one for the first inlet in all cases and is equal to capture efficiency (E) for subsequent inlets. The constant E can be assumed to be equal to 1 for the first inlet because it receives almost no bypass flow from upslope inlets. To space successive down grade inlets on constant slope, use the capture efficiency (E).
- 43,560 = conversion from acres to square feet

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**Inlet Design Computation Sheet (Diagram A-A Process)**

**Given:**

| | |
|---|---|
| **Surface Description** | |
| Road Classification | |
| Type of Curb and Gutter, see Chapter 8, Curbs and Gutters | |
| Surface Material | |
| Manning's Coefficient, n | |
| **Drainage Area Description** | |
| Runoff Coefficient, C | |
| Hydrologic Zone *(Chapter 7)* | |
| Intensity (i) in/hr *(Chapter 7)* | |
| $C_f$ *(Chapter 7, Appendix F)* | |
| Type of Inlet *(Chapter 13, Appendix C, Table B)* | |
| **Design Method: Method A ☐ Method B ☐** | |
| **Design Criteria:** | |
| **Method A (reference Section 2)** | |
| Minimum Efficiency | 70 Percent |
| Allowable spread *(Appendix D, Table A)* | |
| shoulder width | |
| distance into outside lane | |
| spread (T) = Shoulder Width + Distance into outside lane | |
| Percentage Clogged | |
| **Method B (reference Section 2)** | |
| Allowable spread *(Appendix D, Table A)* | |
| shoulder width | |
| distance into outside lane | |
| spread (T) = Shoulder Width + Distance into outside lane | |
| Percentage Clogged | |

**Sketch:**

| Locate Inlet | | | Calculate Hydrology | | | Calculate Spread | | | | | | | | Calculate Inlet Efficiency | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Drainage Length | Drainage Width | Drainage Area | Flow from Contributing Area | Bypass Flow from Upstream Inlet | Total Flow | Spread | Long. Slope | Cross Slope | | Gutter Width | Total Gutter Flow | Design Inlet Length | Design Inlet Width | Inlet Interception Efficiency | Intercepted Flow | Bypass Flow | Number | Station | Comments |
| | | | | | | | | | | | | | | | | | | | |
| L | $W_p$ | A | $Q_r$ | $Q_b$ | $Q_t$ | T | $S_L$ | $S_x$ | $S_w$ | W | $Q_t$ | $L_i$ | $W_i$ | E | $Q_i$ | $Q_b$ | | | |
| (ft) | (ft) | (ac) | (cfs) | (cfs) | (cfs) | (ft) | (ft/ft) | (ft/ft) | (ft/ft) | (ft) | (cfs) | (ft) | (ft) | | (cfs) | (cfs) | | | |
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**Figure 1 – Computation Sheet For Inlet Design**

**Inlet Design Computation Sheet (Diagram B-B Process)**

**Given:**

| | |
|---|---|
| **Surface Description** | |
| Road Classification | |
| Type of Curb and Gutter, see Chapter 8, Curbs and Gutters | |
| Surface Material | |
| Manning's Coefficient, n | |
| **Drainage Area Description** | |
| Runoff Coefficient, C | |
| Hydrologic Zone *(Chapter 7)* | |
| Intensity (i) in/hr *(Chapter 7)* | |
| $C_f$ *(Chapter 7, Appendix F)* | |
| Type of Inlet *(Chapter 13, Appendix C, Table B)* | |
| **Design Method: Method A ☐ Method B ☐** | |
| **Design Criteria:** | |
| **Method A (reference Section 2)** | |
| Minimum Efficiency | 70 Percent |
| Allowable spread *(Appendix D, Table A)* | |
| shoulder width | |
| distance into outside lane | |
| spread (T) = Shoulder Width + Distance into outside lane | |
| Percentage Clogged | |
| **Method B (reference Section 2)** | |
| Allowable spread *(Appendix D, Table A)* | |
| shoulder width | |
| distance into outside lane | |
| spread (T) = Shoulder Width + Distance into outside lane | |
| Percentage Clogged | |

**Sketch:**

| Assume Spread | | | | | | | | Calculate Hydrology | | | Calculate Inlet Efficiency | | | Locate Inlet | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Spread | Long. Slope | Cross Slope | | Gutter Width | Total Gutter Flow | Design Inlet Length | Design Inlet Width | Flow from Contributing Area | Bypass Flow from Upstream Inlet | Total Flow | Inlet Interception Efficiency | Intercepted Flow | Bypass Flow | Drainage Length | Drainage Width | Drainage Area | Number | Station | Comments |
| T | $S_L$ | $S_x$ | $S_w$ | W | $Q_t$ | $L_i$ | $W_i$ | $Q_r$ | $Q_b$ | $Q_t$ | E | $Q_i$ | $Q_b$ | L | $W_p$ | A | | | |
| (ft) | (ft/ft) | (ft/ft) | (ft/ft) | (ft) | (cfs) | (ft) | (ft) | (cfs) | (cfs) | (cfs) | | (cfs) | (cfs) | (ft) | (ft) | (ac) | | | |
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**Figure 2 – Computation Sheet for Inlet Design**

## 4.0 Clogging Factors

Refer to Table A for on-grade and sag inlet clogging factors. Table A provides guidance on the clogging factor recommended based on road classification. Then go to the appropriate ODOT inlet design parameter clogging tables (Table B for 30 percent clogged or Table C for 50 percent clogged) for grate, curb-opening or slot width and length to be used for designing inlets partially clogged. Hydraulic capacity curves or data for available grates would have to be obtained from a manufacturer when evaluating trench drains.

Assume 50 percent clogging for the following inlet types when using inlet design method B due to the tendency of these inlets to plug:

- Slotted drains
- Trench drains
- Scuppers

Using a 50 percent clogging factor will result in providing twice the calculated required length than if no clogging factor were used.

*Note: Some assumptions must be made regarding the nature of the clogging in order to compute the capacity of a partially clogged grate. If the area of a grate is 50 percent covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50 percent.*

### 4.1 Grate Clogging Calculation Example

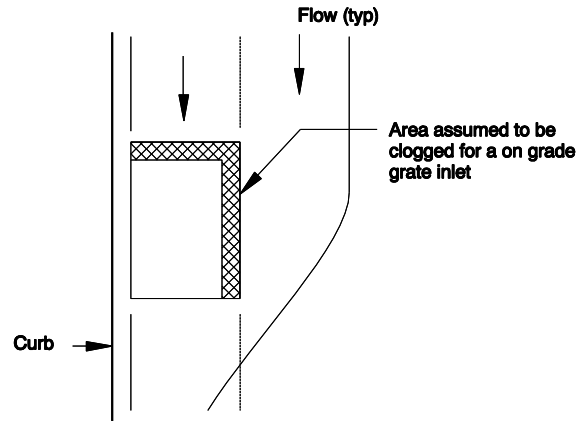
The following example illustrates the application of clogging factors.

#### 4.1.1 Grate configuration assuming 50 percent clogged

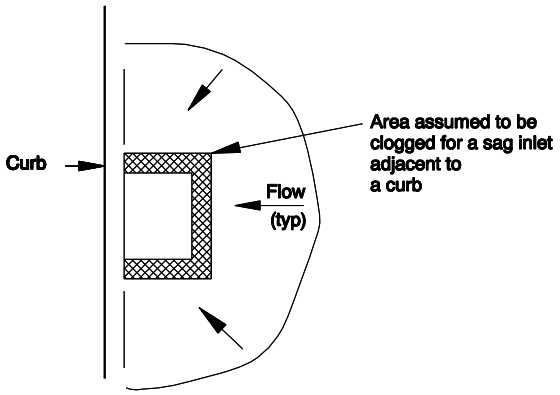
Tables B and C in this appendix identify adjusted widths to account for 30 percent and 50 percent clogging as required for Method B. The following is included to illustrate how inlet dimensions are adjusted to account for clogging. Reference Figure 3 of this Appendix to gain a conceptual understanding of the clogged inlets.

If a 24-inch by 48-inch grate is located next to a curb at a sag and is assumed to be 50 percent clogged. The effective width is 12-inches, reduced from 24-inches to 12-inches. Likewise, the perimeter is reduced from 96-inches to 72-inches (12-inches + 48-inches + 12-inches). The area of the opening would be reduced by 50 percent and the perimeter by 25 percent. To provide an equivalent interception capacity of a grate with a perimeter of 96-inches several alternatives may be available. These alternatives include a grate measuring 48-inches by 48-inches, a grate measuring 24-inches by 72-inches, or a grate measuring 36-inches by 60-inches would meet these requirements.

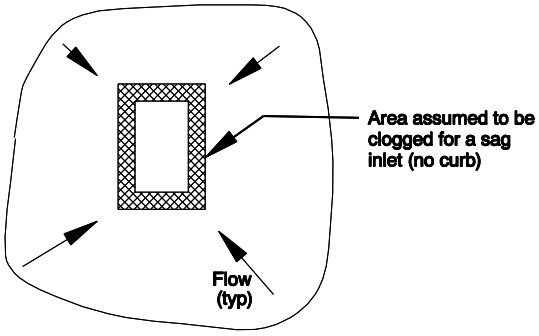
*Note: Figure 3 illustrates typical inlet grate areas assumed to be clogged if placed on-grade, sag with curb or sag with no curb.*




a) On grade




b) Sag with curb




c) Sag no curb

**Figure 3 - Grate Clogging**

**Table B - ODOT Inlet Design Parameters with 30 Percent Clogging**

| Identification | Next to Curb | Grate, curb-opening or slot Width (feet) | Grate, curb-opening or slot Length (feet) | Grate Perimeter (feet) | Clear Opening | |
|---|---|---|---|---|---|---|
| | | | | | Area (square feet) | Ratio (percent) |
| G-1 & CG-1 | Y | 1.39 | 2.31 | 5.10 | 2.41 | 75 |
| G-1 | N | 1.39 | 2.31 | 7.41 | 2.41 | 75 |
| G-2 & CG-2 | Y | 1.84 | 2.26 | 5.94 | 2.99 | 72 |
| G-2 | N | 1.84 | 2.26 | 8.20 | 2.99 | 72 |
| G-2M & G-2MA | N | 1.84 | 2.26 | 8.20 | 3.33 | 80 |
| CG-3 | Y | h = 0.36 | 1.75 | N/A | 0.63 | N/A |
| Curb Inlet Channel | Y | h = 0.33 | 4.20 | N/A | 1.39 | N/A |
| D | N | 1.58 | h = 1.38 | N/A | 1.74 | 80 |
| Type 3 Catch Basin | Y | 1.89 | 1.89 | 5.68 | 1.43 | 40 |
| Type 3 Catch Basin | N | 1.89 | 1.89 | 7.57 | 1.43 | 40 |
| Area Drainage Basin or Field Inlet (Round) | N | 1.65 | 1.65 | 5.18 | 1.45 | 68 |
| Slotted Drain | Y/N | 0.15 | 70 percent of length shown on plans | N/A | N/A | N/A |
| Deck Drain Type A | Y | 0.86 | 2.40 | 4.12 | 1.16 | 56 |

**Table C - ODOT Inlet Design Parameters with 50 Percent Clogging**

| Identification | Next To Curb | Grate, curb-opening or slot Width (feet) | Grate, curb-opening or slot Length (feet) | Grate Perimeter (feet) | Clear Opening | |
|---|---|---|---|---|---|---|
| | | | | | Area (square feet) | Ratio (percent) |
| G-1 & CG-1 | Y | 1.13 | 2.05 | 4.31 | 1.74 | 75 |
| G-1 | N | 1.13 | 2.05 | 6.36 | 1.74 | 75 |
| G-2 & CG-2 | Y | 1.53 | 1.95 | 5.00 | 2.15 | 72 |
| G-2 | N | 1.53 | 1.95 | 6.96 | 2.15 | 72 |
| G-2M & G-2MA | N | 1.53 | 1.95 | 6.96 | 2.39 | 80 |
| CG-3 | Y | h = 0.36 | 1.25 | N/A | 0.45 | N/A |
| Curb Inlet Channel | Y | h = 0.33 | 3.00 | N/A | 0.99 | N/A |
| D | N | 1.12 | h = 1.38 | N/A | 1.24 | 80 |
| Type 3 Catch Basin | Y | 1.60 | 1.60 | 4.80 | 1.02 | 40 |
| Type 3 Catch Basin | N | 1.60 | 1.60 | 6.40 | 1.02 | 40 |
| Area Drainage Basin or Field Inlet (Round) | N | 1.39 | 1.39 | 4.38 | 1.03 | 68 |
| Slotted Drain | Y/N | 0.15 | 50 percent of length shown on plans | N/A | N/A | N/A |
| Deck Drain Type A | Y | 0.67 | 2.21 | 3.56 | 0.83 | 56 |

## 5.0 Inlets on Grade

Inlet interception capacity ($Q_i$) is the flow intercepted by an inlet under a given set of conditions. The efficiency (E) of an inlet is the percent of total flow that the inlet will intercept for those conditions. The efficiency of an inlet is dependent on the cross slope, longitudinal slope, total gutter flow, and pavement roughness. Efficiency is defined by the following equation:

$$E = \frac{Q_i}{Q_t} \qquad \text{(Equation 2)}$$

Where:

$E$ = inlet efficiency
$Q_t$ = total gutter flow in cubic feet per second
$Q_i$ = intercepted flow in cubic feet per second

Flow that is not intercepted by an inlet is termed carryover or bypass flow and is defined as follows:

$$Q_b = Q_t - Q_i \qquad \text{(Equation 3)}$$

Where:

$Q_b$ = flow that is not intercepted by the inlet and must be included in the evaluation of downstream gutters, channels, and inlets

The interception capacity of all inlet configurations increases with increasing flow rates, and inlet efficiency generally decreases with increasing flow rates. Factors affecting gutter flow also affect inlet interception capacity. The depth of water next to the curb is the major factor in the interception capacity of both grate inlets and curb-opening inlets. The interception capacity of a grate inlet depends on the amount of water flowing over the grate, the size and configuration of the grate and the velocity of flow in the gutter.

Interception capacity of a curb-opening inlet is largely dependent on flow depth at the curb and curb-opening length. Flow depth at the curb and consequently, curb-opening inlet interception capacity and efficiency, is increased by the use of a local gutter depression at the curb-opening or a continuously depressed gutter to increase the proportion of the total flow adjacent to the curb.

Slotted drain inlets function in essentially the same manner as curb-opening inlets, i.e., as weirs with flow entering from the side. Interception capacity is dependent on flow depth and inlet length. Efficiency is dependent on flow depth, inlet length and total gutter flow.

The interception capacity of an equal length combination inlet consisting of a grate placed alongside a curb-opening on a grade does not differ materially from that of a grate only. Interception capacity and efficiency are dependent on the same factors which affect grate capacity and efficiency. A combination inlet consisting of a curb-opening inlet placed upstream of a grate inlet has a capacity equal to that of the curb-opening length upstream of the grate plus that of the grate, taking into account the reduced spread and depth of flow over the grate because

of the interception by the curb-opening. This inlet configuration has the added advantage of intercepting debris that might otherwise clog the grate and deflect water away from the inlet.

## 5.1 Grate Inlets (On-grade)

The capacity of a grate inlet depends upon its geometry, cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of gutter inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, a portion of the frontal flow may tend to splash over the end of the grate for some grates. Chart 1 in Appendix H can be utilized to determine splash-over velocities ($V_o$) for various grate configurations and the portion of frontal flow intercepted by the grate.

*Note:*

- *Standard grate inlets used in ODOT storm drainage systems are noted in Appendix C.*
- *The parallel bar grates are the most efficient grates on steep slopes but are not bicycle safe.*

The ratio of frontal flow to total gutter flow ($E_o$) for straight cross slope is expressed by the following equation:

$$E_o = \frac{Q_w}{Q_t} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \qquad \text{(Equation 4)}$$

Where:

$Q_t$ = total gutter flow in cubic feet per second
$Q_w$ = flow in width (W) in cubic feet per second
W = width of depressed gutter or grate in feet
T = total spread of water in the gutter in feet

Chart 2 in Appendix H provides a graphical solution of ($E_o$) for either straight cross slopes or depressed gutter sections.

The ratio of side flow ($Q_s$) to total gutter flow is:

$$\frac{Q_s}{Q_t} = \frac{1 - Q_w}{Q_t} = 1 - E_o \qquad \text{(Equation 5)}$$

The ratio of frontal flow intercepted to total frontal flow ($R_f$) is expressed by the following equation:

$$R_f = 1 - 0.09\,(V - V_o) \qquad \text{(Equation 6)}$$

Where:

V = velocity of flow in the gutter in feet per second
$V_o$ = gutter velocity where splash-over first occurs in feet per second

*Note: $R_f$ can not exceed 1.0. If V is less than $V_o$, $R_f$ =1, meaning that all flow is intercepted. If V is greater than $V_o$, $R_f$ is less than 1, meaning that a portion of frontal flow is intercepted.*

This ratio is equivalent to frontal flow interception efficiency. Chart 1 in Appendix H provides a solution of which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Chart 1 in Appendix H is total gutter flow divided by the area of flow.

*Note: Chart 3 in Appendix H is a Nomograph to solve for velocity in a triangular gutter section with known cross slope, slope and spread. Additional nomographs are located in Chapter 8.*

The ratio of side flow intercepted to total side flow ($R_s$) or side flow interception efficiency, is expressed by:

$$R_s = \frac{1}{1+\left(\frac{0.15\,V^{1.8}}{S_x L^{2.3}}\right)} \qquad \text{(Equation 7)}$$

Where:

V = velocity of flow in gutter in feet per second
L = length of the grate in feet
$S_x$ = cross slope in foot per foot

Chart 4 in Appendix H provides a solution to Equation 7.

The efficiency (E) of a grate is expressed as:

$$E = R_f E_o + R_s(1 - E_o) \qquad \text{(Equation 8)}$$

The interception capacity ($Q_i$) of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

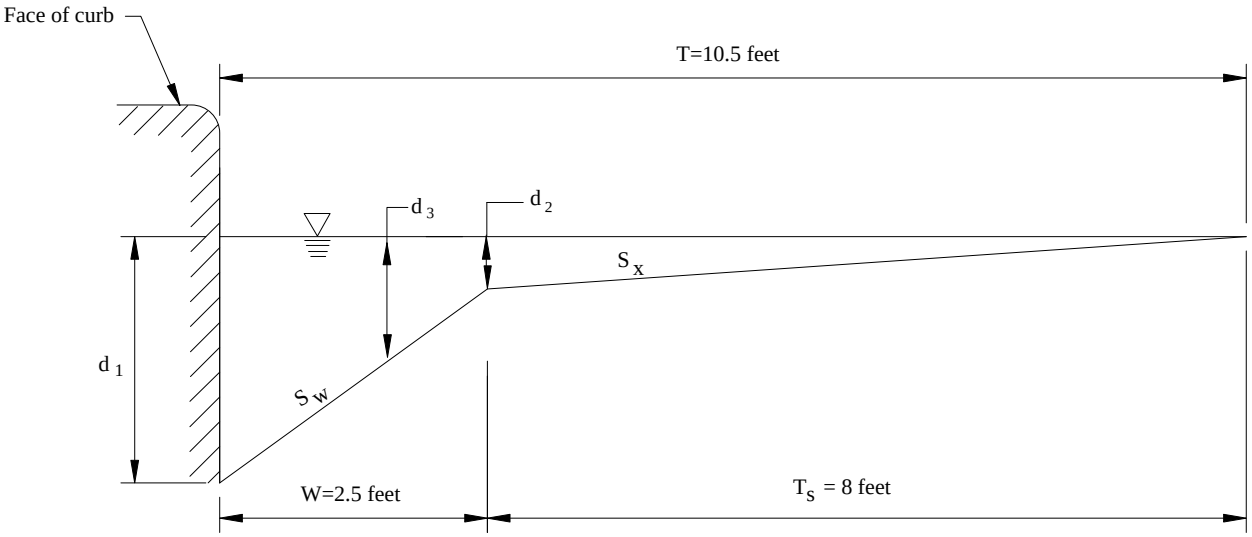
$$Q_i = E\,Q = Q\,[\,R_f E_o + R_s\,(1 - E_o)\,] \qquad \text{(Equation 9)}$$

### 5.1.1 Grate Inlet Capacity (On-grade) Example

The following example illustrates the application of inlet spacing, efficiency and by-pass equations.

#### 5.1.1.1 Determine maximum allowable flow, interception flow, bypass flow, location of first inlet, and spacing of subsequent inlets.

Sketch:




Given:

- Highway on-grade section (greater than or equal to 45 miles per hour)
- Curb and Gutter (reference **Chapter 8**)
- n = 0.016 (Manning's coefficient for asphalt pavement)
- n = 0.014 (Manning's coefficient for concrete gutter w/ AC)
- $S_x$ = 0.02 foot per foot (shoulder and roadway cross slope)
- $S_L$ = 0.035 foot per foot (roadway longitudinal slope)
- $W_p$ = 46.5 feet (width of contributing drainage area, 2 – 12-foot lanes with 10.5 foot shoulder/gutter, 5 foot sidewalk, 7 foot median)
- W = 2.5 feet (gutter width)
- $S_w$ = 0.05 foot per foot (gutter slope)
- C = 0.90 (rational method runoff coefficient for pavement)
- 10-year design, Hydrologic Zone 8 (IDR curves are located in **Chapter 7**)
- G-2 inlet with P-1 $^{7}/_{8}$ – 4 grate (reference Table B, Appendix C)
- $W_g$ = 2.25 feet (grate width for G-2 grate inlet)
- $L_g$ = 2.67 feet (grate length for G-2 grate inlet)

### 5.1.1.2 Method A

Criteria:

- Allowable spread is 0 feet into the outside lane (reference Table A, Appendix D)
- Therefore; T = allowable spread
  = shoulder width + gutter width
  = 8 feet + 2.5 feet = 10.5 feet
- E = 0.70 (minimum efficiency for Method A)
- $L_2$ = 400 feet (maximum distance between successive inlets, reference Section 2, Appendix D)

Solution:

**Step A.1-** Solve for ($d_1$, $d_2$, $d_3$):

$d_1$ = flow depth at curb = (T-W) $S_x$ + W$S_w$
$d_2$ = flow depth at break in gutter cross slope = (T-W) $S_x$ (Chapter 8)
$d_3$ = flow depth at edge of inlet = (T-W) $S_x$ + (W-grate width) $S_w$

$d_1$ = (10.5 – 2.5)(0.02) + (2.5)(0.05) = 0.285 feet
$d_2$ = (10.5 – 2.5)(0.02) = 0.16 feet
$d_3$ = (10.5 – 2.5) (0.02) + (2.5 – 2.25)(0.05) = 0.1725 feet

**Step A.2-** Solve for ($Q_t$):

$Q_t$ = total gutter flow (cubic feet per second)

$$Q_t = \frac{0.56\, S_L^{0.5}\, d_2^{2.67}}{n\, S_x} + \frac{0.56\, S_L^{0.5}\, (d_1^{2.67} - d_2^{2.67})}{n\, S_w} \qquad \left[\begin{array}{c}\text{Chapter 8 or Chart 2}\\ \text{and 5, Appendix H}\end{array}\right]$$

$$Q_t = \frac{0.56\,(0.035)^{0.5}\,(0.16)^{2.67}}{(0.016)(0.02)} + \frac{0.56\ (0.035)^{0.5}\,(0.285^{2.67} - 0.16^{2.67})}{(0.014)(0.05)}$$

$Q_t$ = 2.46 + 4.25 = 6.71 cubic feet per second

**Step A.3-** Solve for ($Q_w$):

$Q_w$ = flow in width (W) or frontal flow, equal to the grate width (in cubic feet per second)

$$Q_w = \frac{0.56\, S_L^{0.5}\, (d_1^{2.67} - d_3^{2.67})}{n\, S_w}$$ (**Chapter 8** or Chart 5, in Appendix H)

$$= \frac{0.56(0.035)^{0.5}(0.285^{2.67} - 0.1725^{2.67})}{(0.014)(0.05)}$$

$Q_w$ = 3.87 cubic feet per second

**Step A.4-** Solve for (V):

V = average gutter flow velocity (in feet per second)

$$V = \frac{Q_t}{A} = \frac{2Q_t}{T_s^2 S_x + W^2 S_w + 2WT_s S_x} = \frac{2Q_t}{\left[W(d_1 + d_2) + \frac{d_2^2}{S_x}\right]}$$ (**Chapter 8**)

$$V = \frac{2\,(6.71)}{(8)^2(0.02) + (2.5)^2(0.05) + (2)(2.5)(8)(0.02)} = 5.61 \text{ feet per second}$$

**Step A.5-** Solve for ($E_o$):

$E_o$ = the ratio of frontal flow to total gutter flow

$$E_o = \frac{Q_w}{Q_t} = \frac{3.87}{6.71} = 0.577$$ (Equation 4 or Chart 2 in Appendix H)

**Step A.6-** Solve for ($R_f$):

$R_f$ = the ratio of frontal flow intercepted to total frontal flow

$R_f = 1 - 0.09\,(V - V_o)$ (Equation 6 or Chart 1 in Appendix H)

Where:

V = average flow velocity in the gutter (in feet per second)

$V_o$ = gutter velocity where splash over first occurs (in feet per second)

V = 5.61 feet per second

G-2 Grate length = 2.67 feet

Grate type = P – $1^7/_8$ – 4

$V_o$ = 5.9 feet per second (per Chart 1, Appendix H)

$R_f = 1 - 0.09\,(5.61 - 5.9)$

$R_f = 0.97$

$R_f$ cannot exceed 1.0, therefore,

$R_f = 0.97$

**Step A.7-** Solve for ($R_s$):

$R_s$ = the ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1 + \left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)}$$ (Equation 7 or Chart 4, Appendix H)

$$R_s = \frac{1}{1+\left(\frac{0.15\,(5.61)^{1.8}}{(0.02)(2.67)^{2.3}}\right)}$$

$R_s = 0.054$

**Step A.8-** Solve for (E):

E = the inlet interception efficiency

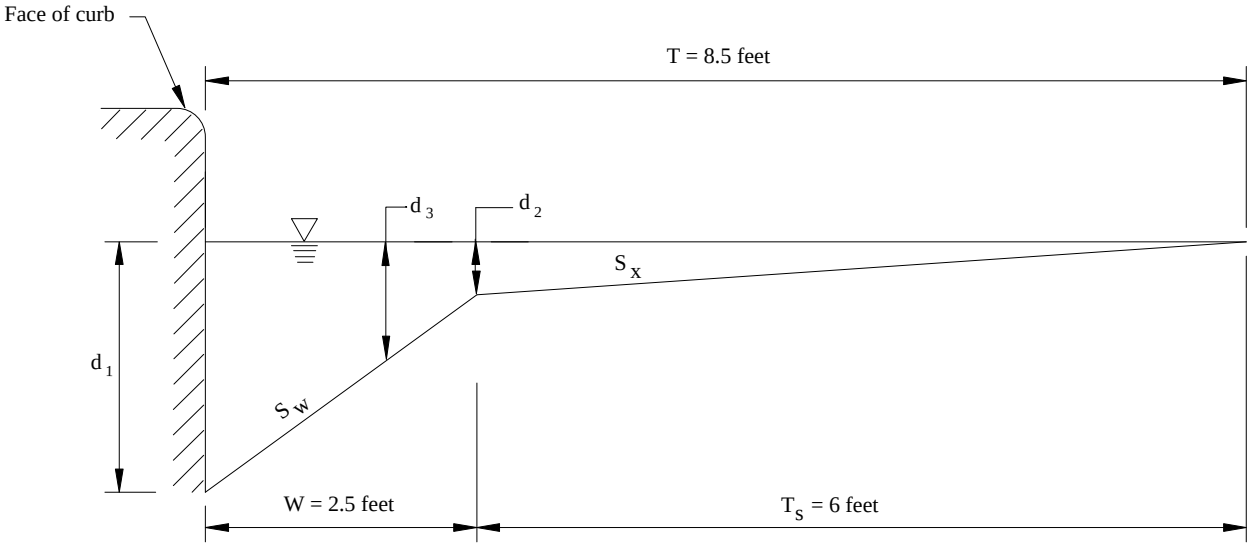
$E = R_f E_o + R_s (1 - E_o)$ (Equation 8)

$E = (1.0)(0.56) + .054\,(1 - 0.56) = 0.58$

E is less than 0.70, therefore, more than 30 percent would bypass inlet. A narrower flow width (T) would be assumed and steps 1 through 8 would be repeated until the minimum efficiency of 0.70 is obtained.

Try T = allowable spread = 8.5 feet

Sketch:




**Step A1.2-** Solve for ($d_1$, $d_2$, $d_3$):

$d_1$ = flow depth at curb = (T-W) $S_x$ + W$S_w$
$d_2$ = flow depth at break in gutter cross slope = (T-W) $S_x$ (**Chapter 8**)
$d_3$ = flow depth at edge of inlet = (T-W) $S_x$ + (W-grate width) $S_w$

$d_1$ = (8.5 – 2.5)(0.02) + (2.5)(0.05) = 0.245 feet
$d_2$ = (8.5 – 2.5)(0.02) = 0.12 feet
$d_3$ = (8.5 – 2.5) (0.02) + (2.5 – 2.25)(0.05) = 0.13 feet

**Step A2.2-** Solve for ($Q_t$):

$Q_t$ = total gutter flow (cubic feet per second)

$$Q_t = \frac{0.56\, S_L^{0.5}\, d_2^{2.67}}{n\, S_x} + \frac{0.56\, S_L^{0.5}\, (d_1^{2.67} - d_2^{2.67})}{n\, S_w} \qquad \left[\begin{array}{c}\text{Chapter 8 or Chart 2}\\ \text{and 5, Appendix H}\end{array}\right]$$

$$Q_t = \frac{0.56\,(0.035)^{0.5}\,(0.12)^{2.67}}{(0.016)(0.02)} + \frac{0.56\ (0.035)^{0.5}\,(0.245^{2.67} - 0.12^{2.67})}{(0.014)(0.05)}$$

$Q_t$ = 1.14 + 2.98 = 4.12 cubic feet per second

**Step A3.2-** Solve for ($Q_w$):

$Q_w$ = flow in width (W) or frontal flow, equal to the grate width (in cubic feet per second)

$$Q_w = \frac{0.56\, S_L^{0.5}\, (d_1^{2.67} - d_3^{2.67})}{n\ S_w}$$ (**Chapter 8** or Chart 5, Appendix H)

$$= \frac{0.56(.035)^{0.5}(0.245^{2.67} - 0.13^{2.67})}{(0.014)(0.05)}$$

$Q_w$ = 2.86 cubic feet per second

**Step A4.2-** Solve for (V):

V = average gutter flow velocity (in feet per second)

$$V = \frac{Q_t}{A} = \frac{2Q_t}{T_s^2 S_x + W^2 S_w + 2WT_s S_x} = \frac{2Q_t}{\left[W(d_1 + d_2) + \frac{d_2^2}{S_x}\right]}$$ (**Chapter 8**)

$$V = \frac{2\,(4.12)}{(6)^2(0.02) + (2.5)^2(0.05) + (2)(2.5)(6)(0.02)} = 5.05 \text{ feet per second}$$

**Step A5.2-** Solve for ($E_o$):

$E_o$ = the ratio of frontal flow to total gutter flow

$$E_o = \frac{Q_w}{Q_t} = \frac{2.86}{4.12} = 0.69$$ (Equation 4 or Chart 2, Appendix H)

**Step A6.2-** Solve for ($R_f$):

$R_f$ = the ratio of frontal flow intercepted to total frontal flow

$R_f$ = $1 - 0.09 (V - V_o)$ (Equation 6 or Chart 1, Appendix H)

Where:
V = average flow velocity in the gutter (in feet per second)
$V_o$ = gutter velocity where splash over first occurs (in feet per second)
V = 5.05 feet per second

G-2 Grate length = 2.67 feet
Grate type = P – $1^7/_8$ – 4
$V_o$ = 5.9 feet per second (per Chart 1, Appendix H)

$R_f$ = $1 - .09 (5.05 - 5.9)$
$R_f$ = 0.92
$R_f$ can not exceed 1.0, therefore,
$R_f$ = 0.92

**Step A7.2-** Solve for ($R_s$):

$R_s$ = the ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1+\left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)}$$ (Equation 7 or Chart 4, Appendix H)

$$= \frac{1}{1+\left(\frac{0.15\,(5.05)^{1.8}}{(0.02)(2.67)^{2.3}}\right)}$$

$R_s$ = 0.065

**Step A8.2-** Solve for (E):

E = the inlet interception efficiency

$E = R_f E_o + R_s (1 - E_o)$ (Equation 8)

E = (1.0)(0.67) + (0.065) (1 - 0.67)

E = 0.69 Okay

**Step A9.2-** Solve for ($L_1$):

$L_1$ = first inlet from the crest

$$L_1 = \frac{43{,}560\,Q_t}{C\,C_f\,i\,W_p}\,E$$ (Equation 1)

i = 2.28 inches per hour (Assume 5 minute $t_c$ using a 10-year storm design, Hydrologic Zone 8, use IDR curve in **Chapter 7**)
C = 0.90
$C_f$ = 1.0 (per Appendix F, **Chapter 7**)
$W_p$ = 24 feet (2 – 12 foot lanes) + 8 foot shoulder + 2.5 gutter + 5 foot sidewalk + 7 foot median = 46.5 feet (width of contributing drainage area)
E = 1.0

$$L_1 = \frac{43560\,(4.12)}{(0.90)(1.0)(2.28)(46.5)}(1.0) = 1{,}881 \text{ feet}$$

**Step A10.2-** Solve for ($L_2$):

$L_2$ = distance to successive inlets

$$L_2 = \frac{43{,}560\,Q_i}{C\,C_f\,i\,W_p} \qquad \text{(Equation 1)}$$

$Q_i$ = intercepted flow (in cubic feet per second)
$Q_i = EQ_t = (0.70)\,(4.12) = 2.88$ cubic feet per second

$$L_2 = \frac{43{,}560\,(2.88)}{(0.9)(1.0)(2.28)(46.5)} = 1{,}315 \text{ feet}$$

Maximum $L_2$ = 400 feet (per Appendix D)
Therefore, $L_1$ = 1,881 feet and $L_2$ = 400 feet

### 5.1.1.3 Method B

- Assume T = allowable spread
  = shoulder width + gutter width
  = 8 + 2.5 = 10.5 feet (per Table A for highway on-grade section; greater than or equal to 45 miles per hour)
- Assume 30 percent clogged (per Table A).
- $L_g$ = 2.26 feet (grate length for G-2 grate inlet, 30 percent clogged, Table B)
- $W_g$ = 1.84 feet (grate width for G-2 grate inlet, 30 percent clogged, Table B)
- $L_2$ = 400 feet, maximum distance to successive inlets (reference Appendix D)

Solution:

**Step B.1-** Solve for ($d_1$, $d_2$, $d_3$):

$d_1$ = flow depth at curb = $(T\text{-}W)S_x + WS_w$ (**Chapter 8**)
$d_2$ = flow depth at break in gutter cross-slope = $(T\text{-}W)S_x$
$d_3$ = flow depth at edge of inlet = $(T\text{-}W)S_x + (W - \text{grate width})S_w$

$d_1$ = (10.5 – 2.5)(0.02) + (2.5)(.05) = 0.285 feet
$d_2$ = (10.5 – 2.5)(0.02) = 0.16 feet
$d_3$ = (10.5 -2.5)(0.02) + (2.5 – 1.84)(.05) = 0.193 feet

**Step B.2-** Solve for ($Q_t$):

$Q_t$ = total gutter flow (cubic feet per second)

$$Q_t = \frac{0.56\, S_L^{0.5}\, d_2^{2.67}}{n\, S_x} + \frac{0.56\, S_L^{0.5}\, (d_1^{2.67} - d_2^{2.67})}{n\, S_w} \qquad \left[\begin{array}{c}\text{Chapter 8 or Charts 2}\\ \text{and 5, Appendix H}\end{array}\right]$$

$$Q_t = \frac{0.56\,(0.035)^{0.5}\,(0.16)^{2.67}}{(0.016)(0.02)} + \frac{0.56\,(0.035)^{0.5}\,(0.285^{2.67} - 0.16^{2.67})}{(0.014)(0.05)}$$

$Q_t$ = 2.46 + 4.12 = 6.58 cubic feet per second

**Step B.3-** Solve for ($Q_w$):

$Q_w$ = flow in width (W) or frontal flow, equal to the clogged grate width (in cubic feet per second)

$$Q_w = \frac{0.56\, S_L^{0.5}\, (d_1^{2.67} - d_3^{2.67})}{n\, S_w} \qquad \text{(\textbf{Chapter 8} or Chart 5, Appendix H)}$$

$$Q_w = \frac{0.56(0.035)^{0.5}\,(0.285^{2.67} - 0.193^{2.67})}{(0.014)(0.05)} = 3.39 \text{ cubic feet per second}$$

**Step B.4-** Solve for (V):

V = average gutter flow velocity (in feet per second)

$$V = \frac{Q_t}{A} = \frac{2\,Q_t}{T_s^2 S_x + W^2 S_w + 2WT_s S_x} = \frac{2\,Q_t}{\left[W(d_1 + d_2) + \frac{d_2^2}{S_x}\right]} \qquad \text{(\textbf{Chapter 8})}$$

$$V = \frac{2\,(6.58)}{(8)^2(0.02) + (2.5)^2\,(0.05) + 2(2.5)(8)(0.02)} = 5.50 \text{ feet per second}$$

**Step B.5-** Solve for ($E_o$):

$E_o$ = the ratio of frontal flow to total gutter flow

$$E_o = \frac{Q_w}{Q_t} = \frac{3.39}{6.58} = 0.515 \qquad \text{(Equation 4 or Chart 2, Appendix H)}$$

**Step B.6-** Solve for ($R_f$):

$R_f$ = the ratio of frontal flow intercepted to total frontal flow

$R_f = 1 - 0.09\,(V - V_o)$ (Equation 6 or Chart 1, Appendix H)

$V$ = average flow velocity in the gutter (in feet per second)
$V_o$ = gutter velocity where splash over first occurs (in feet per second)

$V$ = 5.50 feet per second
$V_o$ = 5.1 feet per second (per Chart 1, Appendix H, using 30 percent clogged grate length equal to 2.26 feet)

$R_f = 1 - 0.09\,(V - V_o)$
$R_f = 1 - 0.09\,(5.50 - 5.1)$
$R_f = 0.96$

$R_f$ can not exceed 1.0, therefore,
$R_f = 0.96$

**Step B.7-** Solve for ($R_s$):

$R_s$ = the ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1 + \left( \frac{0.15\,V^{1.8}}{S_x\,L^{2.3}} \right)}$$ (Equation 7 or Chart 4, Appendix H)

$L$ = 2.26 feet (30 percent clogged grate length)

$$R_s = \frac{1}{1 + \left( \frac{0.15\,(5.50)^{1.8}}{(0.02)(2.26)^{2.3}} \right)}$$

$R_s = 0.039$

**Step B.8-** Solve for (E):

$E$ = the inlet interception efficiency

$E = R_f\,E_o + R_s\,(1 - E_o)$ (Equation 8)

$E = (1.0)(0.489) + 0.039\,(1 - 0.489)$

$E = 0.51$

Okay (70 percent minimum efficiency criteria does not apply to Method B)

**Step B.9-** Solve for ($L_1$):

$L_1$ = first inlet from the crest

$$L_1 = \frac{43560\, Q_t}{C\, C_f\, i\, W_p} E \qquad \text{(Equation 1)}$$

i = 2.28 inches per hour (Assume 5-minute $t_c$ using a 10-year storm design, Hydrologic Zone 8; use IDR curve in **Chapter 7**)

C = 0.90

$C_f$ = 1.0 (per Appendix F, **Chapter 7**)

$W_p$ = 24 feet (2 – 12-foot lanes) + 10.5-foot shoulder/gutter + 5-foot sidewalk + 7-foot median = 46.5 feet

E = 1.0

$$L_1 = \frac{43{,}560\,(6.58)}{(0.90)(1.0)(2.28)(46.5)}(1) = 3{,}004 \text{ feet}$$

**Step B.10-** Solve for ($L_2$):

$L_2$ = distance to successive inlets

$$L_2 = \frac{43{,}560\, Q_i}{C\, C_f\, i\, W_p} \qquad \text{(Equation 1)}$$

$Q_i$ = intercepted flow (in cubic feet per second)

$Q_i$ = E $Q_t$ = (0.51)(6.58) = 3.36 cubic feet per second

$$L_2 = \frac{43{,}560\,(3.36)}{(0.90)(1.0)(2.28)(46.5)} = 1{,}534 \text{ feet}$$

$L_2$ = 400 feet, maximum distance to successive inlets (per Appendix D)

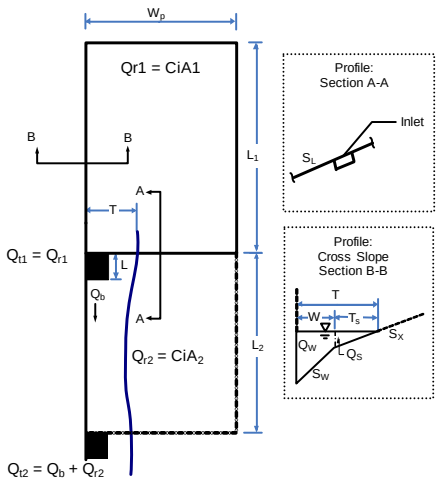
Therefore, $L_1$ = 3,004 feet and $L_2$ = 400 feet

**Inlet Design Computation Sheet (Diagram B-B Process)**

**Given:**

| | |
|---|---|
| **Surface Description** | |
| Road Classification | Highway on-grade section (greater than or equal to 45 miles per hour) |
| Type of Curb and Gutter, see Chapter 8, Curbs and Gutters | Curb and Gutter |
| Surface Material | concrete gutter w/ pavement |
| Manning's Coefficient, n | 0.014 |
| **Drainage Area Description** | |
| Runoff Coefficient, C | 0.9 |
| Hydrologic Zone *(Chapter 7)* | 8 |
| Intensity (i) in/hr *(Chapter 7)* | 2.28 in/hr, 5-minute $T_c$, 10-year design storm |
| $C_f$ *(Chapter 7, Appendix F)* | 1 |
| Type of Inlet *(Chapter 13, Appendix C, Table B)* | G-2 Inlet with P-1 7/8 - 4 grate |
| **Design Method:** **Method A** ☑ **Method B** ☐ | |
| **Design Criteria:** | |
| **Method A** (reference Section 2) | |
| Minimum Efficiency | 70 Percent |
| Allowable spread *(Appendix D, Table A)* | shoulder + gutter |
| shoulder width | 8.00 feet |
| distance into outside lane | 0, but 2.5-foot gutter |
| spread (T) = Shoulder Width + Distance into outside lane | 10.5 feet |
| Percentage Clogged | 0% |
| **Method B** (reference Section 2) | |
| Allowable spread *(Appendix D, Table A)* | |
| shoulder width | |
| distance into outside lane | |
| spread (T) = Shoulder Width + Distance into outside lane | |
| Percentage Clogged | |

**Sketch:**




| Assume Spread | | | | | | | | Calculate Hydrology | | | Calculate Inlet Efficiency | | | Locate Inlet | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Spread | Long. Slope | Cross Slope | | Gutter Width | Total Gutter Flow | Design Inlet Length | Design Inlet Width | Flow from Contributing Area | Bypass Flow from Upstream Inlet | Total Flow | Inlet Interception Efficiency | Intercepted Flow | Bypass Flow | Drainage Length | Drainage Width | Drainage Area | Number | Station | Comments |
| T | $S_L$ | $S_x$ | $S_w$ | W | $Q_t$ | $L_i$ | $W_i$ | $Q_r$ | $Q_b$ | $Q_t$ | E | $Q_i$ | $Q_b$ | L | $W_p$ | A | | | |
| (ft) | (ft/ft) | (ft/ft) | (ft/ft) | (ft) | (cfs) | (ft) | (ft) | (cfs) | (cfs) | (cfs) | | (cfs) | (cfs) | (ft) | (ft) | (ac) | | | |
| | | | | | | | | | | | | | | | | | | 34+00 | High point |
| 10.50 | 0.035 | 0.02 | 0.05 | 2.50 | 6.07 | 2.67 | 2.25 | 6.07 | 0.00 | 6.07 | 0.59 | | | | | | | | Because E is less than 70 percent, more flow would bypass than allowed, reduce flow width and try again |
| 8.50 | 0.035 | 0.02 | 0.05 | 2.50 | 3.75 | 2.67 | 2.25 | 3.75 | 0.00 | 3.75 | 0.70 | 2.63 | 1.12 | 1,711 | 46.50 | 1.83 | 2 | 16+89 | First inlet from crest |
| 8.50 | 0.035 | 0.02 | 0.05 | 2.50 | 3.75 | 2.67 | 2.25 | 2.63 | 1.12 | 3.75 | 0.70 | 2.63 | 1.12 | 1,200 | 46.50 | 1.28 | 1 | 12+89 | This is the maximum distance (1,200 feet) allowed between catchbasins with bypass flows based on hydraulic conditions only; however, due to maintenance requirements, the maximum distance between catchbasins is 400 feet. |

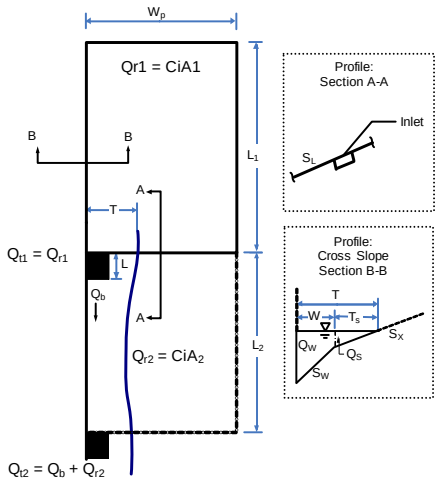
**Figure 4a – Example Problem 5.1.1 (Diagram B-B, Method A)**

**Inlet Design Computation Sheet (Diagram B-B Process)**

**Given:**

| | |
|---|---|
| **Surface Description** | |
| Road Classification | Highway on-grade section (greater than or equal to 45 miles per hour) |
| Type of Curb and Gutter, see Chapter 8, Curbs and Gutters | Curb and Gutter |
| Surface Material | concrete gutter w/ AC |
| Manning's Coefficient, n | 0.014 |
| **Drainage Area Description** | |
| Runoff Coefficient, C | 0.9 |
| Hydrologic Zone *(Chapter 7)* | 8 |
| Intensity (i) in/hr *(Chapter 7)* | 2.28 in/hr, 5-minute $T_c$, 10-year design storm |
| $C_f$ *(Chapter 7, Appendix F)* | 1 |
| Type of Inlet *(Chapter 13, Appendix C, Table B)* | G-2 Inlet with P-1 7/8 - 4 grate |
| **Design Method: Method A ☐ Method B ☑** | |
| **Design Criteria:** | |
| **Method A (reference Section 2)** | |
| Minimum Efficiency | 70 Percent |
| Allowable spread *(Appendix D, Table A)* | |
| shoulder width | |
| distance into outside lane | |
| spread (T) = Shoulder Width + Distance into outside lane | |
| Percentage Clogged | |
| **Method B (reference Section 2)** | |
| Allowable spread *(Appendix D, Table A)* | shoulder + gutter + 0 feet into lane |
| shoulder width | 8.00 feet |
| distance into outside lane | 0.00, but 2.5-foot gutter |
| spread (T) = Shoulder Width + Distance into outside lane | 10.5 feet |
| Percentage Clogged | 30% |

**Sketch:**




| Assume Spread | | | | | | | | Calculate Hydrology | | | Calculate Inlet Efficiency | | | Locate Inlet | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Spread | Long. Slope | Cross Slope | | Gutter Width | Total Gutter Flow | Design Inlet Length | Design Inlet Width | Flow from Contributing Area | Bypass Flow from Upstream Inlet | Total Flow | Inlet Interception Efficiency | Intercepted Flow | Bypass Flow | Drainage Length | Drainage Width | Drainage Area | Number | Station | Comments |
| T | $S_L$ | $S_x$ | $S_w$ | W | $Q_t$ | $L_i$ | $W_i$ | $Q_r$ | $Q_b$ | $Q_t$ | E | $Q_i$ | $Q_b$ | L | $W_p$ | A | | | |
| (ft) | (ft/ft) | (ft/ft) | (ft/ft) | (ft) | (cfs) | (ft) | (ft) | (cfs) | (cfs) | (cfs) | | (cfs) | (cfs) | (ft) | (ft) | (ac) | | | |
| | | | | | | | | | | | | | | | | | | 34+00 | High point |
| 10.50 | 0.035 | 0.02 | 0.05 | 2.50 | 6.07 | 2.26 | 1.84 | 6.07 | 0.00 | 6.07 | 0.51 | 3.10 | 2.97 | 2,771 | 46.50 | 2.96 | 2 | 6+29 | First inlet from crest |
| 10.50 | 0.035 | 0.02 | 0.05 | 2.50 | 6.07 | 2.26 | 1.84 | 3.10 | 2.97 | 6.07 | 0.51 | 3.10 | 2.97 | 1,415 | 46.50 | 1.51 | 1 | 2+29 | This is the maximum distance (1,415 feet) allowed between catchbasins with bypass flows based on hydraulic conditions only; however, due to maintenance requirements, the maximum distance between catchbasins is 400 feet. |

**Figure 4b – Example Problem 5.1.1 (Diagram B-B, Method B)**

## 5.2 Curb-Opening Inlets (On-grade)

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb-openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

*Note:*

- *Standard curb-opening inlets used in ODOT storm drainage systems are noted in Appendix C.*
- *Curb-opening inlets lose capacity rapidly with an increase in longitudinal grade.*

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = 0.6\, Q_t^{0.42}\, S_L^{0.3} \left( \frac{1}{n\, S_x} \right)^{0.6} \qquad \text{(Equation 10)}$$

Where:

$L_T$ = curb-opening length required to intercept 100 percent of the gutter flow in feet
$S_x$ = cross slope in foot per foot
$n$ = Manning's coefficient
$Q_t$ = total gutter flow in cubic feet per second
$S_L$ = longitudinal slope in foot per foot

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by:

$$E = 1 - \left( 1 - \frac{L}{L_T} \right)^{1.8} \qquad \text{(Equation 11)}$$

Where:

$L$ = curb-opening length in feet

Chart 6 in Appendix H is a nomograph for the solution of Equation 10, and Chart 7 (Appendix H) provides a solution of Equation 11.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope ($S_e$) in Equation 12.

$$S_e = S_x + S'_w E_o \qquad \text{(Equation 12)}$$

Where:

| | | |
|---|---|---|
| $S'_w$ | = | cross slope of the gutter measured from the cross slope of the pavement |
| | = | a/W in foot per foot |
| a | = | gutter depression in feet |
| W | = | width of the depressed gutter section (W = 2 feet for CG-3 inlets) in feet |
| $E_o$ | = | ratio of flow in the depressed section to total gutter flow. It is determined by the gutter configuration upstream of the inlet. |
| $S_x$ | = | roadway cross slope in foot per foot |

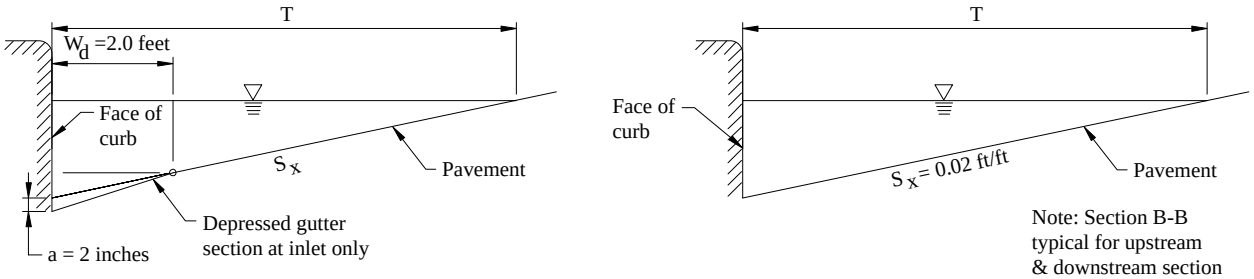
*Note: $S_e$ can be used to calculate the length of curb-opening by substituting $S_e$ for $S_x$ in Equation 10.*

### 5.2.1 Curb-Opening Inlet (On-grade) Example

The following example illustrates the application of the inlet interception equation.

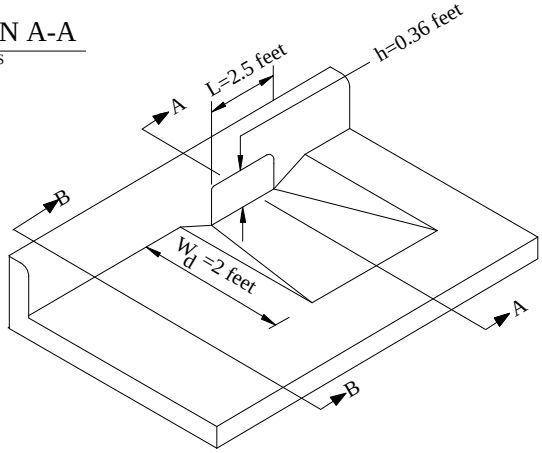
#### 5.2.1.1 Determine location of first inlet and spacing of subsequent inlets

Sketch:




SECTION A-A
NTS

SECTION B-B
NTS




CURB OPENING INLET
NTS

Given:

- Highway on-grade section (less than 45 miles per hour)
- Standard Curb (reference **Chapter 8**)
- n = 0.016 (Manning's coefficient for asphalt pavement)
- $S_x$ = 0.02 foot per foot (roadway cross slope)
- $S_L$ = 0.0025 foot per foot (roadway longitudinal slope)
- $W_p$ = 31 feet (width of contributing drainage area, 2 – 12-foot lanes with 7-foot shoulder)
- CG-3 inlet (reference Table B, Appendix C)
- h = 0.36 feet (curb opening height for CG-3 inlet, see sketch above)
- L = 2.5 feet (curb opening length for CG-3 inlet, see sketch above)
- a = 2 inches (inlet depression, see sketch above)
- $W_d$ = 2 feet (width of depressed gutter section)
- C = 0.90 (rational method runoff coefficient for pavement)
- 10-year design, Salem area (IDR curves are located in **Chapter 7**)

Criteria:

- Allowable spread is 2 feet into the outside lane (per Table A for highway on-grade section; less than 45 miles per hour)
- Therefore; T = allowable spread = shoulder width + 2 feet
  = 7 feet + 2 feet = 9 feet
- E = 0.70 (minimum efficiency for Method A)
- $L_2$ = 400 feet (maximum distance between successive inlets, reference Section 2, Appendix D)

### 5.2.1.2 Method A

Solution:

**Step A.1-** Solve for ($Q_t$):

$Q_t$ = total gutter flow (cubic feet per second). The approach gutter section to the inlet is used to calculate the total flow

$$Q_t = \frac{0.56}{n}(S_x)^{1.67}(S_L)^{0.5}(T)^{2.67}$$ (**Chapter 8** or Chart 5, Appendix H)

$$Q_t = \frac{0.56}{.016}(0.02)^{1.67}(0.0025)^{0.5}(9)^{2.67}$$

$Q_t$ = 0.90 cubic feet per second

**Step A.2-** Solve for ($E_o$):

$E_o$ = ratio of flow in the depressed section to total gutter flow

$$E_o = 1-\left(1-\frac{W_d}{T}\right)^{2.67} = 1-\left(1-\frac{2}{9}\right)^{2.67} = 0.489$$ [Equation 4 or Chart 2, Appendix H]

**Step A.3-** Solve for ($S_e$):

$S_e$ = equivalent cross-slope (foot per foot)

$S'_w = \frac{a}{W_d}$ (cross slope of the depressed gutter measured from the cross slope of the pavement, $S_x$)

a = 2 inches for CG-3 inlets (gutter depression)

$W_d$ = 2 feet for CG-3 inlets (width of the depressed gutter section)

$$S'_w = \frac{\left(\frac{2\text{ in}}{12\text{ in/ft}}\right)}{2\text{ ft}} = 0.0833 \text{ foot per foot}$$

$S_e = S_x + S'_W E_o$ (Equation 12)

$S_e = 0.02 + (0.0833)(0.489) = 0.0607$

**Step A.4-** Solve for ($L_T$):

$L_T$ = curb-opening length required to intercept 100 percent of the gutter flow

$$L_T = 0.6\, Q^{0.42} S_L^{0.3} \left(\frac{1}{n\, S_e}\right)^{0.6}$$ ($S_e$ is substituted for $S_x$ in Equation 10)

$$L_T = 0.6\,(0.90)^{0.42}\,(0.0025)^{0.3}\left(\frac{1}{(0.016)(0.0607)}\right)^{0.6} = 6.11 \text{ feet}$$

**Step A.5-** Solve for (E):

E = the curb-opening interception efficiency

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8}$$ (Equation 11 or Chart 7, Appendix H)

L = 2.5 feet (curb-opening length)

$$E = 1 - \left(1 - \frac{2.5}{6.11}\right)^{1.8}$$

$E = 0.612$

The curb-opening interception efficiency (E) is less than 0.70 therefore, more than 30 percent would bypass the inlet. A narrower flow width (T) would be assumed and steps 1 through 5 would be repeated until the minimum efficiency of 0.70 is obtained.

Try T = allowable spread = 8 feet

**Step A1.2-** Solve for ($Q_t$):

$Q_t$ = total flow in shoulder and gutter (cubic feet per second)

$$Q_t = \frac{0.56}{n}(S_x)^{1.67}(S_L)^{0.5}(T)^{2.67}$$ (**Chapter 8** or Chart 5, Appendix H)

$$Q_t = \frac{0.56}{0.016}(0.02)^{1.67}(0.0025)^{0.5}(8)^{2.67}$$

$Q_t$ = 0.66 cubic feet per second

**Step A2.2-** Solve for ($E_o$):

$$E_o = 1-\left(1-\frac{W_d}{T}\right)^{2.67}$$ (Equation 4 or Chart 2, Appendix H)

$$E_o = 1-\left(1-\frac{2}{8}\right)^{2.67} = 0.536$$

**Step A3.2-** Solve for ($S_e$):

$$S_e = S_x + S'_W E_o$$ (Equation 12)

$S_e$ = 0.02 + (0.0833)(0.536) = 0.0646

**Step A4.2-** Solve for ($L_T$):

$$L_T = 0.6\,Q^{0.42}\,S_L^{0.3}\left(\frac{1}{n\,S_e}\right)^{0.6}$$ ($S_e$ is substituted for $S_x$ in Equation 10)

$$L_T = 0.6\,(.66)^{0.42}(0.0025)^{0..3}\left[\frac{1}{(.016)\,(.0646)}\right]^{0.6} = 5.17 \text{ feet}$$

**Step A5.2-** Solve for (E):

$$E = 1-\left(1-\frac{L}{L_T}\right)^{1.8}$$ (Equation 11 or Chart 7, Appendix H)

$$E = 1-\left(1-\frac{2.5}{5.16}\right)^{1.8} = 0.70 \text{ Okay}$$

**Step A6.2-** Solve for ($L_1$):

$L_1$ = first inlet from the crest

$$L_1 = \frac{43{,}560\,Q_t}{C\;C_f\;i\;W_p}E$$ (Equation 1)

i = 2.10 inches per hour (Salem area, assume 5 minute $t_c$ using a 10-year storm design)
C = 0.90
$C_f$ = 1.0 (per **Chapter 7**, Appendix F)
$W_p$ = 24 feet (2 – twelve foot lanes) + 7 foot shoulder = 31 feet
E = 1.0

$$L_1 = \frac{43{,}560\,(0.66)}{(0.90)(1.0)(2.10)(31)}(1) = 490 \text{ feet}$$

**Step A7.2-** Solve for ($L_2$):

$L_2$ = distance to successive inlets

$$(1)\ L_2 = \frac{43{,}560\,Q_i}{C\ C_f\ i\ W_p} \qquad \text{(Equation 1)}$$

$$(2)\ L_1 = \frac{43{,}560\,Q_t}{C\ C_f\ i\ W_p} \qquad \text{(Equation 1)}$$

$$(3)\ Q_i = EQ_t \qquad \text{(Equation 9)}$$

Substituting $Q_i = EQ_t$ into Equation 1 and equating Equations 1 and 2 simplifies to:

$L_2 = (E)\ (L_1)$

$L_2 = (0.70)(490) = 343$ feet

Therefore $L_1 = 490$ feet and $L_2 = 343$ feet

### 5.2.1.3 Method B

Criteria:

- Allowable spread is 2 feet into the outside lane (per Table A for highway on-grade section; less than 45 miles per hour)
- T = shoulder width + 2 feet
  = 7 feet + 2 feet
  = 9 feet (total allowable spread, per Table A)
- Assume 30 percent clogged (per Table A)
- Curb-opening inlet 30 percent clogged:
- h = 0.36 feet (curb opening height for CG-3 inlet, 30 percent clogged per Table B)
- L = 1.75 feet (curb opening length for CG-3 inlet, 30 percent clogged per Table B)
- $L_2$ = 400 feet (maximum distance between successive inlets, reference Section 2, Appendix D)

**Step B.1-** Solve for ($Q_t$):

$$Q_t = \frac{0.56}{n}(S_x)^{1.67}(S_L)^{0.5}(T)^{2.67} \quad \text{(\textbf{Chapter 8} or Chart 5, Appendix H)}$$

$$Q_t = \frac{0.56}{0.016}(0.02)^{1.67}(0.0025)^{0.5}(9)^{2.67}$$

$$Q_t = 0.90 \text{ cubic feet per second}$$

**Step B.2-** Solve for ($E_o$):

$$E_o = 1-\left(1-\frac{W_d}{T}\right)^{2.67} = 1-\left(1-\frac{2}{9}\right)^{2.67} = 0.489 \quad \left[\begin{array}{r}\text{Equation 4 or Chart 2,}\\ \text{Appendix H}\end{array}\right]$$

**Step B.3-** Solve for ($S_e$):

$$S'_w = \frac{a}{W_d}$$

$a$ = 2 inches for CG-3 inlets (gutter depression)

$W_d$ = 2 feet for CG-3 inlets (width of the depressed gutter section)

$$S'_w = \frac{\left(\frac{2\text{ in}}{12\text{ in/ft}}\right)}{2\text{ ft}} = 0.0833 \text{ foot per foot}$$

$S_e = S_x + S'_W E_o$ (Equation 12)

$S_e = 0.02 + (0.0833)(0.488) = 0.0607$

**Step B.4-** Solve for ($L_T$):

$$L_T = 0.6\,Q^{0.42}\,S_L^{0.3}\left(\frac{1}{n\,S_e}\right)^{0.6} \quad (S_e \text{ is substituted for } S_x \text{ in Equation 10})$$

$$L_T = 0.6\,(0.90)^{0.42}\,(0.0025)^{0.3}\left(\frac{1}{(0.016)\,(0.0607)}\right)^{0.6}$$

$L_T$ = 6.11 feet

**Step B.5-** Solve for (E):

$$E = 1-\left(1-\frac{L}{L_T}\right)^{1.8} \quad \text{(Equation 11 or Chart 7, Appendix H)}$$

$L$ = 1.75 feet (30 percent clogged curb-opening length)

$$E = 1 - \left(1 - \frac{1.75}{6.11}\right)^{1.8}$$

$$E = 0.456$$

Okay (70 percent minimum efficiency criteria does not apply to method B)

**Step B.6-** Solve for ($L_1$):

$L_1$ = first inlet from the crest

$$L_1 = \frac{43{,}560\, Q_t}{C\ C_f\ i\ W_p} E \qquad \text{(Equation 1)}$$

$i$ = 2.10 inches per hour (10-year precipitation intensity for Salem area)
$C$ = 0.90
$C_f$ = 1.0 (per **Chapter 7**, Appendix F)
$W_p$ = 24 feet (2 – 12-foot lanes) + 7-foot shoulder = 31 feet
$E$ = 1.0

$$L_1 = \frac{43{,}560\,(0.90)}{(0.90)(1.0)(2.10)(31)}(1) = 669 \text{ feet}$$

**Step B.7-** Solve for ($L_2$):

$L_2$ = distance to successive inlets

$L_2$ = (E) ($L_1$) = (0.456) (669 feet) = 304 feet

## 5.3 Slotted Inlets (On-grade)

Slotted inlets are effective pavement drainage inlets which have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations. They can be placed longitudinally in the gutter or transversely to the gutter. Slotted inlets should be connected into inlet structures or provide clean-out ports on both ends for maintenance access in case of plugging or freezing.

The minimum cleanout velocity of 3 feet per second should be provided for slotted drain systems. A minimum slope of 0.89 percent for an 18-inch corrugated pipe and 1.5 percent slope for a 12-inch diameter corrugated pipe would provide the desired velocity of 3 feet per second. The designer should assume 50 percent clogging when using inlet design method B, which will result in twice the calculated required length for flow interception because slotted inlets tend to plug.

*Note: Standard slotted inlet used in ODOT Storm drain systems is noted in Appendix C*

### 5.3.1 Longitudinal Placement

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Slotted inlets may have economic advantages in some cases and could be very useful on widening and safety projects where right of way is narrow and existing inlet capacity must be supplemented. It is much less expensive to add length to an existing slotted inlet to increase interception capacity than it is to add length to an existing curb-opening inlet. In some cases curbs could be eliminated as a result of utilizing slotted inlets.

The Federal Highway Administration has performed tests of slotted inlets with 1.75 inch slot widths and these tests have determined that the length of slotted inlet required for total interception can be computed by Equation 10. Therefore, Chart 6 should be used to evaluate slotted inlets. Equation 11 is also applicable to slotted inlets and Chart 7 can be used to obtain the inlet efficiency for the selected length of slotted inlet.

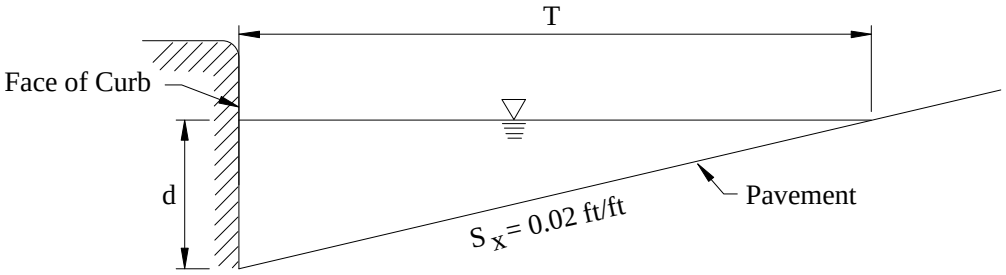
*Note: The determination of total interception and efficiencies are the same for slotted and curb-opening inlets. Therefore, the equations and figures used for the design of curb-opening inlets should be used in the design of slotted inlets. When using Equation 10 for slot inlet design, it should only be used with a slot width of 1.75 inches.*

### 5.3.2 Slotted Inlet (On-grade) Example

The following example illustrates the application of the inlet efficiency equations for slotted inlets.

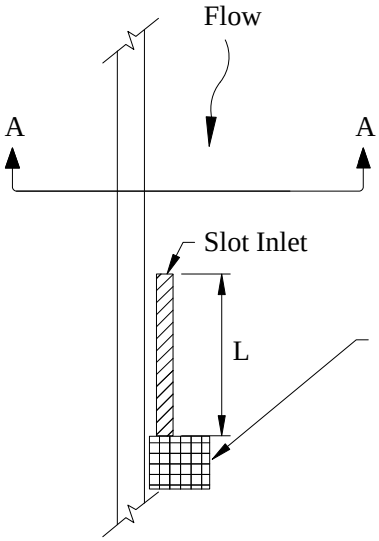
#### 5.3.2.1 Determine the inlet interception efficiency (E), the intercepted flow ($Q_i$), and bypass flow ($Q_b$) for the following conditions:

Sketch:




SECTION A-A

NTS




Grate Inlet (ignored throughout this example. Emphasis is on analyzing a slotted inlet. Inlet is needed at downsteam end of slotted inlet to function as a junction and cleanout)

SLOTTED INLET

NTS

Given:

- Collector road, on-grade section, less than 45 miles per hour
- Standard Curb
- Longitudinal placement of a slotted inlet adjacent to curb.
- L = 10 feet (slotted inlet length)
- $S_L$ = 0.01 foot per foot (roadway longitudinal slope)
- $S_x$ = 0.02 foot per foot (roadway cross slope)
- n = 0.016 (Manning's coefficient for asphalt pavement)
- 10-year design storm, Hydraulic Zone 8 (IDR curves located in **Chapter 7**)
- $W_p$ = 31 feet (width of contributing area, 2 – 11.5 foot lanes with 8-foot shoulder)
- $L_2$ = 400 feet (distance to upstream catch basin)
- i = 2.28 inches per hour (Salem area, assume 5 minute time of concentration, using a 10-year storm)

### 5.3.2.2 Method A

Criteria:

- Bypass flow is limited to a maximum of 30 percent
- Assume no clogging, therefore L = 10 feet (slotted inlet length)
- T = shoulder width + 2 feet
  = 8 feet + 2 feet
  = 10 feet (total allowable spread per Table A)
- E = 0.70 (minimum efficiency for Method A)

Solution:

**Step A.1-** Solve for ($Q_{10}$):

$Q_{10}$ = flowrate to slotted inlet in cubic feet per second

$Q = CiA$ (**Chapter 7**)

$A = (400)(31) = 12{,}400$ square feet = 0.28 acres

$Q = (0.90)(2.28)(0.28) = 0.57$ cubic feet per second

**Step A.2-** Solve for (T):

T = total spread at the slotted inlet (**Chapter 8**)

$$T = \left( \frac{Q_{10} n}{0.56 (S_L)^{0.5} (S_X)^{1.67}} \right)^{0.375}$$

$$T = \left( \frac{(0.57)(0.016)}{0.56 (0.01)^{0.5} (0.02)^{1.67}} \right)^{0.375} = 5.9 \text{ feet}$$

Spread at the slotted inlet = 5.9 feet

Allowable spread = 10 feet.

Placing this slotted inlet 400 feet downstream is acceptable because maximum spread (5.9 feet) is less then the allowable spread (10 feet).

**Step A.3-** Solve for ($L_T$):

$L_T$ = length of opening to intercept 100 percent of the gutter flow in feet

$$L_T = 0.6 Q_{10}^{0.42} S_L^{0.30} \left( \frac{1}{n S_x} \right)^{0.6} \quad \text{(Equation 10)}$$

$$L_T = 0.6(0.57)^{0.42}(0.01)^{0.30}\left(\frac{1}{(0.016)(0.02)}\right)^{0.6}$$

$L_T$ = 14.9 feet

**Step A.4-** Solve for (E):

E = inlet interception efficiency

$$E = 1-\left(1-\frac{L}{L_T}\right)^{1.8} \quad \text{(Equation 11)}$$

$$E = 1-\left(1-\frac{10}{14.9}\right)^{1.8} = 0.86$$

E = 86 percent

Okay, 70 percent minimum efficiency criteria is satisfied.

**Step A.5** Solve for ($Q_i$):

$Q_i$ = intercepted flow

$Q_i$ = EQ (Equation 9)

= (0.86)(0.57) = 0.49 cubic feet per second

**Step A.6** Solve for ($Q_b$):

$Q_b$ = flow that is not intercepted by the slotted inlet and must be included in the evaluation of downstream gutter and inlet

$Q_b$ = Q - $Q_i$ (Equation 3)

$Q_b$ = 0.57 – 0.49 = 0.08 cubic feet per second

### 5.3.2.3 Method B

Criteria:

- Assume 50 percent clogging per Table A, therefore L = 10(0.50) = 5 feet
- T = shoulder width + 2 feet
  = 8 feet + 2 feet
  = 10 feet (total allowable spread per Table A)

**Step B.1-** Solve for ($Q_{10}$):

$Q_{10}$ = flowrate to slotted inlet in cubic feet per second

Q = CiA (**Chapter 7**)

A = (400)(31) = 12,400 square feet = 0.28 acres

Q = (0.90)(2.28)(0.28) = 0.57 cubic feet per second

**Step B.2-** Solve for (T):

T = total spread at the slotted inlet (**Chapter 8**)

$$T = \left( \frac{Q_{10} n}{0.56(S_L)^{0.5}(S_X)^{1.67}} \right)^{0.375}$$

$$T = \left( \frac{(0.57)(0.016)}{0.56(0.01)^{0.5}(0.02)^{1.67}} \right)^{0.375} = 5.9 \text{ feet}$$

Spread at the slotted inlet = 5.9 feet

Allowable spread = 10 feet.

Placing this slotted inlet 400 feet downstream is acceptable because maximum spread is (5.9 feet) less than allowable spread (10 feet).

**Step B.3-** Solve for ($L_T$):

$L_T$ = length of opening to intercept 100 percent of the gutter flow in feet

$$L_T = 0.6 Q_{10}^{0.42} S_L^{0.30} \left( \frac{1}{n S_x} \right)^{0.6} \quad \text{(Equation 10)}$$

$$L_T = 0.6(0.57)^{0.42}(0.01)^{0.30} \left( \frac{1}{(0.016)(0.02)} \right)^{0.6}$$

$L_T$ = 14.9 feet

**Step B.4-** Solve for (E):

E = inlet interception efficiency

L = 5 feet (50 percent clogged slotted inlet length)

$$E = 1 - \left( 1 - \frac{L}{L_T} \right)^{1.8} \quad \text{(Equation 11)}$$

$$E = 1 - \left( 1 - \frac{5}{14.9} \right)^{1.8} = 0.52$$

E = 52 percent

Okay, 70 percent minimum efficiency criteria does not apply to Method B

**Step B.5** Solve for ($Q_i$):

$Q_i$ = intercepted flow

$Q_i$ = EQ (Equation 9)

= (0.52)(0.57) = 0.3 cubic feet per second

**Step B.6** Solve for ($Q_b$):

$Q_b$ = flow that is not intercepted by the slotted inlet and must be included in the evaluation of downstream gutter and inlet

$Q_b = Q - Q_i$ (Equation 3)

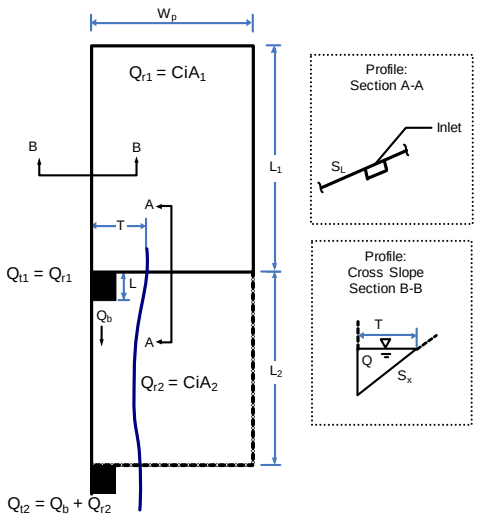
$Q_b = 0.57 – 0.3 = 0.27$ cubic feet per second

**Inlet Design Computation Sheet (Diagram A-A Process)**

**Given:**

| | |
|---|---|
| **Surface Description** | |
| Road Classification | Collector road, on grade, less than 45 miles per hour |
| Type of Curb and Gutter, see Chapter 8, Curbs and Gutters | Standard Curb |
| Surface Material | asphalt |
| Manning's Coefficient, n | 0.016 |
| **Drainage Area Description** | |
| Runoff Coefficient, C | 0.9 |
| Hydrologic Zone *(Chapter 7)* | 8 |
| Intensity (i) in/hr *(Chapter 7)* | 2.28 in/hr, 5-minute $T_c$, 10-year design storm |
| $C_f$ *(Chapter 7, Appendix F)* | 1 |
| Type of Inlet *(Chapter 13, Appendix C, Table B)* | slotted inlet |
| **Design Method:** **Method A** ☑ **Method B** ☐ | |
| **Design Criteria:** | |
| **Method A** (reference Section 2) | |
| Minimum Efficiency | 70 Percent |
| Allowable spread *(Appendix D, Table A)* | shoulder + 2 feet |
| shoulder width | 8.00 feet |
| distance into outside lane | 2.00 |
| spread (T) = Shoulder Width + Distance into outside lane | 10.00 feet |
| Percentage Clogged | 0% |
| **Method B** (reference Section 2) | |
| Allowable spread *(Appendix D, Table A)* | |
| shoulder width | |
| distance into outside lane | |
| spread (T) = Shoulder Width + Distance into outside lane | |
| Percentage Clogged | |

**Sketch:**



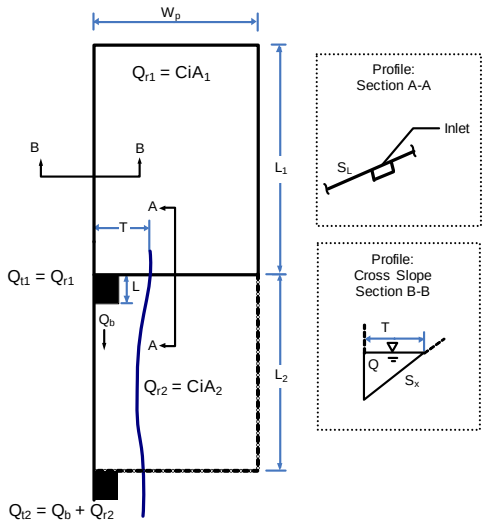

| Locate Inlet | | | Calculate Hydrology | | | Calculate Spread | | | | | | | | Calculate Inlet Efficiency | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Drainage Length | Drainage Width | Drainage Area | Flow from Contributing Area | Bypass Flow from Upstream Inlet | Total Flow | Spread | Long. Slope | Cross Slope | | Gutter Width | Total Gutter Flow | Design Inlet Length | Design Inlet Width | Inlet Interception Efficiency | Intercepted Flow | Bypass Flow | Number | Station | Comments |
| L | $W_p$ | A | $Q_r$ | $Q_b$ | $Q_t$ | T | $S_L$ | $S_x$ | $S_w$ | W | $Q_t$ | $L_i$ | $W_i$ | E | $Q_i$ | $Q_b$ | | | |
| (ft) | (ft) | (ac) | (cfs) | (cfs) | (cfs) | (ft) | (ft/ft) | (ft/ft) | (ft/ft) | (ft) | (cfs) | (ft) | (ft) | | (cfs) | (cfs) | | | |
| 400 | 31.00 | 0.28 | 0.57 | 0.00 | 0.57 | 5.90 | 0.0100 | 0.02 | N/A | N/A | 0.57 | 10.00 | 0.15 | 0.86 | 0.49 | 0.08 | 1 | 4+00 | Efficiency is greater than minimum and spread is less than allowable. |

**Figure 5a – Example Problem 5.3.2 (Diagram A-A, Method A)**

**Inlet Design Computation Sheet (Diagram A-A Process)**

**Given:** **Sketch:**

| | |
|---|---|
| **Surface Description** | |
| Road Classification | Collector road, on grade, less than 45 miles per hour |
| Type of Curb and Gutter, see Chapter 8, Curbs and Gutters | Standard Curb |
| Surface Material | asphalt |
| Manning's Coefficient, n | 0.016 |
| **Drainage Area Description** | |
| Runoff Coefficient, C | 0.9 |
| Hydrologic Zone *(Chapter 7)* | 8 |
| Intensity (i) in/hr *(Chapter 7)* | 2.28 in/hr, 5-minute $T_c$, 10-year design storm |
| $C_f$ *(Chapter 7, Appendix F)* | 1 |
| Type of Inlet *(Chapter 13, Appendix C, Table B)* | slotted inlet |
| **Design Method:** **Method A** ☐ **Method B** ☑ | |
| **Design Criteria:** | |
| **Method A** (reference Section 2) | |
| Minimum Efficiency | 70 Percent |
| Allowable spread *(Appendix D, Table A)* | |
| shoulder width | |
| distance into outside lane | |
| spread (T) = Shoulder Width + Distance into outside lane | |
| Percentage Clogged | |
| **Method B** (reference Section 2) | |
| Allowable spread *(Appendix D, Table A)* | shoulder + 2.00 feet ft into outside lane |
| shoulder width | 8.00 feet |
| distance into outside lane | 2.00 feet |
| spread (T) = Shoulder Width + Distance into outside lane | 10.00 feet |
| Percentage Clogged | 50% |




| Locate Inlet | | | Calculate Hydrology | | | Calculate Spread | | | | | | | | Calculate Inlet Efficiency | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Drainage Length | Drainage Width | Drainage Area | Flow from Contributing Area | Bypass Flow from Upstream Inlet | Total Flow | Spread | Long. Slope | Cross Slope | | Gutter Width | Total Gutter Flow | Design Inlet Length | Design Inlet Width | Inlet Interception Efficiency | Intercepted Flow | Bypass Flow | Number | Station | Comments |
| L | $W_p$ | A | $Q_r$ | $Q_b$ | $Q_t$ | T | $S_L$ | $S_x$ | $S_w$ | W | $Q_t$ | $L_i$ | $W_i$ | E | $Q_i$ | $Q_b$ | | | |
| (ft) | (ft) | (ac) | (cfs) | (cfs) | (cfs) | (ft) | (ft/ft) | (ft/ft) | (ft/ft) | (ft) | (cfs) | (ft) | (ft) | | (cfs) | (cfs) | | | |
| 400 | 31.00 | 0.28 | 0.57 | 0.00 | 0.57 | 5.90 | 0.01 | 0.02 | N/A | N/A | 0.57 | 5.00 | 0.15 | 0.52 | 0.30 | 0.27 | 1 | 4+00 | Efficiency is greater than minimum and spread is less than allowable. |

**Figure 5b – Example Problem 5.3.2 (Diagram A-A, Method B)**

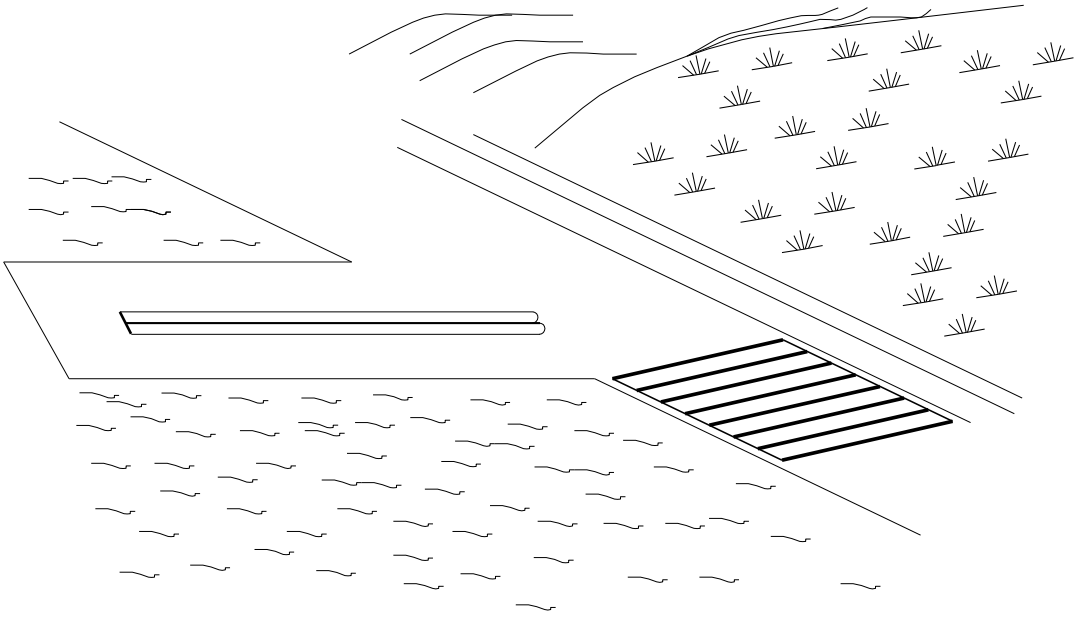
### 5.3.3 Transverse Placement

A slotted vane drain can be installed in conjunction with a grate inlet at locations where it is necessary to capture virtually all of the pavement runoff. The ideal installation would utilize a grate inlet to capture the flow in the gutter and the slotted vane drain to collect the flow extending into the shoulder.

Slotted vane drains should be designed by considering one of the following options:

- when a slotted vane drain is installed perpendicular to flow, use 0.075 cubic feet per second per lineal foot on longitudinal slopes of 0 percent to 6 percent, or
- obtain capacity curves from a manufacturer to aid in design

*Note: A slotted vane drain is shaped and rounded to increase inlet efficiency and should not be confused with a standard vertical riser type slotted inlet (ODOT standard drawing No. RD 328).*



**Slotted Vane Drain (traverse placement)**

## 5.4 Combination Inlets (On-grade)

Combination inlets consist of a grate and curb-opening. The two combination inlets commonly used in storm drainage design are equal length inlets and sweeper inlets.

Equal length inlets refer to a grate inlet placed along side a curb-opening inlet and both have the same length. Standard ODOT inlets CG-1 and CG-2 are equal length inlets as noted in

Appendix C. On-grade equal length combination inlets interception capacity is no greater than that of an on grate inlet because the capacity is computed by neglecting the curb-opening.

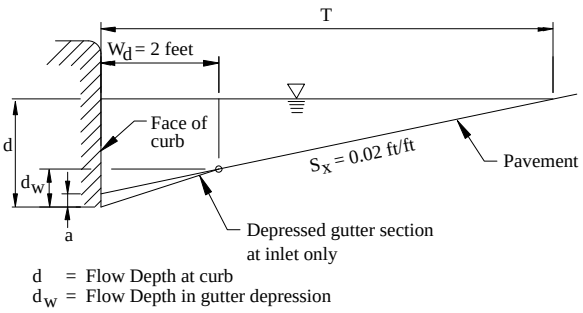
A sweeper inlet refers to a grate inlet placed at the downstream end of a curb-opening extension. Standard ODOT inlets CG-1 and CG-2 can be modified to include a curb-opening extension as noted in Appendix C. The curb-opening in a sweeper inlet is longer than the grate and intercepts gutter flow before the flow reaches the grate. The curb-opening extension has the ability to intercept any debris which might otherwise clog the grate inlet. The interception capacity of sweeper inlets is equal to the sum of the curb-opening upstream of the grate plus the grate capacity.

### 5.4.1 Sweeper Combination Inlet (On-grade) Example

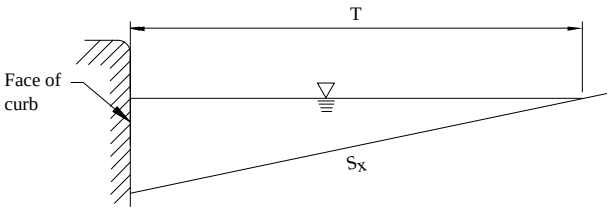
The following example illustrates the computation of interception capacity and spacing of a sweeper combination inlet.

#### 5.4.1.1 Determine maximum allowable flow, interception flow, bypass flow, location of first inlet, and spacing of subsequent inlets

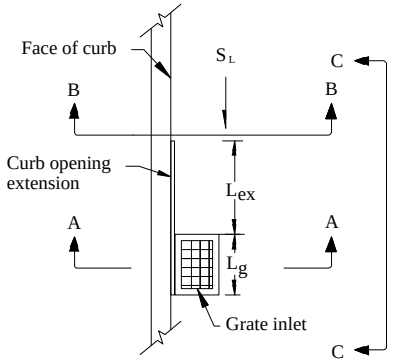
Sketch:



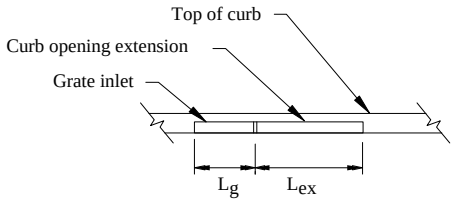

SECTION A-A
NTS




SECTION B-B
NTS




SWEEPER COMBINATION INLET
NTS




SECTION C-C
NTS

Given:

- $S_x$ = 0.02 foot per foot (roadway cross slope)
- a = 1.5 inches (inlet depression only)
- CG-1 inlet with P-$1^7/_8$-4 grate (reference Table B, Appendix C)
- $W_g$ = 1.75 feet (CG-1 grate width)
- $L_g$ = 2.67 feet (CG-1 grate length)
- $L_{ex}$ = 6 feet (curb opening extension length, reference Table B, Appendix C)
- $h_{ex}$ = 0.33 feet (curb-opening height for curb opening extension)
- $S_L$ = 0.0125 foot per foot (roadway longitudinal slope)
- Highway on-grade section (less than 45 miles per hour)
- n = 0.016 (Manning's coefficient for asphalt pavement)
- Standard Curb (reference **Chapter 8**)
- $W_p$ = 46.5 feet (width of contributing area, 2 – 12-foot lanes with 10.5-foot shoulder/gutter, 5-foot sidewalk, 7-foot median)
- $W_d$ = 2 feet (width of the depressed gutter section, at inlet only)
- 10-year design, hydrologic zone 8 (IDR curves located in **Chapter 7**)

### 5.4.1.2 Method A

Criteria:

- T = allowable spread = shoulder width + 2 feet
  = 10.5 feet + 2 feet
  = 12.5 feet (reference Table A, Appendix D)
- E = 0.70 (minimum efficiency for Method A)
- $L_2$ = 400 feet (maximum distance between successive inlets. Reference Section 2, Appendix D)

Solution:

**Step A.1-** Solve for ($Q_t$):

$Q_t$ = total gutter flow (cubic feet per second) - the approach gutter section to the inlet is used to calculate the total flow.

T = 12.5 feet (flow width in shoulder)

$$Q_t = \frac{0.56}{n} S_x^{1.67} S_L^{0.5} T^{2.67}$$ (**Chapter 8** or Chart 5, Appendix H)

$$Q_t = \frac{0.56}{0.016} (0.02)^{1.67} (0.0125)^{0.5} (12.5)^{2.67} = 4.82 \text{ cubic feet per second}$$

**Step A.2-** Solve for ($E_o$):

$E_o$ = ratio of flow in the depressed section to total gutter flow

$$E_o = 1-\left(1-\frac{W}{T}\right)^{2.67} = 1-\left(1-\frac{2}{12.5}\right)^{2.67} = 0.372 \qquad \left[\text{Equation 4 or Chart 2, Appendix H}\right]$$

**Step A.3-** Solve for ($S_e$):

$S_e$ = equivalent cross-slope (foot per foot)

$$S'_w = \frac{a}{W_d}$$

$a$ = 1.5 inches (gutter depression per standard drawing No. RD 366)

$W_d$ = 2 feet (width of the depressed gutter section)

$$S'_w = \frac{\left(\frac{1.5\,\text{in}}{12\,\text{in/ft}}\right)}{2\,\text{ft}} = 0.0625 \text{ foot per foot}$$

$S_e = S_x + S'_W E_o$ (Equation 12)

$S_e = 0.02 + (0.0625)(0.372) = 0.0432$

**Step A.4-** Solve for ($L_T$):

$L_T$ = curb-opening length required to intercept 100 percent of gutter flow

$$L_T = 0.6Q_t^{0.42}S_L^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} \qquad (S_e \text{ substituted for } S_x \text{ in Equation 10})$$

$$L_T = 0.6(4.82)^{0.42}(0.0125)^{0.3}\left(\frac{1}{(0.016)(0.0432)}\right)^{0.6} = 24.5 \text{ feet}$$

**Step A.5-** Solve for (E):

E = efficiency of curb opening extension (curb opening adjacent to grate is ignored throughout this example)

$$E = 1-\left(1-\frac{L_{ex}}{L_T}\right)^{1.8} \qquad \text{(Equation 11 or Chart 7, Appendix H)}$$

$L_{ex}$ = length of curb-opening extension (reference standard drawing No. RD 366)

$L_{ex}$ = 6 feet

$$E = 1-\left(1-\frac{6.0}{24.5}\right)^{1.8} = 0.396 = 39.6 \text{ percent}$$

**Step A.6-** Solve for ($Q_{iex}$)

$Q_{iex}$ = flow intercepted by curb-opening extension

$Q_{iex} = EQ_t$

$= (0.396)(4.82) = 1.91$ cubic feet per second

**Step A.7-** Solve for ($Q_b$):

$Q_b$ = bypass flow from curb-opening extension and total flow toward grate inlet

$Q_b = Q_t - Q_{iex}$

$= 4.82 - 1.91 = 2.91$cubic feet per second

**Step A.8-** Solve for (T):

T = total spread at grate (after interception by curb-opening extension)

This is done by trial and error. Start by assuming an initial value of T = 10.35 feet.

$T_s = T - W = 10.35 - 2.0 = 8.35$ feet

$$Q_s = \frac{0.56}{n} S_x^{1.67} S_L^{0.5} T_s^{2.67}$$ (**Chapter 8** or Chart 5, Appendix H)

$$Q_s = \frac{0.56}{0.016}(0.02)^{1.67}(0.0125)^{0.5}(8.35)^{2.67} = 1.65 \text{ cubic feet per second}$$

$$E_o = 1-\left(1-\frac{W}{T}\right)^{2.67} = 1-\left(1-\frac{2}{10.35}\right)^{2.67} = 0.436$$ [Equation 4 or Chart 2, Appendix H]

$$Q_t = \frac{Q_s}{1-E_o} = \frac{1.65}{1-0.436} = 2.93 \text{ cubic feet per second}$$

Bypass flow from curb opening extension is 2.91 cubic feet per second. Total flow toward catch basin grate at a spread of 10.35 feet is 2.93 cubic feet per second. Therefore, the spread at the catch basin grate is 10.35 feet.

**Step A.9-** Solve for (V):

V = average gutter flow velocity at grate (feet per second)

$$V = \frac{Q_t}{A} = \frac{2Q_t}{T^2 S_x}$$ (**Chapter 8**)

$$V = \frac{2(2.91)}{(10.35)^2(0.02)} = 2.72 \text{ feet per second}$$

**Step A.10-** Solve for ($R_f$)

$R_f$ = ratio of frontal flow intercepted to total frontal flow

$R_f = 1 - 0.09 (V - V_o)$ (Equation 6 or Chart 1, Appendix H)

$V_o$ = gutter velocity where splash over first occurs (in feet per second)

CG-1 grate length = 2.67 feet

Grate type = P-1-$^7/_8$-4

From Chart 1 (Appendix H) $V_o$ = 5.9 feet per second

$R_f = 1 - 0.09 (2.72 - 5.9) = 1.29$

$R_f$ can not exceed 1.0, therefore

$R_f = 1.0$

**Step A.11-** Solve for ($R_s$)

$R_s$ = ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1+\left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)}$$ (Equation 7 or Chart 4, Appendix H)

$$R_s = \frac{1}{1+\left(\frac{0.15\,(2.72)^{1.8}}{0.02(2.67)^{2.3}}\right)} = 0.17$$

**Step A.12-** Solve for ($E_g$):

$E_g$ = interception efficiency of the inlet grate

$E_g = R_f E_o + R_s (1 - E_o)$ (Equation 8)

$E_g = (1.0)(0.436) + 0.\ 17(1 - 0.436) = 0.53 = 53$ percent

**Step A.13-** Solve for ($Q_{ig}$):

$Q_{ig}$ = flow intercepted by grate inlet

$Q_t$ = 2.91 cubic feet per second (bypass flow from curb opening extension and is total flow toward grate inlet)

$Q_{ig} = E_g Q_t = 0.53(2.91) = 1.54$ cubic feet per second

**Step A.14-** Solve for ($Q_i$):

$Q_i$ = total interception flow (both grate inlet and curb-opening extension)

$Q_i$ = interception flow of curb-opening extension ($Q_{iex}$) + interception flow of grate inlet ($Q_{ig}$)

$Q_i = Q_{iex} + Q_{ig}$

$= 1.91 + 1.54 = 3.45$ cubic feet per second

**Step A.15-** Solve for (E):

E = combination inlet efficiency

$$E = \frac{Q_i}{Q_t} = \frac{3.45}{4.82} = 0.72 = 72 \text{ percent}$$

Okay, 70 percent minimum efficiency criteria is satisfied.

**Step A.16-** Solve for ($L_1$):

$L_1$ = distance to first inlet

$$L_1 = \frac{43{,}560Q_t}{CC_f iW_p}E \qquad \text{(Equation 1)}$$

E = 1.0 (no bypass flow to first inlet)

i = 2.28 inches per hour (assume 5-minute $t_c$ using a 10-year storm design, Hydrologic Zone 8, using IDR curve in **Chapter 7**)

$C_f$ = 1.0 (per Appendix F, Chapter 7)

$$L_1 = \frac{43{,}560(4.82)(1.0)}{0.9(1.0)(2.28)(46.5)} = 2{,}200 \text{ feet}$$

**Step A.17-** Calculate (L):

L = distance to successive inlets

$$L = \frac{43{,}560Q_tE}{CC_f iW_p} \qquad \text{(Equation 1)}$$

E = 0.72 (distance to subsequent inlets should account for bypass flow)

$$L = \frac{43{,}560(4.82)(0.72)}{0.9(1.0)(2.28)(46.5)} = 1{,}584 \text{ feet}$$

Maximum inlet spacing shall be limited to 400 feet.
Therefore, $L_1$ = 2200 feet and $L_2$ = 400 feet

### 5.4.1.3 Method B

- T = allowable spread width = shoulder width + 2 feet
  = 10.5 feet + 2 feet
  = 12.5 feet (per Table A for highway on-grade section, less than 45 miles per hour)
- Assume inlet is 30 percent clogged (per Table A)

- $W_g$ = 1.39 feet (grate width for CG-1 inlet 30 percent clogged per Table B, Appendix D.)
- $L_g$ = 2.31 feet (grate length for CG-1 inlet 30 percent clogged per Table B, Appendix D)
- $L_2$ = 400 feet (maximum spacing between successive inlets per Appendix D)
- $L_{ex}$ = 4.20 feet (curb opening extension length, 30 percent clogged, Table B, Appendix D)
- $h_{ex}$ = 0.33 feet (curb opening height for curb opening extension 30 percent clogged, Table B, Appendix D)

**Step B.1-** Solve for ($Q_t$):

$Q_t$ = total gutter flow (The approach gutter section to the inlet is used to calculate the total flow.)

$$Q_t = \frac{0.56}{n} S_x^{1.67} S_L^{0.5} T^{2.67}$$ (**Chapter 8** or Chart 5, Appendix H)

$$Q_t = \frac{0.56}{0.016}(0.02)^{1.67}(0.0125)^{0.5}(12.5)^{2.67} = 4.82 \text{ cubic feet per second}$$

**Step B.2-** Solve for ($E_o$):

$E_o$ = ratio of flow in the depressed section to total gutter flow

$$E_o = 1-\left(1-\frac{W}{T}\right)^{2.67} = 1-\left(1-\frac{2}{12.5}\right)^{2.67} = 0.372$$ [Equation 4 or Chart 2, Appendix H]

**Step B.3-** Solve for ($S_e$):

$S_e$ = equivalent cross-slope (foot per foot)

$$S'_w = \frac{a}{W_d}$$

a = 1.5 inches (gutter depression per standard drawing RD 366)

$W_d$ = 2 feet (width of the depressed gutter section)

$$S'_w = \frac{\left(\frac{1.5\text{ in}}{12\text{ in/ft}}\right)}{2\text{ ft}} = 0.0625 \text{ foot per foot}$$

$S_e = S_x + S'_W E_o$ (Equation 12)

$S_e = 0.02 + (0.0625)(0.372) = 0.0432$

**Step B.4-** Solve for ($L_T$):

$L_T$ = curb-opening length required to intercept 100 percent of gutter flow

$$L_T = 0.6Q_t^{0.42}S_L^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} \quad (S_e \text{ substituted for } S_x \text{ in Equation 10})$$

$$L_T = 0.6(4.82)^{0.42}(0.0125)^{0.3}\left(\frac{1}{(0.016)(0.0432)}\right)^{0.6} = 24.5 \text{ feet}$$

**Step B.5-** Solve for (E):

E = efficiency of curb opening extension (curb opening adjacent to grate is ignored throughout this example)

$$E = 1-\left(1-\frac{L_{ex}}{L_T}\right)^{1.8} \quad \text{(Equation 11 or Chart 7, Appendix H)}$$

$L_{ex}$ = length of curb-opening extension (reference standard drawing No. RD 366)

$L_{ex}$ = 4.20 feet (curb opening extension length, 30 percent clogged, Table B, Appendix D)

$$E = 1-\left(1-\frac{4.2}{24.5}\right)^{1.8} = 0.29 = 29 \text{ percent}$$

**Step B.6-** Solve for ($Q_{iex}$):

$Q_{iex}$ = flow intercepted by curb-opening extension

$Q_{iex} = EQ_t = (0.29)(4.82) = 1.40$ cubic feet per second

**Step B.7-** Solve for ($Q_b$):

$Q_b$ = bypass flow from curb-opening extension and is total flow toward grate inlet

$Q_b = Q_t - Q_{iex}$

$= 4.82 - 1.40 = 3.42$ cubic feet per second

**Step B.8-** Solve for (T):

T = spread at grate (after interception by curb-opening extension)

This is done by trial and error. Start by assuming an initial value of T = 11.0 feet

$T_s = T - W$

$= 11.0 - 2.0 = 9.0$ feet

$Q_s = \frac{0.56}{n} S_x^{1.67} S^{0.5} T_s^{2.67}$ (**Chapter 8** or Chart 5, Appendix H)

$Q_s = \frac{0.56}{0.016} (0.02)^{1.67} (0.0125)^{0.5} (9.0)^{2.67} = 2.01$ cubic feet per second

$E_o = 1 - \left(1 - \frac{W}{T}\right)^{2.67} = 1 - \left(1 - \frac{2}{11.0}\right)^{2.67} = 0.415$ [Equation 4 or Chart 2, Appendix H]

$Q_t = \frac{Q_s}{(1 - E_o)} = \frac{2.01}{1 - 0.415} = 3.44$ cubic feet per second

Bypass flow from the curb opening extension is 3.42 cubic feet per second. Total flow toward the catch basin grate at a spread of 11.0 feet is 3.44 cubic feet per second. Therefore the spread at the catch basin grate is 11.0 feet.

**Step B.9-** Solve for (V):

V = average gutter flow velocity at grate in feet per second

$V = \frac{Q_t}{A} = \frac{2Q_t}{T_s^2 S_x}$ (**Chapter 8**)

$V = \frac{2\,(3.44)}{(11)^2\,(0.02)} = 2.84$ feet per second

**Step B.10-** Solve for ($R_f$):

$R_f$ = ratio of frontal flow intercepted to total frontal flow

$R_f = 1 - 0.09\,(V - V_o)$ (Equation 6 or Chart 1, Appendix H)

$V_o$ = gutter velocity where splash over first occurs (in feet per second)

$L_g$ = 2.31 feet (CG-1 grate length, 30 percent clogged per Table B, Appendix D)

Grate type = P-1-$^7/_8$-4

Therefore from Chart 1 (Appendix H), $V_o = 5.0$ feet per second

$R_f = 1 - 0.09\,(2.84 - 5.0) = 1.19$

$R_f$ = cannot exceed 1.0, therefore

$R_f = 1.0$

**Step B.11-** Solve for ($R_s$):

$R_s$ = ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1+\left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)}$$ (Equation 7 or Chart 4, Appendix H)

$$R_s = \frac{1}{1+\left(\frac{0.15\,(2.84)^{1.8}}{(0.02)(2.31)^{2.3}}\right)} = 0.12$$

**Step B.12-** Solve for ($E_g$):

$E_g$ = interception efficiency of the inlet grate

$E_g = R_f E_o + R_s\,(1 - E_o)$ (Equation 8)

$E_g = (1.0)(0.415) + 0.12\,(1 - 0.415) = 0.49 = 49$ percent

**Step B.13-** Solve for ($Q_{ig}$):

$Q_{ig}$ = interception flow of grate inlet

$Q_{ig} = E_g Q_t = 0.49(3.42) = 1.68$ cubic feet per second

**Step B.14-** Solve for ($Q_i$):

$Q_i$ = total interception flow (both grate inlet and curb-opening extension)

$Q_i$ = interception flow of curb-opening extension ($Q_{iex}$) + interception flow of grate inlet ($Q_{ig}$)

$Q_i = Q_{iex} + Q_{ig}$

$Q_i = 1.40 + 1.68 = 3.08$ cubic feet per second

**Step B.15-** Solve for (E):

E = combination inlet efficiency

$E = Q_i/Q_t$

$= 3.08/4.82 = 0.64 = 64$ percent

Okay (70 percent minimum interception efficiency does not apply to Method B.)

**Step B.16-** Calculate ($L_1$):

$L_1$ = distance to first inlet

$$L_1 = \frac{43{,}560 Q_t E}{C C_f i W_p}$$ (Equation 1)

E = 1.0 (no bypass flow for first inlet)

$i$ = 2.28 inches per hour (assume 5-minute $t_c$ using a 10-year storm design, Hydrologic Zone 8, using IDR curve in **Chapter 7**)

$C_f$ = 1.0 (per Chapter 7, Appendix F)

$$L_1 = \frac{43{,}560(4.82)}{0.9(1.0)(2.28)(46.5)}(1.0) = 2{,}200 \text{ feet}$$

**Step B.17-** Calculate (L):

$L$ = distance to successive inlets

$$L = \frac{43{,}560Q_t}{CC_f iW_p}E \qquad \text{(Equation 1)}$$

$E$ = 0.64 (distance to subsequent inlets should account for bypass flow)

$$L = \frac{43{,}560(4.82)}{0.9(1.0)(2.28)(46.5)}(0.64) = 1{,}408 \text{ feet}$$

Maximum inlet spacing shall be limited to 400 feet

Therefore $L_1$ = 2,200 feet and $L_2$ = 400 feet

## 5.5 Trench Drain System (On-grade)

Trench drain systems with its continuous grate configuration are best suited for intercepting flow on very flat surfaces with little or no longitudinal grade. They can be used on curbed or uncurbed sections and offer little interference to traffic operations. The primary advantage of using trench drains is their ability to intercept flow over a wide section. However, trench drains are susceptible to clogging from sediments and debris and are not recommended for use in environments where significant sediment or debris loads may be present.

*Note: General information on trench drain systems is noted in Appendix C.*

To aid in the design of trench drains, the designer should consider the following guidelines:

- The equations and figures used for the design of grate inlets (reference Section 5.1) should be used in the design of trench drains.
- Consider hydraulic conveyance capacity curves or data from a manufacturer.
- Consider hydraulic capacity curves or data for available grates from a manufacturer.
- Trench drain grate shall be bicycle safe and rated for heavy highway traffic.
- When using design Method B, the designer should assume 50 percent clogging which will result in providing twice the calculated required length for flow interception because trench drains tend to plug.
- The minimum cleanout velocity for trench drain systems should be 3 feet per second.
- Trench drains should not be used in areas with high traffic speeds greater than or equal to 45 miles per hour.

## 6.0 Inlets in Sag Location

Inlets in sag locations operate as weirs under low head conditions. When greater head conditions develop they function as an orifice. Flow may fluctuate between weir and orifice control depending on the grate size, the curb-opening height, or the slot width of the inlet. Flow is in a transition stage at depths between when weir flow definitely prevails and when orifice flow prevails. Control is ill-defined and flow may fluctuate between weir and orifice control at these depths.

The efficiency of inlets in sag locations is critical because all runoff which enters the sag must be passed through the inlet. Total or partial clogging of inlets in these locations can result in hazardous ponding conditions. Grate inlets alone are not recommended for use in sag locations because of the tendencies of grates to become clogged. Combination inlets or curb-opening inlets are recommended for use in these locations because of its better hydraulic capacity and debris handling capabilities.

Grate inlets can be used successfully in sag locations although curb-opening inlets are generally preferred. Grate inlets without a curb box (combination inlet) can be utilized at minor sag points where debris potential is limited. For example a minor sag point might be on a ramp as it joins a mainline. In sag locations where debris is likely, and a grate will be used, it is recommended to install a combination inlet (curb-opening and grate). When designing grates in sag locations, it is recommended to assume half the grate is clogged with debris (as noted in Table A).

It is good engineering practice to place a minimum of one flanking inlet on each side of the sag inlet in locations such as underpasses and in sag vertical curves in depressed sections where significant ponding can occur. The flanking inlets should be placed so they will limit spread on low gradient approaches to the low point and act in relief of the sag inlet if it should become clogged or if the allowable spread is exceeded. Further discussion and methodology of flanking inlets are presented in Section 6.6.

*Note:*

- *If ponding depths exceed the curb height the designer should verify adjacent properties will suffer no flood damage.*
- *The maximum spread width in the sag shall be limited as noted in Table A.*
- *The design discharge shall be the 25-year or 50-year bypass flow (reference Table A) from the first upstream inlet on each side of the sag plus the runoff from any additional areas draining to the sag.*

### 6.1 Grate Inlets (Sag)

A grate inlet in a sag operates as a weir up to a depth of about 4.5 inches and as an orifice for depths greater than 17 inches. A transition from weir to orifice flow occurs between these depths. Chart 8 (Appendix H) is a plot of Equation 13 and Equation 14 for various grate sizes. The effect of grate size on the depth at which a grate operates as an orifice is apparent from the

figure. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equations. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

The capacity of a grate inlet operating as a weir is:

$$Q_i = C_w P d^{1.5} \quad \text{(Equation 13)}$$

Where:

$Q_i$ = grate inlet capacity (cubic feet per second)
P = perimeter of grate excluding bar widths and side against curb in feet
d = depth of water at curb measured from the normal cross slope gutter flow line in feet
$C_w$ = 3.0

The capacity of a grate inlet operating as an orifice is:

$$Q_i = CA\,(2gd)^{0.5} \quad \text{(Equation 14)}$$

Where:

$Q_i$ = grate inlet capacity (cubic feet per second)
C = 0.67 (orifice coefficient)
A = clear opening area of the grate in square feet
g = 32.2 feet per second squared

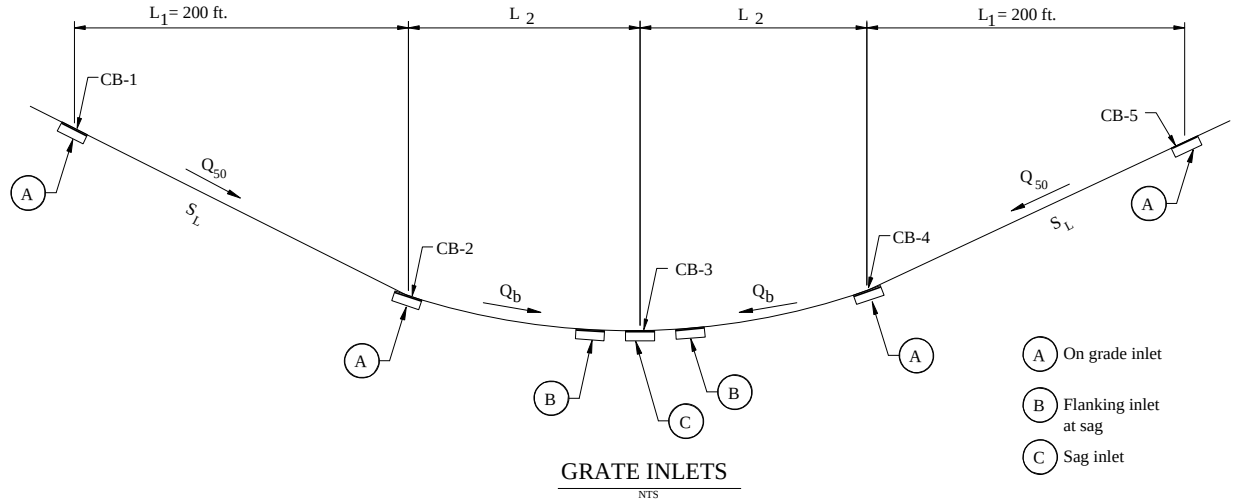
*Note: Standard grate inlets used in ODOT storm drainage systems are noted in Appendix C.*

### 6.1.1 Grate Inlet (Sag) Example

The following example illustrates the design of grate inlets in a sag.

#### 6.1.1.1 Determine the appropriate inlet spacing in a sag

Sketch:




GRATE INLETS
NTS

Given:

- Inlet design Method B (Appendix D)
  *Note: Method A or B could be used. Method selected should be used for entire design.*
- Highway section, main line sag point (greater than or equal to 45 miles per hour)
- Standard Curb (reference **Chapter 8**)
- n = 0.016 (Manning's coefficient for asphalt pavement)
- $S_L$ = 0.035 foot per foot (roadway longitudinal slope)
- $W_P$ = 46.5 feet (width of contributing drainage area, 2 – 12-foot lanes with 10.5-foot shoulder/gutter, 5-foot sidewalk, 7-foot median)
- $S_x$ = 0.02 foot per foot (shoulder and roadway cross slope)
- Flanking inlets ignored in calculations
- 50-year design, Hydrologic Zone 8 (IDR curves located in **Chapter 7**)
- G-2 inlets
- For on grade inlets(CB-1, CB-2, CB-4, and CB-5) assume 30 percent clogging
  - $W_G$ = 1.84 feet (G-2 grate width, 30 percent clogged, Table B, Appendix D)
  - $L_G$ = 2.26 feet (G-2 grate length, 30 percent clogged, Table B, Appendix D)
  - P = 5.94 feet (G-2 grate perimeter next to curb, 30 percent clogged, Table B, Appendix D)
- For low point (sag) inlets (CB-3) assume 50 percent clogging
  - $W_G$ = 1.53 feet (G-2 grate width, 50 percent clogged, Table C, Appendix D)
  - $L_G$ = 1.95 feet (G-2 grate length, 50 percent clogged, Table C, Appendix D)
  - P = 5.0 feet (G-2 grate perimeter next to curb, 50 percent clogged, Table C, Appendix D)
- $L_1$ = 200 feet
- T = shoulder width = 10.5 feet

Criteria:

- Maximum pond width = allowable spread width
- Allowable spread width is 0 feet into the outside lane (reference Table A, Appendix D)
- For on-grade inlets assume a 30 percent clogging factor because Method B has been selected to solve the example problem.
- For low point inlet in a main line sag, assume a 50 percent clogging factor because Method B has been selected to solve the example problem

Solution:

**Step B.1-** Solve for ($d_1$):

$d_1$ = low point flow depth at curb (at allowable spread width)

$d_1 = TS_x$ (**Chapter 8**)

$d_1 = (10.5)0.02 = 0.21$ feet

$d_1 = 0.21$ feet = 2.52 inches

From Section 6.1, d equals 2.52 inches which is less than 4.5 inches; therefore sag inlet operates as a weir.

**Step B.2-** Solve for ($Q_i$):

$Q_i$ = low point grate inlet (CB-3) capacity at maximum allowable spread

$Q_i = C_w P d_1^{1.5}$ (Equation 13)

$d_1$ = 0.21 feet

P = 5.0 feet (G-2 grate perimeter next to curb, 50 percent clogged, Table B, Appendix D)

$Q_i = 3.0(5.0)(0.21)^{1.5} = 1.44$ cubic feet per second

**StepB.3-** Solve for ($Q_{50}$):

$Q_{50}$ = 50-year peak flow (cubic feet per second) draining toward catch basins CB-2 and CB-4

$Q_{50}$ = CiA

C = 0.9 (runoff coefficient for pavement)

i = 3.0 inches per hour (50-year rainfall intensity, IDR curves located in **Chapter 7**)

A = $W_P$ L

= (46.5 feet)(200 feet) = 9,300 square feet

= 0.21 acre

$Q_{50}$ = (0.9)(3.0)(0.21) = 0.57 cubic feet per second

Therefore, 0.57 cubic feet per second drains toward catch basins CB-2 and CB-4.

**Step B.4-** Solve for (T):

T = total spread of water in feet just upstream of inlets CB-2 and CB-4

$Q = \frac{0.56}{n} S_X^{1.67} S_L^{0.5} T^{2.67}$ (**Chapter 8** or Chart 5, Appendix H)

Use Chart 5 (Appendix H) or rearrange the above equation to solve for T.

$$T = \left[\frac{Qn}{0.56 S_L^{0.5} S_x^{1.67}}\right]^{0.375} = \left[\frac{(0.57)(0.016)}{0.56(0.035)^{0.5}(0.02)^{1.67}}\right]^{0.375}$$

T = 4.6 feet (spread just upstream of inlets CB-2 and CB-4)

Okay because this is less than the allowable spread of 10.5 feet.

**Step B.5-** Solve for (V):

V = average gutter flow velocity (in feet per second)

$$V = \frac{Q_t}{A} = \frac{2Q_t}{T^2 S_x} = \frac{2(0.57)}{(4.6)^2(0.02)} = 2.65 \text{ feet per second}$$ (**Chapter 8**)

**Step B.6-** Solve for ($E_o$):

$E_o$ = ratio of frontal flow to total flow

$$E_o = 1 - \left(1 - \frac{W_g}{T}\right)^{2.67}$$ (Equation 4 or Chart 2, Appendix H)

$W_g$ = 1.84 feet (G-2 grate width, 30 percent clogged, Table B, Appendix D)

$$E_o = 1 - \left(1 - \frac{1.84}{4.6}\right)^{2.67}$$

$E_o$ = 0. 74 = 74 percent

**Step B.7-** Solve for ($R_f$):

$R_f$ = ratio of frontal flow intercepted to total frontal flow

$R_f = 1 - 0.09\,(V - V_o)$ (Equation 6 or Chart 1, Appendix H)

$V$ = average flow velocity in the gutter (in feet per second)

$V_o$ = gutter velocity where splash over first occurs (in feet per second)

$V$ = 2.65 feet per second

$L_g$ = 2.26 feet (G-2 grate length, 30 percent clogged, Table B,)

$V_o$ = 5.1 feet per second (per Chart 1, Appendix H)

Grate type = P – $1^7/_8$ – 4

$R_f = 1 - 0.09\,(2.65 - 5.1)$

$R_f$ = 1.22

$R_f$ can not exceed 1.0, therefore,

$R_f$ = 1.0

**Step B.8-** Solve for ($R_s$):

$R_s$ = ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1 + \left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)}$$ (Equation 7 or Chart 4, Appendix H)

$V$ = 2.65 feet per second

$L$ = 2.26 feet (G-2 grate length, 30 percent clogged, Table B, Appendix D)

$$R_s = \frac{1}{1 + \left(\frac{0.15\,(2.65)^{1.8}}{(0.02)(2.26)^{2.3}}\right)}$$

$R_s$ = 0.131

**Step B.9-** Solve for (E):

$E$ = inlet interception efficiency

$E = R_f E_o + R_s (1 - E_o)$ (Equation 8)

$E = (1.0)(0.74) + 0.131\ (1 - 0.74) = 0.77$ Okay

*Note: When using Method A, E must be equal to or greater than 0.70.*

**Step B.10-** Solve for ($Q_b$):

$Q_b$ = bypass flow from adjacent catch basins

$Q_b = Q - Q_i$

$Q_i = EQ = (0.77)(0.57) = 0.44$

$Q_b = 0.57 - 0.44 = 0.13$ cubic feet per second

**Step B.11-** Calculate ($L_2$):

$L_2$ = distance between sag inlet CB-3 and CB-2, and distance between sag inlet CB-3 and CB-4

$$L_2 = \frac{43{,}560Q}{CC_f iW_p}E \quad \text{(Equation 1)}$$

Equation 1 is modified to calculate $L_2$ in sags, therefore

$$L_2 = \frac{43{,}560(Q_i - 2Q_b)}{iCC_f\,2W_p}$$

$i$ = 3.0 inches per hour (assume 5-minute $t_c$ using a 50-year storm design, Hydrologic Zone 8, use IDR curve in **Chapter 7**)

$C$ = 0.90

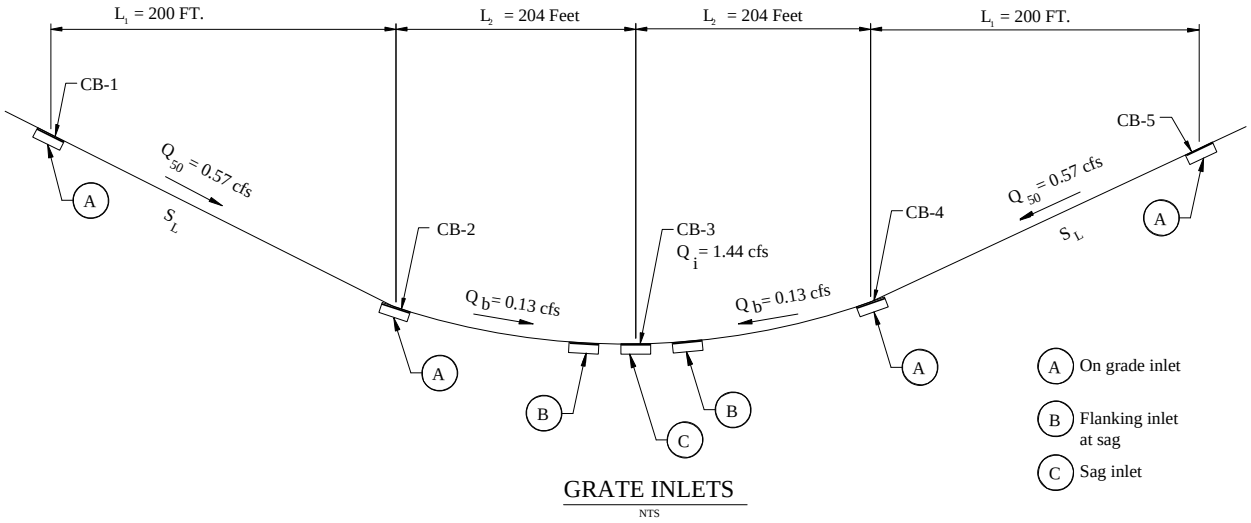
$C_f$ = 1.0 (per **Chapter 7**, Appendix F)

$W_p$ = 46.5 feet

$Q_i$ = 1.44 cubic feet per second (low point inlet (CB-3) interception capacity)

$$L_2 = \frac{43{,}560[1.44 - 2(0.13)]}{(0.9)(1.0)(3.0)(2)(46.5)} = 204 \text{ feet}$$

Maximum distance between inlets is 204 feet.




GRATE INLETS
NTS

## 6.2 Curb-Opening Inlets (Sag)

The capacity of a curb-opening inlet in a sag depends on water depth at the curb, the curb-opening length, and the height of the curb-opening (see the horizontal throat figure below). The inlet operates as a weir to depths equal to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

*Note: Standard curb-opening inlets used in ODOT storm drainage systems are noted in Appendix C*

The equation for the interception capacity of a depressed curb-opening inlet operating as a weir is:

$$Q_i = 2.3\,(L + 1.8\,W)d^{1.5} \qquad \text{(Equation 15)}$$

Where:

L = length of curb-opening in feet
W = width of depression in feet
d = depth of water at curb measured from the normal cross slope gutter flow line in feet

See Chart 9 (Appendix H) for a definition sketch.

The weir equation for curb-opening inlets without depression becomes:

$$Q_i = 3\,L\,d^{1.5} \qquad \text{(Equation 16)}$$

The depth limitation for operation as a weir becomes: d less than h (h = height of curb-opening inlet in feet)

Chart 10, Appendix H should be used for designing undepressed Curb-Opening Inlet Capacity in Sump Locations.

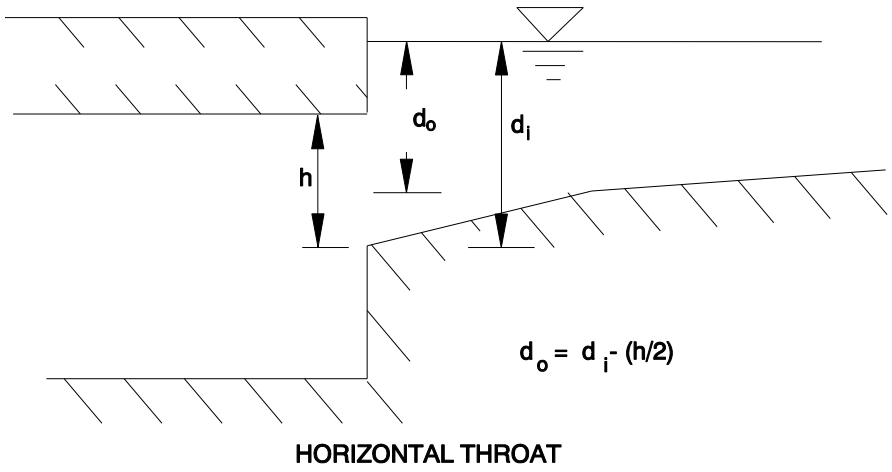
Curb-opening inlets operate as orifices at depths greater than approximately 1.4 times the opening height. The interception capacity can be computed by:

$$Q_i = C_o A \left[ 2g \left( d_i - \frac{h}{2} \right) \right]^{0.5} \quad \text{(Equation 17)}$$

Where:

$C_o$ = orifice coefficient (0.67)
h = height of curb-opening orifice in feet (figure below)
$d_i$ = depth at lip of curb-opening in feet (figure below)

*Note: Equation 17 is applicable to depressed and undepressed curb-opening inlets and the depth at the inlet includes any gutter depression.*



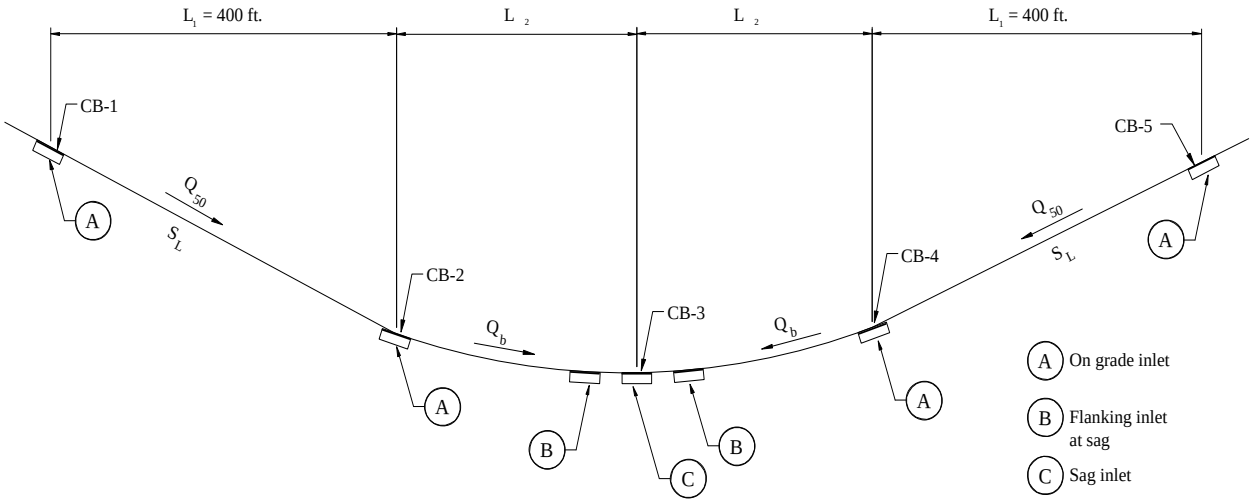

**Curb-Opening Inlet**

### 6.2.1 Curb-Opening Inlet (Sag) Example

The following example illustrates the design of curb-opening inlets in a sag.

#### 6.2.1.1 Determine inlet spacing in a sag location

Sketch:




CURB OPENINGS
NTS

Given:

- Inlet design method = B (Appendix D)
  *Note: Method A or B could be used. Method selected should be used for the entire design.*
- Standard Curb (reference Chapter 8)
- Highway-main line sag, less than 45 miles per hour (per Table A, Appendix D)
- $S_L$ = 0.0025 foot per foot (roadway longitudinal slope)
- n = 0.016 (Manning's coefficient for asphalt pavement)
- $W_p$ = 36 feet (width of contributing area, 2 – 12-foot lanes with 7-foot shoulder, 5-foot sidewalk)
- $S_x$ = 0.02 foot per foot (roadway cross slope)
- CG-3 inlets
- 50-year design (per Table A), Salem area (Hydrologic Zone 7)
- $L_c$ = 1.25 feet (length of curb opening, reference Table C, 50 percent clogged)
- $L_c$ = 1.75 feet ( length of curb opening, reference Table B, 30 percent clogged)
- a = 2 inches (inlet depression)
- $W_d$ = 2.0 feet (width of depressed gutter section at inlet only)
- $h_c$ = 0.36 feet (curb opening height, reference Table B and C)
- $L_1$ = 400 feet
- Flanking inlets ignored in calculations

Criteria:

- For on-grade inlets, assume a 30 percent clogging factor because Method B has been selected to solve the example problem.
- For low point inlet in a mainline sag use a 50 percent clogging factor because Method B has been selected to solve the example problem (see Tables A and C, Appendix D)
- T = shoulder width + 2 feet = 7 feet + 2 feet = 9 feet (allowable spread width, see Table A, Appendix D)

Solution: Solve inlet design using Method B

**Step B.1** Solve for ($d_1$):

$d_1$ = low point flow depth at curb (at allowable spread)

$d_1 = TS_x + a$ (**Chapter 8**)

$= 9(0.02) + 2/12 = 0.35$ feet $= 4.16$ inches

Because flow depth (0.35 feet) is less than curb opening height (0.36 feet) inlet CB-3 operates as a weir.

**Step B.2-** Solve for ($Q_i$):

$Q_i$ = interception flow of low point curb-opening inlet CB-3

$Q_i = 2.3(L+1.8W)d^{1.5}$ (Equation 15)

(Equation 15 is for depressed curb-opening inlets operating as a weir.)

L = 1.25 feet (length of curb opening, 50 percent clogged)

W = 2 feet (width of depression)

$Q_i = 2.3[1.25 + 1.8(2)](0.35)^{1.5} = 2.31$ cubic feet per second

(maximum allowable flow to low point inlet CB-3)

**Step B.3-** Solve for ($Q_{50}$):

$Q_{50}$ = 50-year peak flow (cubic feet per second) draining toward catch basins CB-2 and CB-4

$Q_{50} = CiA$ (**Chapter 7**)

C = 0.9 (runoff coefficient for pavement)

i = 2.8 inches per hour (50-year rainfall intensity, IDR curves located in **Chapter 7**)

A = WL

= (36 feet)(400 feet) 14,400 square feet

= 0.33 acre

$Q_{50}$ = (0.9)(2.8)(0.33) = 0.83 cubic feet per second

Therefore, 0.83 cubic feet per second drains toward catch basins CB-2 and CB-4.

**Step B.4-** Solve for (T):

T = total spread of water in feet just upstream of inlets CB-2 and CB-4

$$Q = \frac{0.56}{n} S_X^{1.67} S_L^{0.5} T^{2.67}$$ (**Chapter 8** or Chart 5, Appendix H)

Use Chart 5 (Appendix H) or rearrange the above equation to solve for T.

$$T = \left[\frac{Qn}{0.56 S_L^{0.5} S_x^{1.67}}\right]^{0.375} = \left[\frac{(0.83)(0.016)}{0.56(0.0025)^{0.5}(0.02)^{1.67}}\right]^{0.375}$$

T = 8.75 feet (spread just upstream of catch basins CB-2 and CB-4)

Okay (because this is less than the allowable spread of 9 feet)

**Step B.5-** Calculate ($E_o$):

$E_o$ = ratio of flow in the depressed section to total gutter flow

$$E_o = 1 - \left(1 - \frac{W_d}{T}\right)^{2.67}$$ (Equation 4 or Chart 2, Appendix H)

$$E_o = 1 - \left(1 - \frac{2}{8.75}\right)^{2.67}$$

= 0.50 = 50 percent

**Step B.6-** Calculate ($S_e$):

$S_e$ = equivalent cross slope (foot per foot)

$$S_e = S_x + S'_w E_o$$ (Equation 12)

$S_x$ = roadway Cross Slope

$S'_w$ = $\frac{a}{W}$ (cross slope of the gutter measured from the cross slope of the pavement, $S_x$)

$$= \frac{a}{W} = \frac{(2/12)}{2}$$ = 0.0833 foot per foot

$S_e$ = 0.02 + (0.083)(0.50) = 0.0615

**Step B.7-** Calculate ($L_T$):

$L_T$ = curb opening length required to intercept 100 percent of the flow

$$L_T = 0.6\, Q_t^{0.42}\, S_L^{0.3} \left(\frac{1}{n\, S_e}\right)^{0.6}$$ ($S_e$ is substituted for $S_x$ in Equation 10)

$$L_T = 0.6\,(0.83)^{0.42}\,(0.0025)^{0.3}\left(\frac{1}{(0.016)(0.0615)}\right)^{0.6}$$

$L_T$ = 5.86 feet

**Step B.8-** Solve for (E):

E = curb opening efficiency

$L_c$ = 1.75 feet (length of curb opening 30 percent clogged)

$$E = 1-\left[1-\frac{L_c}{L_T}\right]^{1.8} = 1-\left[1-\frac{1.75}{5.86}\right]^{1.8}$$ (Equation 11 or Chart 7, Appendix H)

E = 0.47

Okay (70 percent minimum efficiency criteria does not apply to Method B)

**Step B.9-** Solve for ($Q_b$):

$Q_b$ = flow that is not intercepted by an inlet and must be included in the evaluation of downstream gutter and inlet (cubic feet per second)

$Q_b = Q - Q_i$

$Q_i$ = EQ = (0.47)(0.83) = 0.39 cubic feet per second

$Q_b$ = 0.83 – 0.39 = 0.44 cubic feet per second

**Step B.10-** Solve for ($L_2$):

$L_2$ = distance between sag inlet CB-3 and CB-2, and

= distance between sag inlet CB-3 and CB-4

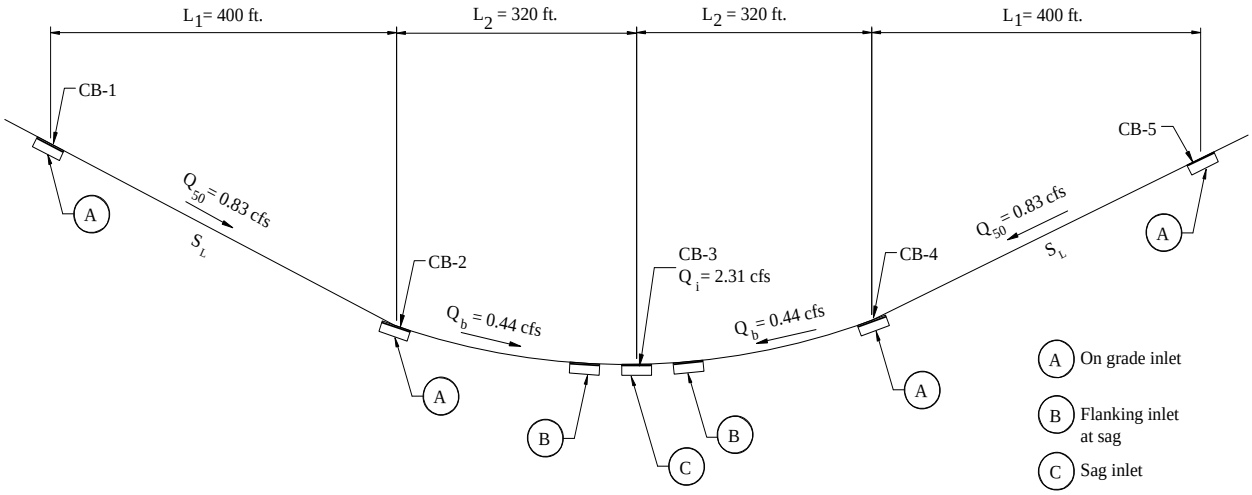
$$L_2 = \frac{43{,}560Q}{CC_f iW_p}E$$ (Equation 1)

Equation 1 is modified to calculate $L_2$ in sags, therefore

$$L_2 = \frac{43{,}560(Q_i - 2Q_b)}{iCC_f\,2W_p}$$

$Q_i$ = 2.31 cubic feet per second (low point inlet (CB-3) interception capacity)

$$L_2 = \frac{43{,}560[2.31 - 2(0.44)]}{(0.9)(1.0)(3.0)(2)(36.0)} = 320 \text{ feet}$$




CURB OPENINGS

NTS

## 6.3 Slotted Inlets (Sag)

The use of slotted drain inlets in sag configurations is generally discouraged because of the tendency of such inlets in sags to intercept debris and clog. However, there may be locations where it is desirable to supplement an existing low point inlet with the use of a slotted drain. It is not recommended to place slotted drain inlets in sags unless a tapered slot is provided.

*Note:*

- *Standard slotted inlets used in ODOT storm drain systems are noted in Appendix C.*
- *When using inlet design Method B, the designer should assume 50 percent clogging which will result in providing twice the calculated required length for flow interception because slotted inlets tend to plug.*
- *A tapered slot inlet has a sloped invert to provide positive drainage. This is necessary in applications when the finished grade slot elevation is constant.*

Slotted inlets in sag locations perform as weirs to depths of about 0.2 feet (2.4 inches). This condition is dependent on slot width and length. Slotted inlets perform as orifices at depths greater than about 0.4 feet (4.8 inches). Flow is in a transition stage between these depths.

The interception capacity of a slotted inlet operating as a weir can be computed by the following equation:

$$Q_i = 2.48\ L\ d^{1.5} \qquad \text{(Equation 18)}$$

Where:

L = length of slot in feet
d = depth of water at slot in feet

The interception capacity of a slotted inlet operating as an orifice can be computed by the following equation:

$$Q_i = 0.8\ L\ W\ (2gd)^{0.5} \qquad \text{(Equation 19)}$$

Where:

W = width of slot in feet
L = length of slot in feet
d = depth of water at slot in feet
g = 32.2 feet per second squared

For a slot width of 1.75 inches, Equation 19 simplifies to:

$$Q_i = 0.94\ L\ d^{0.5} \qquad \text{(Equation 20)}$$

The orifice equation noted in this section to compute the interception capacity of slotted inlets should be used at depths between 2.4 inches and 4.8 inches. The orifice coefficient varies with depth, slot width, and the length of the slotted inlet. Chart 11, Appendix H, provides solutions for weir flow and a plot representing data at depths between weir and orifice flow.

## 6.4 Combination Inlets (Sag)

Combination inlets consist of a grate and curb-opening. These types of inlets are recommended for use in sags because of their better hydraulic capacity and debris handling capabilities. The two combination inlets commonly used in storm drainage design are equal length inlets and sweeper inlets.

Equal length inlets refer to a grate inlet placed along side a curb-opening inlet and both have the same length. Standard ODOT inlets CG-1 and CG-2 are equal length inlets as noted in Appendix C. The interception capacity of these types of inlets is essentially equal to that of a grate only inlet in weir flow. The capacity of equal length combination inlets in orifice flow is equal to the capacity of the grate plus the capacity of the curb-opening.

A sweeper inlet refers to a grate inlet placed at the downstream end of a curb-opening extension. Standard ODOT inlets CG-1 and CG-2 can be modified to include a curb-opening extension as noted in Appendix C. The curb-opening in a sweeper inlet is longer than the grate and intercepts gutter flow before the flow reaches the grate. This type of inlet is more efficient than the equal length combination inlet and the curb-opening has the ability to intercept any debris which may clog the grate inlet.

Equation 13 and Chart 8, Appendix H, can be used for weir flow in combination inlets in sag locations. Assuming complete clogging of the grate, Equation 15, Equation 16, Equation 17 and Charts 9 and 10 (Appendix H) for curb-opening inlets are applicable.

Where depth at the curb is such that orifice flow occurs, the interception capacity of the inlet is computed by adding Equation 14 and Equation 17 as follows:

$$Q_i = 0.67\ A_g\ (2\ g\ d)^{0.5} + 0.67\ h\ L\ (2g\ d_o)^{0.5} \quad \text{(Equation 21)}$$

Where:

- $A_g$ = clear area of the grate in square feet
- g = 32.2 feet per second squared
- d = depth at the curb in feet
- h = height of curb-opening orifice in feet (reference the horizontal throat figure in Section 6.2)
- L = length of curb-opening in feet
- $d_o$ = effective depth at the center of the curb-opening orifice in feet (reference the horizontal throat figure in Section 6.2)

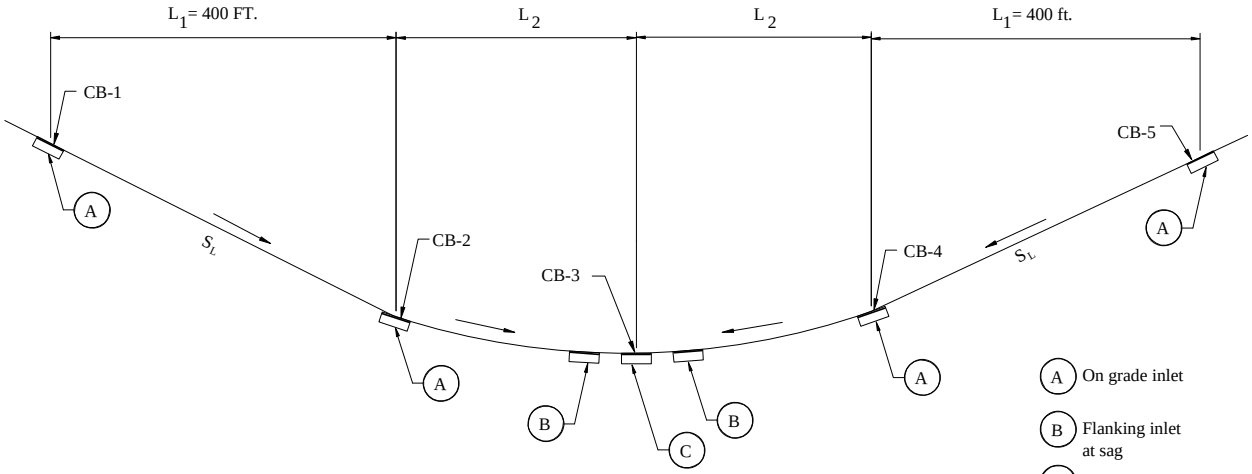
Trial and error solutions are necessary for determining the depth at the curb for a given flow rate using Chart 8, Chart 9, and Chart 10 (Appendix H) for orifice flow. Different assumptions for clogging of the grate can also be examined using these charts.

### 6.4.1 Sweeper Combination Inlet (Sag) Example

The following example illustrated the design of combination inlets in a sag.

#### 6.4.1.1 Determine the appropriate inlet spacing in a sag.

Sketch:




SWEEPER COMBINATION INLET
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Given:

- Inlet design method = B (Appendix D)
  *Note: Method A or B could be used. Method selected should be used for the entire design.*
- Highway main line sag (less than 45 miles per hour)

- Standard Curb (reference **Chapter 8**)
- $W_p$ = 75.5 feet (width of contributing drainage area, 4-12-foot lanes with 10.5-foot shoulder, 5-foot sidewalk, 12-foot median)
- W = 2 feet (width of depressed section, at inlet only)
- $S_L$ = 0.018 (roadway longitudinal slope)
- $S_x$ = 0.02 (shoulder and roadway cross slope)
- CG-1 inlets width P-1$^7/_8$-4 grate (reference Table B, Appendix C)
- $L_{ex}$ = 4.20 feet (curb opening extension 30 percent clogged, Table B, Appendix D)
- $L_{ex}$ = 3.0 feet (curb opening extension 50 percent clogged, Table C, Appendix D)
- $h_{ex}$ = 0.33 feet (curb opening height for curb opening extension)
- a = 1.5 inches (inlet depression)
- n = 0.016 (Manning's coefficient for asphalt pavement)
- $L_1$ = 400 feet
- Flanking inlets ignored in calculations
- 50-year design, Hydrologic Zone 8 (IDR curves in **Chapter 7**)
- $W_g$ = 1.39 feet (CG-1 grate width, 30 percent clogged, per Table B, Appendix D)
- $L_g$ = 2.31 feet (CG-1 grate length, 30 percent clogged, per Table B, Appendix D)
- $P_g$ = 5.10 feet (CG-1 grate perimeter, 30 percent clogged, per Table B, Appendix D)
- $W_g$ = 1.13 feet (CG-1 grate width, 50 percent clogged, per Table C, Appendix D)
- $L_g$ = 2.05 feet (CG-1 grate length, 50 percent clogged, per Table C, Appendix D)
- $P_g$ = 4.31 feet (CG-1 grate perimeter, 50 percent clogged, per Table C, Appendix D)

Criteria:

- For on-grade inlets, assume a 30 percent clogging factor because Method B has been selected to solve the example problem.
- For low point inlet in a main line sag, assume a 50 percent clogging factor because Method B has been selected.
- T = shoulder width + 2 feet
  = 10.5 feet + 2 feet
  = 12.5 feet (allowable flow spread per Table A, Appendix D)

Solution:

**Step B.1-** Solve for ($d_1$):

$d_1$ = low point flow depth at curb (at allowable spread width)

$d_1 = (T-W)S_x + WS_w$ (**Chapter 8**)

$d_1 = (12.5 - 2)(0.2) + (2)[(1.5/12)/2]$

$= 0.21 + 0.125$ feet

$d_1 = 0.335$ feet = 4.02 inches

Because flow depth (0.34 feet) is approximately equal to the curb opening height and not greater than 4.5 inches (Section 6.1), assume inlet operates as a weir.

**Step B.2-** Solve for ($Q_i$):

$Q_i$ = low point grate inlet (CB-3) and curb opening extension capacity at maximum allowable spread

$Q_{ig}$ = grate inlet capacity at maximum allowable spread

= $C_w P d^{1.5}$ (Equation 13)

P = 4.31 feet (CG-1 grate perimeter, 50 percent clogged, Table B, Appendix D)

d = 0.335 feet

$Q_{ig}$ = $3.0(4.31)(0.335)^{1.5}$ = 2.51 cubic feet per second

$Q_{iex}$ = low point curb opening extension (CB-3) capacity at maximum allowable spread

= $2.3(L + 1.8W)d^{1.5}$ (Equation 15)

L = 3.0 feet (curb opening extension 50 percent clogged, Table B, Appendix D)

W = 2 feet (width of depression)

d = 0.335 feet

$Q_{iex}$ = $2.3\ [3.18 + 1.8(2)]\ (0.335)^{1.5}$

= 2.94 cubic feet per second

$Q_i$ = $Q_{ig} + Q_{iex}$

= 2.51 + 2.94 = 5.45 cubic feet per second

**Step B.3-** Solve for ($Q_{50}$):

$Q_{50}$ = 50-year peak flow (cubic feet per second) draining toward catch basins CB-2 and CB-4

Q = CiA (**Chapter 7**)

i = 3.0 inches/hour (IDR curves in **Chapter 7**)

C = 0.9

A = $W_p$ L

= (75.5)(400) = 30,200 square feet = 0.70 acres

$Q_{50}$ = (0.9)(3.0)(0.70) = 1.90 cubic feet per second

Therefore, 1.90 cubic feet per second drains toward catch basins CB-2 and CB-4.

**Step B.4-** Solve for (T):

T = total spread of water in feet just upstream of inlets CB-2 and CB-4

$$Q_{50} = \frac{0.56}{n} S_x^{1.67} S_L^{0.5} T^{2.67}$$

Use Chart 5 (Appendix H) or rearrange the previous equation to solve for T.

$$T = \left(\frac{nQ_{50}}{0.56(S_x)^{1.67}(S_L)^{0.5}}\right)^{0.375}$$ (**Chapter 8** or Chart 5, Appendix H)

$$T = \left(\frac{0.016(1.90)}{0.56(0.02)^{1.67}(0.018)^{0.5}}\right)^{0.375} = 8.25 \text{ feet}$$

Okay (because this is less than the allowable spread of 12.5 feet)

**Step B.5-** Solve for ($E_o$):

$E_o$ = ratio of flow in the depressed section to total gutter flow

$$E_o = 1-\left[1-\frac{W}{T}\right]^{2.67}$$ (Equation 4 or Chart 2, Appendix H)

$$E_o = 1-\left[1-\frac{2}{8.25}\right]^{2.67}$$

$E_o = 0.52$

**Step B.6-** Solve for ($S_e$):

$S_e$ = equivalent cross slope (foot per foot)

$S_e = S_x + S'_w E_o$ (Equation 12)

Where:

$$S'_w = \frac{a}{W}$$

$a$ = 1.5 inches (inlet depression)

$W$ = 2 feet (width of depression)

$$S'_w = \frac{a}{W} = \frac{1.5/12}{2} = 0.0625 \text{ foot per foot}$$

$$S_e = 0.02 + 0.0625(0.52) = 0.0525 \text{ foot per foot}$$

**Step B.7-** Solve for ($L_T$):

$L_T$ = curb-opening length required to intercept 100 percent of gutter flow

$$L_T = 0.6\, Q_t^{0.42}\, S_L^{0.3}\left(\frac{1}{n\, S_e}\right)^{0.6}$$ [$S_e$ is substituted for $S_x$ in Equation 10 or Chart 6, Appendix H]

$$L_T = 0.6(1.9)^{0.42}(0.018)^{0.3}\left(\frac{1}{(0.016)(0.0525)}\right)^{0.6}$$

$L_T$ = 16.5 feet

**Step B.8-** Solve for (E):

E = curb-opening extension efficiency (neglect curb opening adjacent to grate)

$$E = 1-\left(1-\frac{L_{ex}}{L_T}\right)^{1.8}$$ (Equation 11 or Chart 7, Appendix H)

$L_{ex}$ = length of curb-opening extension prior to grate inlet, 30 percent clogged

$L_{ex}$ = 4.2 feet

$$E = 1-\left(1-\frac{4.2}{16.5}\right)^{1.8} = 0.41 = 41\,\text{percent}$$

**Step B.9-** Solve for ($Q_{iex}$):

$Q_{iex}$ = flow intercepted by curb-opening extension

$Q_{iex} = EQ_{50} = (0.41)(1.9) = 0.78$ cubic feet per second

**Step B.10-** Solve for ($Q_b$):

$Q_b$ = bypass flow from curb-opening extension and total flow toward grate inlet

$Q_b = Q_{50} - Q_{iex} = 1.90 - 0.78 = 1.12$ cubic feet per second

**Step B.11-** Solve for (T):

T = total spread at grate (after interception by curb-opening extension)

This is done by trial and error. Start by assuming an initial value of T = 6.75 feet

$T_s = T - W = 6.75 - 2.0 = 4.75$ feet

$$Q_s = \frac{0.56}{n} S_x^{1.67} S_L^{0.5} T_s^{2.67}$$ (**Chapter 8** or Chart 5, Appendix H)

$$Q_s = \frac{0.56}{0.016}(0.02)^{1.67}(0.018)^{0.5}(4.75)^{2.67} = 0.44 \text{ cubic feet per second}$$

$$E_o = 1-\left[1-\frac{W}{T}\right]^{2.67} = 1-\left[1-\frac{2}{6.75}\right]^{2.67}$$ (Equation 4 or Chart 2, Appendix H)

$E_o = 0.61$

$$Q_t = \frac{Q_s}{1-E_o} = \frac{0.44}{1-0.61} = 1.13 \text{ cubic feet per second}$$

Bypass flow from curb opening extension is 1.13 cubic feet per second. Total flow toward the catch basin grate at a spread of 6.75 feet is 1.13 cubic feet per second. Therefore, the spread at the catch basin grate is 6.75 feet.

**Step B.12-** Solve for (V):

V = average gutter flow velocity at grate

$$V = \frac{Q_t}{A} = \frac{2Q_t}{T^2 S_x}$$ (**Chapter 8**)

$$V = \frac{2\,(1.13)}{6.75^2\,(0.02)} = 2.48 \text{ feet per second}$$

**Step B.13-** Solve for ($R_f$)

$R_f$ = ratio of frontal flow intercepted to total frontal flow

$R_f = 1 - 0.09\,(V - V_o)$ (Equation 6 or Chart 1, Appendix H)

$V_o$ = gutter velocity where splash over first occurs (in feet per second)

CG-1 grate length = 2.31 feet (30 percent clogged, Table B)

Grate type = P-1-$^7/_8$-4

Therefore from Chart 1 (Appendix H), $V_o$ = 5.0 feet per second

$R_f = 1 - 0.09\,(2.48 - 5.0) = 1.23$

$R_f$ cannot exceed 1.0, therefore

$R_f = 1.0$

**Step B.14-** Solve for ($R_s$)

$R_s$ = ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1 + \left( \frac{0.15V^{1.8}}{S_x L^{2.3}} \right)}$$ (Equation 7 or Chart 4, Appendix H)

V = 2.48 feet per second

L = 2.31 (CG-1 length, 30 percent clogged, Table B, Appendix D)

$$R_s = \frac{1}{1 + \left( \frac{0.15\,(2.48)^{1.8}}{0.02(2.31)^{2.3}} \right)} = 0.15$$

**Step B.15-** Solve for ($E_g$):

$E_g$ = interception efficiency of the inlet grate

$E_g = R_f E_o + R_s (1 - E_o)$ (Equation 8)

$E_g = (1.0)(0.61) + 0.15(1 - 0.61) = 0.67 = 67$ percent

**Step B.16-** Solve for ($Q_{ig}$):

$Q_{ig}$ = interception flow of grate inlet

$Q_{ig} = E_g Q_t = 0.67(1.13) = 0.76$ cubic feet per second

**Step B.17-** Solve for ($Q_i$):

$Q_i$ = total interception flow (both grate inlet and curb-opening extension)

$Q_i$ = interception flow of curb-opening extension + interception flow of grate inlet

$Q_i = Q_{iex} + Q_{ig}$

$Q_i = 0.78 + 0.76 = 1.54$ cubic feet per second

**Step B.18-** Solve for ($Q_b$):

$Q_b$ = total bypass flow of upstream inlet (both grate inlet and curb-opening extension)

$Q_b = Q_{50} - Q_i$

$Q_b = 1.90 - 1.54 = 0.36$ cubic feet per second

**Step B.19-** Calculate ($L_2$):

$L_2$ = distance between sag inlet CB-3 and CB-2, and

= distance between sag inlet CB-3 and CB-4

$$L_2 = \frac{43{,}560Q}{CC_f iW_p}E \quad \text{(Equation 1)}$$

Equation 1 is modified to calculate $L_2$ in sags, therefore

$$L_2 = \frac{43{,}560\,(Q_i - 2Q_b)}{C\ C_f\ i\ 2W_p}$$

i = 3.0 inches per hour (assume 5-minute $t_c$ using a 50-year storm design, Hydrologic Zone 8, using IDR curve in **Chapter 7**)

$C_f$ = 1.0

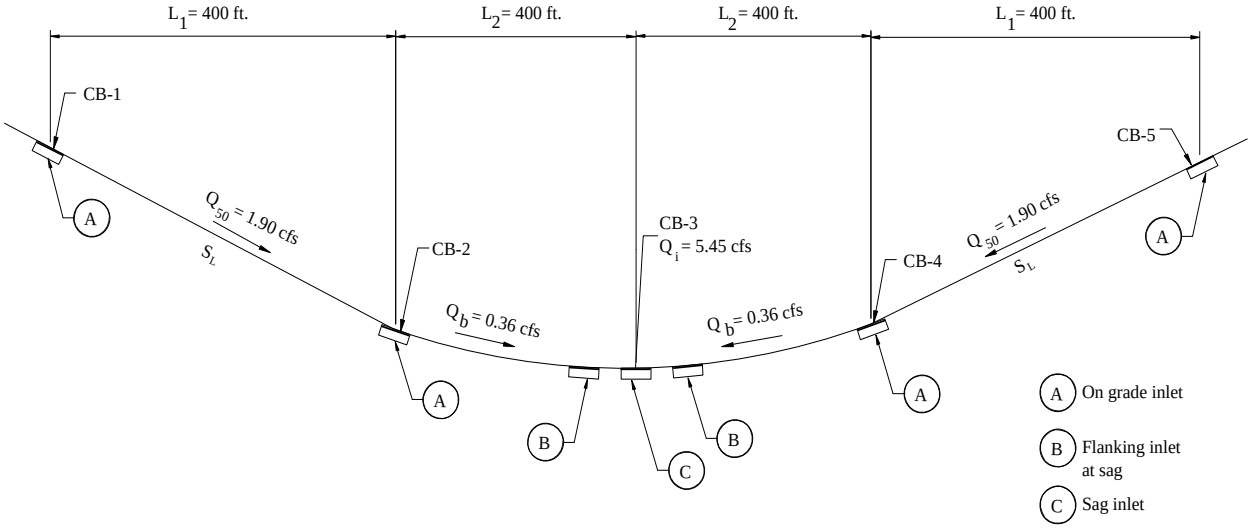
$W_p$ = 75.5 feet

$Q_i$ = 5.45 cubic feet per second (low point inlet (CB-3) interception capacity)

$$L_2 = \frac{43{,}560\,(5.45 - 2(0.36))}{(0.9)(1.0)(3.0)(2)(75.5)} = 505 \text{ feet}$$

Maximum inlet spacing shall be limited to 400 feet

505 feet is greater than 400 feet, therefore inlet spacing shall be limited to 400 feet.




SWEEPER COMBINATION INLETS
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## 6.5 Trench Drain Systems (Sag)

The use of trench drains in sags is generally discouraged because of the tendency of such inlets to intercept debris and clog. However, there may be locations where it is desirable to supplement an existing low point inlet with the use of a trench drain.

*Note: General information on trench drain systems is noted in Appendix C.*

To aid in the design of trench drains, the designer should consider the following guidelines:

- Equations and figures used for the design of grate inlets (reference Section 6.1) should be used in the design of trench drains.
- Consider trench inlet hydraulic conveyance capacity curves or data from a manufacturer.
- Consider trench inlet hydraulic capacity curves or data for available grates from a manufacturer.
- Trench drain grate shall be a bicycle safe grate and is rated for heavy highway traffic.
- When using design Method B, the designer should assume 50 percent clogging which will result in providing twice the calculated required length for flow interception because trench drains tend to plug.
- The minimum cleanout velocity for trench drain systems should be 3 feet per second.

- Trench drains should not be used in areas with high traffic speeds greater than or equal to 45 miles per hour.

## 6.6 Flanking Inlets

Inlets should always be located at the low points of sag vertical curves. In addition to providing low point inlets, it is good engineering practice to place flanking inlets on each side of low point inlets when in a depressed area that stormwater has no outlet except through the drainage system. This is illustrated in Figure 6.

The purpose of flanking inlets is to act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded as noted in Table A. Flanking inlets can be located so they will function before water spread exceeds the allowable spread at the sump location. Also flanking inlets should be located so that they will receive all of the flow when the primary inlet at the bottom of the sag is clogged.

*Note: When designing inlets in sag locations flanking inlets are neglected in the calculations.*

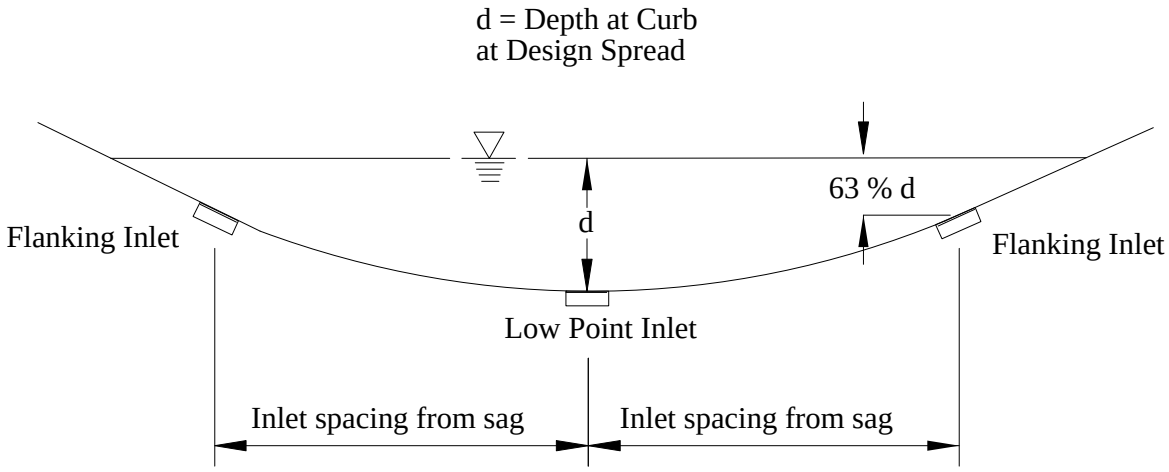


**Figure 6 - Flanking Inlets**

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Flanking inlet design shall meet one of the following methods:

**Method 1:**
Flanking inlets should be placed 10-15 feet from the sag inlet and should be no more than 0.1 foot higher than the sag inlet. If this criteria does not fit the site conditions, determine the locations of the flanking inlets as described in Method 2.

**Method 2:**

Flanking inlets can be located so they will function before water spread exceeds the allowable spread at the sump location. The flanking inlets should be located so that they will receive all of the flow when the primary inlet at the bottom of the sag is clogged. They should do this without exceeding the allowable spread at the bottom of the sag. If the flanking inlets are the same dimension as the primary inlet, they will each intercept one-half the design flow when they are located so that the depth of ponding at the flanking inlets is 63 percent of the depth of ponding at the low point. If the flanking inlets are not the same size as the primary inlet, it will be necessary to either develop a new factor or do a trial and error solution using assumed depths with the weir equation to determine the capacity of the flanking inlet at the given depths.

Table D shows the spacing required for various depths at curb criteria and vertical curve lengths defined by:

$$K = \frac{L}{G_2 - G_1} \qquad \text{(Equation 22)}$$

Where:

- $K$ = rate of vertical curvature
- $L$ = length of the vertical curve in feet
- $G_1$ = beginning approach grade
- $G_2$ = ending approach grade

**Table D - Flanking Inlet Locations**

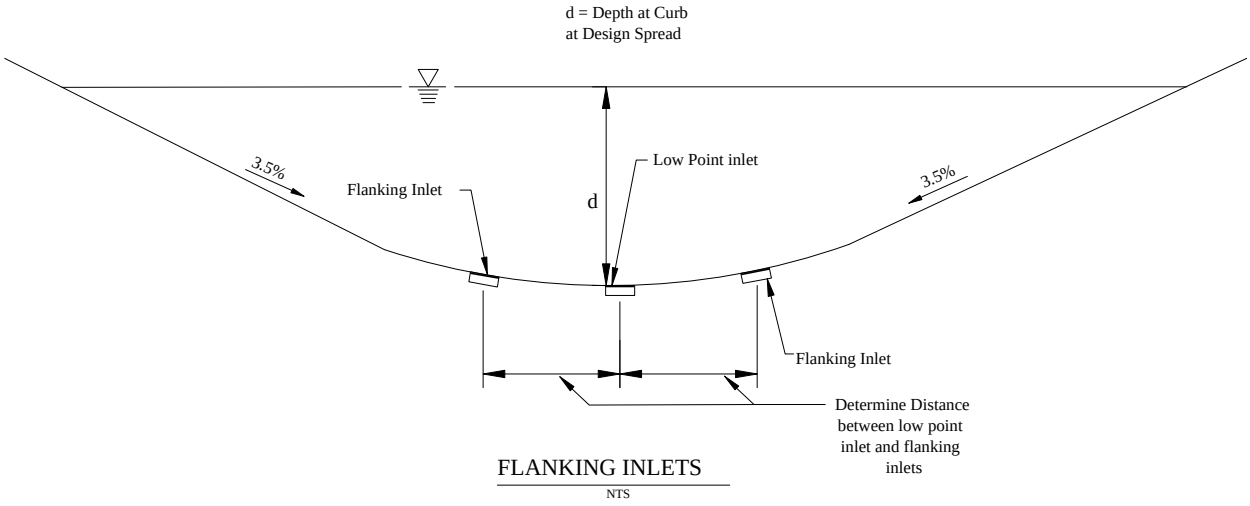
| Distance to flanking inlets in sag vertical curve using depth at curb criteria (feet). | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| K → (feet per percent) / (feet) d ↓ | 20 | 30 | 40 | 50 | 70 | 90 | 110 | 130 | 160 | 167 |
| 0.1 | 20 | 24 | 28 | 32 | 37 | 42 | 47 | 51 | 57 | 58 |
| 0.2 | 28 | 35 | 40 | 45 | 53 | 60 | 66 | 72 | 80 | 82 |
| 0.3 | 35 | 42 | 49 | 55 | 65 | 73 | 81 | 88 | 98 | 100 |
| 0.4 | 40 | 49 | 57 | 63 | 75 | 85 | 94 | 102 | 113 | 116 |
| 0.5 | 45 | 55 | 63 | 71 | 84 | 95 | 105 | 114 | 126 | 129 |
| 0.6 | 49 | 60 | 69 | 77 | 92 | 104 | 115 | 125 | 139 | 142 |
| 0.7 | 53 | 65 | 75 | 84 | 99 | 112 | 124 | 135 | 150 | 153 |
| 0.8 | 57 | 69 | 80 | 89 | 106 | 120 | 133 | 144 | 160 | 163 |
| NOTES 1. $X = (200dK)^{0.5}$, where X = distance from the sag point (feet). 2. $d = Y - Y_f$ where Y = depth of ponding and $Y_f$ = depth at the flanker inlet. 3. Drainage maximum K = 167 feet per percent. | | | | | | | | | | |

### 6.6.1 Flanking Inlets Example

The following example illustrates the procedure presented for Method 2.

#### 6.6.1.1 Determine the location of the flanking inlets if located to function in relief of the inlet at the low point when the inlet at the low point is clogged

Sketch:




Given:

- Highway on-grade section, main line sag (45 miles per hour or greater)
- The spread at design flow (Q) is is not to exceed the shoulder width of 10 feet (per Table A).
- $W_p$ = 46.5 feet (width of contributing drainage area, 4-12-foot lanes with 10-foot shoulder, 5-foot sidewalk, 7-foot median)
- L = 500 feet (vertical curve length at sag)
- $S_x$ = 0.02 foot per foot (shoulder and lane cross section)
- $G_1$ = -3.5 percent (roadway slope entering sag)
- $G_2$ = 3.5 percent (roadway slope exiting sag)

Criteria:

- Flanking inlets should be designed so that they will receive all of the flow when the primary inlet at the bottom of the sag is clogged.
- Ponding at the flanking inlet should be 63 percent of the depth of ponding at the low point (for inlets of the same size)
- T = shoulder width + 0 feet = 10 feet (allowable total spread at inlet opening, per Table A, Appendix D)

Solution:

**Step B.1-** Solve for (K):

K = rate of vertical curvature

$$K = \frac{L}{(G_2 - G_1)} \qquad \text{(Equation 22)}$$

L = 500 feet (vertical curve length)

$$K = \frac{500}{[3.5 - (-3.5)]}$$

K = 71.4 feet per percent

**Step B.2-** Solve for (Y):

Y = depth of ponding at design spread.

$Y = S_x T$

$= (0.02)(10)$

Y = 0.20 feet

**Step B.3-** Solve for ($Y_f$):

$Y_f$ = depth at flanker locations

Flanker inlet should be located so that depth at flanker is 63 percent of the depth at the sag inlet

$Y_f = 0.20(0.63) = 0.126$ feet

**Step B.4-** Solve for (X):

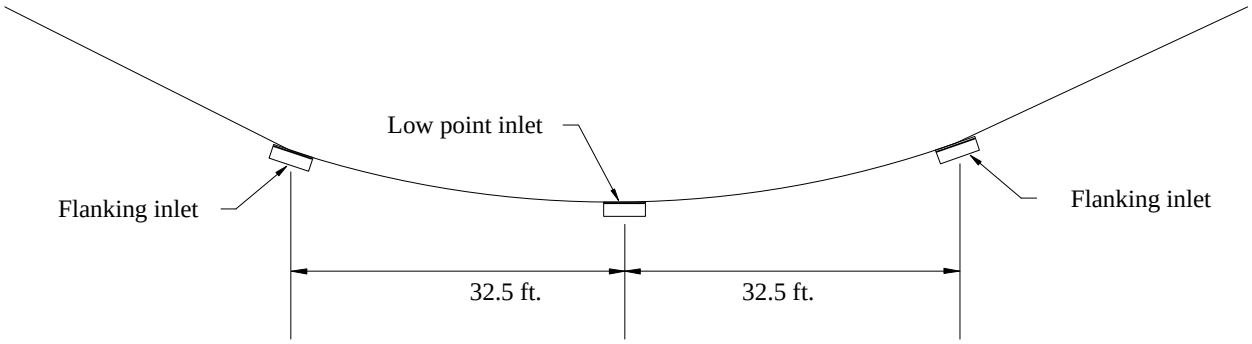
X = distance from flanker inlet to sag inlet

$X = (200dK)^{0.5}$

$d = Y - Y_f = 0.20 - 0.126 = 0.074$

$X = [(200)(0.074)(71.4)]^{0.5} = 32.5$ feet

Therefore, space flanking inlets at a distance no more than 32.5 feet from sag point.




FLANKING INLETS
NTS

Example problems in Section 6.0 illustrate the total interception capacity of inlets in sag locations. Except where inlets become clogged, spread on low gradient approaches to the low point is a more stringent criterion for design than the interception capacity of the sag inlet. It is recommended that a gradient of 0.3 percent be maintained within 50 feet of the level point in order to provide adequate drainage. Standard inlet locations may need to be adjusted to avoid excessive spread in the sag curve. Inlets may also be needed between the flankers and the ends of the curves. For major sag points, the flanking inlets are added as a safety factor, and are not considered as intercepting flow to reduce the bypass flow to the sag point. They are installed to assist the sag point inlet in the event of clogging.

## 7.0 Median Inlets

Standard ODOT drop inlets G-2M and G-2MA are recommended median and roadside ditch inlets as noted in Appendix C. These inlets are intended for portions of highways that have a very long continuous grade. When these inlets are placed in the clear zone the designer should evaluate that the proposed inlet location would not cause an errant vehicle to overturn.

The design examples in this section demonstrate the use of Chart 1, Chart 4, Chart 12, and Chart 13 located in Appendix H. These figures should be used in analyzing drop inlets in medians or roadside ditches on continuous grade. Charts 1 and 4 are used to estimate the ratios of frontal and side flow intercepted by the grate to total flow. The interception capacity of drop inlets in median ditches on continuous grades can be estimated by use of Charts 12 and 13 to estimate the flow depth and the ratio of frontal flow to total flow in the ditch.

Small dikes downstream of drop inlets (Figure 7) can be provided to impede bypass flow in an attempt to cause complete interception of the approach flow. The dikes usually need not be more than a few inches high and should have traffic safe slopes. The height of dike required for complete interception on continuous grades or the depth of ponding in sag vertical curves can be computed by use of Chart 8. The effective perimeter of a grate in an open channel with a dike should be taken as 2(L + W) since one side of the grate is not adjacent to a curb.

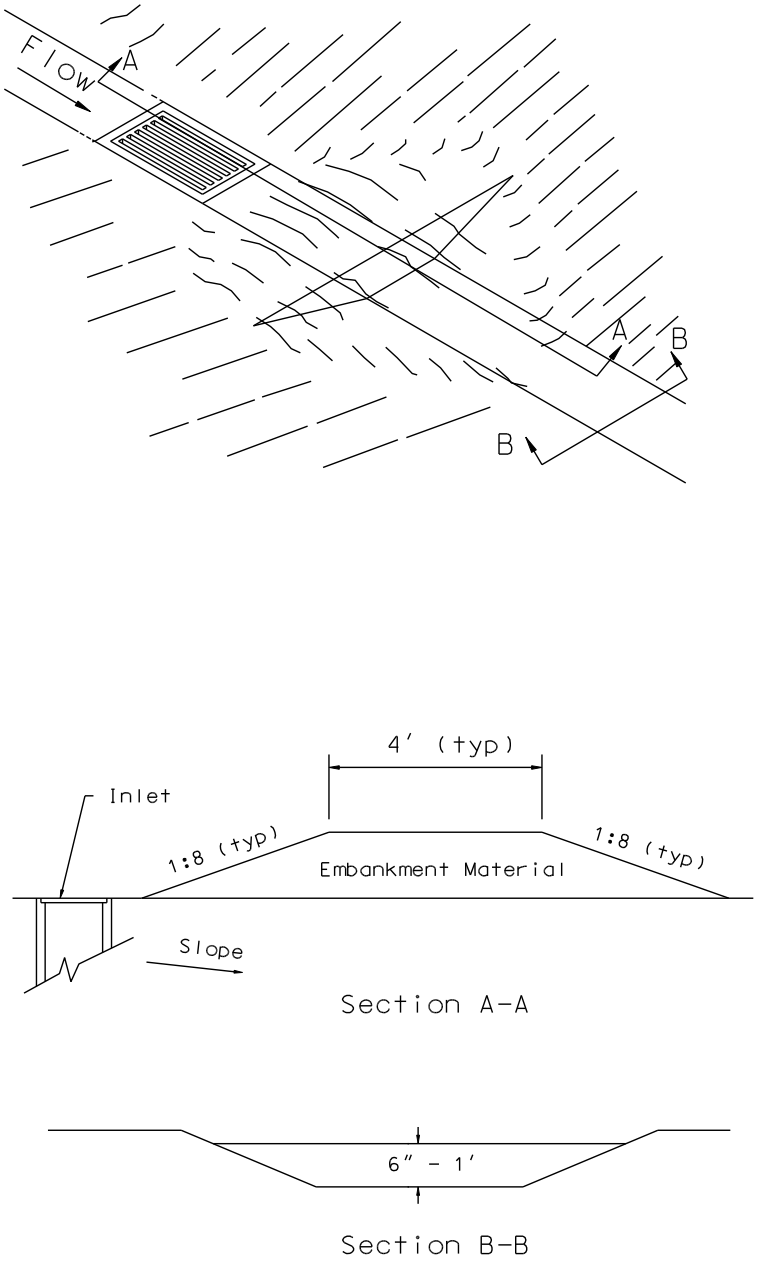


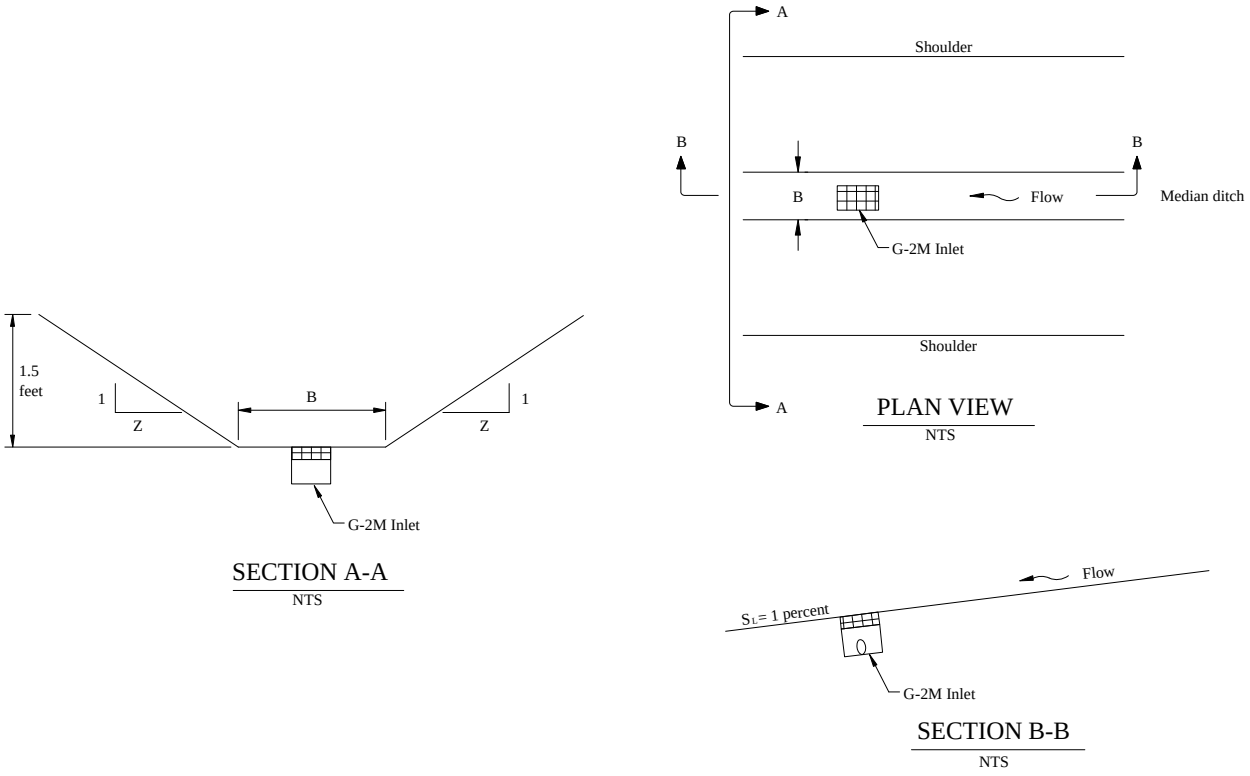

**Figure 7 – Median Drop Inlet**

## 7.1 Median Drop Inlet (On-grade) Example

The following example illustrates the design of a median drop inlet (on grade) with a grate width less than the bottom width of the ditch.

### 7.1.1 Determine the maximum allowable flow, interception flow, and bypass flow.

Sketch:




Given:

- Urban collector, less than 45 miles per hour, on grade section
- $S_L$ = 0.01 foot per foot (longitudinal slope of roadway and channel)
- $W_p$ = 58 feet (width of contributing area, 2-12 foot travel lanes, 2-6 foot shoulders, 1-22 foot wide median)
- $W_g$ = 2.25 feet (G-2M inlet grate width)
- $L_g$ = 2.67 feet (G-2M inlet grate length)
- $P_g$ = 9.83 feet (G-2M inlet grate perimeter)
- i = 2.28 inches per hour (10-year design, Hydrologic Zone 8 (IDR curves are located in **Chapter 7**))
- C = 0.42 (weighted rational method runoff coefficient for pavement and grass cover)
- B = 4 feet (channel bottom width)
- n = 0.073 (Manning's coefficient for grass lined channel, reference **Chapter 8**)
- z = 6 feet (side slope, horizontal run per one foot of height)

- $D_{channel}$ = 1.5 feet (total channel depth)
- A = 1.01 Acres (total contributing area to median inlet)

### 7.1.2 Method A

Criteria:

- E = 0.70 (minimum efficiency for Method A)
- $y_{max}$ = 0.5 feet (maximum flow depth to maintain minimum of 1 foot freeboard in ditch)

Solution:

**Step A.1-** Solve for (Q):

Q = peak runoff rate to inlet during 10-year event

Q = CiA (**Chapter 7**)

= (0.42)(2.28)(1.01)

= 0.96 cubic feet per second

**Step A.2-** Solve for (y):

Y = flow depth in channel during 10-year event

Use Manning's equation

$$Q = \left(\frac{1.486}{n}\right) R^{2/3} A S_L^{1/2}$$ (**Chapter 8**)

Solve for flow depth by trial and error by assuming a flow depth until Q equals 0.96 cubic feet per second.

y = 0.25 feet

Okay (because 0.25 feet is less than maximum allowable depth of 0.5 feet)

**Step A.3-** Solve for (P):

P = wetted perimeter in feet

$$P = B + 2y\sqrt{Z^2 + 1}$$ (**Chapter 8**)

y = 0.25 feet

$$P = 4 + 2(0.25)\sqrt{6^2 + 1} = 7.04 \text{ feet}$$

**Step A.4-** Solve for (A):

A = flow area in square feet.

$A = By + Zy^2$ (**Chapter 8**)

$= (4)(0.25) + (6)(0.25)^2$

= 1.38 square feet

**Step A.5-** Solve for (R):

R = hydraulic radius, feet

R = A/P
= 1.38 / 7.04
= 0.196 feet

**Step A.6-** Solve for (Q):

Q = flow rate in channel at assumed flow depth of 0.25 feet

$$Q = \left(\frac{1.486}{n}\right) R^{2/3} A S^{1/2}$$ (**Chapter 8**)

$$Q = \left(\frac{1.486}{0.073}\right)(0.196)^{2/3}(1.38)(0.01)^{1/2} = 0.96 \text{ cubic feet per second}$$

The flow calculated in this step is the same as that calculated in Step A.1. Therefore this flow depth of 0.25 feet is correct.

**Step A.7-** Solve for (V):

V = average flow velocity in channel at flow depth of 0.25 feet

V = Q / A (**Chapter 8**)

$$V = \left(\frac{Q}{A}\right) = \frac{0.96}{1.38} = 0.70 \text{ feet per second}$$

*Note: To assure the channel is stable, the designer must evaluate that the maximum shear stress caused by the design flow is less than the maximum shear stress the lining can resist. The designer is directed to Chapter 8 for additional information.*

**Step A.8-** Solve for ($R_f$):

$R_f$ = ratio of frontal flow intercepted to total frontal flow

G-2M inlet equivalent grate type = P-1-$^7/_8$ (Table B, Appendix C)

$L_g$ = 2.67 feet (G-2M grate length)

$V_o$ = 9.7 feet per second (Chart 1, Appendix H)

$R_f = 1 - 0.09 (V - V_o)$ (Equation 6)

$R_f = 1 - 0.09 (0.70 - 9.7) = 1.81$

$R_f$ cannot be greater than 1, therefore $R_f = 1.0$

**Step A.9-** Solve for ($R_s$):

$R_s$ = ratio of side flow intercepted to total side flow

$$R_s = R_s = \frac{1}{1+\left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)} \quad \text{(Equation 7)}$$

$$R_s = \frac{1}{1+\left(\frac{0.15(0.70)^{1.8}}{(0.01)(2.67)^{2.3}}\right)} = 0.55$$

*Note: $S_x$ is assumed to be 1 percent because the ditch bottom is wider than the grate and has no cross-slope, therefore, per FHWA HEC-22 recommends using the least cross-slope (1 percent) noted in Chart 4, Appendix H.*

**Step A.10-** Solve for ($E_o$):

$E_o$ = ratio of frontal flow to total flow in trapezoidal channel

$$E_o = \frac{W_g}{B + yZ} \quad \text{(Chart 13, Appendix H)}$$

$W_g$ = 2.25 feet (G-2M grate width) (Table B, Appendix C)

$$E_o = \frac{2.25}{\left[4 + 0.25(6)\right]} = 0.41$$

**Step A.11-** Solve for (E):

E = inlet efficiency

$E = R_f E_o + R_s (1 - E_o)$ (Equation 8)

$E = 1 (0.41) + 0.55(1 - 0.41) = 0.73$

E = 0.73 is greater than the minimum allowable interception efficiency, therefore criteria of minimum efficiency is met.

**Step A.12-** Solve for ($Q_i$):

$Q_i$ = intercepted flow in cubic feet per second

$Q_i = EQ_t$

$Q_i$ = (0.73) (0.96) = 0.70 cubic feet per second

**Step A.13-** Solve for ($Q_b$):

$Q_b$ = flow that is not intercepted by the inlet and must be included in the evaluation of downstream channels and inlets

$Q_b = Q_t - Q_i$

$Q_b$ = 0.96 – 0.70 = 0.26 cubic feet per second

### 7.1.3 Method B

Criteria:

- Assume 30 percent clogging of a type G-2M grate (per Table A)
- $L_g$ = 2.26 feet (G-2M grate length, 30 percent clogged)
- $W_g$ = 1.84 feet (G-2M grate width, 30 percent clogged)
- $y_{max}$ = 0.5 feet (maximum flow depth to maintain minimum of 1 foot freeboard in ditch)

Solution:

**Step B.1-** Solve for (Q):

Q = peak runoff rate to inlet during 10-year event

Q = CiA (**Chapter 7**)

= (0.42)(2.28)(1.01)

= 0.96 cubic feet per second

**Step B.2-** Solve for (y):

y = flow depth in channel during 10-year event

Use Manning's equation

$$Q = \left(\frac{1.486}{n}\right) R^{2/3} A S_L^{1/2}$$ (**Chapter 8**)

Solve for flow depth by trial and error by assuming a flow depth until Q equals 0.96 cubic feet per second.

y = 0.25 feet

Okay (because 0.25 feet is less than maximum allowable depth of 0.5 feet)

**Step B.3-** Solve for (P):

P = wetted perimeter in feet

$$P = B + 2y\sqrt{Z^2 + 1}$$ (**Chapter 8**)

y = 0.25 feet

$$P = 4 + 2(0.25)\sqrt{6^2 + 1} = 7.04 \text{ feet}$$

**Step B.4-** Solve for (A):

A = flow area in square feet.

$A = By + Zy^2$ (**Chapter 8**)

$= (4)(0.25) + (6)(0.25)^2$

= 1.38 square feet

**Step B.5-** Solve for (R):

R = hydraulic radius, feet

R = A/P
= 1.38/7.04
= 0.196 feet

**Step B.6-** Solve for (Q):

Q = flow rate in channel at assumed flow depth of 0.25 feet

$$Q = \left(\frac{1.486}{n}\right) R^{2/3} A S^{1/2}$$ (**Chapter 8**)

$$Q = \left(\frac{1.486}{0.073}\right)(0.196)^{2/3}(1.38)(0.01)^{1/2} = 0.96 \text{ cubic feet per second}$$

The flow calculated in this step is the same as that calculated in Step B.1. Therefore this flow depth of 0.25 feet is correct.

**Step B.7-** Solve for (V):

V = average flow velocity in channel at flow depth of 0.25 feet

$V = Q / A$ (**Chapter 8**)

$$V = \left(\frac{Q}{A}\right) = \frac{0.96}{1.38} = 0.70 \text{ feet per second}$$

*Note: To assure the channel is stable, the designer must evaluate that the maximum shear stress caused by the design flow is less than the maximum shear stress the lining can resist. The designer is directed to **Chapter 8** for additional information.*

**Step B.8-** Solve for ($R_f$):

$R_f$ = ratio of frontal flow intercepted to total frontal flow

G-2M inlet equivalent grate type = P-1-$^7/_8$ (Table B, Appendix C)

$L_g$ = 2.26 feet (G-2M grate length, 30 percent clogged)

$V_o$ = 8.6 feet per second (Chart 1, Appendix H)

$R_f = 1 - 0.09 (V - V_o)$ (Equation 6)

$R_f = 1 - 0.09 (0.70 - 8.6) = 1.71$

$R_f$ cannot be greater than 1, therefore $R_f = 1.0$

**Step B.9-** Solve for ($R_s$):

$R_s$ = ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1+\left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)} \quad \text{(Equation 7)}$$

$$R_s = \frac{1}{1+\left(\frac{0.15(0.70)^{1.8}}{(0.01)(2.26)^{2.3}}\right)} = 0.45$$

**Step B.10-** Solve for ($E_o$):

$E_o$ = ratio of frontal flow to total flow in trapezoidal channel

$$E_o = \frac{W_g}{B+yZ} \quad \text{(Chart 13, Appendix H)}$$

$W_g$ = 1.84 feet (G-2M grate width, 30 percent clogged) (Table B, Appendix C)

$$E_o = \frac{1.84}{(4+0.25(6))} = 0.34$$

**Step B.11-** Solve for (E):

E = inlet efficiency

$E = R_f E_o + R_s (1 - E_o)$ (Equation 8)

$E = 1(0.34) + 0.45(1 - 0.34) = 0.64$

Okay because 70 percent interception criteria is not required for Method B.

**Step B.12-** Solve for ($Q_i$):

$Q_i$ = intercepted flow in cubic feet per second

$Q_i = EQ_t$

$Q_i = (0.64)\ (0.96)$

$Q_i$ = 0.61 cubic feet per second

**Step B.13-** Solve for ($Q_b$):

$Q_b$ = flow that is not intercepted by the inlet and must be included in the evaluation of downstream channels and inlets

$Q_b = Q_t - Q_i$

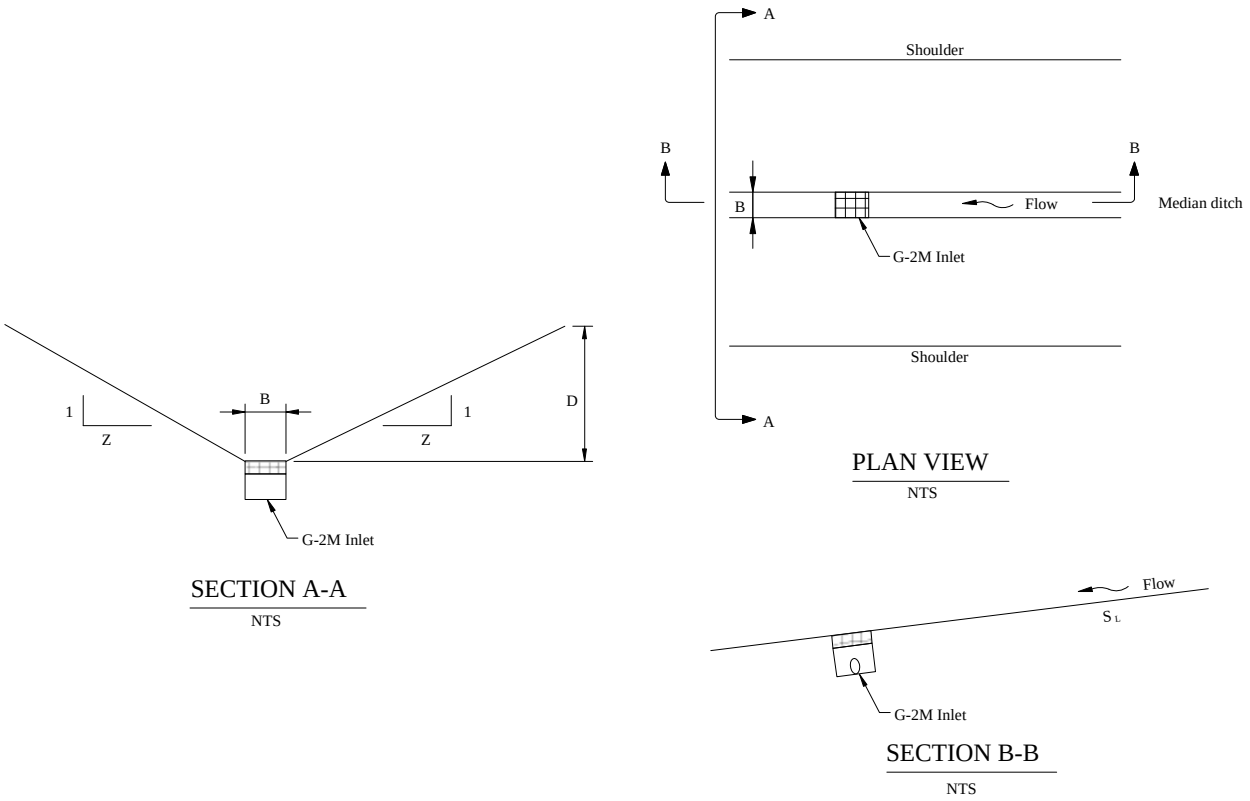
$Q_b = 0.96 - 0.61 = 0.35$ cubic feet per second

## 7.2 Median Drop Inlet (On-grade) Example

The following example illustrates the use of Chart 4 (Appendix H) when the grate width is equal to the bottom width of the ditch. Use Chart 4 by substituting ditch side slopes for values of $S_x$ as illustrated in the following example:

### 7.2.1 Determine the intercepted flow ($Q_i$) and the bypassed flow ($Q_b$) for the following conditions.

Sketch:




Given:

*Note: Method A or Method B could be used. Method selected should be used for entire design.*

- Q = 20.0 cubic feet per second (10-year peak runoff)
- Urban collector, greater than 45 milers per hour, on-grade section
- B = 2.25 feet (channel width)
- n = 0.027 (Manning's roughness coefficient)
- z = 6 (channel side slope)
- $S_L$ = 0.0125 feet per feet (longitudinal slope of channel and roadway)
- D = 2.0 feet (total ditch depth)
- The flow in the median ditch is to be intercepted by a drop inlet
- Type G-2M inlet with a P-1-$^7/_8$ parallel bar grate
- $W_g$ = 2.25 feet (grate width for G-2M inlet)
- $L_g$ = 2.67 (grate length for G-2M inlet)
- There is no dike downstream of the inlet.

### 7.2.2 Method A

Solution:

**Step A.1-** Solve for (y):

y = flow depth in channel during a 10-year event.

Use Manning's equation

$$Q = \left(\frac{1.486}{n}\right) R^{2/3} A S_L^{1/2}$$ (**Chapter 8**)

Solve for flow depth by trial and error. Start by assuming a flow depth.

y = 0.78 feet

**Step A.2-** Solve for (P):

P = wetted perimeter, feet

$$P = B + 2y\sqrt{Z^2 + 1}$$ (**Chapter 8**)

y = 0.78 feet

$$P = 2.25 + 2(0.78)\sqrt{6^2 + 1} = 11.74 \text{ feet}$$

**Step A.3-** Solve for (A):

A = flow area, square feet.

$A = By + Zy^2$ (**Chapter 8**)

$= (2.25)(0.78) + (6)(0.78)^2$

= 5.41 square feet

**Step A.4-** Solve for (R):

R = hydraulic radius, feet

R = A/P = 5.41/11.74 = 0.46 feet

**Step A.5-** Solve for (Q):

Q = flow rate in channel at assumed flow depth of 0.78 feet

$$Q = \left(\frac{1.486}{n}\right) R^{2/3} A S^{1/2}$$ (**Chapter 8**)

$$Q = \left(\frac{1.486}{0.027}\right)(0.46)^{2/3}(5.41)(0.0125)^{1/2} = 20.0 \text{ cubic feet per second}$$

The flow calculated in this step is the same as that given in the problem statement. Therefore this flow depth is correct.

**Step A.6-** Solve for (V):

V = average flow velocity in channel at flow depth of 0.78 feet

V = Q/A (**Chapter 8**)

= 20.0/5.41 = 3.70 feet per second

**Step A.7-** Solve for ($R_f$):

$R_f$ = ratio of frontal flow intercepted to total frontal flow

G-2M inlet equivalent grate type = P-1-$^7/_8$ parrallel pipe. (Table B, Appendix C)

$L_g$ = 2.67 feet (G-2M grate length)

$V_o$ = 9.7 feet per second (Chart 1, Appendix H)

$R_f = 1 - 0.09 (V - V_o)$ (Equation 6)

$R_f = 1 - 0.09 (3.70 - 9.7) = 1.54$

$R_f$ cannot be greater than 1, therefore $R_f = 1.0$

**Step A.8-** Solve for ($R_s$):

$R_s$ = ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1+\left(\frac{0.15(V)^{1.8}}{S_x(L)^{2.3}}\right)}$$ (Equation 7, or Chart 4, Appendix H)

$$R_s = \frac{1}{1+\left(\frac{0.15(3.70)^{1.8}}{0.17(2.67)^{2.3}}\right)} = 0.51$$

*Note:* $S_x = \frac{1}{Z}$ *per FHWA HEC-22*

**Step A.9-** Solve for ($E_o$)

$E_o$ = ratio of frontal flow to total flow in trapezoidal channel

$$E_o = \frac{W_g}{B + yZ}$$ (Chart 13, Appendix H)

$W_g$ = 2.25 feet (G-2M grate width) (Table B, Appendix C)

$$E_o = \frac{2.25}{(2.25 + 0.78(6.0))} = 0.32$$

**Step A.10-** Solve for (E):

E = total efficiency.

$E = E_o R_f + R_s (1 - E_o)$ (Equation 8)

$E = (0.32)(1.0) + (0.51)(1-0.32) = 0.67$

The minimum allowable inlet efficiency when using Method A is 70 percent. Therefore the design would have to be modified to meet efficiency requirements such as adding an additional inlet upstream to reduce peak flow to this inlet or use Method B or by constructing a barrier downstream of the inlet to force the water into the inlet. This method is demonstrated in example problem 7.3.

### 7.2.3 Method B

Criteria:

- Assume 30 percent clogged type G-2M inlet with a P-1-$^7/_8$ parallel bar grate
- $W_g$ = 1.84 feet (grate width for G-2M inlet, per Table B)
- $L_g$ = 2.26 (grate length for G-2M inlet, per Table B)

Solution:

**Step B.1-** Solve for (y):

y = flow depth in channel during a 10-year event.

Use Manning's equation

$$Q = \left(\frac{1.486}{n}\right) R^{2/3} A S_L^{1/2}$$ (**Chapter 8**)

Q = 20 cubic feet per second

Solve for flow depth by trial and error. Start by assuming a flow depth.

y = 0.78 feet

**Step B.2-** Solve for (P):

P = wetted perimeter, feet

$P = B + 2y\sqrt{Z^2 + 1}$ (**Chapter 8**)

y = 0.78 feet

$P = 2.25 + 2(0.78)\sqrt{6^2 + 1} = 11.74$ feet

**Step B.3-** Solve for (A):

A = flow area, square feet.

$A = By + Zy^2$ (**Chapter 8**)

$= (2.25)(0.78) + (6)(0.78)^2$

= 5.41 square feet

**Step B.4-** Solve for (R):

R = hydraulic radius, feet

R = A/P = 5.41/11.74 = 0.46 feet

**Step B.5-** Solve for (Q):

Q = flow rate in channel at assumed flow depth of 0.78 feet

$Q = \left(\frac{1.486}{n}\right) R^{2/3} A S^{1/2}$ (**Chapter 8**)

$Q = \left(\frac{1.486}{0.027}\right)(0.46)^{2/3}(5.41)(0.0125)^{1/2} = 20.0$ cubic feet per second

The flow calculated in this step is the same as that given in the problem statement. Therefore this flow depth is correct.

**Step B.6-** Solve for (V):

V = average flow velocity in channel at flow depth of 0.78 feet

$V = Q/A$ (**Chapter 8**)

= 20.0/5.41 = 3.70 feet per second

**Step B.7-** Solve for ($R_f$):

$R_f$ = ratio of frontal flow intercepted to total frontal flow

G-2M inlet equivalent grate type = P-1-$^7/_8$ parrallel pipe. (Table B, Appendix C)

$L_g$ = 2.26 feet (G-2M grate length 30 percent clogged, Table B)

$V_o$ = 8.6 feet per second (Chart 1, Appendix H)

$R_f = 1 - 0.09 (V - V_o)$ (Equation 6)

$R_f = 1 - 0.09 (3.70 - 8.6) = 1.44$

$R_f$ cannot be greater than 1, therefore $R_f = 1.0$

**Step B.8-** Solve for ($R_s$):

$R_s$ = ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1+\left(\frac{0.15(V)^{1.8}}{S_x(L)^{2.3}}\right)} \quad \text{(Equation 7)}$$

$$R_s = \frac{1}{1+\left(\frac{0.15(3.70)^{1.8}}{0.17(2.26)^{2.3}}\right)} = 0.41$$

**Step B.9-** Solve for ($E_o$)

$E_o$ = ratio of frontal flow to total flow in trapezoidal channel

$$E_o = \frac{W}{B+yZ} \quad \text{(Chart 13, Appendix H)}$$

$W_g$ = 1.84 feet (G-2M grate width, 30 percent clogged, Table B)

$$E_o = \frac{1.84}{(2.25+0.78(6.0))} = 0.27$$

**Step B.10-** Solve for (E):

E = total efficiency.

$$E = E_o R_f + R_s (1 - E_o) \quad \text{(Equation 8)}$$

$$E = (0.27)(1.0) + (0.41)(1\text{-}0.27) = 0.57$$

**Step B.11-** Solve for ($Q_i$, $Q_b$ ):

$Q_i$ = interception flow in cubic feet per second

$Q_b$ = flow that is not intercepted by the inlet and must be included in the evaluation of downstream channels and inlets

$Q_i = EQ = (0.57)(20.0)$

$Q_i$ = 11.4 cubic feet per second

$Q_b = Q - Q_i = 20.0 - 11.4$

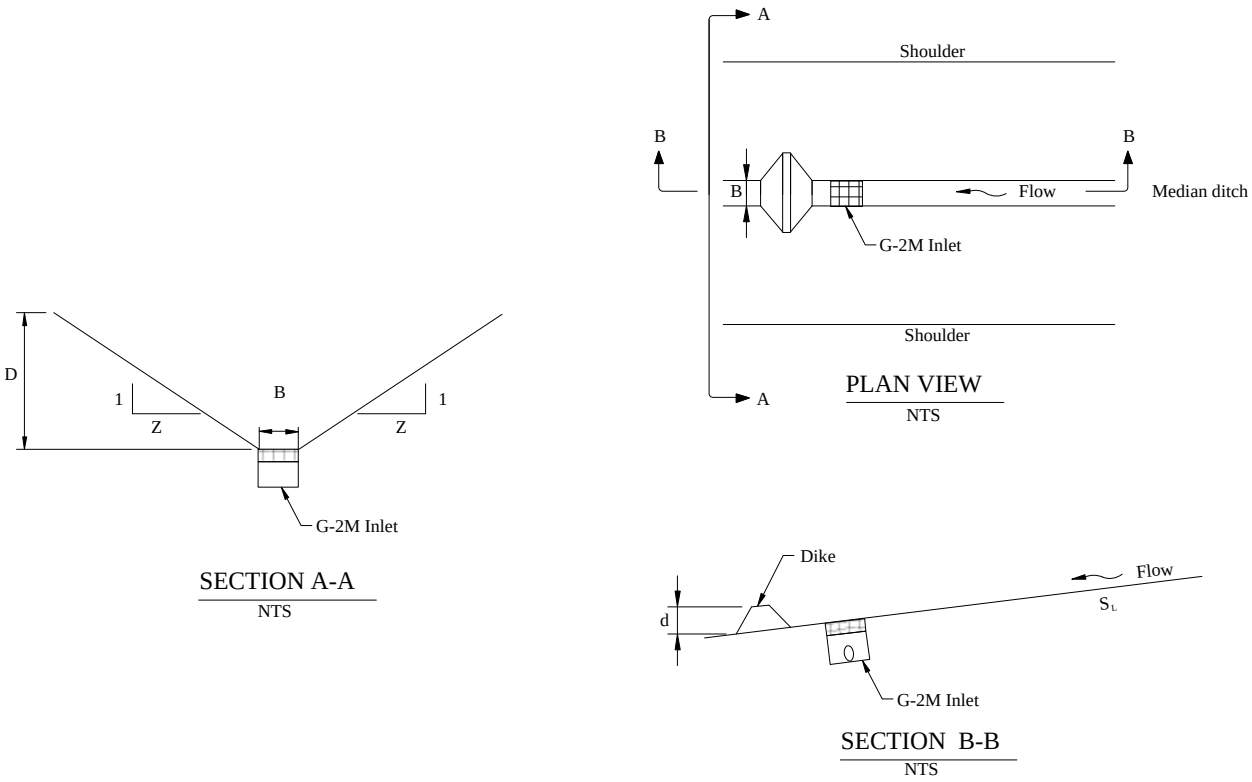
$Q_b$ = 8.6 cubic feet per second

## 7.3 Median Drop Inlet (On-grade) Example

### 7.3.1 Determine the dike height required downstream of the grate inlet to provide total interception of ditch flow

The following example uses the conditions provided in Example 7.2.1 to illustrate the procedure of designing a dike downstream of a drop inlet that would provide adequate obstruction to cause total interception of ditch flow.

Sketch:




Given:

- Q = 20 cubic feet per second (10-year peak runoff)
- B = 2.25 feet (channel width)
- $L_g$ = 2.67 feet (type G-2M inlet length)
- $W_g$ = 2.25 feet (type G-2M inlet width)
- n = 0.027 (Manning's roughness coefficient)
- z = 6 (channel side slope)
- $S_L$ = 0.0125 feet per feet (longitudinal slope of channel and roadway)
- The flow in the median ditch is to be intercepted by a drop inlet (type G-2M measuring 2.25 feet by 2.67 feet with a P-1-$^7/_8$ parallel bar grate).

Solution:

**Step 1-** Solve for ($P_g$):

$P_g$ = grate perimeter

$P_g = 2(L_g+W_g)$

$P_g = 2(2.67 + 2.25) = 9.84$ feet

**Step 2-** Solve for (d):

d = depth of flow required for 100 percent interception

$d = [Q_i / (C_w P_g)]^{0.67}$ (Equation 13 or Chart 8, Appendix H)

For 100 percent interception, $Q_i = Q = 20$ cubic feet per second

$C_w = 3.0$

$P_g = 9.84$

$d = [(20) / ((3.0)(9.84))]^{0.67} = 0.77$ feet

An obstruction berm will need to have a minimum height of 0.77 feet for total interception.

## 8.0 Embankment Inlets/Downdrains

There will be locations where disposal of runoff must be provided at the drainage inlet. This is often the case along fill slopes or to intercept water upgrade or downgrade of bridges. The flow intercepted is usually discharged into open chutes or pipe downdrains that terminate at the toe of the fill slope.

Listed below are additional guidelines that need to be addressed when designing these inlets:

- Water quality treatment – runoff collected by these drainage inlets need to address the most stringent local standards or refer to ODOT's water quality design guidelines.
- Energy dissipation – the designer is referred to Chapter 11 to design outfalls meeting ODOT design guidelines.
- Conveyance capacity – the designer is referred to Appendix F for storm pipe design and this section for downdrain design guidelines.

Example problem solutions in other sections of this manual illustrate the difficulty in providing for near total interception on-grade. Grate inlets intercept little more than the flow conveyed by the gutter width occupied by the grate. Combination curb-opening and grate inlets can be designed to intercept total flow if the length of curb-opening upstream of the grate is sufficient to reduce spread in the gutter to the width of the grate used. Depressing the inlets or curb-opening would significantly reduce the length of inlet required. Therefore, the most practical procedure for use where near total interception is necessary are sweeper inlets, increased grate width, or slotted inlets of sufficient length.

*Note: Design charts and procedures in Section 5.0 are applicable to the design of inlets on embankments.*

Drainage piping systems used to convey intercepted flow from inlets to the toe of a fill slope may be open or closed chutes.

Pipe downdrains (closed chutes) are preferable because the flow is confined and cannot cause erosion along the sides. Open chutes are often damaged by erosion from water splashing over the sides of the chute due to oscillation in the flow and from spill over the sides at bends in the chute.

*Note: It is recommended that these types of pipes be buried to minimize bank erosion and eliminate interference with maintenance operations.*

Erosion at the ends of downdrains or chutes can be a problem if not anticipated. To control the potential for pipe outlet erosion the designer is referred to **Chapter 11** for information on energy dissipation guidelines.

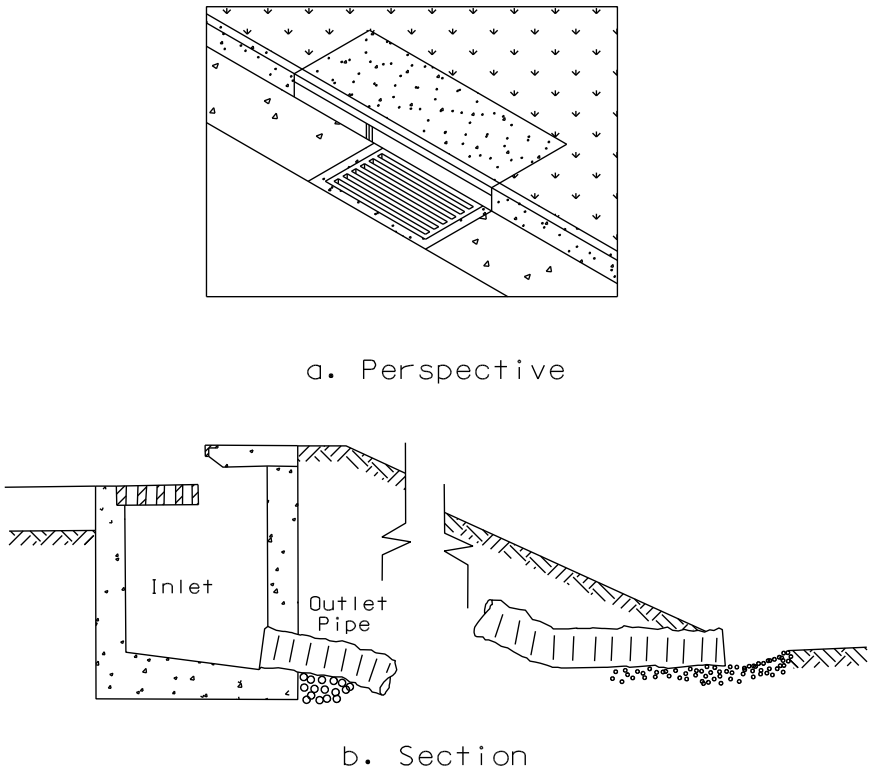



**Figure 8 - Embankment inlet and downdrain.**

The capacity of downdrain pipes are most commonly restricted by inlet control as described in **Chapter 9**. Capacities are shown in Table E assuming the headwater depth does not exceed the diameter of the pipe (HW/D=1). Downdrain pipes controlled by pipe capacity can be sized using Manning's equation as described in **Chapter 8**.

**Table E - Downdrain Pipe Capacity**

| Diameter of Slope Pipe (inches) | Maximum Design Flow (cubic feet per second) |
|---|---|
| 12 | 2.0 |
| 15 | 3.7 |
| 18 | 5.7 |
| 21 | 8.1 |
| 24 | 11.8 |

## 9.0 Scuppers

Scuppers are another way of removing stormwater from roadway pavement and bridge decks. The primary advantage of using scuppers is their ability to intercept flow over a wide section and offer little interference to traffic operations. However, scuppers are not recommended for use in environments where significant sediment or debris loads may be present because they are susceptible to clogging.

*Note: Reference Roadway Concrete Barrier section of ODOT's Standard Drawings (current version) for typical horizontal scuppers used in ODOT storm drainage systems. For a more detailed discussion of horizontal and vertical scuppers, the reader is referred to FHWA's "Design of Bridge Deck Drainage" - Hydraulic Engineering Circular No 21.*

Listed below are additional guidelines that need to be addressed when designing scuppers:

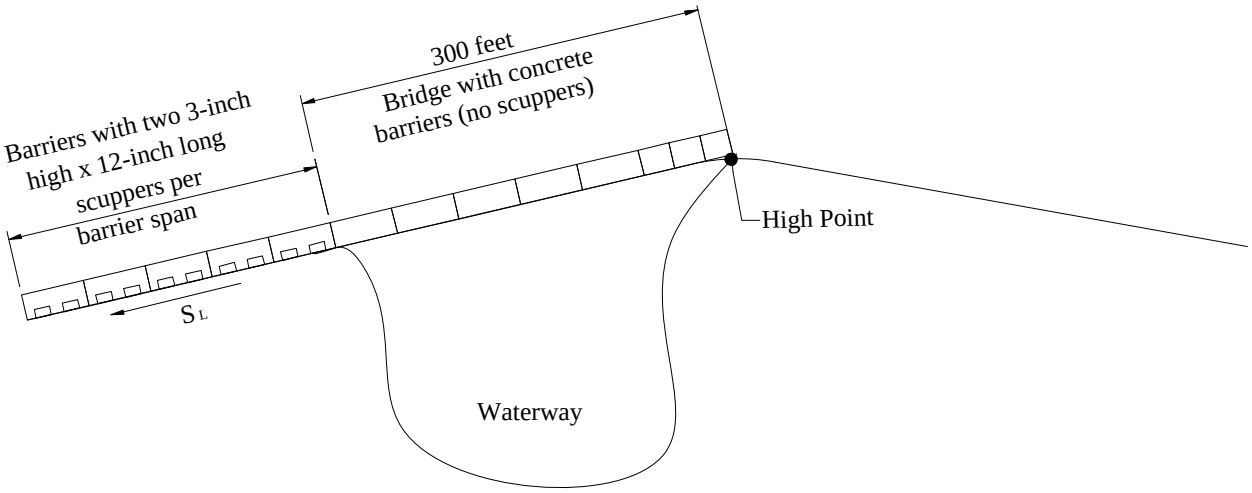
- The equations and figures used for the design of curb-opening inlets should be used in the design of horizontal scuppers.
- The equations and figures used for the design of grate inlets should be used in the design of vertical scuppers.
- Water quality treatment – runoff collected by scuppers needs to address the most stringent local standards or refer to ODOT's water quality design guidelines. Due to environmental regulations it is discouraged to directly discharge from bridge decks using scuppers onto waterways, riparian areas, wetlands, or other sensitive areas. It is good practice to assume all bridge deck runoff will have water quality treatment unless permit or environmental personnel give permission to do otherwise.
- When using inlet design method B, the designer should assume 50 percent clogging which will result in providing twice the calculated required length for flow interception because scuppers tend to plug.
- Energy dissipation – flow exiting horizontal scuppers should be closely evaluated for erosion potential because the flow is not directed to a storm drain system and is often released directly onto down slopes. Reference ODOT's Erosion Control Manual, **Chapter 8,** and **Chapter 11** for recommended methods to prevent erosion.

## 9.1 Scupper (On-grade) Example

The following example illustrates the design of horizontal scuppers.

### 9.1.1 Determine inlet efficiency of concrete barrier scuppers

Sketch:




SCUPPERS
NTS

Given:

- Highway section on-grade (greater than 45 miles per hour)
- $L_b$ = 5.5 feet (length of barrier between scuppers)
- $W_P$ = 31 feet (width of contributing area, 2 – 12-foot lanes with 7-foot shoulder)
- $S_L$ = 0.01 feet per feet
- $S_X$ = 0.02 feet per feet
- n = 0.016 (Manning's coefficient for asphalt placement)
- 10-year design (per Table A), Salem area (Hydrologic Zone 7)
- i = 2.10 inches per hour (Salem area, assume 5-minute $t_c$ using a 10-year storm design)
- Type A curb and gutter (reference Chapter 8)

### 9.1.2 Method A

Criteria:

- T = allowable spread = shoulder width + 0 feet = 7 feet (per Table A)
- $L_S$ = scupper length = (1 foot)
- $h_S$ = 3 inches (scupper height)

Solution:

**Step A.1-** Solve for (Q):

Q = flow rate to first upstream scupper in cubic feet per second

Q = CiA (**Chapter 7**)

A = (300)(31) = 9300 square feet = 0.21 acres

Q = (.90)(2.10)(0.21) = 0.40 cubic feet per second

**Step A.2-** Solve for (T):

T = total spread at upstream scupper (**Chapter 8**)

$$T = \left( \frac{Q_{10} n}{0.56(S_L)^{.5}(S_X)^{1.67}} \right)^{0.375}$$

$$T = \left( \frac{(0.40)(0.016)}{0.56(0.01)^{.5}(0.02)^{1.67}} \right)^{0.375} = 5.1 \text{ feet}$$

Spread at first upstream scupper = 5.1 feet

Allowable spread = 7 feet. Therefore, no deck drains will be needed between the high point and first upstream scupper because maximum spread is less than allowable spread.

**Step A.3-** Solve for ($L_T$):

$L_T$ = length of opening to intercept 100 percent of the gutter flow in feet

$$L_T = 0.6Q^{0.42} S_L^{0.30} \left( \frac{1}{nS_x} \right)^{0.6} \quad \text{(Equation 10)}$$

$$L_T = 0.6(0.40)^{0.42}(0.01)^{0.30} \left( \frac{1}{(0.016)(0.02)} \right)^{0.6}$$

$L_T$ = 13 feet

**Step A.4-** Solve for (E):

E = scupper-opening interception efficiency

$$E = 1 - \left( 1 - \frac{L}{L_T} \right)^{1.8} \quad \text{(Equation 11)}$$

$$E = 1 - \left( 1 - \frac{1.0}{13} \right)^{1.8} = 0.13$$

E = 13 percent

The minimum allowable inlet efficiency when using Method A is 70 percent. Therefore, the design would have to be modified to meet efficiency requirements such as adding additional inlets as necessary, using barriers with no scuppers and using inlets, or using Method B

### 9.1.3 Method B

Criteria:

- For scuppers use a 50 percent clogging factor (per Table A)
- T = allowable spread = shoulder width + 0 feet = 7 feet (per Table A)
- $L_S$ = scupper length = (1 foot)(50 percent) = 0.5 feet
- $h_S$ = 3 inches (scupper height)

Solution:

**Step B.1-** Solve for (Q):

Q = flowrate to first upstream scupper in cubic feet per second

Q = CiA (**Chapter 7**)

A = (300)(31) = 9300 square feet = 0.21 acres

Q = (.90)(2.10)(0.21) = 0.40 cubic feet per second

**Step B.2-** Solve for (T):

T = total spread at upstream scupper (**Chapter 8**)

$$T = \left( \frac{Q_{10} n}{0.56 (S_L)^{.5} (S_X)^{1.67}} \right)^{0.375}$$

$$T = \left( \frac{(0.40)(0.016)}{0.56 (0.01)^{.5} (0.02)^{1.67}} \right)^{0.375} = 5.1 \text{ feet}$$

Spread at first upstream scupper = 5.1 feet

Allowable spread = 7 feet. Therefore, no deck drains will be needed between the high point and first upstream scupper because maximum spread is less than allowable spread.

**Step B.3-** Solve for ($L_T$):

$L_T$ = length of opening to intercept 100 percent of the gutter flow in feet

$$L_T = 0.6 Q^{0.42} S_L^{0.30} \left( \frac{1}{n S_x} \right)^{0.6} \quad \text{(Equation 10)}$$

$$L_T = 0.6(0.40)^{0.42}(0.01)^{0.30}\left(\frac{1}{(0.016)(0.02)}\right)^{0.6}$$

$L_T$ = 13 feet

**Step B.4-** Solve for (E):

E = scupper-opening interception efficiency

$$E = 1-\left(1-\frac{L}{L_T}\right)^{1.8} \quad \text{(Equation 11)}$$

$$E = 1-\left(1-\frac{0.5}{13}\right)^{1.8} = 0.068$$

E = 6.8 percent

Okay, 70 percent minimum efficiency criteria does not apply to Method B

**Step B.5-** Solve for ($Q_i$):

$Q_i$ = intercepted flow by the first scupper

$Q_i$ = EQ = (0.068)(0.40) = 0.0272 cubic feet per second

**Step B.6-** Solve for ($Q_b$):

$Q_b$ = bypass flow from the first scupper

$Q_b = Q - Q_i$

$Q_b$ = 0.40 – 0.0272 = 0.37 cubic feet per second

**Step B.7-** Solve for (Q):

Q = flow to second scupper in cubic feet per second

$Q = Q_b + Q_2$

$Q_b$ = 0.37 cubic feet per second

$Q_2$ = CiA (**Chapter 7**)

$L_b$ = 5.5 feet (length of barrier between scuppers)

A = $L_bW$ = (31 feet)(5.5 feet) = 171 $ft^2$ = 0.004 acres

$Q_2$ = (0.90)(2.10)(0.004) = 0.008 cubic feet per second

Q = 0.37 + 0.008 = 0.378 cubic feet per second

**Step B.8-** Solve for (T):

T = flow spread at second scupper

$$T = \left( \frac{Qn}{0.56(S_L)^{0.5}(S_X)^{1.67}} \right)^{0.375} \quad \textbf{(Chapter 8)}$$

$$T = \left( \frac{(0.378)(0.016)}{0.56(0.01)^{0.5}(0.02)^{1.67}} \right)^{0.375} = 5.0 \text{ feet}$$

Spread at second and subsequent scuppers is approximately 5.0 feet. Allowable spread is 7 feet; this is acceptable because maximum spread is less than allowable spread.

# APPENDIX E
# ACCESS STRUCTURES

## 1.0 Introduction

Access structures are utilized to provide entry to continuous underground storm drains for inspection and cleanout. A manhole or grate inlet may be used for access. Typical locations where access structures should be specified are:

- where two or more storm drains converge,
- where pipe size changes,
- where an abrupt change in alignment occurs, and
- where an abrupt change of the grade occurs.

## 2.0 Spacing

Spacing of access structures on small lines up to and including 36 inches shall not exceed 400 feet. Spacing of access structures on lines larger than 36 inches shall not exceed 900 feet. The spacing shall not exceed 400 feet when inlets are used in lieu of manholes.

## 3.0 Types

A listing of standard ODOT access structures and their associated drawing numbers is presented in Table A.

## Table A ODOT Access Structures

| I.D. | Configuration (Max Pipe Diameter; Max Depth) | ODOT Standard Drawing No. |
|---|---|---|
| Standard MH | Precast Manhole with cone top (24 inch; N/A) | RD336 |
| Shallow MH | Shallow Precast Manhole with flat top and manhole cover. (24 inch; N/A) | RD342 |
| Drop Manhole | Outside drop to manholes added when pipes enter the structure at significantly different elevations. | RD352 |
| Large Precast Manhole | Manhole Base for large pipes (8 feet; N/A). Large manholes are required for pipes from 2.5 feet through 8 feet. | RD346 |
| B | Precast inlet box with flat top and concrete slab cover (24 inch; 4 feet) | RD368 |
| B-SL | Precast inlet box with inclined flat top and concrete slab cover (24 inch; 4 feet) | RD368 |

## 4.0 Sizing

When determining the minimum access hole size required for various pipe sizes and locations, two general criteria must be met.

- The manhole or inlet structure must be large enough to accept the maximum pipe as shown in Table B.
- A 12-inch minimum distance between each pipe should be maintained (measured from the outside of the pipe).

Figure 1 displays a typical layout for a manhole with inflow and outflow pipes.

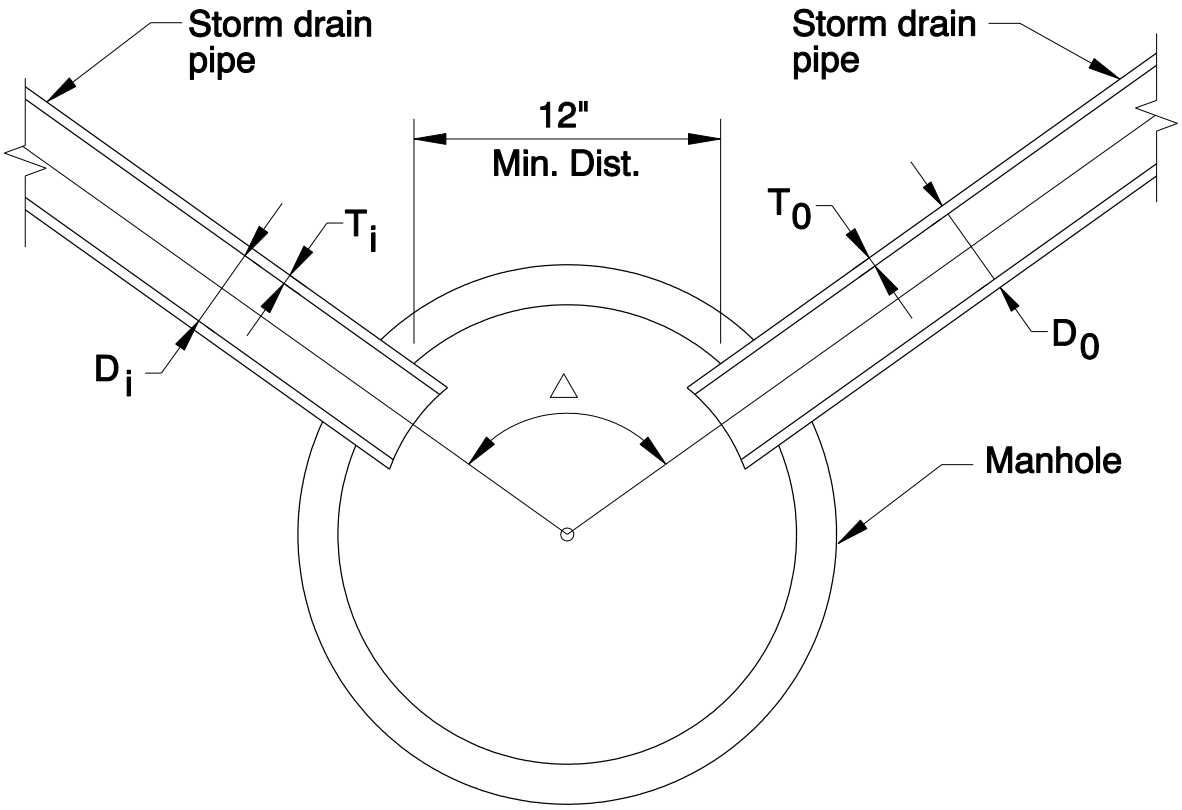



**Figure 1 Manhole Sizing**

Equation 1 is used for determining the proper manhole diameter based on incoming pipe diameters, pipe wall thickness, and the angle between the pipes in degrees.

$$K = \frac{\frac{D_i}{2} + T_i + \frac{D_o}{2} + T_o + 12}{\Delta}$$ (Equation 1)

Where:

$D_i$ and $T_i$ are interior diameter and wall thickness of inlet Pipe, inches

$D_o$ and $T_o$ are interior diameter and wall thickness of outlet Pipe, inches

$\Delta$ = angle between the pipes (measured from the centerline of each pipe), degrees

**Table B Manhole Sizing**

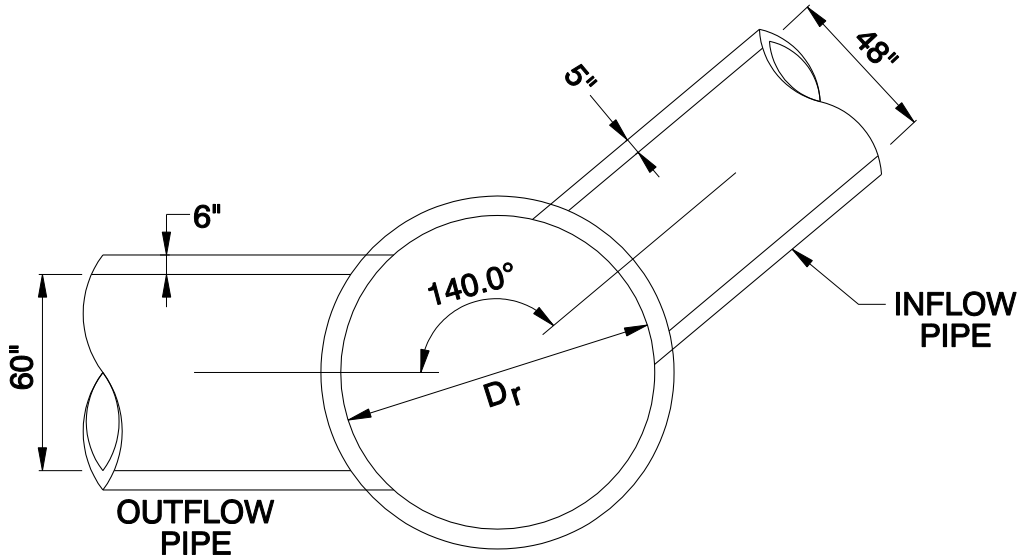
| Manhole Diameter inches | K inches/degree | Maximum Pipe Size inches | Maximum Pipe Size inches |
|---|---|---|---|
| | | AASHTO | ODOT |
| 42 | 0.37 | 28 | 24 |
| 48 | 0.42 | 30 | 24 |
| 60 | 0.52 | 42 | 30 |
| 72 | 0.63 | 48 | 42 |
| 84 | 0.73 | 60 | 54 |
| 96 | 0.84 | 72 | 66 |
| 108 | 0.94 | 84 | 78 |
| 120 | 1.05 | 0 | 90 |
| 126 | 1.10 | 0 | 96 |

## 4.1 Manhole Sizing Example Problem

The following example illustrates using Table B to determine the minimum access hole size required.

### 4.1.1 Determine the minimum access hole size required for the following conditions:

Sketch:




Manhole
NTS

Given:

$D_i$ = 48 inches (inflow pipe diameter)
$T_i$ = 5 inches (inflow pipe wall thickness)
$D_o$ = 60 inches (outflow pipe diameter)
$T_o$ = 6 inches (outflow pipe wall thickness)
$\Delta$ = $140^o$ (pipe deflection at manhole)
$D_r$ = required manhole diameter in inches

Solution:

**Step 1-** Solve for (K):

K = access hole (manhole) sizing coefficient

$$K = \frac{\frac{D_i}{2} + T_i + \frac{D_o}{2} + T_o + 12}{\Delta} \quad \text{(Equation 1)}$$

$$K = \frac{\frac{48}{2} + 5 + \frac{60}{2} + 6 + 12}{140}$$

K = 0.55 inch/degree

**Step 2-** Determine ($D_r$):

$D_r$ = Required Manhole Diameter in inches

Use Table B to obtain appropriate manhole diameter. For K = 0.55 minimum manhole diameter is 72 inches (6 feet). However, the largest pipe has a diameter of 60 inches, and ODOT requires a minimum diameter manhole of 96 inches for this size of pipe. Therefore, a 96-inch manhole should be selected.

For this example, spacing is not critical and the pipe size governs. Had the Δ angle been 115 degrees or less, the following sizing coefficient would be calculated.

$$K(115) = \frac{\frac{48}{2} + 5 + \frac{60}{2} + 6 + 12}{115}$$

K (115) = 0.70

Therefore, from Table B, a minimum access hole diameter of 84-inches (7 feet) would be required based on spacing. However, similar to the example discussed above, the pipe size of 60 inches would have required the use of a 96 inch access hole.

Had the Δ angle been 90 degrees or less, the following sizing coefficient would be calculated:

$$K(90) = \frac{\frac{48}{2} + 5 + \frac{60}{2} + 6 + 12}{90}$$

K (90) = 0.86

Therefore, from Table B, a minimum access hole diameter of 108 inches (9 feet) would be required based on spacing. In this case, spacing is the limiting factor (instead of the pipe diameters used in the previous calculations) and a 9-foot

diameter access hole would be required and would not need to be sized further to accommodate larger pipes.

## 5.0 Blind Connections

Design of completely new storm sewers should not include blind connections. Single inlets may be blind-connected to 18-inch diameter or larger storm drains without constructing a manhole in projects involving retrofitting existing systems. The maximum diameter of intersecting storm drain shall be 6 inches less than the trunk storm drain. When blind connections are used, they should not constrict or obstruct the trunk line.

## 6.0 Drop Manhole

Drop manholes shall be used where the difference in flowline elevations between intersecting storm drains, except inlet runs, exceeds 4 feet. The purpose of a drop manhole is to prevent splashing which might interfere with work in the manhole. They also prevent water from dropping on the workers in the manhole. Drop manholes are only effective during low flows.

## 7.0 Special Manholes

Special manholes shall be designed when conditions prevent the use of standard manholes. For example, a special manhole is needed for pipes larger than 8 feet in diameter, and for splitter manholes to water quality facilities. As discussed above, larger diameter manholes are required for systems with pipe diameters exceeding 2.5 feet.

## 8.0 Other Considerations

- Access structures should not be located in traffic lanes; however, when it is impossible to avoid locating an access hole in a traffic lane, care should be taken to insure it is not in the normal vehicle wheel path.
- Access structures over 20 feet in depth should be avoided due to the limitations of ODOT vactor trucks. Adjacent access to the structure is needed for the vactor to operate to this maximum depth.
- The Oregon Occupational Safety and Health Administration have requirements for manhole rest platforms for structures deeper than 20 feet, and the design should satisfy these requirements. These requirements can be obtained from the Oregon Occupational Safety and Health Administration website at http://www.orosha.org/

- Other applicable confined space design guidelines can be obtained from the Oregon Occupational Safety and Health Administration website noted above.

## 9.0 Utility Issues

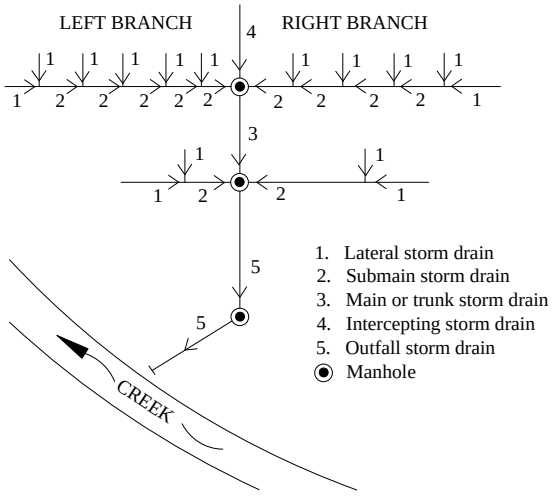
Utility conflicts are presented in Appendix F of this chapter.

# APPENDIX F
# STORM DRAINS

## 1.0 Introduction

This appendix provides information for the planning and hydraulic design of storm drainage systems. The methodology is intended for those with an understanding of basic hydrologic and hydraulic concepts and principles. Hydrologic concepts were discussed in **Chapter 7**. Important hydraulic principles include flow classification, conservation of mass, conservation of momentum, and conservation of energy. These elements were introduced in **Chapter 8**. Guidance on procedures to evaluate energy losses associated with storm drain systems is provided in Appendix G.

A storm drain is that portion of the highway drainage system which receives surface water through inlets and conveys the water through a network of pipes to an outfall (see figure below). It is composed of different lengths and sizes of pipe connected by appurtenant structures. A section of pipe connecting one inlet or appurtenant structure to another is termed a "segment" or "run". Appurtenant structures include inlet structures (excluding the actual inlet opening), access holes, junction chambers, and other miscellaneous structures.




**Typical Closed Conduit Storm Drain System**

Storm drain system design begins after inlet capacity and spacing (Appendix D) have been determined. Then, on a drainage system map, the connecting pipes, access holes, direction of flow, outfall location, existing drainage features and any necessary horizontal/vertical adjustments to avoid existing utilities would be documented. The design would advance into

calculating site hydrology (sub-basin delineation, land use types, runoff coefficients, time of concentration, and peak discharges) and hydraulic analysis (pipe flow capacity and hydraulic grade line). Hand calculations or computer programs and spreadsheets would be used to evaluate and document a storm drain system design.

*Note: Hand calculations are included in this appendix for full and partial full flow design. Hand calculations for surcharged-full flow design are discussed in Appendix G. Step-by-step procedures have been included to aid the designer in understanding the analysis process, but the most efficient means of evaluating storm drain systems is with computer programs.*

## 2.0 Design Guidelines

The following design guidelines to be used in storm drainage system design.

- Storm drainage systems shall not alter the existing drainage pattern without the written permission of the affected property owners. Right-of-way negotiations should cover this aspect of the sewer design and written releases obtained from the property owners where changes in drainage patterns will occur.

- Storm drainage systems shall be designed for the runoff from the level of development present at the time of design with the following exceptions:

    A. Projects within the boundaries of an urban growth or master drainage plan will be designed for runoff from future development subject to the following agreement:

        - The cost of this system will be shared as follows:

            - The Highway Division will pay for a system designed for runoff from the level of development present at the time of design.
            - Additional cost for a system designed for future development is the responsibility of city, county or developer as appropriate.

    B. If the system is not designed for future development, connections to the system will not be allowed unless the following conditions are met:

        - The connecting system collects runoff from areas which naturally drain to the highway storm system.

        - Runoff from the connecting system will be limited to the runoff the storm system was designed to accept from the areas drained by the connecting system.

        - Storm drainage systems shall be designed for future anticipated improvements to the highway which would increase the runoff to the system. Adding curbs to an uncurbed section of highway is an example of an improvement which would increase the runoff to the system.

  - Storm drainage systems shall be designed to operate under full or partial-full flow when conditions permit. Surcharge flow shall be permitted only when back water from the receiving stream will submerge the storm sewer.

- Storm drainage systems shall be designed to operate at full or partial-full flow (open-channel flow) for the design storms described in **Chapter 3**. Open channel design only accounts for major losses (pipe friction losses) as demonstrated in the example problem at the end of this appendix. An increase to the pipe roughness coefficient should be applied to account for minor losses (manhole losses, bends in pipes, expansion and contraction losses, etc.). **Chapter 8**, Appendix A contains hydraulic roughness (Manning's n) values for conduits. When applicable, a minimum, normal, and maximum roughness coefficient is provided for various conduits. Use the maximum roughness coefficient to account for minor losses or the normal coefficient when minor losses will be calculated.

- Storm drainage systems operating under surcharge conditions (pressure flow) shall evaluate the hydraulic grade line and minor losses (manhole losses, bends in pipes, expansion and contraction losses, etc.) as discussed in Appendix G. Adjust inlet sizes/locations and pipe size/location to correct hydraulic grade line problems. Pressure flow design must ensure the hydraulic grade line is 0.5 feet or more below the rim elevation of any drainage structure which may be affected. The energy grade line must also be at or below the rim elevation of any drainage structure which may be affected. A pressure flow example problem is provided in Appendix G.

- Water quality – runoff collected by storm drain systems needs to address the most stringent local standards or refer to ODOT's water quality design guidelines (reference **Chapter 14**).

- Water quantity – runoff collected by storm drain systems needs to address the most stringent local standards or refer to ODOT's water quantity design guidelines (reference **Chapter 12**).

## 3.0 Storm Drain Design Flows

Storm drain systems are designed to convey the discharge recurrence intervals listed in **Chapter 3**. Higher design discharges may be justified for critical installations where considerable danger to the public or expense could occur as a result of failure.

*Note: A higher design discharge is required for storm drains that convey runoff from highway sag points (reference **Chapter 3**).*

## 4.0 Storm Drain Outfalls

All flow from a storm drain system is discharged to an outfall. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the storm water away from the highway. Outfall conditions are an important part of storm drain design as described in this section.

The following system features must be evaluated when designing outfalls:

- The flow line or invert elevation of the proposed storm drain outlet
- Tailwater elevations
- The need for energy dissipation
- The orientation of the outlet structure

The flow line or invert elevation of the proposed outlet should be equal to or higher than the flowline of the outfall. If this is not the case, there may be a need to pump the water to the elevation of the outfall.

*Note: Positive gravity flow is the desired method of discharge. All non-pumping options should be evaluated such as relocating the discharge point to an area that would allow positive gravity flow of stormwater before recommending the construction of a pumping system. Relocation of an outfall should not alter the existing drainage pattern without the written permission of the affected property owners.*

There may be instances in which excessive tailwater conditions cause the hydraulic gradeline to be above grade, which in turn pushes flow out of inlets and access holes, possibly creating hazardous flooding conditions. The potential for this hydraulic condition should be examined. Flap gates placed at the outlet can sometimes alleviate this condition; otherwise, it may be necessary to isolate the storm drain from the outfall by use of a pumping system.

Energy dissipators are used to reduce the velocity and, consequently, the erosion potential of flowing water. Protection is usually required at the outlet of storm drain systems that discharge onto embankments, into natural or unlined channels, or into drainage swales and ponds. Typical storm drain outfall energy dissipators meeting ODOT design guidelines are discussed in Chapter 11.

The orientation of the outfall is another important design consideration. Where practical, the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for excessive erosion. If the outfall structure can not be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of a receiving channel, care must be taken to avoid erosion on the opposite

channel bank. Armoring may be necessary along this scour critical area and should be protected by revetment or suitable biotechnical methods or both (reference **Chapter 15**).

Listed below are additional guidelines that need to be considered when designing outfalls:

- Runoff should discharge from a single outfall. Avoid multiple outlet pipes from a single outfall structure.
- When feasible, the outfall location selected should not be submerged.
- Avoid outfall locations that are subject to high velocities.
- Consideration should be given to erosion protection along the receiving stream and where storm drain outfall discharge occurs.
- Be aware of environmentally sensitive areas when selecting an outfall location.

## 5.0 Open Channel vs. Pressure Flow

Two design approaches may be used for sizing storm drains under the steady uniform flow assumption. The first is referred to as open channel or gravity flow design. To maintain open channel flow, the segment must be sized so that the water surface within the conduit remains open to atmospheric pressure. For open channel flow, flow energy is derived from the flow velocity (kinetic energy), depth (pressure), and elevation (potential energy). If the water surface throughout the conduit is to be maintained at atmospheric pressure, the flow depth must be less than the height of the conduit.

Pressure flow design requires that the flow in the conduit be at a pressure greater than atmospheric. Under this condition, there is no exposed flow surface within the conduit. In pressure flow, flow energy is again derived from the flow velocity, depth, and elevation. The significant difference here is that the pressure head will be above the top of the conduit, and will not equal the depth of flow in the conduit. In this case, the pressure head rises to a level represented by the hydraulic grade line (see Appendix G for a discussion of the hydraulic grade line).

The question of whether open channel or pressure flow should control design has been debated. For a given flow rate, design based on open channel flow requires larger conduit sizes than those sized based on pressure flow. While it may be more expensive to construct storm drainage systems designed based on open channel flow, this design procedure provides a margin of safety by providing additional headroom in the conduit to accommodate an increase in flow above the design discharge. This factor of safety is often desirable since the methods of runoff estimation are not exact, and once placed, storm drains are difficult and expensive to replace.

There may be situations where pressure flow design is a desirable hydraulic condition. For example, on some projects, there may be adequate headroom between the conduit and inlet/access hole elevations to tolerate pressure flow. In this case, significant costs savings may be realized over the cost of a system designed to maintain open channel flow. Also, in some cases it may be necessary to use an existing system which must be placed under pressure flow to accommodate the proposed design flow rates. In instances such as these, there may be advantages in making a cursory hydraulic and economic analysis of a storm drain using both design methods before making a final selection.

Under most ordinary conditions, it is recommended that storm drains be sized based on a gravity flow criteria at flow full or near full. Designing for full flow is a conservative assumption since the peak flow actually occurs at 93 percent of full flow. However, the designer should maintain awareness that pressure flow design may be justified in certain instances. When pressure flow is allowed, special emphasis should be placed on the proper design of the joints so that they are able to withstand the pressure flow.

## 6.0 Hydraulic Capacity

The most widely used formula to calculate the velocity and discharge in storm drains flowing full, but not under pressure flow is the Manning's equation.

The Manning's equation for full flow velocity is:

$$V_{full} = \left[\frac{1.486}{n}\right] R^{2/3} S^{1/2}$$ (Equation 1 and **Chapter 8**)

The Manning's equation for full flow discharge is:

$$Q_{full} = \left[\frac{1.486}{n}\right] A\, R^{2/3} S^{1/2}$$ (Equation 2 and **Chapter 8**)

Where:

- V = mean velocity in feet per second
- Q = rate of flow in cubic feet per second
- n = Manning roughness coefficient (**Chapter 8**)
- D = storm drain diameter in feet
- S = slope of the energy grade line in foot per foot (S = conduit slope for steady uniform flow)
- A = cross-sectional area in square feet
- R = hydraulic radius = A/P in feet
- P = wetted perimeter in feet

A complete discussion of closed conduit (storm drains) hydraulic properties was introduced in **Chapter 8**. This chapter contains nomographs to assist in the solution of the Manning's equation for full flow in storm drains. It also contains a hydraulic elements chart to assist in the solution of the Manning's equation for partial-full flow in storm drains.

## 7.0 Minimum Velocity and Grades

Storm drains should be designed to maintain full-flow pipe velocities of 3.0 feet per second or greater to maintain a self-cleaning velocity in the storm drain to prevent deposition of sediments and subsequent loss of capacity. Minimum slopes required for a velocity of 3 feet per second can be computed using the form of Manning's equation given in Equation 3 or use values provided in Table A.

$$S = 2.9\left[\frac{nV}{D^{0.67}}\right]^2$$ (Equation 3 and **Chapter 8**)

Where:

S = slope of the energy grade line in feet per feet (S = conduit slope for steady uniform flow)
n = Manning's coefficient
V = mean velocity in feet per second
D = diameter of pipe in feet

**Table A Minimum Pipe Slopes**

***(To ensure 3 feet per second velocity in storm drains flowing full)***

| | | Minimum Slopes, foot per foot | | |
|---|---|---|---|---|
| **Pipe Size** | **Full Pipe Flow** | **n** | **n** | **n** |
| Inches | Cubic feet per second | 0.013 | 0.024 | 0.027 |
| 6 | 0.59 | 0.0112 | 0.0381 | 0.0482 |
| 8 | 1.06 | 0.0076 | 0.0259 | 0.0328 |
| 10 | 1.65 | 0.0056 | 0.0192 | 0.0243 |
| 12 | 2.37 | 0.0044 | 0.0150 | 0.0190 |
| 16 | 4.21 | 0.0030 | 0.0102 | 0.0129 |
| 18 | 5.33 | 0.0026 | 0.0087 | 0.0111 |
| 21 | 7.25 | 0.0021 | 0.0071 | 0.0090 |
| 24 | 9.47 | 0.0017 | 0.0059 | 0.0075 |
| 30 | 14.78 | 0.0013 | 0.0044 | 0.0056 |
| 36 | 21.28 | 0.0010 | 0.0034 | 0.0044 |
| 48 | 37.79 | 0.0007 | 0.0023 | 0.0030 |
| 60 | 59.00 | 0.0005 | 0.0017 | 0.0022 |

## 8.0 Minimum Pipe Diameter

The minimum diameter for a storm drain pipe is 12 inches. Storm drain pipes should not decrease in size in a downstream direction regardless of the available pipe gradient.

## 9.0 Curved Alignment

Curved pipe alignment should only be used when it is not practical to change the pipe alignment with manholes. Check with the appropriate District Maintenance Supervisor before specifying a curved pipe alignment.

Long radius bend sections are available from many suppliers and are the preferable means of changing direction in pipes 48 inches and larger. Short radius bend sections are also available

and can be utilized if there is not room for the long radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. Utilizing large access holes solely for changing direction may not be cost effective on large size storm drains.

## 10.0 Storm Drains on Steep Slopes

Storm drain systems that are designed for steep slopes need to take into consideration a number of design elements such as pipe materials to be used, pipe anchors, severe abrasion, high velocities, type of pipe joints, spacing of joints, use of armoring materials, and types of bed load materials flowing in the pipe system. Many of these design features are referenced in **Chapter 5**.

Listed below are additional guidelines that need to be considered when designing storm drains on steep slopes:

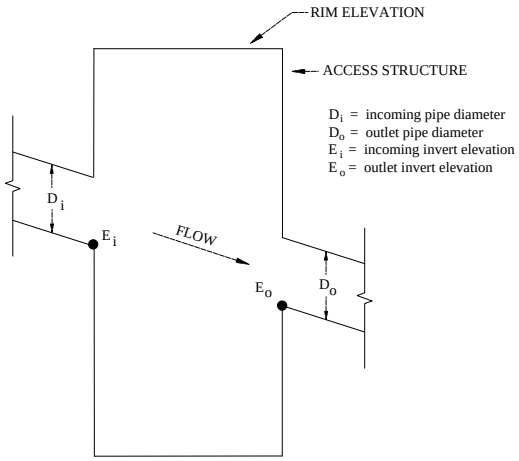
- Concrete pipes should not be used when the slope exceeds 20 percent.
- Differential settlement or movement of joints.
- Metal and plastic pipes which are used on steep slopes that exceed 20 percent should be anchored to prevent movement.
- Refer to ODOT Standard Drawing RD332 for pipe anchor details.
- Energy Dissipation is an important element for storm drain systems on steep slopes. Design elements related to the types and selection of energy dissipators is discussed in **Chapter 11**.

## 11.0 Cleanouts

Permanent cleanouts are not allowed unless concurrence is obtained by the ODOT District maintaining the storm drain system. Temporary clean outs will be evaluated on a case by case basis.

## 12.0 Pipe Connections in a Junction

The crowns of all incoming pipes at a junction should be at the same elevation or higher than the crown of the outlet pipe when the outlet pipe is the same size or larger than the incoming pipes.




**Pipe Connections in a Junction**

$D_o$ is always greater than or equal to the diameter $D_i$ and $E_o$ is less than $E_i$. This is the recommended case for pipe connections to a junction. When the elevation difference in a junction exceeds 4 feet a drop manhole shall be used.

## 13.0 TV Pipe Inspections

After construction of storm drain systems a video inspection should be conducted. It is recommended to inspect 100 percent of the system. A minimum of 25 percent of the total storm drain system is required to be inspected with video equipment. It is recommended that the equipment include a way to record distances and can record the inspection activities onto video tape or DVD.

## 14.0 Utility Conflicts

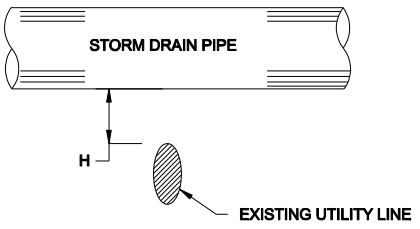
Storm drain alignment, both vertical and horizontal, is often affected by utilities located in close proximity or direct conflict with existing utilities, such as sanitary sewers, fiber optic cable, gas, water, etc. ODOT provides utilities a permit to install facilities within ODOT right-of-way. A condition of the permit requires the utility to relocate if there is a conflict with ODOT facilities. In some instances, ODOT may redesign its facilities to avoid utility disruption.

A utility facility is in conflict if a highway construction element is within two (2) feet of the outside diameter of the utility or is directly over or under for trenching, drilling or construction activities. The utility owner is required to provide information about these clearances as illustrated in the figure below (examples 1 and 2). The vertical distance (H) may vary depending on the utility type; therefore, early design coordination between ODOT and utility owner is necessary to minimize project delays caused by utility relocation work.

The clearances can be reduced, or sometimes eliminated, if casings or other protection measures are installed around the utility as illustrated in the figure below (examples 3 and 4). Listed below are additional guidelines that need to be considered when casing or proposing other protection measures:

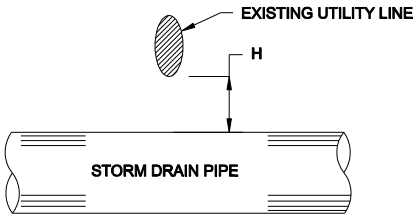
- The design capacity of the storm drain system must be maintained and the potential for blockage by debris must be minimized when a carry-through pipe is used in an access manhole.
- The carry-through pipe sleeve through an access manhole shall be ductile iron pipe. Reference ODOT standard drawing RD354.
- Coordinate casing or other protection measure with utility owner

In all cases, the final storm drain design should not be made until affected utilities are located. In some instances, pothole excavation and survey is needed to determine the exact utility location. This is an important factor in storm drain design because unanticipated utility conflicts are time consuming and expensive when encountered during construction. Furthermore, accidentally encountering gas lines, high voltage power lines or other utilities during construction could have deadly consequences.



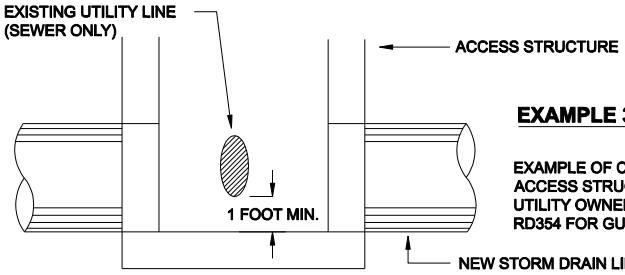

**EXAMPLE 1**

EXAMPLE OF STORM DRAIN CROSSING ABOVE AN EXISTING UTILITY. THE VERTICAL DISTANCE (H) MAY VARY DEPENDING ON THE TYPE OF UTILITY. NO PROTECTION MEASURES ARE REQUIRED WHEN (H) IS GREATER THAN OR EQUAL TO A MINIMUM CLEARANCE SPECIFIED BY THE UTILITY OWNER. CASING, ACCESS STRUCTURE OR OTHER PROTECTION MEASURES ARE REQUIRED WHEN VERTICAL DISTANCE (H) IS LESS THAN THE MINIMUM CLEARANCE SPECIFIED BY THE UTILITY OWNER.



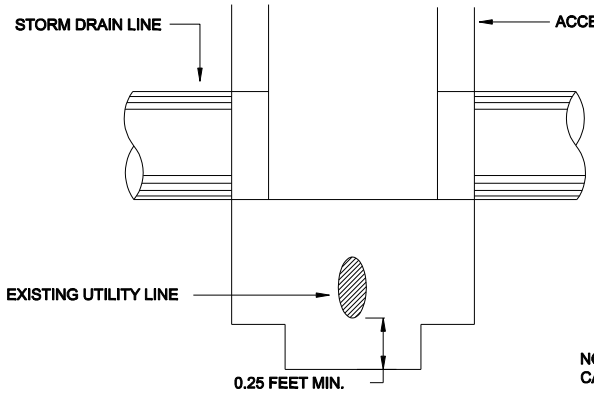

**EXAMPLE 2**

EXAMPLE OF STORM DRAIN CROSSING BELOW AN EXISTING UTILITY. THE VERTICAL DISTANCE (H) MAY VARY DEPENDING ON THE TYPE OF UTILITY. NO PROTECTION MEASURES ARE REQUIRED WHEN (H) IS GREATER THAN OR EQUAL TO A MINIMUM CLEARANCE SPECIFIED BY THE UTILITY OWNER. CASING, ACCESS STRUCTURE OR OTHER PROTECTION MEASURES ARE REQUIRED WHEN VERTICAL DISTANCE (H) IS LESS THAN THE MINIMUM CLEARANCE SPECIFIED BY THE UTILITY OWNER.




**EXAMPLE 3**

EXAMPLE OF CONFLICT UTILITY CROSSING INSIDE A STORM DRAIN ACCESS STRUCTURE BARREL. COORDINATE CONFLICT CROSSING WITH UTILITY OWNER AND/OR REFERENCE ODOT STANDARD DRAWING NO. RD354 FOR GUIDANCE.




**EXAMPLE 4**

EXAMPLE OF CONFLICT UTILITY CROSSING THROUGH THE BOTTOM SLAB OF A STORM DRAIN ACCESS STRUCTURE. COORDINATE CONFLICT CROSSING WITH UTILITY OWNER.

NOTE: CONSULT UTILITY OWNER FOR MINIMUM CLEARANCE, CASING, OR OTHER PROTECTION MEASURE INFORMATION.

## Storm Drain Conflicts

## 15.0 Full and Partial-full Flow

The following is the procedure to design storm drains for full and partial-full flow. The Storm Drain Design Sheet shown on Figure 1 should be used to tabulate the hydraulic data for the design.

**Step 1-** Prepare a map or maps (**Chapter 6**) showing the entire drainage basin contributing storm water to the system. Layout the proposed storm sewer indicating the location of each manhole and inlet.

**Step 2-** Delineate the basin drainage area contributing to each inlet, then divide these areas into subareas having similar RUNOFF COEFFICIENTS (ground cover and slope).

*Note: Layout of the storm drain system, drainage basin map and storm sewer design sheets must be documented as in **Chapter 4**.*

**Step 3-** Starting at the upper end of the system, label each of the subareas with an index number (1, 2, 3, etc.) and circle each.

**Step 4-** Enter in Column 1 of the STORM SEWER DESIGN SHEET (Figure 1) the station for the inlet of the pipe run under consideration.

**Step 5-** Enter in Column 2 the index number for each subarea.

**Step 6-** Enter in column 3 the area in acres of each subarea.

**Step 7-** Select the RUNOFF COEFFICIENT (**Chapter 7**) that best fits the ground cover condition and slope for each subarea. Enter this coefficient in Column 4 opposite the corresponding subarea.

**Step 8-** Multiply AREA in Column 3 by the RUNOFF COEFFICIENT in Column 4. Enter in Column 5.

**Step 9-** Sum all the EQUIVALENT AREAS in Column 5 for the pipe section under consideration. Enter sum in Column 6 opposite the last subarea in Column 5.

**Step 10-** Enter in Columns 1a and 1b (same forms) the station and node I.D. for the outlet of the pipe under consideration.

**Step 11-** Determine the TIME OF CONCENTRATION for the first pipe run only. For overland flow use the Kinematic Wave Equation (see **Chapter 7**) and for channel flow use the Manning Equation for open-channel flow (see **Chapter 8**). In the case of

paved areas, the minimum time of concentration, $T_c$ shall be 5 minutes while the minimum $T_c$ for nonpaved areas shall be 10 minutes.

**Step 12-**. Enter in Column 7 the value in minutes determined in Step 11.

**Step 13-** Add all TIME OF CONCENTRATION or FLOW TIME values in Column 7 for previous pipe runs on the same sewer branch. Enter in Column 8.

**Step 14-** Determine the RAINFALL INTENSITY from the appropriate Intensity-Duration Curve (see **Chapter 7**). Enter in Column 9. NOTE: RAINFALL DURATION is considered equal to the value in Column 8.

**Step 15-** Multiply the RAINFALL INTENSITY in Column 9 by the DRAINAGE AREA in Column 6 to determine the DESIGN DISCHARGE. Enter in Column 10.

**Step 16-** Enter in Column 11 a trial slope for the sewer pipe. Usually, the slope of the roadway can be used.

**Step 17-** Hydraulic pipe characteristics, roughness coefficients, and equations are located in **Chapter 8** to aid manual calculations. Also, nomographs for several types of pipes are located in **Chapter 8**. When the appropriate nomograph is available, determine the size of pipe by laying a straight edge between the DISCHARGE and SLOPE scales using the values in Column 10 and Column 11. The appropriate size pipe will be read directly above the straight edge on the PIPE DIAMETER scale. Enter this size in Column 12.

**Step 18-** Adjust the straight edge on the nomograph such that it lies on the SLOPE (Column 11) and the PIPE DIAMETER (Column 12).

**Step 19-** Read the pipe CAPACITY at full flow on the DISCHARGE scale. If this value is 10 percent larger than the DESIGN FLOW in Column 9, then the pipe slope should be flattened if feasible. The slope at which the pipe will just flow full can be found by laying the straight edge between the DESIGN DISCHARGE and CULVERT DIAMETER scales, and reading the new slope on the SLOPE scale.

**Step 20-** Re-enter in Column 11 the new pipe slope if an adjustment is made.

**Step 21-** Enter in Column 13 the pipe CAPACITY for the slope selected.

**Step 22-** With the straight edge on the DESIGN SLOPE and PIPE DIAMETER scales, read the DESIGN VELOCITY. The minimum DESIGN VELOCITY is 3 feet per second. Enter in Column 14.

**Step 23-** Enter in Column 15 the LENGTH of sewer pipe between stations listed in Column 1.

**Step 24-** Divide the LENGTH in Column 15 by the VELOCITY in Column 14, then divide the resultant by 60 to determine the FLOW TIME in minutes. Enter this value in Column 7.

**Step 25-** Multiply the INVERT SLOPE in Column 11 by the LENGTH in Column 15 to determine the FALL. Enter in Column 16.

**Step 26-** Enter in Column 17 the INVERT ELEVATIONS for both the inlet and outlet ends of the pipe.

**Step 27-** Enter in Column 18 the TOP OF MANHOLE ELEVATION at both the inlet and outlet ends of the pipe.

**Step 28-** Enter in Column 19 REMARKS pertinent to the pipe being considered.

**Step 29-** Repeat procedure for next run of pipe except that the TIME OF CONCENTRATION in Step 11 (Column 7) is replaced by FLOW TIME from Step 24 for the succeeding pipe runs.

## 16.0 Storm Drain Design (Single Branch Full Flow) Example

The following example demonstrates the design process and calculations for a single branch storm drain flowing full. It will be designed to convey stormwater from the drainage basins delineated on Figure 2 (drainage map) to Mill Creek in Salem, Oregon.

**PART A**

The following design steps outline the proposed storm drain system's characteristics (i.e. horizontal/vertical system alignment, delineating drainage basins, preparing a drainage map, collection structure selection, hydrology calculations, etc.) which are necessary prior to pipe sizing design:

**Step A.1-** Determine the inlet type (**Chapter 13**) and curb type (**Chapter 8**) which would provide the most economical method of collecting storm water from the highway and side areas. The CG-2 inlet was used for this design because of the considerable water and debris anticipated and also because it is a multiple lane highway. Calculate the inlet spacing using the methods shown in the inlet spacing and capacity appendix. The following table shows a comparison between curb type and the estimated number of CG-2 inlets from station 0+00 to station 10+00. Zero bypass was used as a design criterion for the storm drain.

| Curb Type | Inlet Number CG-2 |
|---|---|
| Standard Curb | 13 |
| Mountable Curb | 50 |
| Curb & Gutter | 50 |

The Standard Curb would be the most economical based on the estimated number of inlets and will be used for this design.

**Step A.2-** Locate the inlets (see Figure 3) and delineate the drainage area to each storm water collection point. An alternative to the manhole with inlet would be a manhole and a CG-2 inlet located close by. This inlet would be connected into the manhole. The manhole and inlet alternate was not selected based on its additional cost. Numerically identify each delineated drainage area. Connect the inlets as shown on Figure 3 with lines to represent pipe alignment.

**Step A.3-** Locate and sketch the manhole, connecting the outfall pipe and the outfall as shown on Figure 3. The outfall should direct the storm water in the same general direction as the creek. The manhole should be located on the creek bank in a protected location. The outfall should be protected from potential flood damage.

**Step A.4-** Enter in Column 1a and 1b of the "Storm Sewer Design Sheet" (Figure 1) the station and node I.D.s of the inlets, the manhole with inlet grate, the manhole, and the outfall shown on Figure 3.

**Step A.5-** Enter the connecting pipe lengths from Figure 3 onto **Figures 4, 5 and 6** (Column 15).

**Step A.6-** Determine highwater elevation and average creek velocity at the outlet location shown on Figure 3 for the 50-, 25-, 10- and 5-year floods. The Hydraulics Engineer would be one source of obtaining the information listed in the table below.

Mill Creek Data

| Recurrence Interval | Highwater Elevation at Outlet, feet | Average Velocity in Creek at Outlet, feet per second |
|---|---|---|
| 50-Year | 83.0 | 8.0 |
| 25-Year | 81.5 | 7.5 |
| 10-Year | 80.0 | 7.0 |
| 5-Year | 77.0 | 6.0 |

**Step A.7-** The invert elevation of 77.85 feet was entered in Column 17 for the outlet end of the 90-foot long outfall pipe. From a field surveyed cross-section near the outfall, the bottom of the creek was estimated to be at elevation 77.85 feet. An outfall elevation of 77.85 feet was selected for this design and entered in Column 17. The outfall will be submerged by the creek; however, the rest of the system will not.

**Step A.8-** Enter the delineated upstream drainage area to each **trunk line** collection point in Columns 2, 3, 4, 5, and 6.

*Note: A common error in storm sewer design is to arbitrarily assume the limits of the contributing side drainage. The best way to determine the contributing drainage area is to walk the project and observe the limits of side drainage. This is particularly effective during a rain storm.*

**Step A.9-** Using information provided on Figure 2, determine the finish grade of the roadway above the trunk line and enter in Column 18 of **Figures 4, 5, and 6** using the relationship given below. The storm drain trunk line will be located 42 feet right of centerline.

Top of Manhole or Inlet Grate Elevation = Finish Grade at Center Line - (Cross Slope feet/feet)(Highway Width / 2)

= Finish Grade at Center Line - (0.02)(40 feet) + (0.08)(2 feet)

Top of Manhole/Inlet Elevation = Finish Grade at Center Line - 1.0 feet

The results are shown in the table below:

| Station | Finish Grade at Center Line, feet | Top of Inlets or Manhole Elevation, feet (Column 18) |
|---|---|---|
| 0+00 Rt | 110.1 | -- |
| 3+00 Rt | 105.9 | 104.9 |
| 4+50 Rt&Lt | 103.8 | 102.8 |
| 6+00 Rt&Lt | 101.7 | 100.7 |
| 7+00 Rt&Lt | 100.3 | 99.3 |
| 8+00 Rt&Lt | 98.9 | 97.9 |
| 9+00 Rt&Lt | 97.5 | 96.5 |
| 10+00 Rt&Lt | 96.1 | 95.1 |
| | | Elev. Natural Ground |
| 10+00 (392 feet Rt) | -- | 95.5 |

**Step A.10-** Determine the minimum starting invert elevation (Column 17) at the beginning of **the trunk line and each lateral** using the following relationship. Assume 12-inch diameter pipe since it is the smallest allowable pipe size for storm drains. Smooth walled PVC pipe with a minimum allowable cover of one foot will be used with this design. To account for minor losses, a slightly higher roughness coefficient will be used. A roughness coefficient for concrete (n = 0.013) will be used for this design.

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Minimum Invert Elevation | = | Top of Manhole or Inlet Elevation | - | Pavement Thickness | - | Pipe Diameter | - | Shell Thickness of Pipe | - | Minimum Cover |
| Starting Invert Elevation | = | Top of Manhole or Inlet Elevation | | 1.0 feet | - | 1.0 feet | - | 2.5/12 | - | 1.0 feet |
| Starting Invert Elevation | = | Top of Manhole or Inlet Elevation | | 3.21 feet | | | | | | |

Enter the following minimum starting Invert Elevations in Column 17:

| Station | Top of Manhole or Inlet Elevation, feet | Starting Invert Elevation, feet | Remarks | Comment |
|---|---|---|---|---|
| 3+00 Rt | 104.9 | 101.69 | TRUNK | Fig. 4, NODE 2 |
| 4+50 Lt | 102.8 | 99.59 | LATERAL | Fig. 5, NODE 1 |
| 6+00 Lt | 100.7 | 97.49 | LATERAL | Fig. 5, NODE 4 |
| 7+00 Lt | 99.3 | 96.09 | LATERAL | Fig. 5, NODE 6 |
| 8+00 Lt | 97.9 | 94.69 | LATERAL | Fig. 6, NODE 8 |
| 9+00 Lt | 96.5 | 93.29 | LATERAL | Fig. 6, NODE 10 |
| 10+00 Lt | 95.1 | 91.89 | LATERAL | Fig. 6, NODE 12 |

**Step A.11-** Enter a 5-minute time of concentration in Columns 7 and 8 for the beginning of the trunk line (**Figure 4**) and for each lateral (**Figures 5 and 6**). Calculations for the 5-minute time of concentration are shown below for overland and gutter flow over pavement. The gutter is classified as Standard Curb (**Chapter 8**).

Figure 7 shows the estimated drainage paths for overland and gutter flow. There are two drainage paths which need to be investigated. The first path is the sum of $L_o$ and L. The second path is the sum of $L_1$ and $L_2$.

Given:

n = 0.016 (Manning's coefficient for asphalt pavement)
S = 0.014 feet/feet (roadway longitudinal slope)
$S_{x3}$ = 0.02 feet/feet (roadway cross slope)
$S_w$ = 0.083 feet/feet (gutter cross slope)
W = 2 feet (depressed gutter width)
I = 2.12 inches/hour (Zone 7, **Chapter 7**)
Q = CIA (Chapter 7) in cubic feet per second
Q = (0.9)(2.12)(42 x 300 + 100 x ll0) / 43,560
= 1.03 cubic feet per seconds

(The gutter flow variables are defined in **Chapter 8**)

Time of concentration overland flow:

$$T_c = 0.93\left[\frac{L^{0.6}n^{0.6}}{I^{0.4}S^{0.3}}\right] \quad \text{(Chapter 7)}$$

$$T_c = 0.93\left[\frac{(42)^{0.6}(0.016)^{0.6}}{(2.12)^{0.4}(0.02)^{0.3}}\right]$$

= 1.75 minutes for overland path $L_o$

$$T_c = 0.93\left[\frac{(110)^{0.6}(0.016)^{0.6}}{(2.12)^{0.4}(0.01)^{0.3}}\right]$$

= 3.85 minutes for overland path $L_1$

Estimate flow widths with Standard Curb (**Chapter 8**).

By trial and error using the equations from **Chapter 8** for a Standard Curb:

$$Q = \frac{(0.56)S^{0.5}}{n}\left[\frac{d_2^{2.67}}{S_{x3}} + \frac{\left(d^{2.67} - d_2^{2.67}\right)}{S_w}\right] \quad \text{(Chapter 8)}$$

$d_2 = S_{x3}T_3$

$d = S_{x3}T_3 + WS_w$

$$Q = \frac{(0.56)(0.014)^{0.5}}{0.016}\left[\frac{d_2^{2.67}}{S_{x3}} + \frac{\left(d^{2.67} - d_2^{2.67}\right)}{S_w}\right]$$

$$Q = 4.733\left[\frac{d_2^{2.67}}{S_{x3}} + \frac{\left(d^{2.67} - d_2^{2.67}\right)}{S_w}\right]$$

$d_2 = 0.02T_3$

$d = d_2 + (2)(0.083) = d_2 + 0.166$

Assume: $T_3 = 2.5$ feet

$d_2 = 0.02\ (2.5) = 0.05$ feet

$d = 0.05 + 0.166 = 0.216$ feet

$$Q = 4.733\left[\frac{0.05^{2.67}}{0.02} + \frac{\left(0.216^{2.67} - 0.05^{2.67}\right)}{0.083}\right]$$

$Q = 1.01$ cubic feet per second -- close enough

Total flow width = $T_3 + W = 2.5$ feet + 2 feet = 4.5 feet

$$A = \frac{1}{2}\left[w\left(d + d_2\right) + \frac{d_2^2}{S_{x3}}\right]$$

$Q = VA$, discharge, cubic feet per second

$V = Q / A$, velocity, feet per second

$$V = \frac{2(1.03)}{\left[2(0.216 + 0.05) + \frac{0.05^2}{0.02}\right]}$$

$V = 3.13$ feet per second

$T_c = 300 /(3.13)(60) = 1.60$ minutes for channel flow path L

$T_c = 100 /(3.13)(60) = 0.53$ minutes for channel flow path $L_2$

Determine longest time of concentration

Path $L_o + L$: $T_c = 1.75+1.60 = 3.35$ minutes

Path $L_1 + L_2$: $T_c = 3.85+0.53 = 4.38$ minutes

Path $L_1 + L_2$ yields the controlling time of concentration; however, the minimum allowable time of concentration is 5 minutes. Consequently the starting time of concentration is 5 minutes. A similar analysis should be done for all storm sewers to determine the starting time of concentration. When in doubt, use 5 minutes as this will yield the most conservative design.

*Note: Refer to **Chapter 7** for additional discussion on determining time of concentration.*

**PART B**

The following design steps outline the procedure for sizing storm drains. The design analysis is documented on Figures 4, 5, and 6.

**Step B.1-** Obtain the average 10-year rainfall intensity from **Chapter 7** for a 5.0 minute time of concentration. The 10-year rainfall intensity for Salem is 2.12 inches per hour and would be entered in Column 9 in **Figures 4, 5, and 6**.

**Step B.2-** Multiply Column 9 by Column 6 and enter the results in Column 10 in **Figures 5 and 6**.

**Step B.3-** Using Figure 8 by drawing a line through the points for a 12-inch pipe and 3.0 feet per second velocity on the nomograph, determine the pipe invert slope, capacity flowing full, and velocity flowing full and enter the results in Columns 11, 13 and 14 (**Figures 5 and 6**). Verify that the assumed 12-inch pipe has capacity flowing full which is at least equal to the calculated design discharge in Column 10. In this example all lateral pipes have sufficient capacity.

**Step B.4-** Determine and enter the fall in Column 16 for each of the **laterals** using the following relationship:

Fall in feet = Invert Slope in feet/feet x length in feet

For station 4+50 Lt.
Column 16 = Column 11 / 100 x Column 15
Fall = (0.45 / 100)(84) = 0.38 feet

**Step B.5-** The outlet invert elevations for the upstream structures were determined in Step A.10. Now determine the inlet invert elevations of the 12-inch diameter laterals

at the downstream structures by subtracting Column 16 from Column 17. Enter the results in Column 17, **Figures 5 and 6**.

*Note: Steps B.6 through B.7 are to be completed for the outlet at 4+50Rt before completing the remainder of columns.*

**Step B.6-** Determine and enter the fall (Column 16) for the outlet of the **trunk line** at Station 4+50Rt. The fall also includes enough depth so that the crown of the lateral pipe will be at or above the crown of the trunk line using the following relationship. Enter the result in Column 16, **Figure 4**.

$$\text{Minimum Fall (Column 16)} = \text{Longitudinal Highway Slope \% (Figure 5)} \times \frac{\text{Length (Column 15)}}{100} + \text{Lateral Fall (Column 16)}$$

Minimum fall (Column 16) = 1.4 percent x 150 feet / 100 + 0.38 = 2.48 feet

*Note: The addition of the lateral fall value is only for the outlet at 4+50Rt.*

**Step B.7-** Subtract Column 16 from Column 17 (previous station/node) in **Figure 4** and enter results in Column 17 for the present station/node.

**Step B.8-** Calculate the Invert Slope and enter the results in Column 11 in **Figure 4** using the following relationship:

Invert Slope = (Fall, Column 16)(100)/(Length, Column 15)
= 2.48 x l00 / 150 = 1.653 percent

**Step B.9-** Using Figure 8 by drawing a line between the pipe size selected (Column 12) and slope (Column 11) in **Figure 4,** determine the pipe capacity flowing full and velocity flowing full and enter the results in Columns 13 and 14 for **station 4+50Rt**.

Be sure to select a pipe size which has capacity flowing full which is at least equal to the estimated design discharge in Column 10. The 12-inch pipe has capacity (4.55 cubic feet per second) which is greater than the design discharge (1.04 cubic feet per second). Also check to make sure the velocity is greater than or equal to 3.0 feet per second.

**Step B.10-** Calculate and enter in Column 7 the flow time through the **trunk line** using the following relationship.

$T = L/V(60)$ (**Chapter 7**)

$T = 150/(5.84)(60) = 0.4$ minutes

**Step B.11-** Add the flow time calculated above to the previous total time of concentration and enter the result in Column 8, **Figure 4**.

0.4 + 5.0 = 5.4 minutes

**Step B.12-** Obtain the 10-year rainfall intensity corresponding to a duration of 5.4 minutes using graphs in **Chapter 7**. Enter 2.08 inches per hour in Column 9, (**Figure 4**).

**Step B.13-** Multiply Column 9 by Column 6 in **Figure 4** and enter 2.81 cubic feet per second in Column 10 of Figure 4.

**Step B.14-** Assume the **trunk line** is parallel to the profile grade of the highway and enter 1.4 percent in Column 11 from the outlet side of the inlet at station 4+50Rt to the inlet side of the manhole with inlet at station 10+00Rt.

**Step B.15-** Estimate and enter in Column 16 in **Figure 4** the fall from station 4+50Rt (out) to station 6+00Rt (in) as shown below:

Fall = (1.40)(150) / 100 = 2.10 feet

**Step B.16-** Using Figure 8 by drawing a line between pipe size selected (Column 12) and slope (Column 11) in **Figure 4**, select a pipe size that has capacity flowing full which is at least equal to the estimated design discharge (2.81 cubic feet per second) in Column 10. The 12-inch pipe has capacity of 4.25 cubic feet per second which is greater than the design discharge of 2.81 cubic feet per second. Enter 4.25 and 5.40 in Columns 13 and 14, respectively. Also check the velocity to ensure a minimum of 3.0 feet per second.

**Step B.17-** Repeat steps 10 through 16 for the remainder of the system.

**Step B.18-** The available fall of 11 feet between station 10+00Rt and 10+00 (350 feet Rt) was approximated by subtracting elevation 80.0 from elevation 91.0. This pipe could, therefore, be laid on a maximum grade of approximately 2.5 percent. The natural bank elevation where the proposed manhole would be located is higher (95.5 feet) than the top of manhole with inlet at station 10+00Rt (94.7 feet). This pipe must have a minimum flowing full capacity of 13.84 cubic feet per second.

Using Figure 8, an 18-inch pipe with a slope of 1.7 percent would have a capacity of 13.84 cubic feet per second. Full flow velocity is 7.9 feet per second.

**Step B.19-** Enter the design information for the 18-inch pipe into Columns 11, 12, 13, 14, 16 and 17 of **Figure 4 for station 10+00 Rt.**

**Step B.20-** Enter in Column 14 of **Figure 4 for station 10+00 (350 feet) Rt**. an outlet velocity of 3.0 feet per second for the 90-foot outfall. This velocity will retain the

solid material in motion (see Section 7) and still prevent an erosion problem at the outlet of this outfall.

**Step B.21-** Determine the pipe size between station **10+00 (350 feet) Rt** to the outfall. Using Figure 8, determine the slope and pipe size necessary to convey the design discharge, 13.8 cubic feet per second, with a velocity of 3 feet per second. This requires a 30-inch pipe laid on a 0.13 percent slope. Enter the invert slope (0.13 percent), pipe size (30-inch) and capacity flowing full (14.6 cubic feet per second) into Columns 11, 12 and 13, respectively in **Figure 4**.

**Step B.22-** Determine and enter the Fall, Column 6 in **Figure 4**, for the outfall using the following relationship.

Fall = 0.13/100 (90 feet) = 0.12 feet

**Step B.23-** Add and enter the invert elevation, Column 17, **Figure 4**, for the inlet end of the outfall using the following relationship.

Invert Elevation = 77.85 feet + 0.12 feet
= 77.97 feet (invert elevation out at node 14)

FIGURE #1
STORM DRAIN DESIGN SHEET
FULL AND PARTIAL-FULL FLOW

PROJECT: DESIGN FREQUENCY: 50-year* 10-year DESIGNED BY: DATE:

| Station | Node I.D. | Drainge Area | | Runoff Coeff. | Equiv. Area for 100% Runoff | Accumlative Impervious Drainage Area | Time of Concent. or Flow Time | Total Time of Concent. | Average Rainfall Intensity | Design discharge | Invert Slope | Pipe Size | Capacity Flowing Full | Velocity Flowing Full | Length | Fall | Invert Elevation | Top of Manhole or Inlet Elevation | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Index No. | Area | | | | | | | | | | | | | | | | |
| | | | A | C | CA | $\Sigma$CA | $T_c$ | $T_c$ | I | Q | S | D | $Q_f$ | $V_f$ | L | | | | |
| | | | Acres | | Acres | Acres | Minutes | Minutes | in/hr | cfs | % | Inches | cfs | fps | feet | feet | feet | feet | |
| 1a | 1b | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | |

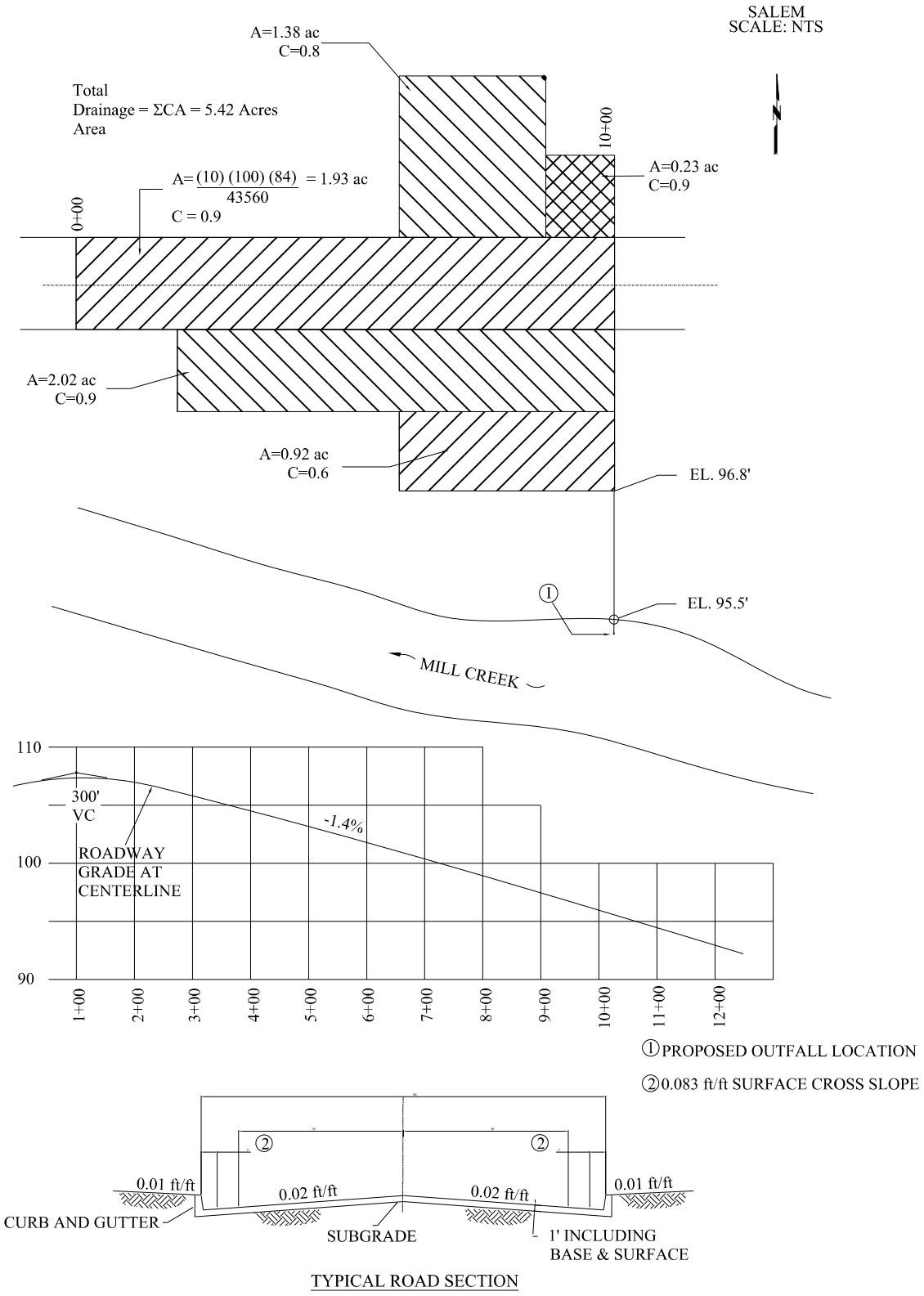



FIGURE #2
DRAINAGE MAP AND ROADWAY GRADE INFORMATION

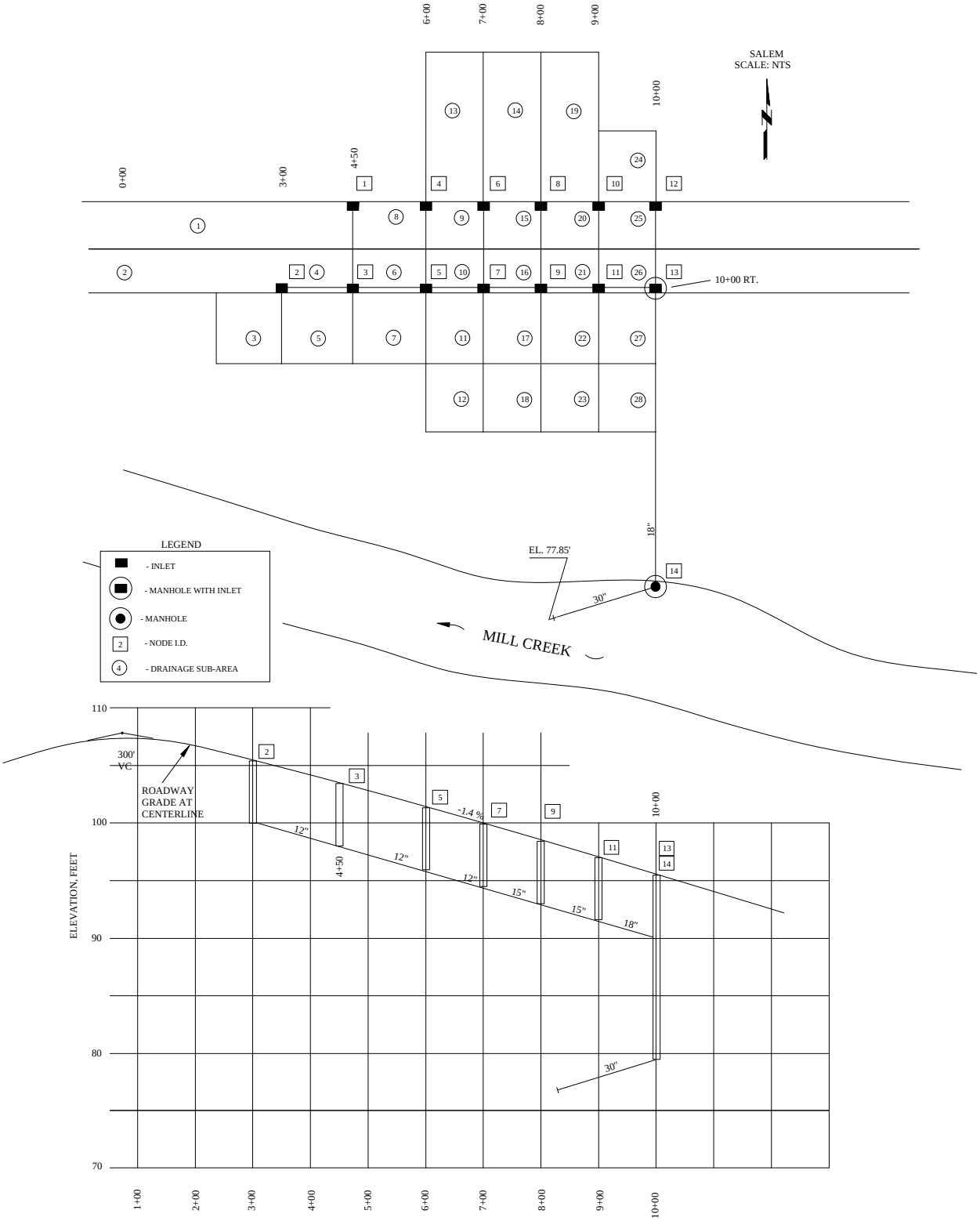



FIGURE #3
SYSTEM LAYOUT

ⓐ = ① ④ ⑤
ⓑ = ⑥ ⑦ ⑧
ⓒ = ⑨ ⑩ ⑪ ⑫ ⑬
ⓓ = ⑭ ⑮ ⑯ ⑰ ⑱
ⓔ = ⑲ ⑳ ㉑ ㉒ ㉓
ⓕ = ㉔ ㉕ ㉖ ㉗ ㉘

FIGURE #4 (**TRUNK**)
STORM DRAIN DESIGN SHEET
FULL AND PARTIAL-FULL FLOW

PROJECT: Example DESIGN FREQUENCY: 10-Year; 50-Year* DESIGNED BY: JOE SMITH DATE: 2005

| Station | Node I.D. | Drainge Area | | Runoff Coeff. | Equiv. Area for 100% Runoff | Accumlative Impervious Drainage Area | Time of Concent. or Flow Time | Total Time of Concent. | Average Rainfall Intensity | Design discharge | Invert Slope | Pipe Size | Capacity Flowing Full | Velocity Flowing Full | Length | Fall | Invert Elevation | Top of Manhole or Inlet Elevation | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Index No. | Area A | C | CA | ΣCA | $T_c$ | $T_c$ | I | Q | S | D | $Q_f$ | $V_f$ | L | | | | |
| | | | Acres | | (3) x (4) | Acres | Minutes | Minutes | in/hr | cfs | % | Inches | cfs | fps | feet | feet | feet | feet | |
| 1a | 1b | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
| 0+00 | | | | | | | | | | | | | | | | | | | |
| | 2 | ②③ | 0.54 | 0.90 | 0.49 | 0.49 | | | | | | | | | | | | | |
| | | | | | | | 5.0 | 5.0 | 2.12 | | | | | | | | | | |
| 3+00 Rt | | | | | | | | | | 1.04 | | | | | | | 101.69 | 104.9 | IE OUT AND TOP OF MH AT NODE 2 |
| | 3 | ⓐ | 0.95 | 0.90 | 0.86 | 1.35 | | | | | 1.653 | 12 | 4.55 | 5.84 | 150 | | | | |
| | | | | | | | 0.4 | 5.4 | 2.08 | | | | | | | 2.48 | | | |
| 4+50 Rt | | | | | | | | | | 2.81 | | | | | | | 99.21 | 102.8 | IE IN AND OUT AND TOP OF MH AT NODE 3 |
| | 5 | ⓑ | 0.67 | 0.90 | 0.60 | 1.95 | | | | | 1.40 | 12 | 4.25 | 5.40 | 150 | | | | |
| | | | | | | | 0.5 | 5.9 | 2.00 | | | | | | | 2.10 | | | |
| 6+00 Rt | | | | | | | | | | 3.90 | | | | | | | 97.11 | 100.7 | IE IN AND OUT AND TOP OF MH AT NODE 5 |
| | 7 | ⓒ | 1.14 | 0.81 | 0.92 | 2.87 | | | | | 1.40 | 12 | 4.25 | 5.40 | 100 | | | | |
| | | | | | | | 0.3 | 6.2 | 1.98 | | | | | | | 1.40 | 95.71 | | IE IN AT NODE 7 |
| 7+00 Rt | | | | | | | | | | 5.68 | | | | | | | 95.46 | 99.3 | IE OUT AT NODE 7 |
| | 9 | ⓓ | 1.14 | 0.81 | 0.92 | 3.79 | | | | | 1.40 | 15 | 7.70 | 6.30 | 100 | | | | |
| | | | | | | | 0.3 | 6.5 | 1.92 | | | | | | | 1.40 | | | |
| 8+00 Rt | | | | | | | | | | 7.28 | | | | | | | 94.06 | 97.9 | IE IN AND OUT AND TOP OF MH AT NODE 9 |
| | 11 | ⓔ | 1.14 | 0.81 | 0.92 | 4.71 | | | | | 1.40 | 15 | 7.70 | 6.30 | 100 | | | | |
| | | | | | | | 0.3 | 6.8 | 1.90 | | | | | | | 1.40 | 92.66 | | IE IN AT NODE 11 |
| 9+00 Rt | | | | | | | | | | 8.95 | | | | | | | 92.41 | 96.5 | IE OUT AT NODE 11 |
| | 13 | ⓕ | 0.91 | 0.84 | 0.76 | 5.47 | | | | | 1.40 | 18 | 12.50 | 7.30 | 100 | | | | |
| | | | | | | | 0.2 | 7.0 | 2.53 * | | | | | | | 1.40 | 91.01 | | IE IN AT NODE 13 |
| 10+00 Rt | | | | | | | | | | 13.84 | | | | | | | 88.97 | 95.1 | IE OUT AT NODE 13 |
| | | | | | | | | | | | 1.70 | 18 | 13.84 | 7.90 | 350 | | | | |
| | | | | | | | | | | | | | | | | 5.95 | 83.02 | | IE IN AT NODE 14 |
| 10+00 (350') Rt | 14 | | | | | | | | | | | | | | | | 77.97 | 95.5 | IE OUTAT NODE 14 |
| | | | | | | | | | | | 0.13 | 30 | 14.60 | 3.00 | 90 | | | | |
| | | | | | | | | | | | | | | | | 0.12 | | | |
| Outfall | | | | | | | | | | | | | | | | | 77.85 | | |
| | | | | | | | | | | | | | | | | | | | |

**FIGURE #5 (LATERALS)**
**STORM DRAIN DESIGN SHEET**
**FULL AND PARTIAL-FULL FLOW**

PROJECT: Example Problem DESIGN FREQUENCY: 10-Year; 50-Year* DESIGNED BY: JOE SMITH DATE: 2005

| Station | Node I.D. | Drainge Area | | Runoff Coeff. | Equiv. Area for 100% Runoff | Accumlative Impervious Drainage Area | Time of Concent. or Flow Time | Total Time of Concent. | Average Rainfall Intensity | Design discharge | Invert Slope | Pipe Size | Capacity Flowing Full | Velocity Flowing Full | Length | Fall | Invert Elevation | Top of Manhole or Inlet Elevation | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Index No. | Area A | C | CA | CA | $T_c$ | $T_c$ | I | Q | S | D | $Q_f$ | $V_f$ | L | | | | |
| | | | Acres | | (3) x (4) Acres | Acres | Minutes | Minutes | in/hr | (6) x (9) cfs | % | Inches | cfs | fps | feet | feet | feet | feet | |
| 1a | 1b | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
| 0+00 Lt | | | | | | | | | | | | | | | | | | | |
| | 1 | ① | 0.43 | .9 | 0.39 | 0.39 | | | | | | | | | | | | | |
| | | | | | | | 5.0 | 5.0 | 2.12 | | | | | | | | | | |
| 4+50 Lt | | | | | | | | | | 0.83 | | | | | | | 99.59 | 102.8 | IE OUT AND TOP OF MH AT NODE 1 |
| | | | | | | | | | | | 0.45 | 12 | 2.40 | 3.00 | 84 | | | | |
| | | | | | | | | | | | | | | | | 0.38 | | | |
| 4+50 Rt | | | | | | | | | | | | | | | | | 99.21 | 102.8 | IE IN AND TOP OF MH AT NODE 3 |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| 4+50 Lt | | | | | | | | | | | | | | | | | | | |
| | 4 | ⑧ | 0.14 | .9 | 0.13 | 0.13 | | | | | | | | | | | | | |
| | | | | | | | 5.0 | 5.0 | 2.12 | | | | | | | | | | |
| 6+00 Lt | | | | | | | | | | 0.28 | | | | | | | 97.49 | 100.7 | IE OUT AND TOP OF MH AT NODE 4 |
| | | | | | | | | | | | 0.45 | 12 | 2.40 | 3.00 | 84 | | | | |
| | | | | | | | | | | | | | | | | 0.38 | | | |
| 6+00 Rt | | | | | | | | | | | | | | | | | 97.11 | 100.7 | IE IN AND TOP OF MH AT NODE 5 |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| 6+00 Lt | | | | | | | | | | | | | | | | | | | |
| | 6 | ⑬ ⑨ | 0.56 | .82 | 0.46 | 0.46 | | | | | | | | | | | | | |
| | | | | | | | 5.0 | 5.0 | 2.12 | | | | | | | | | | |
| 7+00 Lt | | | | | | | | | | 0.98 | | | | | | | 96.09 | 99.3 | IE OUT AND TOP OF MH AT NODE 6 |
| | | | | | | | | | | | 0.45 | 12 | 2.40 | 3.00 | 84 | | | | |
| | | | | | | | | | | | | | | | | 0.38 | | | |
| 7+00 Rt | | | | | | | | | | | | | | | | | 95.71 | 99.3 | IE IN AND TOP OF MH AT NODE 7 |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |

FIGURE #6 (**LATERALS**)
STORM DRAIN DESIGN SHEET
FULL AND PARTIAL-FULL FLOW

PROJECT: Example DESIGN FREQUENCY: 10-Year; 50-Year* DESIGNED BY: JOE SMITH DATE: 2005

| Station | Node I.D. | Drainge Area | | Runoff Coeff. | Equiv. Area for 100% Runoff | Accumlative Impervious Drainage Area | Time of Concent. or Flow Time | Total Time of Concent. | Average Rainfall Intensity | Design discharge | Invert Slope | Pipe Size | Capacity Flowing Full | Velocity Flowing Full | Length | Fall | Invert Elevation | Top of Manhole or Inlet Elevation | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Index No. | Area | | | | | | | | | | | | | | | | |
| | | | A | C | CA | $\Sigma$ CA | $T_c$ | $T_c$ | I | Q | S | D | $Q_f$ | $V_f$ | L | | | | |
| | | | Acres | | (3) x (4) | Acres | Minutes | Minutes | in/hr | (6) x (9) cfs | % | Inches | cfs | fps | feet | feet | feet | feet | |
| 1a | 1b | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
| 7+00 Lt | | | | | | | | | | | | | | | | | | 99.3 | |
| | 8 | (14) (15) | 0.56 | .82 | 0.46 | 0.46 | | | | | | | | | | | | | |
| | | | | | | | 5.0 | 5.0 | 2.12 | | | | | | | | | | |
| 8+00 Lt | | | | | | | | | | 0.98 | | | | | | | 94.69 | 97.9 | IE OUT AND TOP OF MH AT NODE 8 |
| | | | | | | | | | | | 0.45 | 12 | 2.40 | 3.00 | 84 | | | | |
| | | | | | | | | | | | | | | | | 0.38 | | | |
| 8+00 Rt | | | | | | | | | | | | | | | | | 94.31 | 97.9 | IE IN AND TOP OF MH AT NODE 9 |
| | | | | | | | | | | | | | | | | | | | |
| 8+00 Lt | | | | | | | | | | | | | | | | | | 97.9 | |
| | 10 | (19) (20) | 0.56 | .82 | 0.46 | 0.46 | | | | | | | | | | | | | |
| | | | | | | | 5.0 | 5.0 | 2.12 | | | | | | | | | | |
| 9+00 Lt | | | | | | | | | | 0.98 | | | | | | | 93.29 | 96.5 | IE OUT AND TOP OF MH AT NODE 10 |
| | | | | | | | | | | | 0.45 | 12 | 2.40 | 3.00 | 84 | | | | |
| | | | | | | | | | | | | | | | | 0.38 | | | |
| 9+00 Rt | | | | | | | | | | | | | | | | | 92.91 | 96.5 | IE IN AND TOP OF MH AT NODE 11 |
| | | | | | | | | | | | | | | | | | | | |
| 9+00 Lt | | | | | | | | | | | | | | | | | | | |
| | 12 | (24) (25) | 0.33 | .9 | 0.30 | 0.30 | | | | | | | | | | | | | |
| | | | | | | | 5.0 | 5.0 | 2.12 | | | | | | | | | | |
| 10+00 Lt | | | | | | | | | | 0.64 | | | | | | | 91.89 | 95.1 | IE OUT AND TOP OF MH AT NODE 12 |
| | | | | | | | | | | | 0.45 | 12 | 2.40 | 3.00 | 84 | | | | |
| | | | | | | | | | | | | | | | | 0.38 | | | |
| 10+00 Rt | | | | | | | | | | | | | | | | | 91.51 | 95.1 | IE IN AND TOP OF MH AT NODE 13 |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |

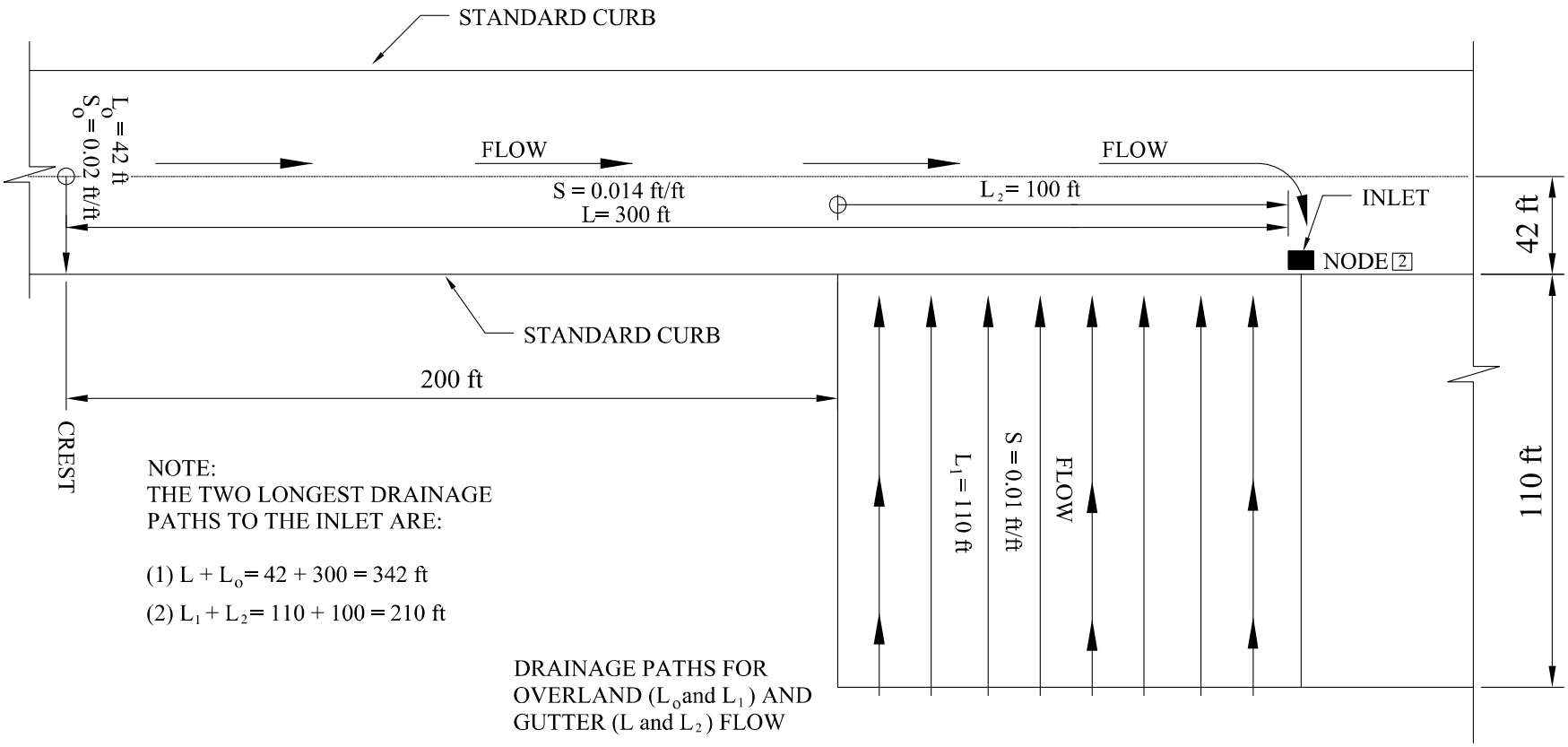
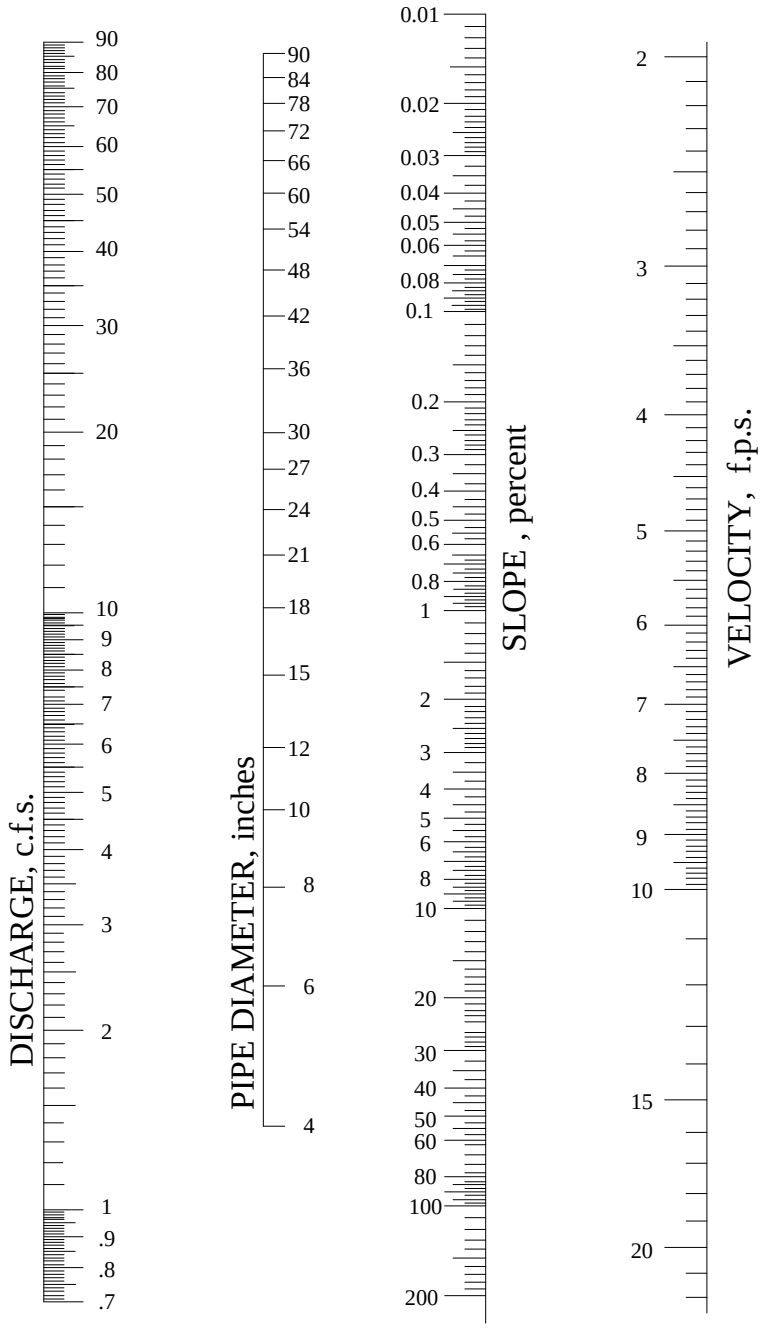

STANDARD CURB
$L_o$ = 42 ft
$S_o$ = 0.02 ft/ft
FLOW
FLOW
S = 0.014 ft/ft
L= 300 ft
$L_2$= 100 ft
INLET
NODE 2
42 ft
STANDARD CURB
200 ft
CREST
NOTE:
THE TWO LONGEST DRAINAGE
PATHS TO THE INLET ARE:
(1) L + $L_o$= 42 + 300 = 342 ft
(2) $L_1$ + $L_2$= 110 + 100 = 210 ft
$L_1$= 110 ft
S = 0.01 ft/ft
FLOW
110 ft
DRAINAGE PATHS FOR
OVERLAND ($L_o$and $L_1$) AND
GUTTER (L and $L_2$) FLOW

FIGURE #7
NTS

FIGURE #8




CAPACITY AND VELOCITY
NOMOGRAPH FOR
CIRCULAR CONCRETE PIPES
PIPE FLOWING FULL
n = 0.013

# APPENDIX G
# HYDRAULIC GRADE LINE

## 1.0 Introduction

The hydraulic grade line is used to aid the designer in determining the acceptability of a proposed or evaluation of an existing storm drainage system by establishing the elevation to which water will rise when the system is operating under design conditions. Detail on this system performance analysis is presented in this appendix.

## 2.0 Definitions

Definitions of terms which will be important in a storm drainage analysis and design are provided in this section. These and other terms defined in the manual glossary will be used throughout the remainder of this appendix in dealing with different aspects of storm drainage analysis.

Energy grade line – Represents the total available energy in the system (potential energy plus kinetic energy).

Hydraulic grade line - The hydraulic grade line (HGL), a measure of flow energy, is a line coinciding with the level of flowing water at any point along an open channel. In closed conduits flowing under pressure, the hydraulic grade line is the level to which water would rise in a vertical tube (open to atmospheric pressure) at any point along the pipe. HGL is determined by subtracting the velocity head ($V^2/2g$) from the energy gradient (or energy grade line). Figure 1 illustrates the energy and hydraulic grade lines for open channel and pressure flow in pipes. As illustrated in Figure 1, if the HGL is above the inside top (crown) of the pipe, pressure flow conditions exist. Conversely, if the HGL is below the crown of the pipe, open channel flow conditions exist.

Critical depth – is defined as the depth for which the specific energy (sum of the flow depth and velocity head) of a given discharge is at a minimum. A slight change in specific energy can result in a significant rise or fall in the water depth when flow is at or near critical depth. Because of this, critical depth is an unstable condition and it rarely occurs for any distance along a water surface profile.

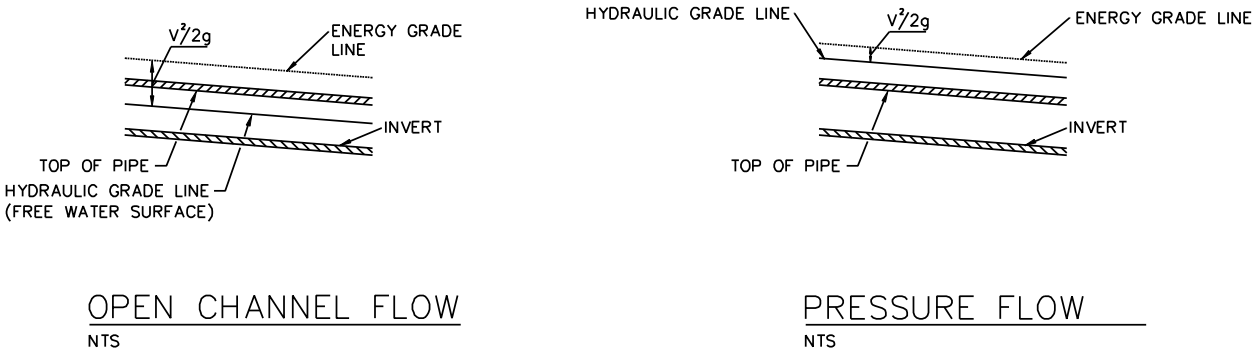



**Figure 1 - Hydraulic and Energy Grade Lines in Pipe Flow**

## 3.0 Design Guidelines

Storm drainage systems operating under surcharge conditions (pressure flow) shall evaluate the hydraulic grade line and minor losses such as manhole losses, bends in pipes, expansion and contraction losses, etc. Adjust inlet sizes/locations and pipe sizes/location to correct hydraulic grade line problems. Pressure flow design must assure the hydraulic grade line is 0.5 feet or more below rim elevation of any drainage structure which may be affected. The energy grade line must also be at or below the rim elevation of any drainage structure which may be affected.

## 4.0 Tailwater

Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation. For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth of the outlet. The tailwater may also occur between the critical depth and the invert of the outlet, however, the starting point for the hydraulic grade line determination should be either the design tailwater elevation or $(d_c + D)/2$, whichever is highest, where $d_c$ is outlet critical depth and D is crown depth.

An exception to the above rule would be for a very large outfall with low tailwater where a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the design tailwater elevation, whichever was highest.

If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by a comparison of the drainage area of the receiving stream to the area of the storm drainage system. For example, if the storm drainage system has a drainage area much smaller than that of the receiving stream, the peak discharge from the storm drainage system may be out of phase with the peak discharge from the receiving watershed. Table A provides a comparison of discharge frequencies for coincidental occurrence for a 10-year and 100-year design storm. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. For example, if the receiving stream has a drainage area of 200 acres and the storm drainage system has a drainage area of 2 acres, the ratio of receiving area to storm drainage area is 200 to 2 which equals 100 to 1. From Table A and considering a 10-year design storm occurring over both areas, the flow rate in the main stream will be equal to that of a five-year storm when the drainage system flow rate reaches its 10-year peak flow at the outfall. Conversely, when the flow rate in the main channel reaches its 10-year peak flow rate, the flow rate from the storm drainage system will have fallen to the 5-year peak flow rate discharge. This is because the drainage areas are different sizes, and the time to peak for each drainage area is different.

**Table A - Frequencies for Coincidental Occurrence**

| AREA RATIO | FREQUENCIES FOR COINCIDENTAL OCCURRENCE | | | |
|---|---|---|---|---|
| | 10-Year Design | | 100-Year Design | |
| | Main Stream | Tributary | Main Stream | Tributary |
| 10,000 to 1 | 1<br>10 | 10<br>1 | 2<br>100 | 100<br>2 |
| 1,000 to 1 | 2<br>10 | 10<br>2 | 10<br>100 | 100<br>10 |
| 100 to 1 | 5<br>10 | 10<br>5 | 25<br>100 | 100<br>25 |
| 10 to 1 | 10<br>10 | 10<br>10 | 50<br>100 | 100<br>50 |
| 1 to 1 | 10<br>10 | 10<br>10 | 100<br>100 | 100<br>100 |

## 5.0 Energy Losses

All energy losses in pipe runs and junctions must be estimated prior to computing the hydraulic grade line. In addition to the principal energy involved in overcoming the friction in each conduit run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access structure. The following sections present relationships for estimating typical energy losses in storm drainage systems. The application of these relationships is included in the design example.

### 5.1 Pipe Friction Losses

The friction slope is the energy gradient in feet per feet for that run.

$$H_f = S_f L \qquad \text{(Equation 1)}$$

Where:

$H_f$ = total headloss due to friction in feet
$S_f$ = friction slope in feet per feet
$L$ = length of pipe in feet

The friction slope in Equation 1 is also the hydraulic gradient for a particular pipe run. The friction loss is simply the energy gradient multiplied by the length of the run in feet as indicated by Equation 1. The friction slope will match the pipe slope since this design procedure assumes steady uniform flow in open channels. Pipe friction losses for pipes flowing full can be determined by Equation 2.

$$S_f = \frac{H_f}{L} = \left( \frac{Qn}{0.46D^{2.67}} \right)^2 \qquad \text{(Equation 2)}$$

Where:
- $S_f$ = friction slope in feet per feet
- n = Manning's roughness coefficient
- D = pipe diameter in feet
- Q = design discharge in cubic feet per second

Equation 3 and Equation 4 present forms of the Manning's equation rearranged to solve for energy loss.

$$H_f = \frac{2.87n^2V^2L}{D^{4/3}} \qquad \text{(Equation 3)}$$

$$H_f = \left[ \frac{29n^2L}{R^{4/3}} \right] \frac{V^2}{2g} \qquad \text{(Equation 4)}$$

Where:
- $H_f$ = total head loss due to friction in feet
- n = Manning's roughness coefficient
- D = diameter of pipe in feet
- L = length of pipe in feet
- V = mean velocity in cubic feet per second
- R = hydraulic radius in feet
- g = gravitational constant (32.16 feet per second squared)

## 5.2 Exit Loss

The exit loss is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as an endwall, the exit loss is:

$$H_o = K\left[ \frac{V^2}{2g} - \frac{V_d^2}{2g} \right] \qquad \text{(Equation 5)}$$

Where:
- V = average outlet velocity in feet per second
- $V_d$ = channel velocity downstream of outlet in feet per second
- g = gravitational constant (32.16 feet per second squared)
- K = 1.0 (loss coefficient)

When $V_d = 0$ as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with moving water, the exit loss may be reduced to virtually zero.

## 5.3 Entrance Losses

Entrance losses need to be considered in storm drain design only at those locations where the storm drain originates at a culvert. The energy losses associated with culvert inlets are related to the type of culvert inlet used. Entrance losses are calculated as discussed in **Chapter 9**.

## 5.4 Bend Loss

The bend loss coefficient for storm drain design is minor but can be evaluated using the following formula:

$$H_b = 0.0033\Delta\left(\frac{V_o^2}{2g}\right) \quad \text{(Equation 6)}$$

Where:
- $\Delta$ = angle of curvature in degrees
- $V_o$ = average outlet velocity in feet per second
- $g$ = gravitational constant (32.16 feet per second squared)

## 5.5 Access Structure Losses

The head loss encountered in going from one pipe to another through an access structure is commonly represented as being proportional to the velocity head at the outlet pipe. Using K to signify this constant of proportionality, the energy loss is approximated by using the following formula.

$$H_{as} = K\left(\frac{V_o^2}{2g}\right) \quad \text{(Equation 7)}$$

Experimental studies have determined that the K value can be approximated as follows:

$$K = K_o\, C_D\, C_d\, C_Q\, C_p\, C_B \quad \text{(Equation 8)}$$

Where:
- $H_{as}$ = head loss at access structure
- $K$ = adjusted loss coefficient
- $K_o$ = initial head loss coefficient based on relative access structure size
- $C_D$ = correction factor for pipe diameter (pressure flow only)
- $C_d$ = correction factor for flow depth (non-pressure flow only)
- $C_Q$ = correction factor for relative flow
- $C_B$ = correction factor for benching
- $C_p$ = correction factor for plunging flow

### 5.5.1 Relative Access Structure Size

$K_o$ is estimated as a function of the relative access structure size and the angle of deflection between the inflow and outflow pipes. See Figure 2.

$$K_o = 0.1\left(\frac{b}{D_o}\right)(1-\sin\theta)+1.4\left(\frac{b}{D_o}\right)^{0.15}\sin\theta \qquad \text{(Equation 9)}$$

Where: $\theta$ = the angle between the inflow and outflow pipes in degrees
$b$ = access structure diameter in feet
$D_o$ = outlet pipe diameter in feet

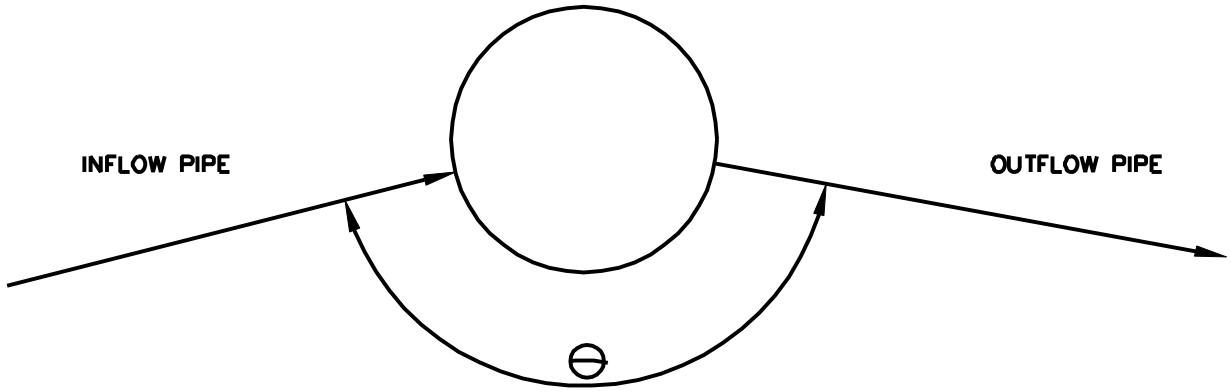



**Figure 2 - Deflection Angle**

### 5.5.2 Pipe Diameter

A change in head loss due to differences in pipe diameter is only significant in pressure flow situations when the depth in the access structure to outlet pipe diameter ratio, $d_{aho}/D_o$, is greater than 3.2. Otherwise, $C_D$ is set equal to 1.0. Use the following equation when this is not the case.

$$C_D = \left(\frac{D_o}{D_i}\right)^3 \qquad \text{(Equation 10)}$$

Where: $C_D$ = correction factor for pipe diameter
$D_i$ = incoming pipe diameter in feet
$D_o$ = outgoing pipe diameter in feet
$d_{aho}$ = water depth in access structure above the outlet pipe invert in feet

### 5.5.3 Flow Depth

The correction factor for flow depth is significant only in cases of free surface flow or low pressures, when $d_{aho}/D_o$ ratio is less than 3.2. The correction factor for flow depth is equal to 1 when this ratio is greater than 3.2. Water depth in the access hole is approximated as the level of hydraulic gradeline at the upstream end of the outlet pipe. The correction factor for flow depth ($C_d$) is calculated by the following:

$$C_d = 0.5\left(\frac{d_{aho}}{D_o}\right)^{0.6} \quad \text{(Equation 11)}$$

Where:
- $C_d$ = correction factor for flow depth
- $d_{aho}$ = water depth in access structure above outlet pipe invert in feet
- $D_o$ = outgoing pipe diameter in feet

### 5.5.4 Relative Flow

The correction factor for relative flow ($C_Q$) is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. The correction factor is only applied where there are three or more pipes entering the structure at approximately the same elevation. Otherwise ($C_Q$) is equal to 1.0. It is computed as follows:

$$C_Q = (1-2\sin\theta)\left[1-\left(\frac{Q_i}{Q_o}\right)\right]^{0.75}+1 \quad \text{(Equation 12)}$$

Where:
- $C_Q$ = correction factor for relative flow
- $\theta$ = the angle between the inflow and outflow pipes in degrees
- $Q_i$ = flow in the inflow pipe in cubic feet per second
- $Q_o$ = flow in the outlet pipe in cubic feet per second

As can be seen from the equation ($C_Q$) is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. To illustrate this effect, consider the access structure shown in the Figure 3 and assume the following two cases to determine the impact of pipe 2 entering the access structure.

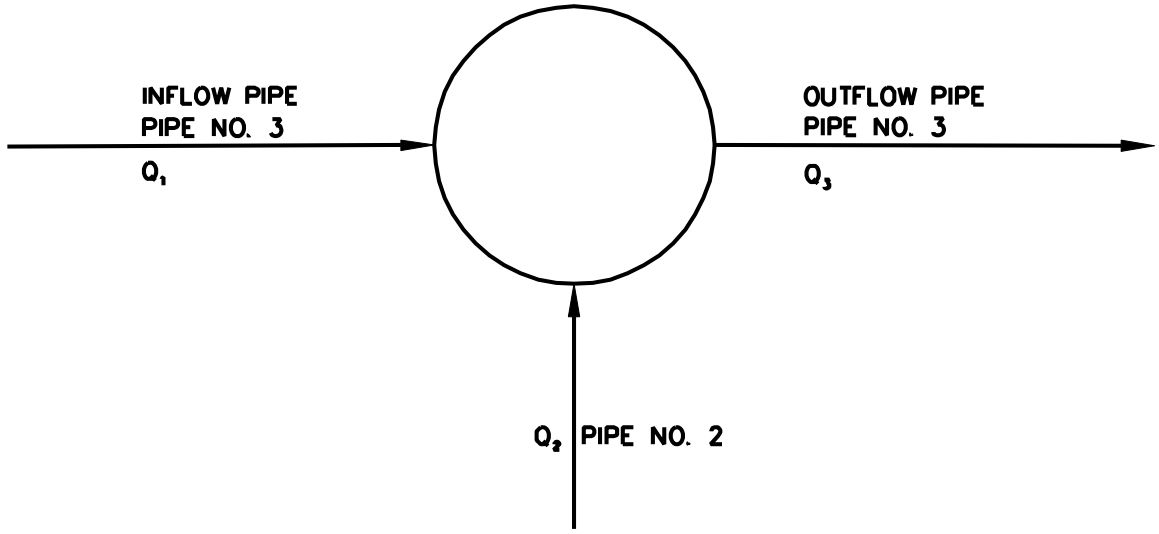



**Figure 3 - Relative Flow Effect**

Case 1: $Q_1 > Q_2$

$Q_1$ = 3 cubic feet per second
$Q_2$ = 1 cubic feet per second
$Q_3$ = 4 cubic feet per second

Using Equation 12,

$$C_{Q3\text{-}1} = \left(1 - 2\sin 180°\right)\left[1 - \left(\frac{3}{4}\right)\right]^{0.75} + 1$$
$$= 1.35$$

Case 2: $Q_2 > Q_1$

$Q_1$ = 1 cubic feet per second
$Q_2$ = 3 cubic feet per second
$Q_3$ = 4 cubic feet per second

Using Equation 12,

$$C_{Q3\text{-}1} = \left(1 - 2\sin 180°\right)\left[1 - \left(\frac{1}{4}\right)\right]^{0.75} + 1$$
$$= 1.81$$

### 5.5.5 Plunging Flow

The correction factor for plunging flow ($C_p$) is calculated by the following:

$$C_p = 1 + 0.2\left(\frac{h}{D_o}\right)\left(\frac{h - d_{aho}}{D_o}\right) \quad \text{(Equation 13)}$$

Where:
- $C_p$ = correction for plunging flow
- $h$ = vertical distance of plunging flow from flow line of incoming pipe to the center of outlet pipe in feet
- $D_o$ = outgoing pipe diameter in feet
- $d_{aho}$ = water depth in access structure above outlet pipe invert in feet

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet, plunging into the access hole, on the inflow pipe for which the head loss is being calculated. Using the notations in the above figure for the example, $C_p$ is calculated for pipe 1 when pipe 2 discharges plunging flow. The correction factor is only applied when h is greater than $d_{aho}$. Otherwise the correction factor for plunging flow is equal to 1.

### 5.5.6 Benching

The correction for benching in the access structure ($C_B$) is obtained from Table B. Benching tends to direct flow through the access structure, resulting in reductions in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed. Table C provides a table of standard ODOT benching details. Figure 4 provides a schematic of the four different types of benching.

**Table B - Correction For Benching**

| Bench Type | Correction Factors ($C_B$) | |
|---|---|---|
| | Submerged* | Unsubmerged** |
| Flat Floor | 1.00 | 1.00 |
| Half Bench | 0.95 | 0.15 |
| Full Bench | 0.75 | 0.07 |
| Improved | 0.40 | 0.02 |
| * pressure flow, $d_{aho}/D_o > 3.2$<br>** free surface flow, $d_{aho}/D_o < 1.0$ | | |

**Table C ODOT Benching**

| Access Structure Type | Bench Type | Drawing |
|---|---|---|
| 4 foot Standard Manhole | Full Bench | RD344 |
| Standard Manhole | Half Bench | RD336 |
| Shallow Manhole | Half Bench | RD342 |
| Large Manhole | Half Bench | RD346 |
| All inlet Boxes | Flat Floor | - - - - - |

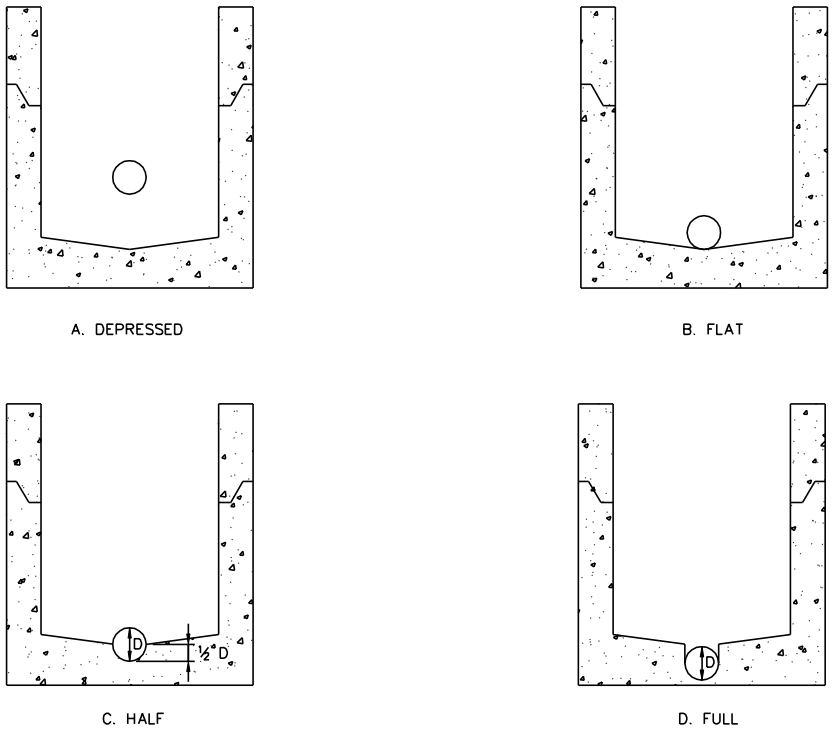



**Figure 4 - Benching Type Schematic**

## 6.0 Surcharged-Full Flow (Pressure Flow Conditions)

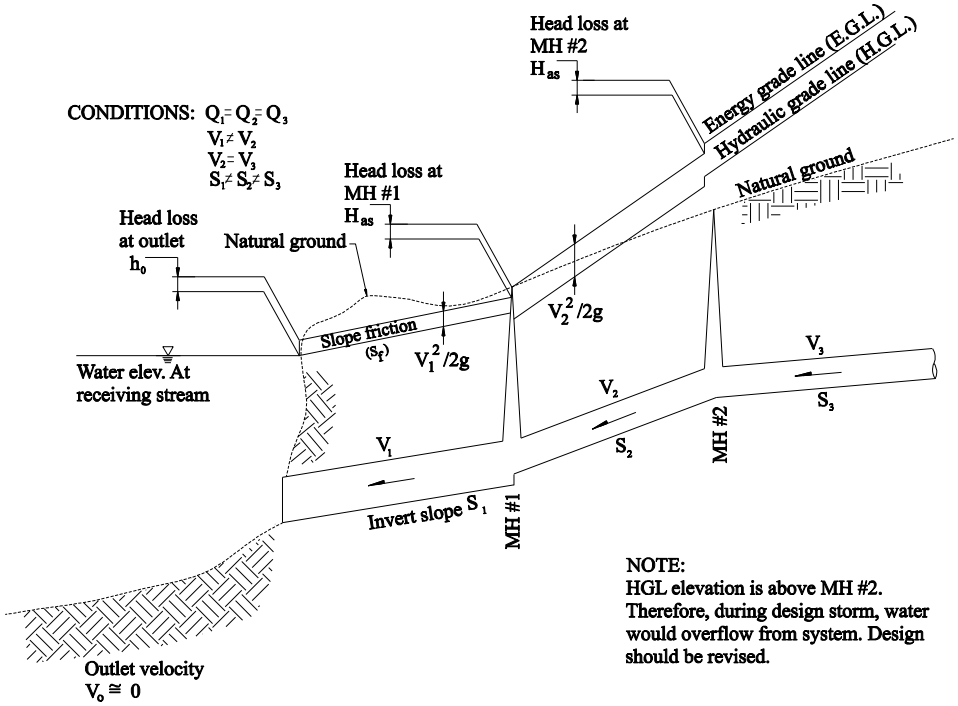
Most storm drains discharge into natural receiving streams or drainages which do not submerge the outlet of the system; however, there are locations where a storm system must discharge into a stream or river where major floods in those streams will submerge the storm drain and back water into the system. During periods of little or no flow in the storm system, backwater from these streams usually is not a problem; however, when a peak flow of design magnitude occurs in the storm system simultaneously with a high flow in the receiving stream, the system becomes "surcharged" and operates under pressure flow conditions.

Under the "surcharged" condition, backwater from the receiving stream may raise the elevation of the water surface (Hydraulic Grade Line or HGL) in the system high enough that water could bubble out of manholes and inlets at low points in the highway grade.

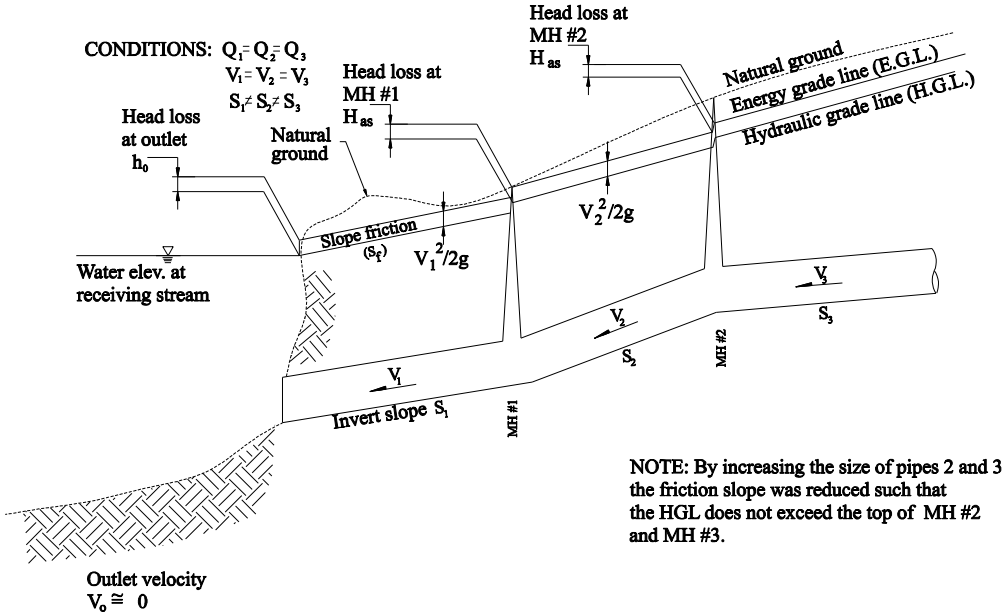
# FIGURE 5

## SURCHARGE FULL FLOW

### IMPROPER DESIGN




### PROPER DESIGN



The elevation of the HGL can be determined only after the Energy Grade Line (EGL) has been established. The EGL is a line showing the total available energy in the system (potential energy plus kinetic energy). If the sewer were of constant cross section with no manholes or angle points the EGL would have a constant and continuous slope equal to the FRICTION SLOPE, $S_f$ . However, storm sewers are not constructed in this manner and each appurtance disrupts the flow pattern in the sewer, producing an energy loss which is indicated by a sudden drop in the elevation of the EGL.

Inundation of the highway can be prevented by increasing the size of the sewer pipes downstream from those areas where the elevation of the hydraulic grade line (HGL) exceeds the elevation of the low manholes and inlets. Increasing the size of the pipe reduces the friction loss in the line. Figure 5 shows a short section of a storm sewer. In the top drawing; Pipe 2 and Pipe 3 are small, thereby inducing large friction losses as indicated by the slope of the EGL. In the bottom drawing, the size of these two pipes has been increased to reduce the friction losses, thereby lowering the elevation of the EGL and HGL to a point below the top of each manhole.

## 6.1 Energy Grade Line and Hydraulic Grade Line

This section presents a step-by-step procedure for manual calculation of the energy grade line (EGL) and the hydraulic grade line (HGL) using the energy loss method. This has been included to aid the designer in understanding the analysis process, but the most efficient means of evaluating storm drain systems is with computer programs.

The following is the procedure to design sewers for surcharged-full flow conditions.

Step 1- Design the storm system with procedures outlined in Appendix F for FULL AND PARTIAL-FULL FLOW. Assume there is free outfall from the storm sewer.

Step 2- Draw a profile of the proposed sewer showing the highway grade and location of each manhole and its cover elevation. Note elevations of low gutters.

Step 3- Use tabular design sheet STORM SEWER DESIGN SHEET – SURCHARGED-FULL FLOW (Pressure Flow Conditions), Figure 6.

Step 4- Using Figure 6. Enter in Column 1 the station of the sewer outfall.

Step 5- Enter in Column 2 the design discharge, Q cfs for the outfall pipe.

Step 6- Enter in Column 3 the size of the outfall pipe, D in inches.

Step 7- Enter in Column 4 the length of the outfall pipe, L in feet

Step 8- Hydraulic structure losses and other minor pipe losses are determined and transferred to Figure 6 in columns 9 and 13. Refer to section 5.0 for Energy Losses.

Step 9- Enter in Column 5 the average flow velocity (V) in feet per second of the outfall pipe.

Step 10- Enter in Column 6 the velocity head for the outfall pipe.

Step 11- Enter in Column 7 the average flow velocity component in the outlet channel (which is parallel to the system) - in most cases this can be considered negligible.

*Note: Column 7 is normally used only once to define conditions downstream from the storm drain system.*

Step 12- Enter in Column 8 the velocity head for velocity in Column 7 - in most cases this can be considered negligible.

*Note: The EGL and HGL at the outfall have now been determined. The next steps apply to the storm drain system.*

Step 13- Hydraulic pipe characteristics, roughness coefficients, and equations are located in **Chapter 8** to aid in manual calculations. Also, nomographs for several types of pipes are located in **Chapter 8**. When the appropriate nomograph is available, use the DISCHARGE (Column 2) and PIPE DIAMETER (Column 3) and note the value on the SLOPE scale. This is the FRICTION SLOPE ($S_f$) of the EGL. Enter this value into Column 10.

Step 14- Multiply the value in Column 10 by the LENGTH in Column 4 to determine the FRICTION HEAD LOSS. Enter in Column 11.

Step 15- Enter in Column 12 the elevation of the energy grade line (Column 16, previous line, plus column 9) at the outlet of the drain.

Step 16- Enter in Column 16 the elevation of the hydraulic grade line at the outlet of the drain by subtracting from the value in Column 12 the value in Column 6.

Step 17- Enter in Column 1 the station identification of the upstream end of the outfall pipe.

Step 18- Enter in Column 12 the elevation of the energy grade line ($EGL_o$) at the upstream end of the outfall pipe by adding to the first value in Column 15 (on the previous line) (FRICTION HEAD LOSS) to the value in Column 11.

Step 19- Enter in Column 14 energy losses for structures, multiply Column 13 by Column 6.

Step 20- Enter in Column 15 the elevation of the energy grade line, $EGL_i$ , Column 14 is added to Column 12 (previous line).

Step 21- Enter in Column 16 the elevation of the hydraulic grade line at the upstream end of the outlet pipe. Note: the EGL and HGL should be parallel and separated by the amount of the velocity head in the storm drain pipe (HGL = EGL – $V^2/2g$).

Step 22- Repeat procedure starting with Step 13 for the next upstream run of pipe.

Step 23- Plot the HGL and EGL on the storm sewer profile to determine if the HGL elevation exceeds the gutter grade elevation of the highway.

Step 24- If the HGL elevation is above the highway grade, the *size* of the pipes downstream from the point of inundation should be enlarged to reduce the elevation of the HGL. The pipes should be enlarged sufficiently to assure the hydraulic grade line is 0.5 feet or more below the rim elevation of any drainage structure which may be affected. The EGL must also be at or below the rim elevation of any drainage structure which may be affected.

## 6.2 Storm Drain Design (Surcharge-full Flow) Example

The following example demonstrates the design process and calculation for a double branch surcharged storm drain that is surcharged-full flow. The drain system will be designed to convey stormwater from the drainage basins displayed on Figure 7 (drainage map) to G. K. Elliot Bay which is located at Tillamook, Oregon. The double branch surcharged-full storm sewer will remove the storm water runoff from the two hatched basins shown on the Drainage Map (Figure 7).

**PART A.**

The following design steps outline the proposed storm drain system's characteristics (i.e., preparing a drainage basin map, delineation of drainage basins, horizontal computations, collection alignments, hydrologic computations, and collection structure selection) which are necessary prior to pipe sizing design.

Step A.1- Locate and sketch the storm sewer system as shown on Figure 8, 9 and 10. Add stations from the most distant points on the basins to the outfall at G. K. Elliott Bay. An equation at the manhole was included to separate the left branch from the right branch.

Step A.2- Enter in Column 1 of "STORM SEWER DESIGN SHEET - FULL AND PARTIAL-FULL FLOW", Figure 11, the stations which had been sketched and located on Figure 8. A Type III non-reinforced concrete pipe will be used with a required minimum cover of 1.5 feet. A Manning's "n" value of 0.013 will be assumed. The outfall pipe from the manhole will be sized to convey the 50-year peak design discharge.

Step A.3- Enter in Columns 2, 3, 4, 5 and 6 of Figure 11 the drainage areas Basin 1 ($\Sigma CA$ = 5.4 acres) and Basin 2 ($\Sigma CA$ = 9.45 acres) which are shown on Figure 7.

Step A.4- The times of concentration of 6.3 minutes (Basin 1) and 29.5 minutes (Basin 2) were used for the design of the storm sewer. These times of concentration were calculated as explained in **Chapter 7**. Enter these times of concentration in Columns 7 and 8 of Figure 11.

Step A.5- Obtain the 10-year rainfall intensity corresponding to durations of 6.3 minutes and 29.5 minutes in **Chapter 7**. Enter 2.5 inches per hour and 1.25 inches per hour in Column 9, Figure 11.

Step A.6.- Multiply Column 9 times Column 6 and enter 11.8 cubic feet per second and 13.5 cubic feet per second in Column 10 of Figure 11.

Step A.7- Two profiles which show the controlling elevation, the manhole location and the G. K. Elliott Bay receiving channel, were drawn on Figures 9 and 10. From these figures enter in Column 18 of Figure 11 the controlling ground elevations of 117.0, 110.0 and 109.5 feet. These are the elevations which must not be exceeded by the hydraulic grade line. If these elevations are exceeded, localized flooding will occur.

Step A.8- Enter in Column 15 of Figure 11 the pipe lengths of 570, 370, and 170 feet.

Step A.9- Estimate the invert slope as follows:

Right Branch Invert Slope = (117.0 – 109.5)(100) / 570 = 1.316 percent
Left Branch Invert Slope = (110 – 109.5)(100) / 170 = 0.294 percent

Step A.10- The previously estimated design discharges and invert slopes were used to tabulate the pipe size, capacity flowing full and velocity flowing full for each branch. This data was determined using Figure 12 (circular concrete pipe nomograph). Enter in Columns 11, 12, 13 and 14 of Figure 11 the following table:

*Note: Additional hydraulic pipe characteristics, roughness coefficients, and nomographs are located in Chapter 8.*

| | Design Discharge, Cubic feet per Second | Invert Slope, Percent | Pipe Size, Inches | Capacity Flowing Full, Cubic Feet per Second | Velocity Flowing Full, Feet per Second |
|---|---|---|---|---|---|
| RIGHT Branch | 11.8 | 1.316 | 18 | 12.2 | 7.0 |
| LEFT Branch | 13.5 | 0.294 | 27 | 16.5 | 4.2 |

Step: A.11- Determine and enter in Column 17 of Figure 11 the minimum invert elevation for the left branch using the following relationship. Type III non-reinforced concrete pipe will be used with a required minimum cover of 1.5 feet.

| Left Branch | | Top of Manhole Elevation | - | Pavement Thickness | - | Pipe Diameter | - | Shell Thickness of Pipe | - | Minimum Cover | = | Minimum Invert Elevation |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Minimum Invert Elev. Inlet End | = | 110.0 | - | 1.0 | - | $\frac{27}{12}$ | - | $\frac{4.0}{12}$ | - | 1.5 | = | 104.92 feet |
| Minimum Invert Elev. Outlet End | = | 109.5 | - | 1.0 | - | $\frac{27}{12}$ | - | $\frac{4.0}{12}$ | - | 1.5 | = | 104.42 feet |

Step A.12- Determine and enter in Column 17 of Figure 11 the minimum invert elevation for the outlet end of the Right Branch as shown below. The same relationship defined above is used to calculate this elevation.

Minimum Invert Elev. Outlet End Right Branch $= 109.5 - 1.0 - \frac{18}{12} - \frac{2.5}{12} - 1.5 = 105.29$ feet

Step A.13- Calculate and enter in Column 17 of Figure 11 the invert elevation for the inlet end of the Right Branch.

Invert Elev. Inlet End Right Branch $= 105.29 + (1.316)(570) / 100 = 112.79$ feet

Step A.14- The outfall pipe from the manhole is to be sized to convey the 50-year peak design discharge. Both the 29.5-minute storm and the 6.3-minute storm must be investigated to determine which storm produces the peak discharge through the outfall pipe. For the longer time of concentration (29.5-minute storm), both basins will contribute runoff from 100 percent of their drainage areas. Enter in Columns 2, 3, 4, 5 and 6 of Figure 11 the total drainage area. ($\Sigma CA = 14.85$ acres)

Step A.15- For the shorter time of concentration (6.3-minute storm), 100 percent of Basin 1 and a portion of Basin 2 will contribute runoff to the outfall (Station 3+70). The following relationship was used to determine the time of contributing runoff from Basin 2 for the 6.3-minute storm. The velocity for the left and right branch was estimated to be 4.2 feet per second and 7.0 feet per second (see Figure 11), respectively.

| Time of Contributing Runoff Basin 2 | + | Pipe Flow Time Right Branch | = | Time of Concentration Basin 1 | + | Pipe Flow Time Left Branch |
|---|---|---|---|---|---|---|

$$\text{Time of Contributing Runoff Basin 2} = 6.3 + \frac{170}{(4.2)(60)} - \frac{570}{(7.0)(60)} = 5.61 \text{ minutes}$$

The kinematic wave equation (see **Chapter 7**) was used to calculate the length of contributing drainage area when $I_{50\text{-yr}}$ = 3.20 inches per hour for the 50-year flood, the 6.3-minute storm and a contributing time of 5.61 minutes.

These calculations are shown below:

$T_c$ = 5.61 minutes
I = 3.20 inches per hour
n = 0.012
s = 0.001 feet per feet.

$$T_c = \frac{0.93L^{0.6}n^{0.6}}{I^{0.4}S^{0.3}}$$ (see **Chapter 7**)

Rearranging the above equation to solve for L

$$L = \left(\frac{T_c I^{0.4} S^{0.3}}{0.93 n^{0.6}}\right)^{1.67}$$

$$L = \left(\frac{(5.61)(3.20)^{0.4}(0.001)^{0.3}}{0.93(0.012)^{0.6}}\right)^{1.67}$$

L = 116 feet use L = 120 feet

Step A.16- Sketch this estimated travel length of 120 feet on Figure 8 which will delineate the contributing drainage area for the 6.3-minute storm (Area A1). Enter in Columns 2, 3, 4, 5, and 6 of Figure 11 the contributing drainage area ($A_1 = \Sigma CA$ = 0.69 acres) from Basin 2.

Step A.17- Calculate and enter in Column 7 the flow time through the right and left branches using the following relationship.

Right branch: $T_c = L / V = 570 /(7.0)(60) = 1.4$ minutes
Left branch: $T_c = L / V = 170 /(4.2)(60) = 0.7$ minutes

Step A.18- Add the flow times calculated above to the previous total time of concentration and enter the following results in Column 8.

29.5 minutes + 1.4 minutes
= 30.9 minutes (total time of concentration for right branch)

6.3 minutes + 0.7 minutes
= 7.0 minutes (total time of concentration for left branch)

Step A.19- Obtain the 50-year rainfall intensity corresponding to durations of 30.9 and 7.0 minutes using the Zone 2 Intensity-Duration-Frequency curve (**Chapter 7**). Enter 1.65 inches per hour and 3.20 inches per hour in Column 9, Figure 11.

Step A.20- Multiply Column 9 times Column 6 and enter 24.5 cubic feet per second and 19.5 cubic feet per second in Column 10 of Figure 11. The outfall pipe will be sized to convey 24.5 cubic feet per second, the greater peak discharge.

Step A.21- For Station 3+ 70, enter in Column 17 the Invert Elevation of 99.6 feet. The elevation places the outlet of the outfall pipe at the bottom of the channel. An outlet velocity of approximately 3.0 feet per second would minimize erosion at the end of the outfall pipe.

Step A.22- Given a design discharge of 24.5 cubic feet per second, determine slope, pipe size, and velocity for the outfall, Station 3+ 70. Using Figure 12, enter in Columns 11, 12, 13 and 14 of Figure 11 for the outfall pipe 0.145 percent, 36-inch, 24.5 cubic feet per second and 3.6 feet per second. The 36-inch diameter pipe is the smallest standard size pipe which can carry the design flow (24.5 cubic feet per second) and also satisfy the minimum velocity criterion (3 feet per second).

Step A.23- For the outfall pipe, Station 0+ 00 Ah to Station 3+ 70, determine fall and enter in Column 16 as shown below:

Fall = (0.145) (370) / 100 = 0.67 feet

Step A.24- Calculate and enter in Column 17 the Invert Elevation as shown below:

99.6 + 0.67 = 100.27 feet
This will be the invert elevation at Station 0+ 00 Ah.

**PART B.**

Check the storm sewer design as a surcharged full flow (pressure flow) system by determining the energy and hydraulic grade lines. The 50-year storm is the design storm for the surcharged condition.

Step B.1- Enter in Column 1 of "STORM SEWER DESIGN SHEET - SURCHARGED FLOW", Figure 13, the stations shown on Figure 8 beginning with the sewer outfall at 3+ 70. Note two entries are required, one for the left branch and one for the right branch.

Step B.2- Enter in Columns 2, 3, and 4 for both branches of Figure 13 the Discharge (24.5 cubic feet per second), the Pipe Size (36-, 18- and 27-inch) and Pipe Lengths (370, 570 and 170 feet) from Figure 11.

Step B.3- Enter in Columns 5 and 6 the velocity of 3.6 feet per second and the velocity head of 0.20 feet. The velocity head is calculated as shown below.

$$\text{Velocity Head} = \frac{V^2}{2g} = \frac{3.6}{64.4} = 0.20 \text{ feet}$$

Step B.4- Since the average velocity component in G. K. Elliott Bay is negligible, enter in Columns 7 and 8 the velocity of 0 feet per second and the velocity head of 0 feet.

Step B.5- Calculate and enter in Columns 2, 5 and 6, the discharge, velocity and velocity head for the right and left branches as shown below.

$Q = Q_{Lt} + Q_{Rt} = 24.5$ cubic feet per second

$Q_{Rt} = CIA = (9.45)(1.65)$

$= 15.59$ cubic feet per second

$Q_{Rt} = CIA = 15.6$ cubic feet per second

$Q_{Lt} = (5.40)(1.65)$

$= 8.91$ cubic feet per second

$Q_{Lt} = 8.9$ cubic feet per second

$Q_{Rt} + Q_{Lt} = 24.5$ cubic feet per second

$A_{18"} = 1.77$ square feet $\qquad V_{Rt} = \frac{Q}{A} = \frac{15.6}{1.77} = 8.8$ feet per second

$A_{27"} = 3.97$ square feet $\qquad V_{Lt} = \frac{Q}{A} = \frac{15.6}{3.97} = 2.2$ feet per second

$\frac{V_{RT}^2}{2g} = 1.21$ feet $\qquad \frac{V_{LT}^2}{2g} = 0.08$ feet

Step B.6- Enter Figure 12 using the discharges of 24.5 cubic feet per second, 15.6 cubic feet per second and 8.9 cubic feet per second (Column 1) and the pipe sizes of 36 inches, 18 inches and 27 inches (Column 2), to obtain 0.145, 2.30 and 0.090 percent from the slope scale. These are the Friction Slopes, $S_f$, of the EGL. Enter these friction slopes in Column 10.

Step B.7- Multiply the Friction Slopes in Column 10 times the Pipe Length in Column 4 and enter in Column 11 the following results.

Outfall
Friction
Head Loss = $\frac{(0.145)(370)}{100}$ = 0.54 feet

Right Branch
Friction
Head Loss = $\frac{(2.30)(570)}{100}$ = 13.11 feet

Left Branch
Friction
Head Loss = $\frac{(0.090)(170)}{100}$ = 0.15 feet

Step B.8- Calculate and enter in Column 14 (Figure 13) the minor energy loss in the manhole (reference section 5.5).

Calculate the adjusted loss coefficient (K) and enter into Column 13 (Figure 13)

$$H_{as} = K\left(\frac{V_o^2}{2g}\right)$$

$$K = K_o C_D C_d C_Q C_P C_B \qquad \text{(Equation 8)}$$

Where:

| | | |
|---|---|---|
| $H_{as}$ | = | energy loss across access hole in feet |
| $V_o$ | = | outlet velocity in feet per second |
| g | = | acceleration due to gravity (32.2 feet per second squared) |
| K | = | adjusted loss coefficient |
| $K_o$ | = | initial headloss coefficient based on relative access hole size |
| $C_D$ | = | correction factor for pipe diameter (pressure flow only) |
| $C_d$ | = | correction factor for flow depth |
| $C_Q$ | = | correction factor for relative flow |
| $C_P$ | = | correction factor for plunging flow |
| $C_B$ | = | correction factor for benching |

Calculate ($K_o$):

$$K_o = 0.1\left(\frac{b}{D_o}\right)(1-\sin\theta)+1.4\left(\frac{b}{D_o}\right)^{0.15}\sin\theta \qquad \text{(Equation 9)}$$

$\theta$ = the angle between inflow and outflow pipes in degrees
b = access structure diameter in feet
$D_o$ = outlet pipe diameter in feet

Left branch calculation:

$\theta$ = 146 degrees (reference Figure 8)

b = 4 feet

$D_o$ = 3 feet

$$K_o = 0.1\left(\frac{4}{3}\right)\left(1-\sin 146°\right)+1.4\left(\frac{4}{3}\right)^{0.15}\sin\ (146°)$$

= 0.059 + 0.817

= 0.876

Right branch calculations:

Compute $K_o$

Where: $\theta$ = 131 degrees (reference Figure 8)

b = 4 feet

$D_o$ = 3 feet

$$K_o = 0.1\left(\frac{4}{3}\right)\left(1-\sin 131°\right)+1.4\left(\frac{4}{3}\right)^{0.15}\sin 131°$$

= 0.033 + 1.103

= 1.136

Use the higher of the $K_o$ values. Therefore,

$K_o$ = 1.136 (right branch)

Calculate ($C_D$):

$$C_D = \left(\frac{D_o}{D_i}\right)^3 \qquad \text{(Equation 10)}$$

$D_o$ = outgoing pipe diameter in feet

$D_i$ = incoming pipe diameter in feet

For Left Branch:

$$C_D = \left(\frac{3}{2.25}\right)^3$$

= 2.370

For Right Branch:

$$C_D = \left(\frac{3}{1.5}\right)^3$$

= 2.00

Is the ratio of $d_{aho}/D_o$ greater than 3.2?

If not, $C_D = 1.0$

$d_{aho}$ = water depth in access hole above the outlet pipe invert, feet

$d_{aho}$ = HGL = invert elevation

Invert EL = 100.27 feet (Figure 11, Column 17)

HGL EL = 107.54 feet (Figure 13, Column 16)

$d_{aho} = 107.54 - 100.27$

$= 7.27$

$\frac{d_{aho}}{D_o} = \frac{7.27}{3} = 2.4$ which is less than 3.2, therefore $C_D = 1.0$

Calculate ($C_d$):

$$C_d = 0.5\left(\frac{d_{aho}}{D_o}\right)^{0.6} \quad \text{(Equation 11)}$$

$$= 0.5\left(\frac{7.27}{3}\right)^{0.6}$$

$$= 0.85$$

However, if the ratio of $d_{aho}/D_o$ is greater than 3.2, then $C_d = 1.0$

$\frac{d_{aho}}{D_o} = \frac{7.27}{3} = 2.4$ which is less than 3.2

Therefore, $C_d = 0.85$

Calculate ($C_Q$):

This correction factor is only applied to situations where there are three or more incoming pipes at approximately the same elevation. Otherwise, $C_Q$ is equal to 1.0. This example problem has two incoming pipes. Therefore, $C_Q = 1.0$.

Calculate ($C_p$):

$$C_p = 1 + 0.2\left(\frac{h}{D_o}\right)\left(\frac{h - d_{aho}}{D_o}\right) \quad \text{(Equation 13)}$$

Where: h = vertical distance of plunging flow from the flow line of the higher elevation inlet pipe to the center of the outflow pipe.

The two inflow pipes are at the same invert elevation and this is not a manhole with an inlet grate. Therefore,

$C_p = 1.0$

Calculate ($C_B$):

To be conservative, the manhole will be assumed to have a flat floor (reference Tables B and C).

$C_B$ = 1.00

Calculate (K):

$K = K_O C_D C_d C_Q C_P C_B$ (Equation 8)

$K = 1.136(1)(0.85)(1)(1)(1) = 0.97$

Calculate ($H_{as}$):

$H_{as} = K\frac{V_o^2}{2g}$

V = 3.6 feet per second (Column 5, Figure 13)

$= 0.97\frac{3.6^2}{2(32.2)} = 0.20$ feet (Column 14, Figure 13)

Step B.9- Reference Section 5.3 and **Chapter 9**. Calculate and enter in Column 9 the entrance loss to the right (9+ 99 RT) and left (6+ 99 LT) branches using the following relationship. Assume right and left branches are projecting from fill, socket end, therefore entrance coefficient $K_e$ is 0.2.

Entrance Loss Right Branch = $\frac{K_e V^2}{2g} = 0.2\left(\frac{8.8^2}{2(32.2)}\right) = 0.24$ feet

Entrance Loss Left Branch = $\frac{K_e V^2}{2g} = 0.2\left(\frac{2.2^2}{2(32.2)}\right) = 0.02$ feet

Step B.10- Enter in Column 16, Figure 13 from Figure 7 the highest known tide elevation of 107.0 feet. This elevation is the HGL at the outlet of the outfall.

Step B.11- Add the outlet loss (0.20 feet) in Column 9, Figure 13 to the water elevation in Column 16 and enter in Column 15 the following energy grade line (EGL) at the outlet of the system for the right and left branches. The outlet loss equals 1.0 $V^2/2g$, V = 3.6 feet per second.

Energy Grade Line (EGL) = 0.20 + 107.0 = 107.20 feet

Step B.12- Enter in Column 12, Figure 13 the elevation of the EGL at the upstream end of the outfall pipe (0+ 00) by adding to the first value in Column 15 with the value in Column 11 as shown below for the right and left branches.

EGL = 107.20 + 0.54 = 107.74 feet

Step B.13- Subtract 0.20 (Column 6) from 107.74 (Column 12) and enter in Column 16 the HGL elevation 107.54 feet for the right and left branches.

Step B.14- Enter in Column 18 the top of manhole elevation 109.5 feet (see Figure 7).

Step B.15- The elevation of the EGL inside the manhole is calculated by adding the manhole loss to the EGL of the outfall pipe.

EGL = 0.20 + 107.74 = 107.94 feet

Step B.16- The EGL and HGL calculations for the right (18-inch pipe) and left branch (27-inch pipe) upstream runs of pipe are shown in the following table.

| FOR 18-INCH PIPE ON RIGHT SIDE | | | | |
|---|---|---|---|---|
| STA. | VELOCITY HEAD | HEAD LOSS, Feet | EGL ELEV RIGHT, Feet | HGL ELEV RIGHT, Feet |
| 0+00 Rt | 0.20 | | 107.74 | 107.54 |
| M.H. | | 0.20 | 107.94 | |
| 15+70 Rt | 1.21 | | 107.94 | 106.73 |
| 10+00 Rt | 1.21 | 13.11 | 121.05 | 119.84 |
| 9+99 Rt | 0.00 | 0.24 | 121.29 | 121.29 |
| FOR 27-INCH PIPE ON LEFT SIDE | | | | |
| 0+00 Lt | 0.20 | | 107.74 | 107.54 |
| M.H. | | 0.20 | 107.94 | |
| 8+70 Lt | 0.08 | | 107.96 | 107.88 |
| 7+00 Lt | 0.08 | 0.15 | 108.11 | 108.03 |
| 6+99 Lt | 0.00 | 0.02 | 108.13 | 108.13 |

Step B.17- Second design attempt with 24-inch pipe for the Right Branch. Need to determine the EGL and HGL. As shown in the table above, the proposed upstream run of 18-inch pipe for the right branch would result in an improper design as shown on Figure 1. The calculated EGL and HGL elevation of 121.29 feet exceeds the natural ground elevation of 117.0 feet shown on Figure 10. For a proper design the EGL and HGL elevation must be below elevation 117.0 feet. A 24-inch will be investigated with its design information tabulated below. The information below will be used in Figure 13.

| | Design Discharge | Invert Slope | Pipe Size | Capacity Flowing Full | Velocity Flowing Full |
|---|---|---|---|---|---|
| RIGHT Branch | 11.8 cfs | 0.27 % | 24-inch | 11.8 cfs | 3.7 fps |

Step B.18- Calculate as shown below the velocity and velocity head for a 24-inch pipe which would convey a discharge of 15.6 cfs (see step 5 above). Enter the values in Columns 5 and 6.

$A_{24\ in.}$ = 3.142 square feet $V_{Rt} = \frac{Q}{A} = \frac{15.6}{3.142}$ = 5.0 feet per second

$\frac{V_{RT}^2}{2g}$ = 0.39 feet

Step B.19- Enter Figure 12 using the discharge of 15.6 cubic feet per second (Column 1) and the 24-inch pipe (Column 2) to obtain 0.47 percent from the slope scale. Enter in Column 10 the friction slope of 0.47 percent.

Step B.20- Multiply the friction slope in Column 10 times the pipe length in Column 4 and enter in Column 11 the following result.

Friction Head Loss = $\frac{(0.47)(570)}{100}$ = 2.68 feet

Step B.21- Estimate the minor energy loss in the manhole with the 24-inch pipe using the following relationships presented in Section 5.5. The minor energy loss will remain the same as computed in Step 8. $H_{as}$ = 0.20 feet

Step B.22- The elevation of the EGL inside the manhole is calculated by adding the manhole loss to the EGL of the outfall pipe.

EGL = 0.20 + 107.74 = 107.94 feet

Step B.23- The EGL and HGL calculations for the right and left branch upstream runs of pipe are shown in Figure 13.

Step B.24- As shown in Figure 13 both the right and left branches are proper designs. The HGL and EGL are below the control points.

Step B.25- Change the entries in Columns 11,12,13 and 14 for the Right Branch from 1.316 percent, 18-inch, 12.2 cubic feet per second and 7.0 feet per second to 0.27 percent, 24-inch, 11.8 cubic feet per second and 3.7 feet per second as shown in Figure 11.

Step B.26- Determine and enter in Column 17 of Figure 11 the minimum invert elevation for the outlet end of the 24-inch pipe as shown below. The relationship from Step 11, Part A, was used to calculate the invert elevation.

Minimum Invert Elev. Outlet End Right Branch = 109.5 - 1.0 - $\frac{24}{12}$ - $\frac{3.0}{12}$ - 1.5 = 104.75 feet

Step B.27- Estimate and enter in Column 17 of Figure 11 the invert elevation for the inlet end of the 24-inch pipe using the following relationship.

$$\text{Invert Elev.}_{\substack{\text{Inlet End}\\\text{Right Branch}}} = 104.75 + \frac{0.27(570)}{100} = 106.29 \text{ feet}$$

Step B.28- Sketch the storm system, the HGL and EGL on Figures 14 (Right Branch) and 15 (Left Branch).

# FIGURE 7

# DRAINAGE MAP

All elevations are assumed Datum

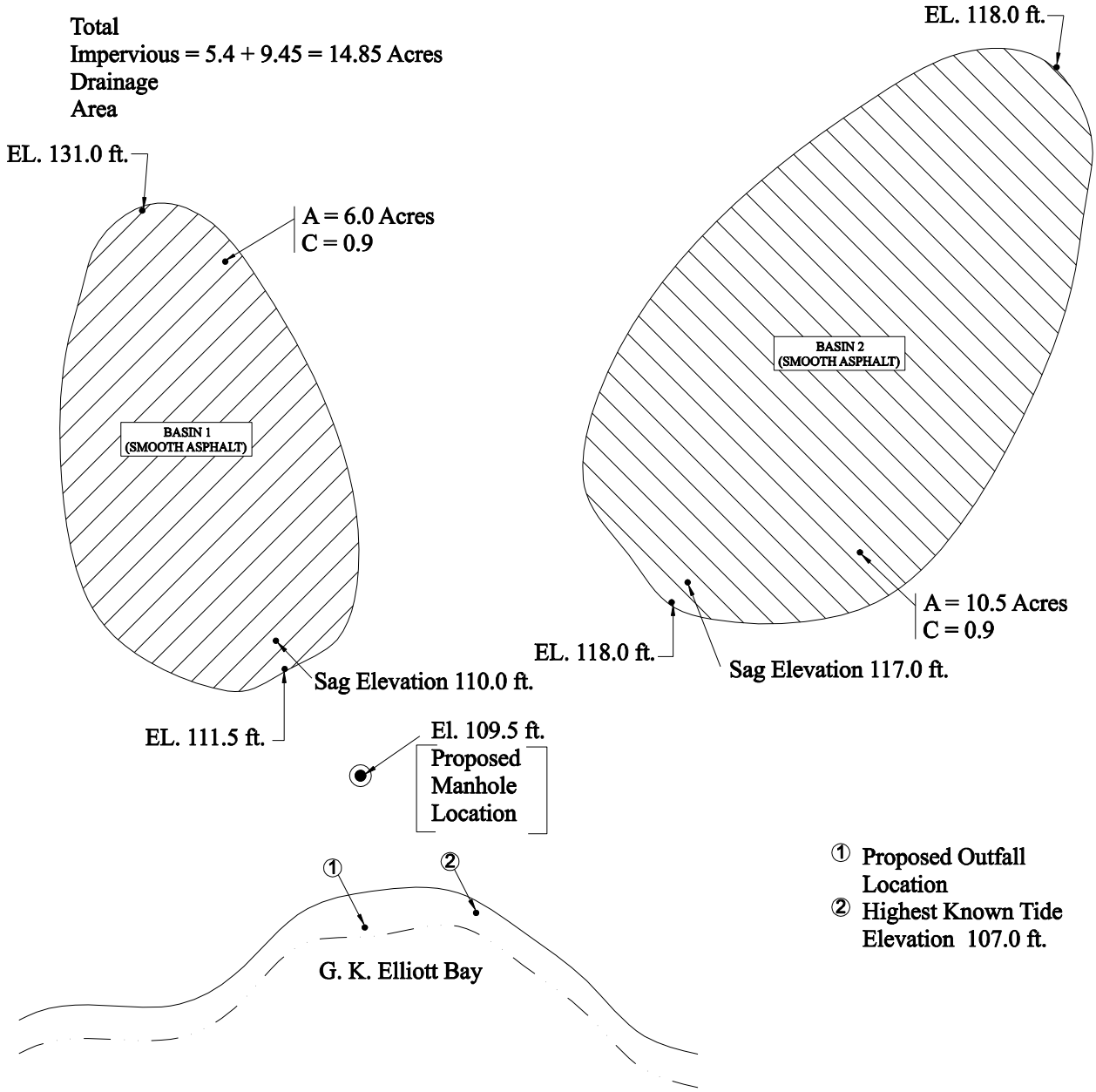
Impervious area to be drained into Storm Sewer

Basin 1 =Σ CA = 5.4 Acres

Basin 2 =Σ CA = 9.45 Acres

Total
Impervious = 5.4 + 9.45 = 14.85 Acres
Drainage
Area

Tillamook, Oregon
No Scale




① Proposed Outfall Location
② Highest Known Tide Elevation 107.0 ft.

# FIGURE 8
# SYSTEM LAYOUT

Tillamook, Oregon
No Scale

$A_1=\Sigma CA_1 = (0.9)(0.77)= 0.69$ Acres

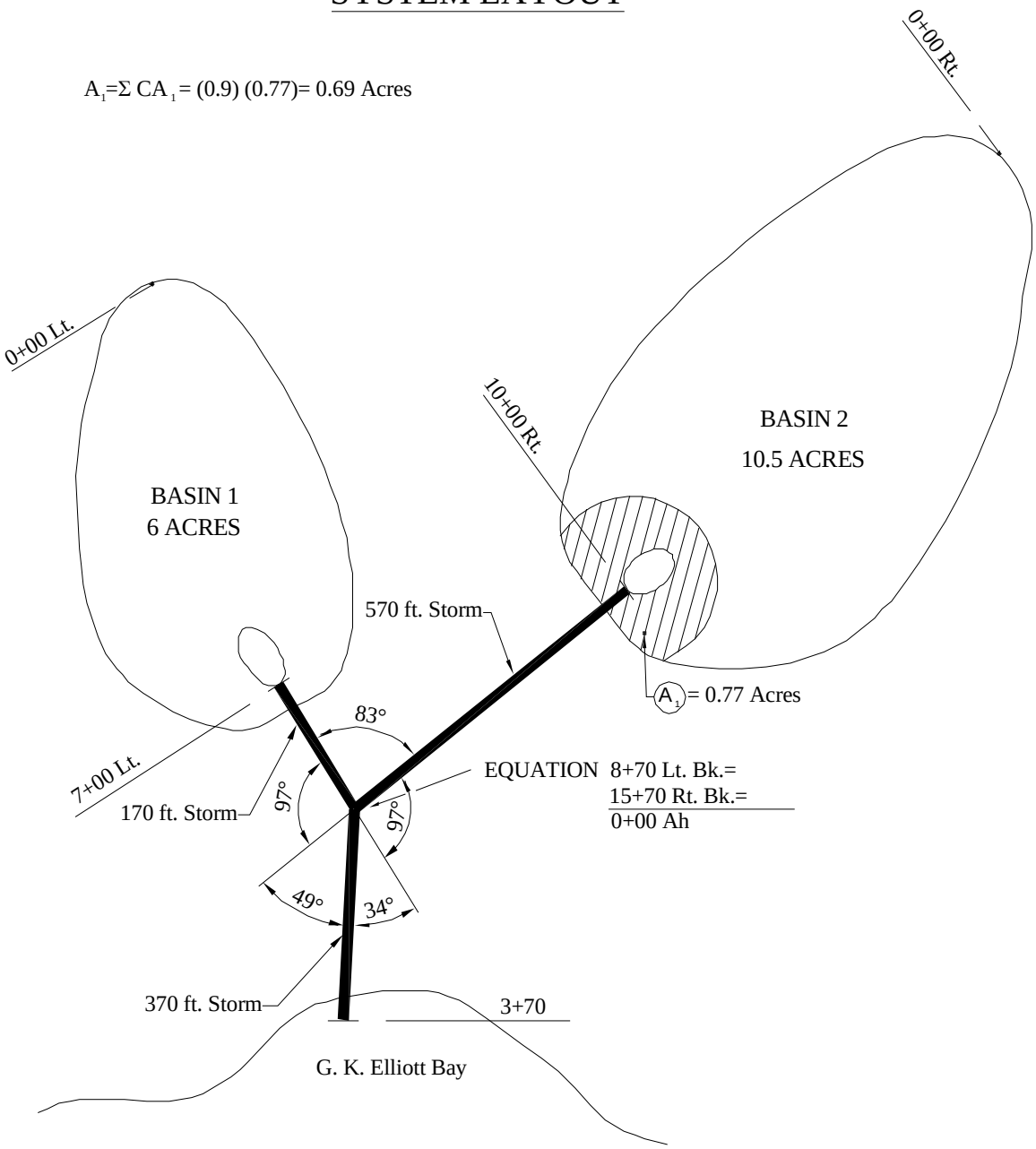


FIGURE 11
STORM DRAIN DESIGN SHEET
FULL AND PARTIAL-FULL FLOW

* 50- Year
PROJECT: Example Problem DESIGN FREQUENCY: 10- Year DESIGNED BY: JOE SMITH DATE: 2005

| Station | Node I.D. | Drainge Area Index No. | Drainge Area Area A | Runoff Coeff. C | Equiv. Area for 100% Runoff CA (3) x (4) Acres | Total Impervious Drainage Area ΣCA Acres | Time of Concent. or Flow Time, $T_c$ Minutes | Total Time of Concent. $T_c$ Minutes | Average Rainfall Intensity I in/hr | Design Discharge Q (6) x (9) cfs | Invert Slope S % | Pipe Size D Inches | Capacity Flowing Full $Q_f$ cfs | Velocity Flowing Full $V_f$ fps | Length L feet | Fall feet | Invert Elevation feet | Top of Manhole Elevation feet | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1a | 1b | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
| 10+00 Rt | | | | | | | | | | | | | | | | | | | |
| | | ② | 10.5 | .9 | 9.45 | 9.45 | | | | | | | | | | | | | |
| | | | | | | | 29.5 | 29.5 | 1.25 | | | | | | | | | | |
| 7+00 Lt | | | | | | | | | | 11.8 | | | | | | | 106.29<br>112.79 | 117.0 | |
| | | ① | 6.0 | .9 | 5.40 | 14.85 | | | | | 0.27<br>1.316 | 24<br>18 | 11.8<br>12.2 | 3.7<br>7.0 | 570 | | | | |
| | | | | | | | 1.4 | 30.9 | *1.65 | | | | | | | 1.54<br>7.50 | 104.75<br>105.29 | | (in) |
| 15+70 Rt Bk → | | | | | | | | | | 24.5 | | | | | | | 100.27 | 109.5 | (out) |
| 0+00 Ah | | | | | | | | | | | 0.145 | 36 | 24.5 | 3.6 | 370 | | | | |
| | | | | | | | | | | | | | | | | 0.67 | | | |
| 3+70 | | | | | | | | | | | | | | | | | 99.6 | | (outfall) |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| 0+00 Lt | | | | | | | | | | | | | | | | | | | |
| | | ① | 6.0 | .9 | 5.40 | 5.40 | | | | | | | | | | | | | |
| | | | | | | | 6.3 | 6.3 | 2.50 | | | | | | | | | | |
| 7+00 Lt | | | | | | | | | | 13.5 | | | | | | | 104.92 | 110.0 | |
| | | A1 | 0.77 | .9 | 0.69 | 6.09 | | | | | 0.294 | 27 | 16.5 | 4.2 | 170 | | | | |
| | | | | | | | 0.7 | 7.0 | *3.20 | | | | | | | 0.50 | 104.42 | | (in) |
| 8+70 Lt Bk → | | | | | | | | | | 19.5 | | | | | | | 100.27 | 109.5 | (out) |
| 0+00 Ah | | | | | | | | | | | 0.145 | 36 | 24.5 | 3.6 | 370 | | | | |
| | | | | | | | | | | | | | | | | 0.67 | | | |
| 3+70 | | | | | | | | | | | | | | | | | 99.6 | | (outfall) |
| | | | | | | | | | | | | | | | | | | | |
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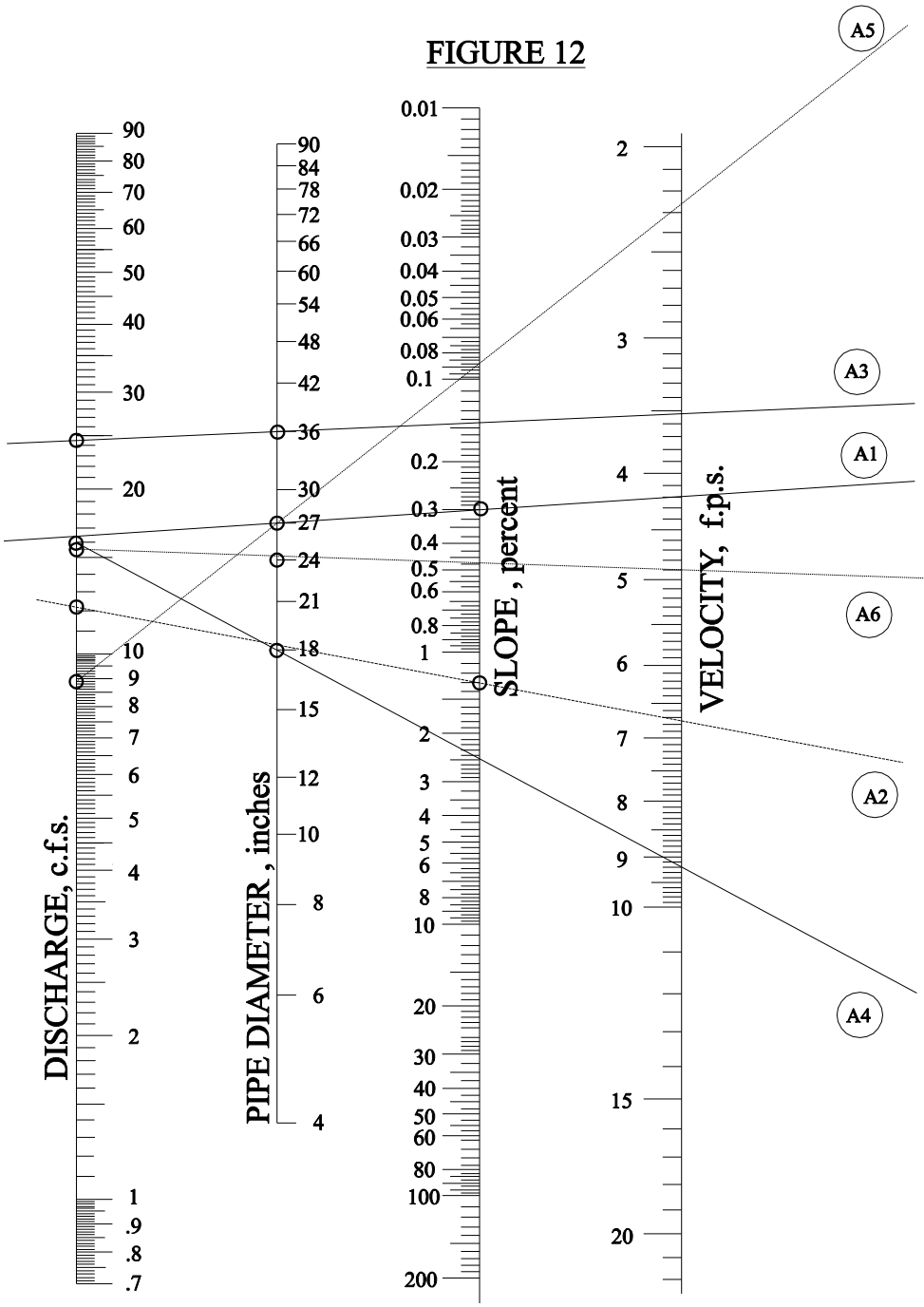
# FIGURE 12




KEY NOTES:

**A1** Step A.10
LT Branch
S=0.3%
27-inch DIA.
16.5 cfs

**A2** Step A.10
RT Branch
S=1.3%
18-inch DIA.
12.2 cfs

**A3** Step B.6
24.5 cfs
36-inch DIA.

**A4** Step B.6
15.6 cfs
18-inch DIA.

**A5** Step B.6
8.9 cfs
27-inch DIA.

**A6** Step B.19
15.6
S=0.47%
24-inch DIA.

## CAPACITY AND VELOCITY NOMOGRAPH FOR CIRCULAR CONCRETE PIPES PIPE FLOWING FULL n = 0.013

FIGURE 13
STORM DRAIN DESIGN SHEET
SURCHARGED FLOW (Pressure Flow Conditions)

PROJECT: Example Problem DESIGN FREQUENCY: 50-Year DESIGNED BY: JOE SMITH DATE: 2005

| PIPE DATA | | | | | HEAD LOSSES | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Station<br>I.D. | Node I.D. | Discharge<br>cfs | Pipe Size<br>D<br>inches | Pipe Length<br>L<br>feet | $V_1$<br>fps | $\frac{V_1^2}{2g}$<br>feet | $V_2$<br>fps | $\frac{V_2^2}{2g}$<br>feet | Minor Pipe Losses<br>feet | Friction Slope<br>$S_f$<br>% | Friction Head Loss<br>(4) x (10)<br>feet | Energy Grade Line<br>$EGL_0$<br>feet | K | $K\frac{V^2}{2g}$<br>feet | $EGL_i$<br>feet | Piezometric or water Surface<br>(HGL)<br>feet | Crown of Pipe<br>feet | Low Gutter/ Surface Elevation<br>feet | Remarks |
| 1a | 1b | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
| Outfall | | | | | | | | | | | | 107.00 | | | | 107.00 | | | Outfall |
| 3+70 | | 24.5 | 36 | | 3.6 | 0.20 | 0.0 | 0.0 | 0.20 | | | | | | 107.20 | 107.0 | 102.60 | | Just inside pipe at outfall pipe |
| | | | | | | | | | | | | | | | | | | | |
| 0+00 Ah | | 24.5 | 36 | 370 | 3.6 | 0.20 | | | 0.0 | 0.145 | 0.54 | 107.74 | | | | 107.54 | 103.27 | | Just inside inlet end of outfall pipe |
| | | | | | | | | | | | | | 0.97 | 0.20 | 107.94 | | | 109.50 | Inside manhole |
| | | | | | | | | | | | | | | | | | | | |
| 15+70 Rt | | 15.6 | 18 | 570 | 8.8 | 1.21 | | | | | | 107.94 | | | | 106.73 | | | Just inside pipe as it enters manhole |
| | | | | | | | | | | | | | | | | | | | |
| 10+00 Rt | | 15.6 | 18 | 570 | 8.8 | 1.21 | | | | 2.30 | 13.11 | 121.05 | | | | 119.84 | | | Just inside outlet of pipe |
| | | | | | | | | | | | | | | | | | | | |
| 9+99 Rt | | | | | | | | | 0.24 | | | | | | 121.29 | 121.29 | | 117.00 | Exceeds ground level, try next size pipe |
| | | | | | | | | | | | | | | | | | | | |
| 15+70 Rt | | 15.6 | 24 | 570 | 5.0 | .39 | | | | | | 107.94 | | | | 107.55 | 106.75 | | 2nd Try |
| | | | | | | | | | | | | | | | | | | | |
| 10+00 Rt | | 15.6 | 24 | 570 | 5.0 | .39 | | | | 0.47 | 2.68 | 110.62 | | | | 110.23 | 108.29 | | |
| | | | | | | | | | | | | | | | | | | | |
| 9+99 Rt | | | | | | | | | 0.08 | | | | | | 110.70 | 110.70 | | 117.00 | Does not exceed ground elevations |
| | | | | | | | | | | | | | | | | | | | Design is okay |
| | | | | | | | | | | | | | | | | | | | |
| Outfall | | | | | | | | | | | | 107.00 | | | | 107.00 | | | Outfall |
| 3+70 Lt | | 24.5 | 36 | | 3.6 | .20 | 0.0 | 0.0 | | | | | | | 107.20 | 107.00 | 102.6 | | Just inside pipe at outfall |
| 0+00 Ah | | | | | | | | | | 0.145 | 0.54 | 107.74 | | | | 107.54 | 103.27 | | Just inside inlet end of outfall pipe |
| | | | | | | | | | | | | | 0.97 | 0.20 | 107.94 | | | 109.50 | Inside manhole |
| 8+70 Lt | | 8.9 | 27 | 170 | 2.2 | .08 | | | 0.02 | | | 107.96 | | | | 107.88 | 106.67 | | Just inside pipe as it enters manhole |
| 7+00 Lt | | 8.9 | 27 | 170 | 2.2 | .08 | | | | 0.090 | 0.15 | 108.11 | | | | 108.03 | | | Just inside outlet pipe |
| 6+99 Lt | | | | | | | | | 0.02 | | | | | | 108.13 | 108.13 | 107.17 | 110.00 | Just outside inlet end of pipe |
| | | | | | | | | | | | | | | | | | | | HGL & EGL do not exceed ground |
| | | | | | | | | | | | | | | | | | | | elevations. Design is OK |
| | | | | | | | | | | | | | | | | | | | |

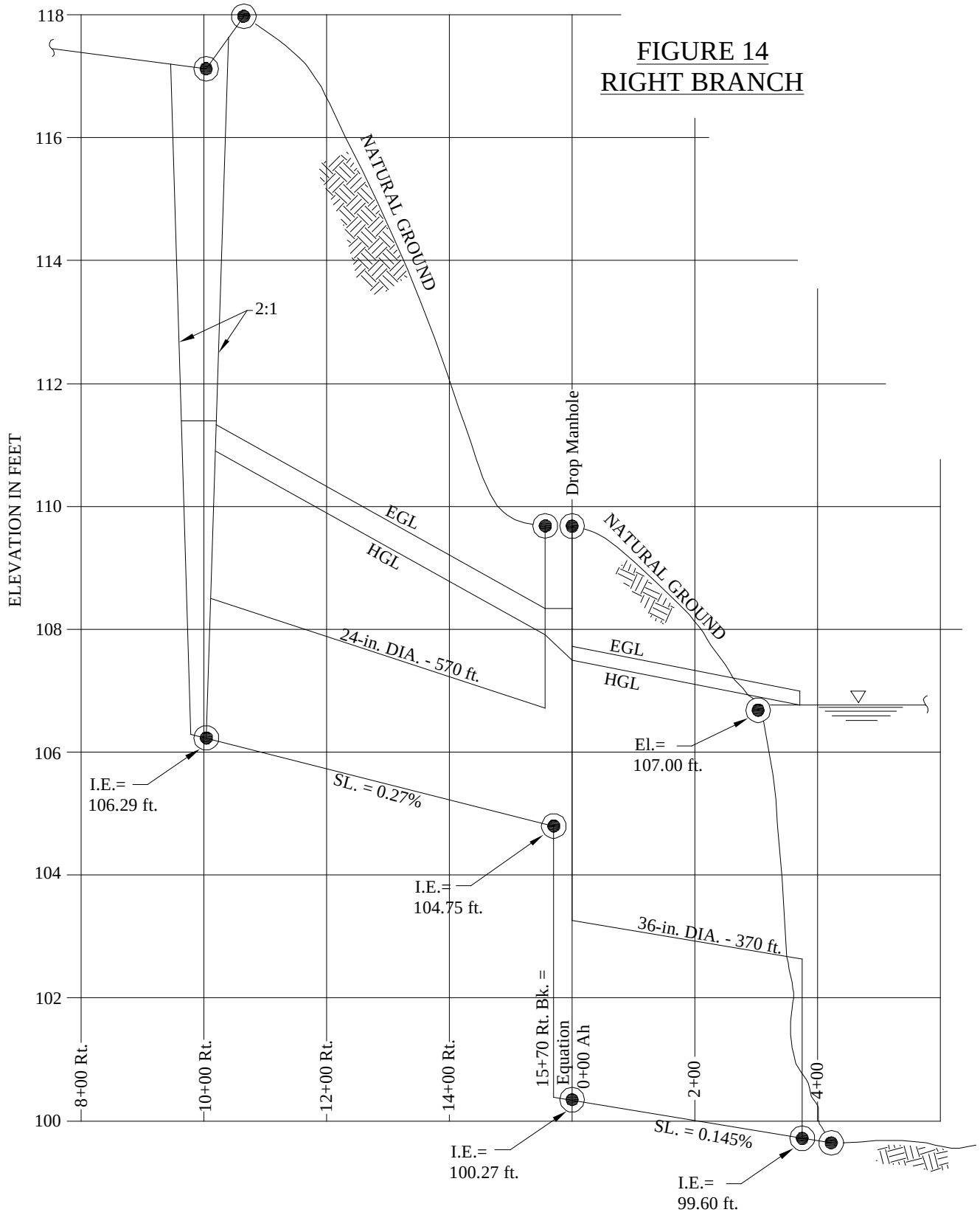


FIGURE 14
RIGHT BRANCH
ELEVATION IN FEET
118
116
114
112
110
108
106
104
102
100
NATURAL GROUND
2:1
Drop Manhole
EGL
HGL
NATURAL GROUND
24-in. DIA. - 570 ft.
EGL
HGL
El.=
107.00 ft.
I.E.=
106.29 ft.
SL. = 0.27%
I.E.=
104.75 ft.
36-in. DIA. - 370 ft.
8+00 Rt.
10+00 Rt.
12+00 Rt.
14+00 Rt.
15+70 Rt. Bk. =
Equation
0+00 Ah
2+00
4+00
SL. = 0.145%
I.E.=
100.27 ft.
I.E.=
99.60 ft.

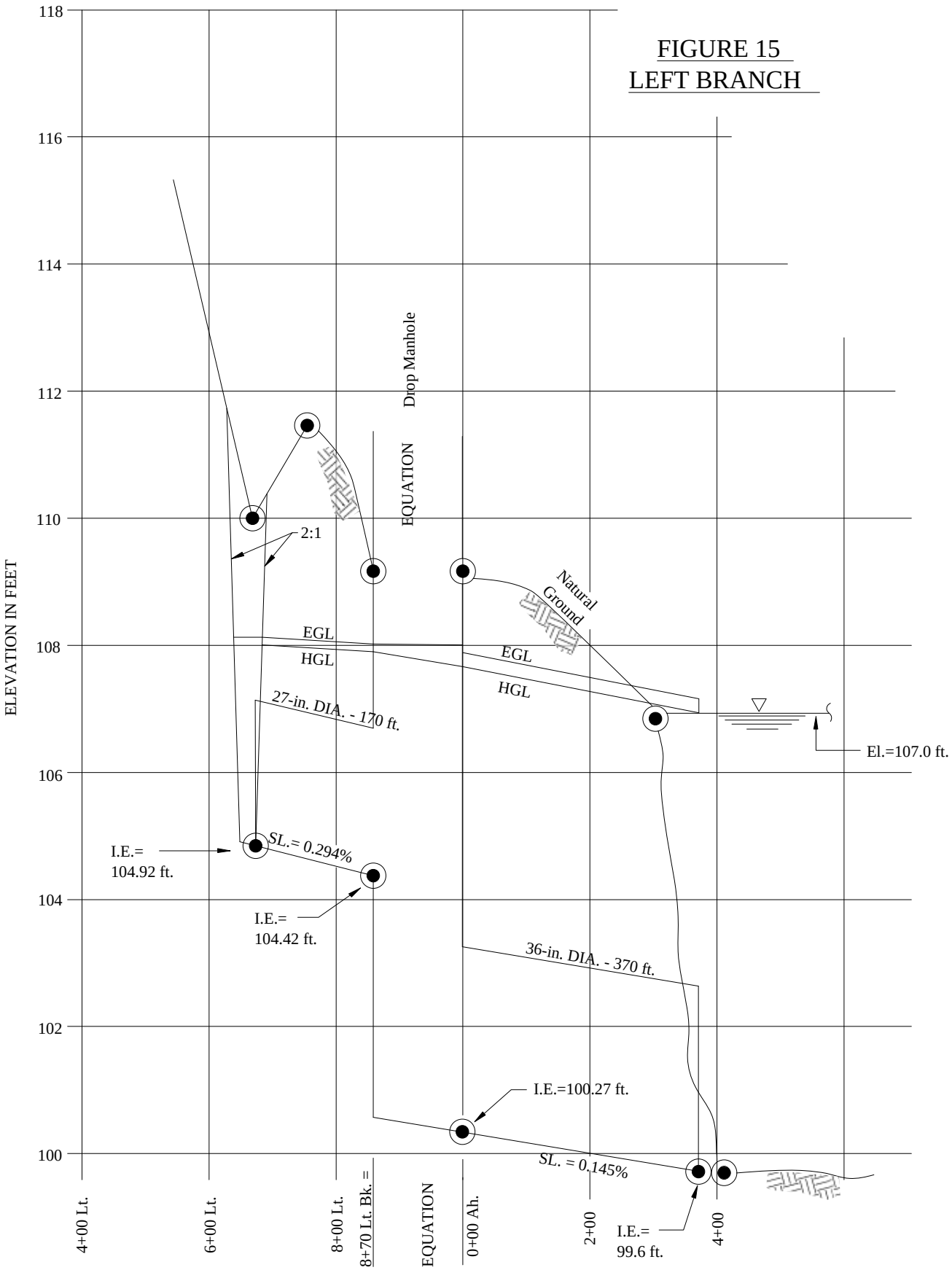


FIGURE 15
LEFT BRANCH
ELEVATION IN FEET
118
116
114
112
110
108
106
104
102
100
Drop Manhole
EQUATION
2:1
Natural Ground
EGL
HGL
EGL
HGL
27-in. DIA. - 170 ft.
El.=107.0 ft.
I.E.= 104.92 ft.
SL.= 0.294%
I.E.= 104.42 ft.
36-in. DIA. - 370 ft.
I.E.=100.27 ft.
SL. = 0.145%
I.E.= 99.6 ft.
4+00 Lt.
6+00 Lt.
8+00 Lt.
8+70 Lt. Bk. =
EQUATION
0+00 Ah.
2+00
4+00

# APPENDIX H – DESIGN CHARTS

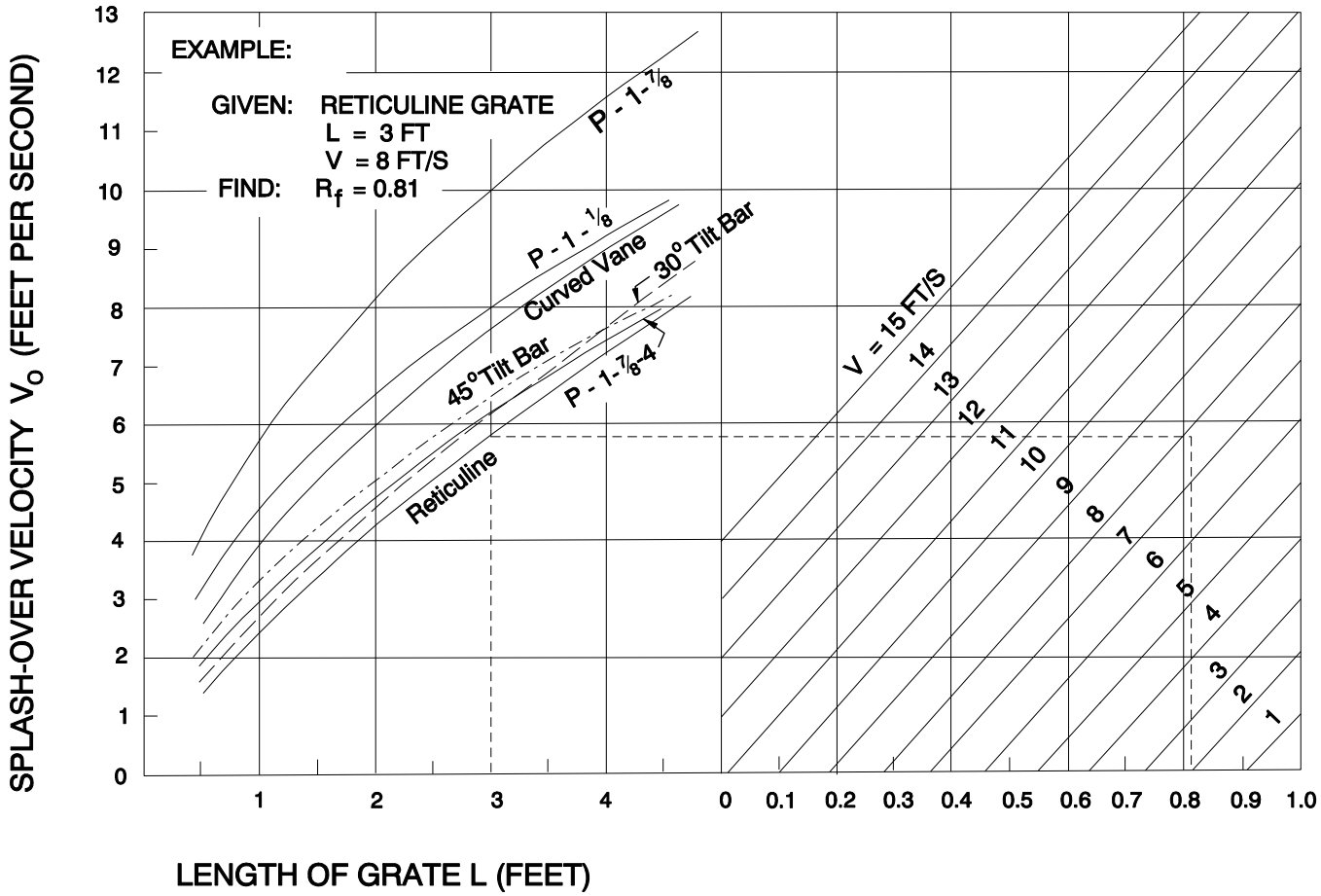



**Chart 1 - Grate Inlet Frontal Flow Interception Efficiency**

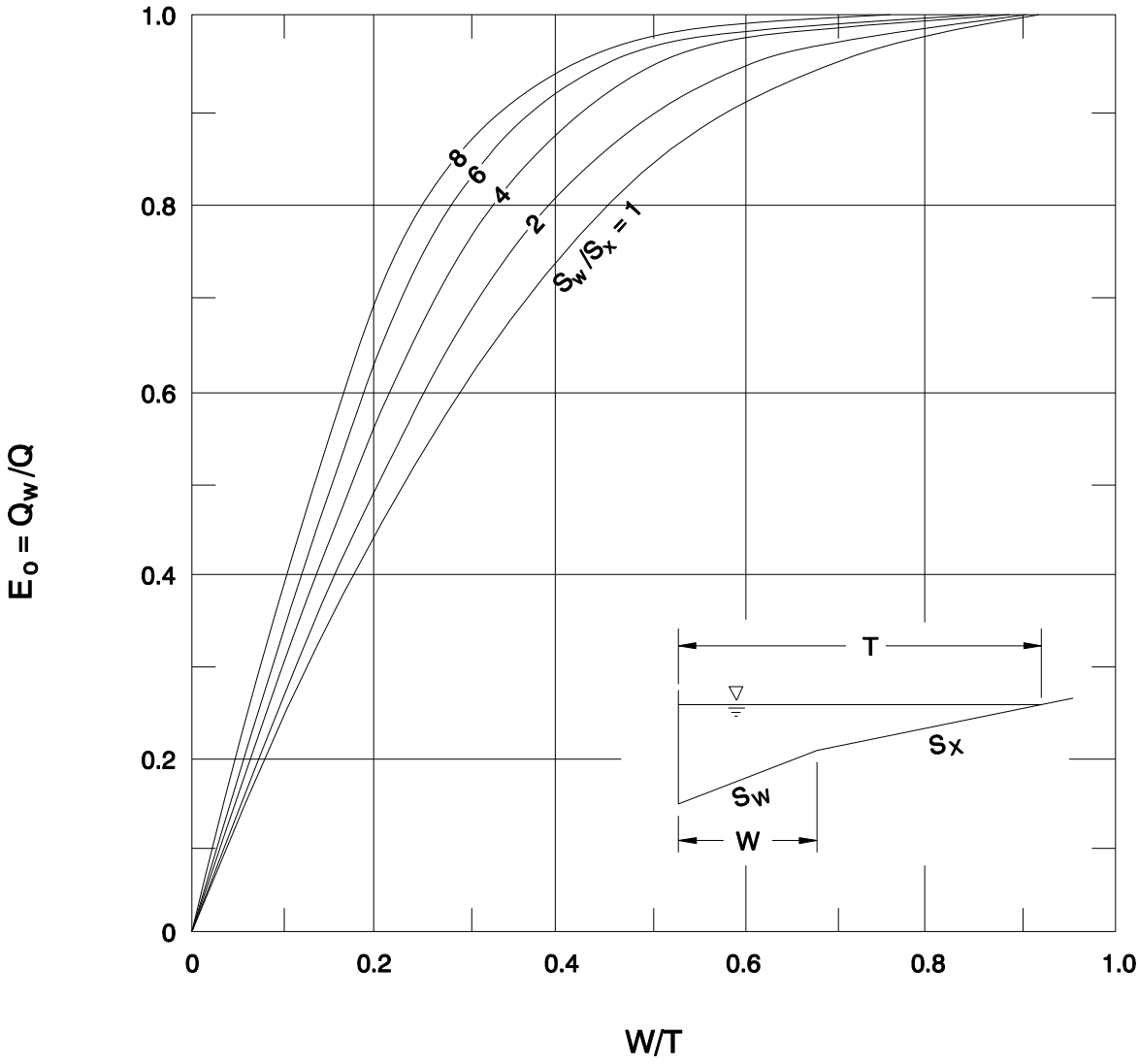



**Chart 2 - Ratio of Frontal Flow to Total Gutter Flow**

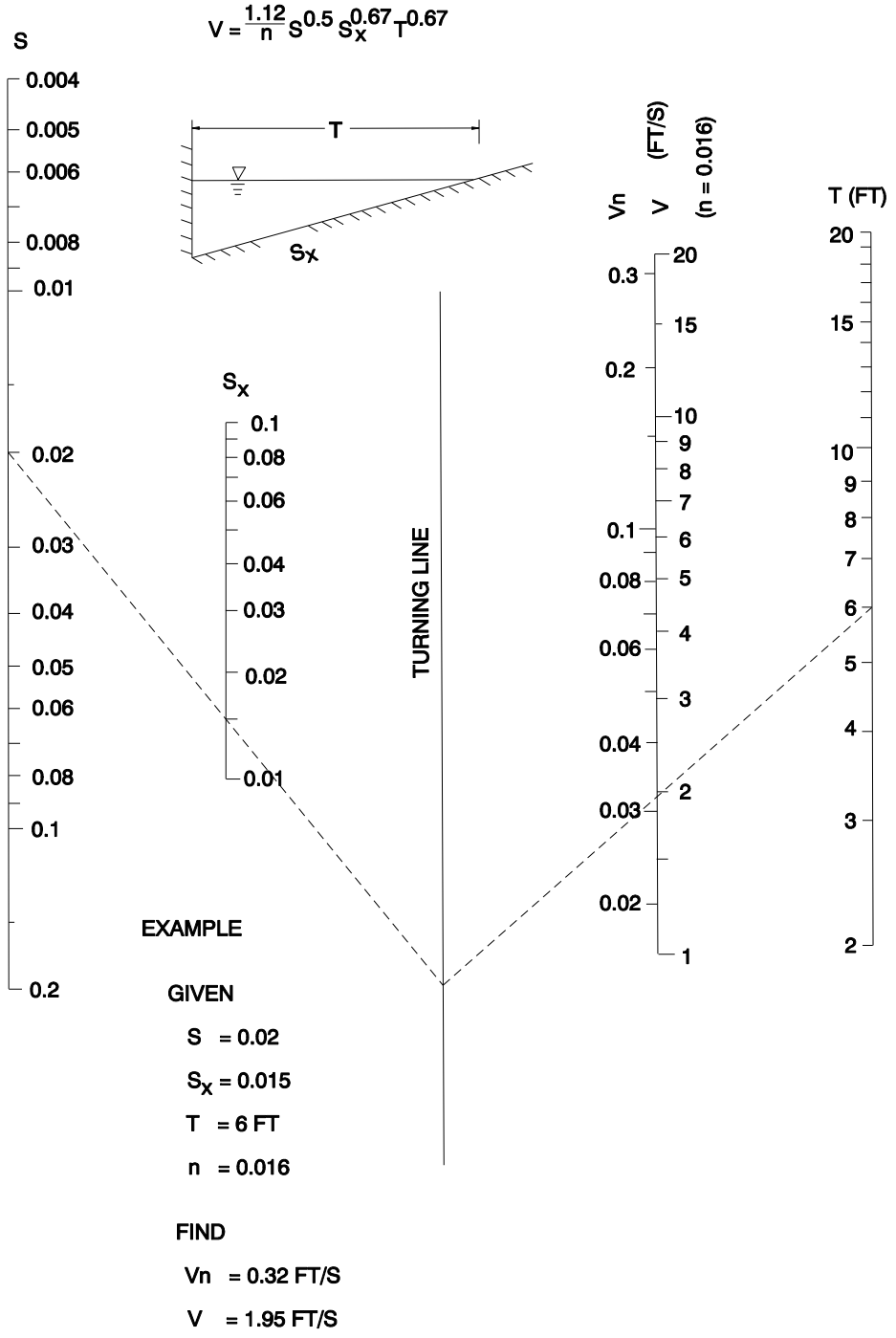



**Chart 3 - Velocity in Triangular Gutter Sections**

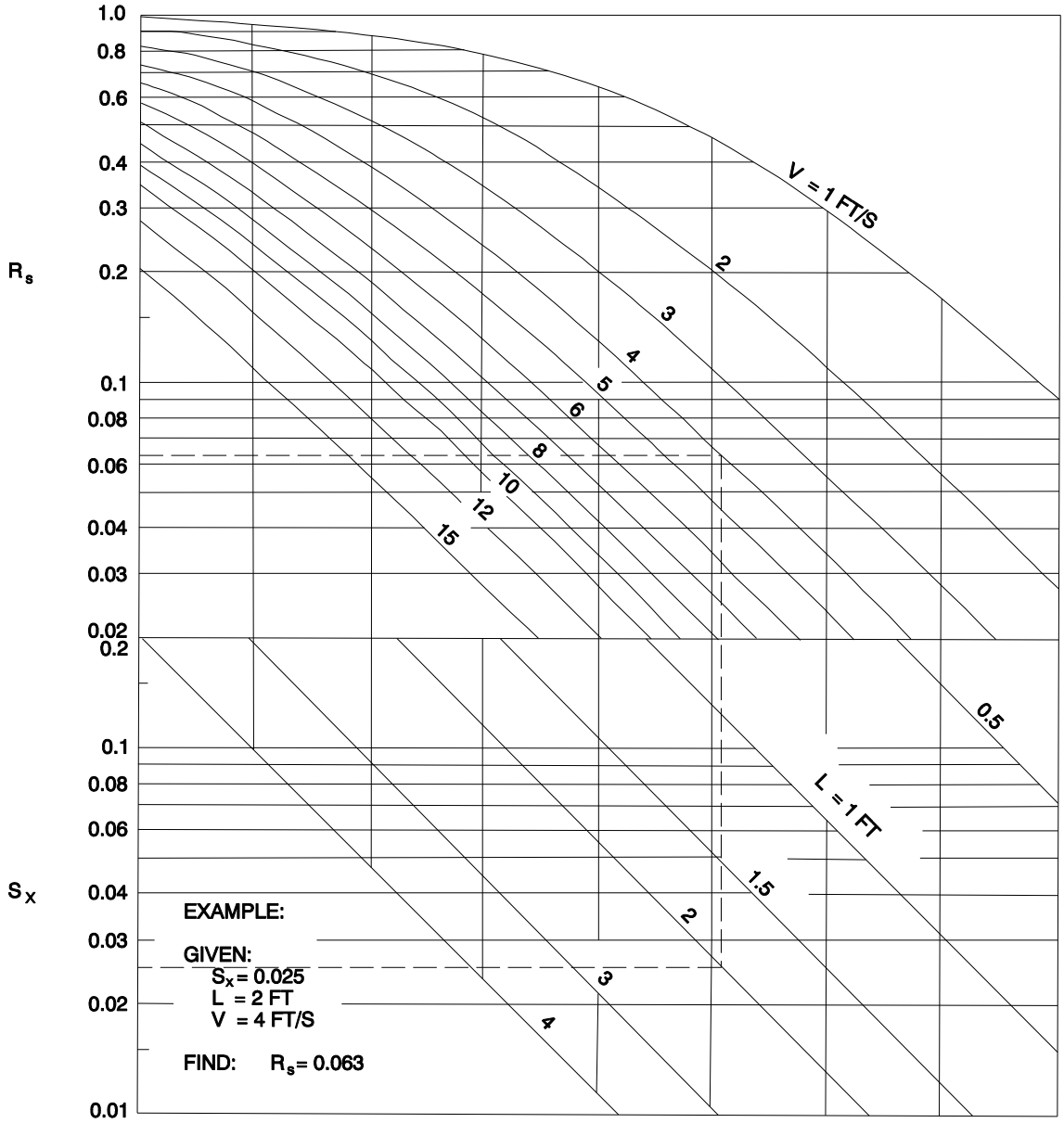



**Chart 4 - Grate Inlet Side Flow Intercept Efficiency**

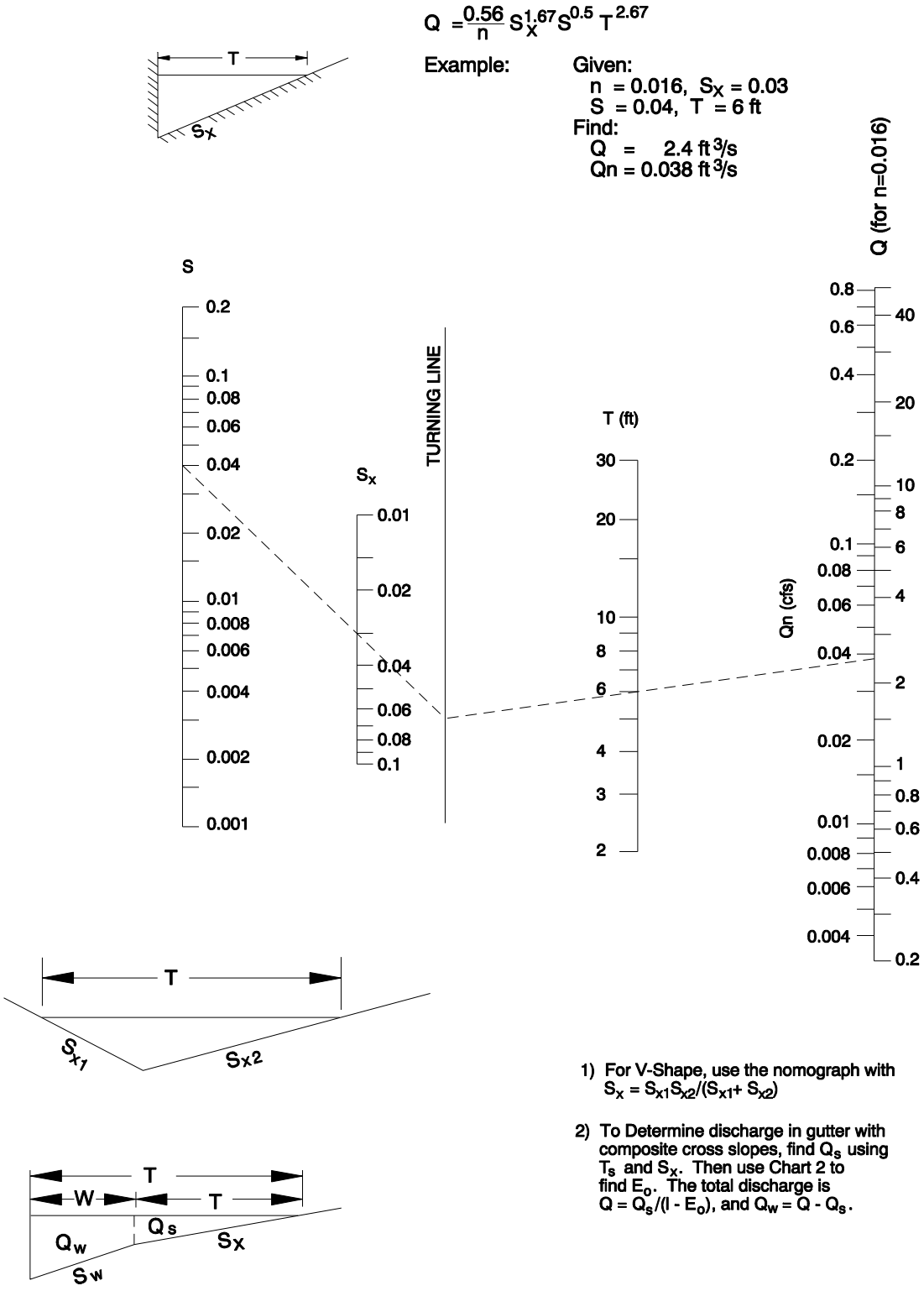



**Chart 5 - Flow in Triangular Gutter Sections**

$L_T = 0.6Q^{0.42}S^{0.3}(1/nS_X)^{0.6}$

For composite cross slopes, use $S_e$ for $S_X$.

$S_e = S_X + S'_W E_o$ ; $S'_W = a/W$

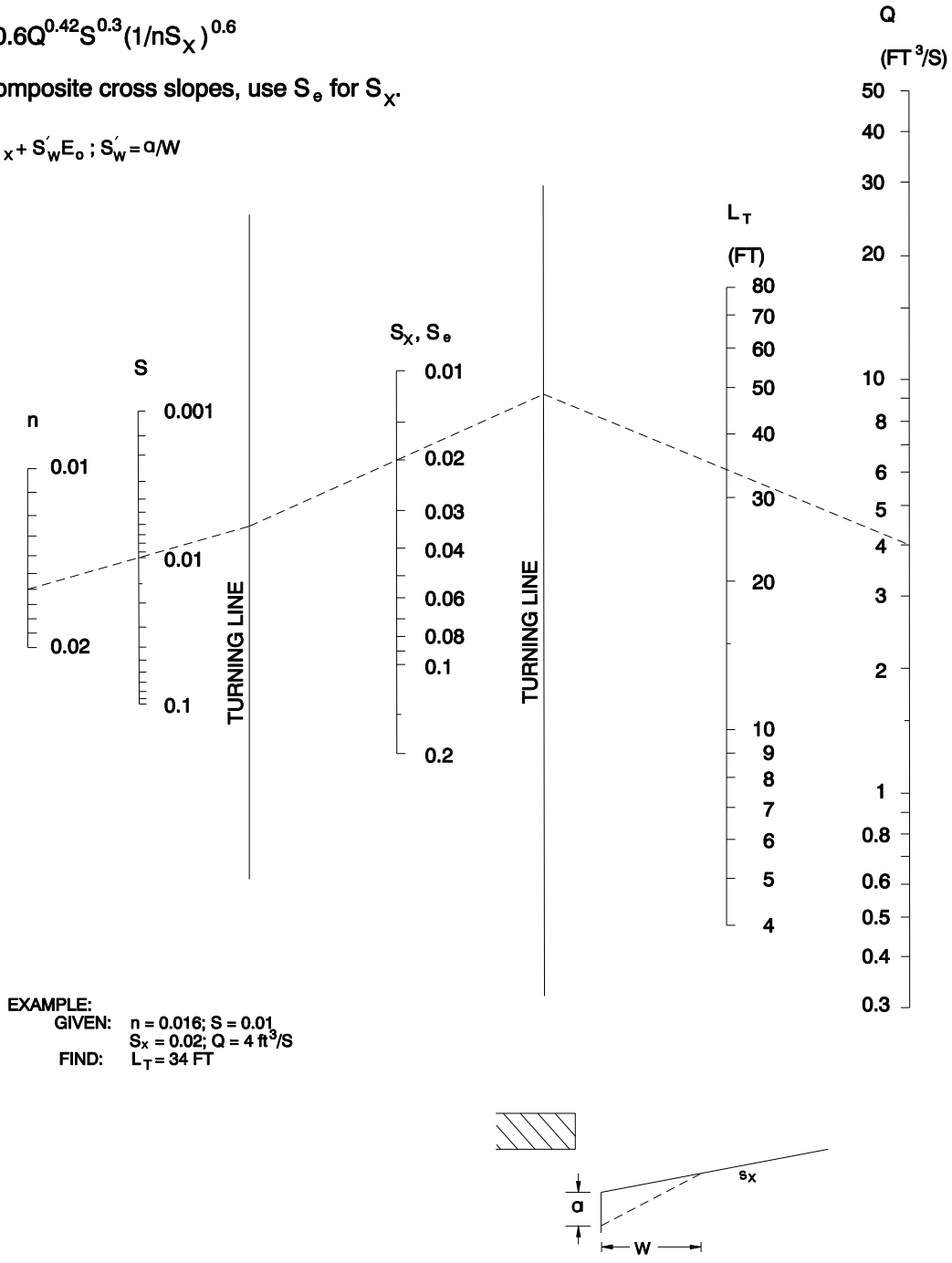



**Chart 6 - Curb-Opening and Slotted Drain Inlet Length for Total Interception**

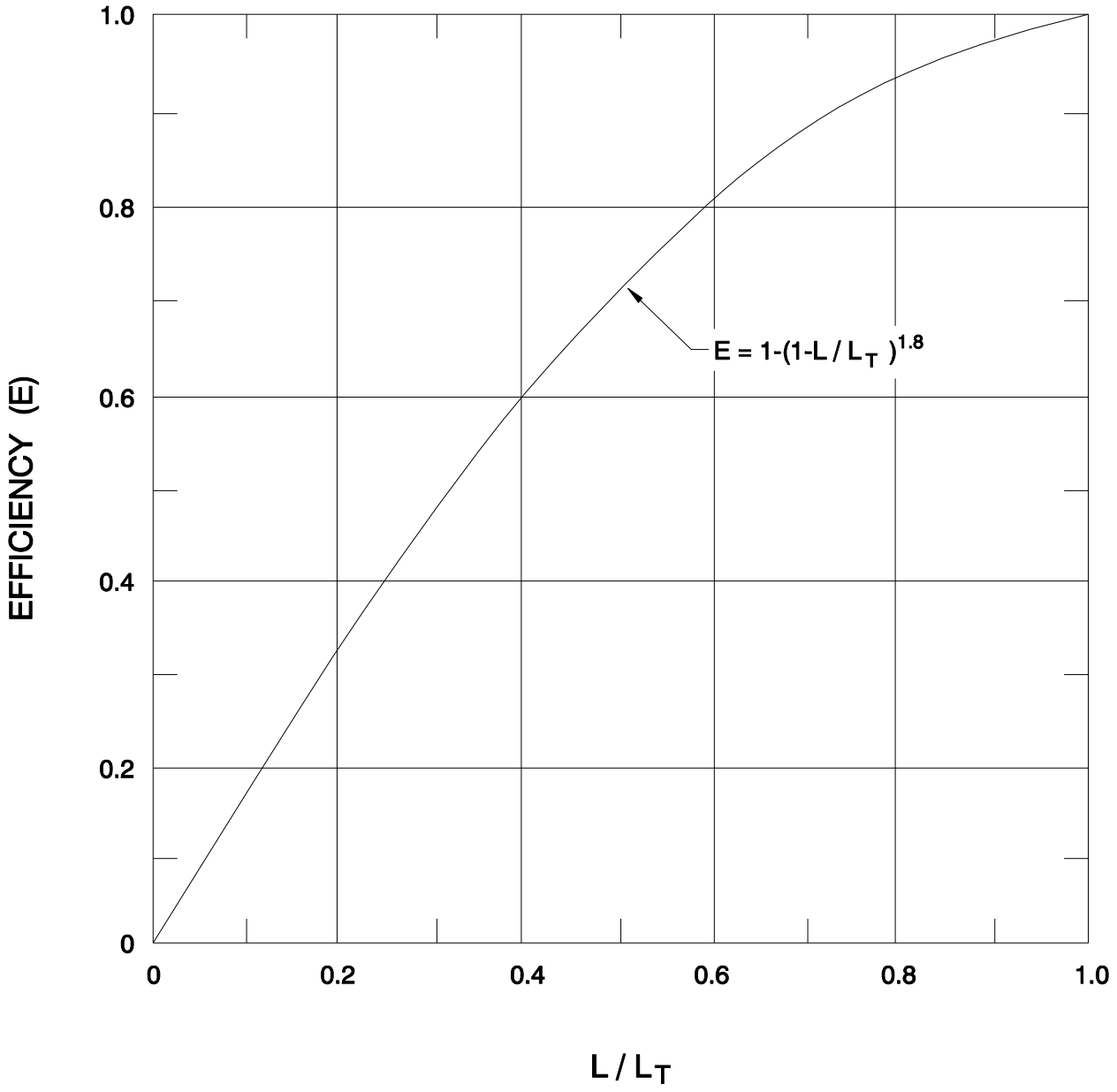



**Chart 7 – Curb-Opening and Slotted Drain Inlet Interception Efficiency**

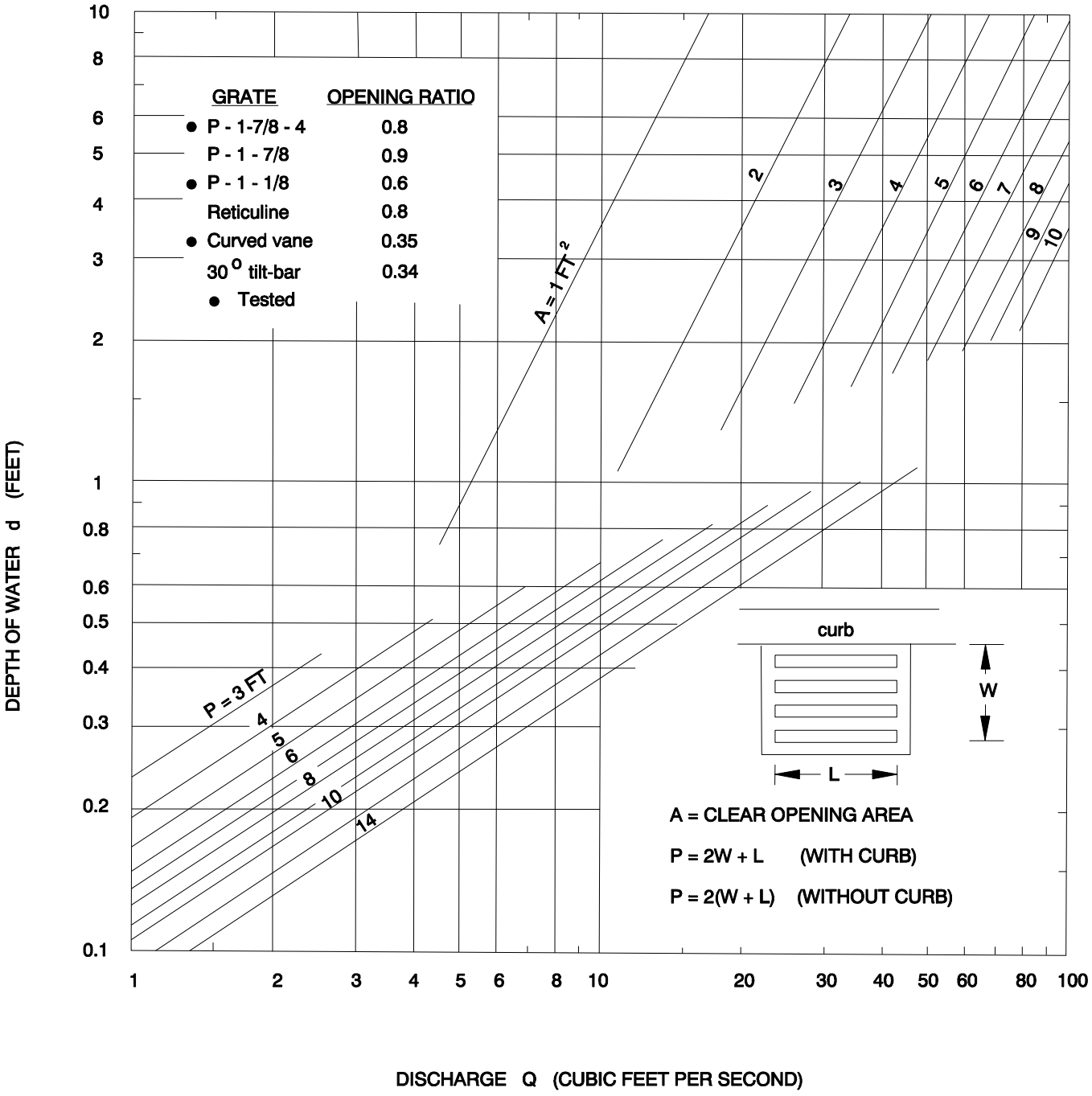



**Chart 8 - Grate Inlet Capacity in Sump Conditions**

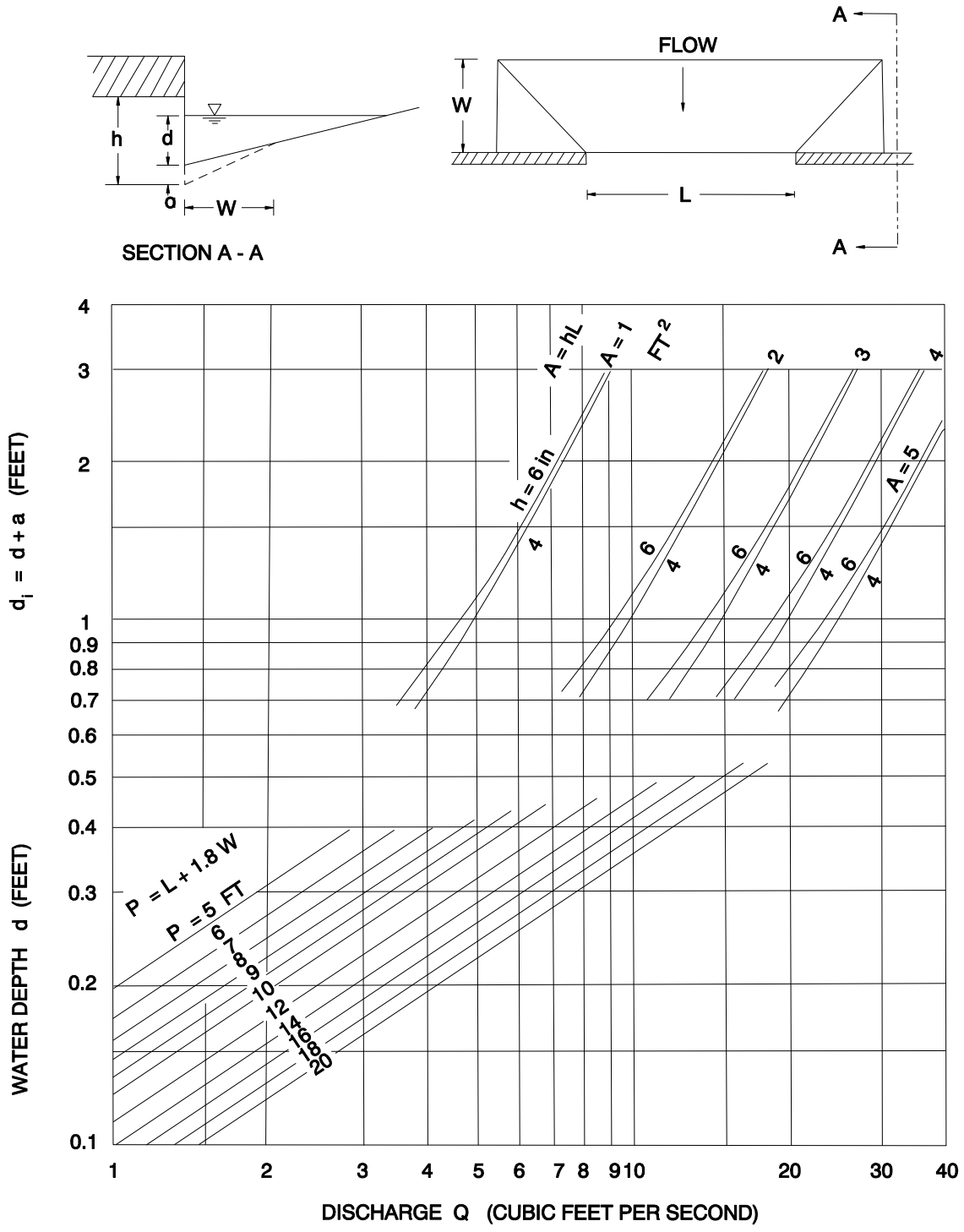



**Chart 9 - Depressed Curb-opening Inlet Capacity in Sump Locations**

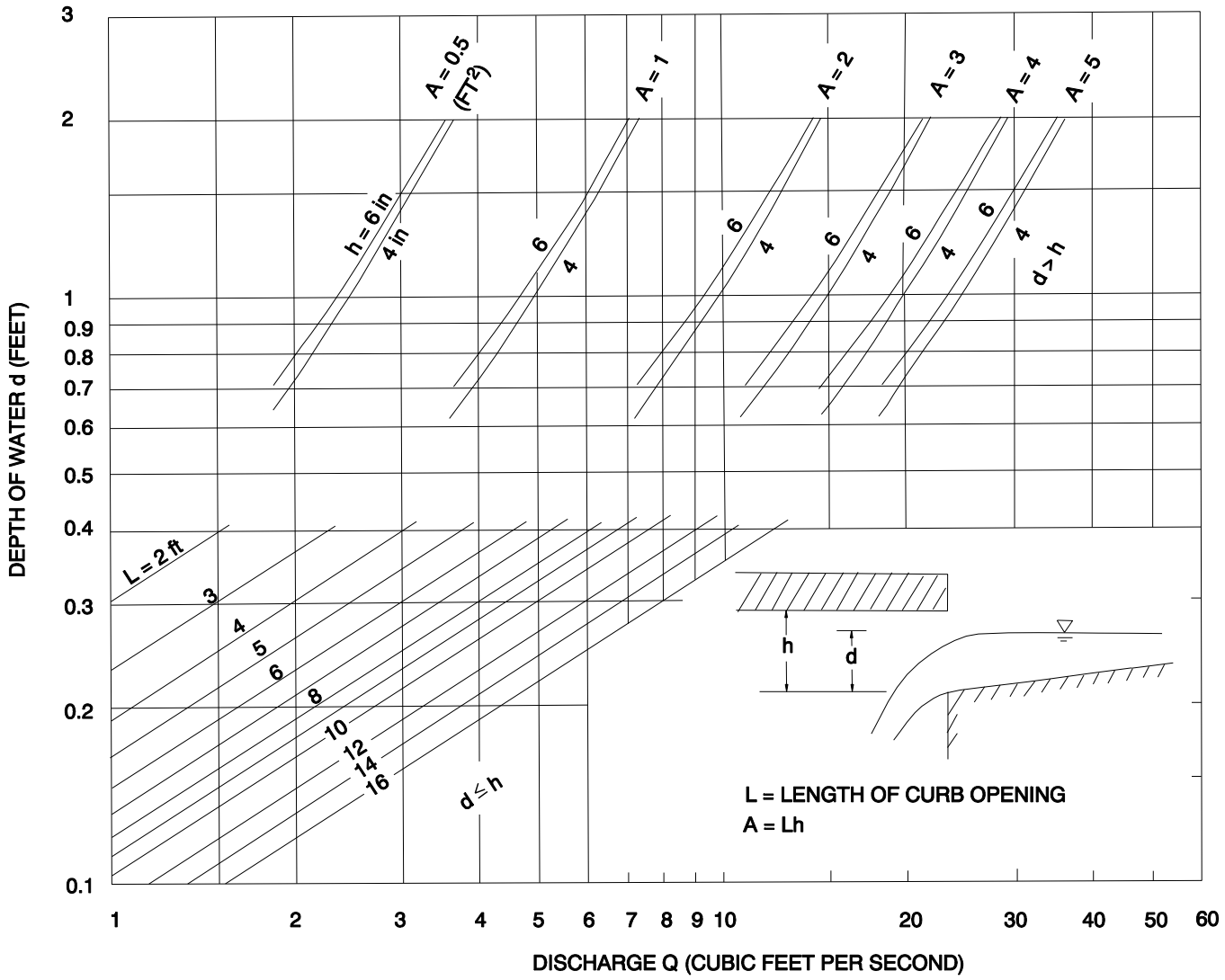



**Chart 10 - Undepressed Curb Opening Inlet Capacity in Sump Conditions**

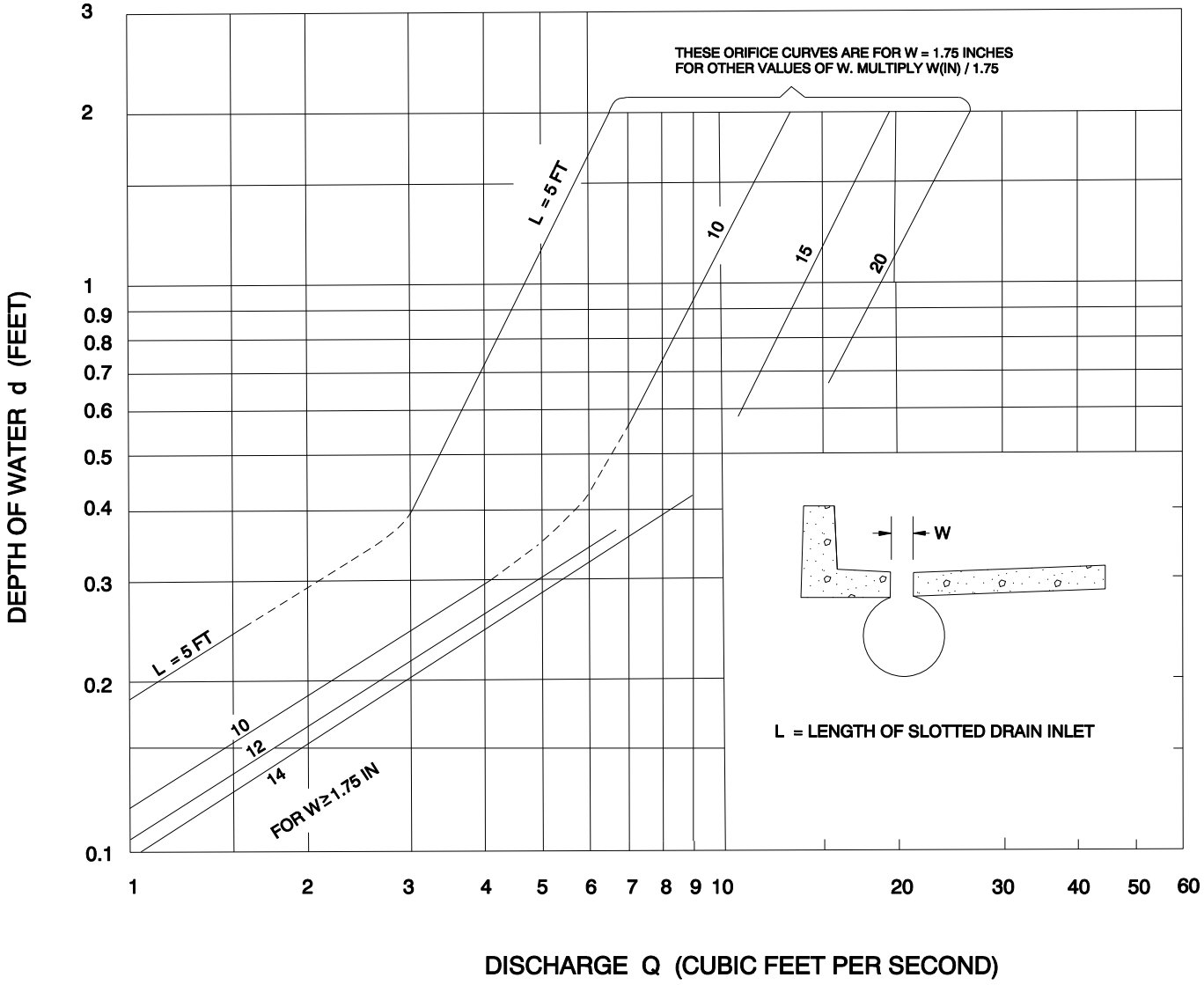



**Chart 11 – Slotted Drain Inlet Capacity in Sump Location**

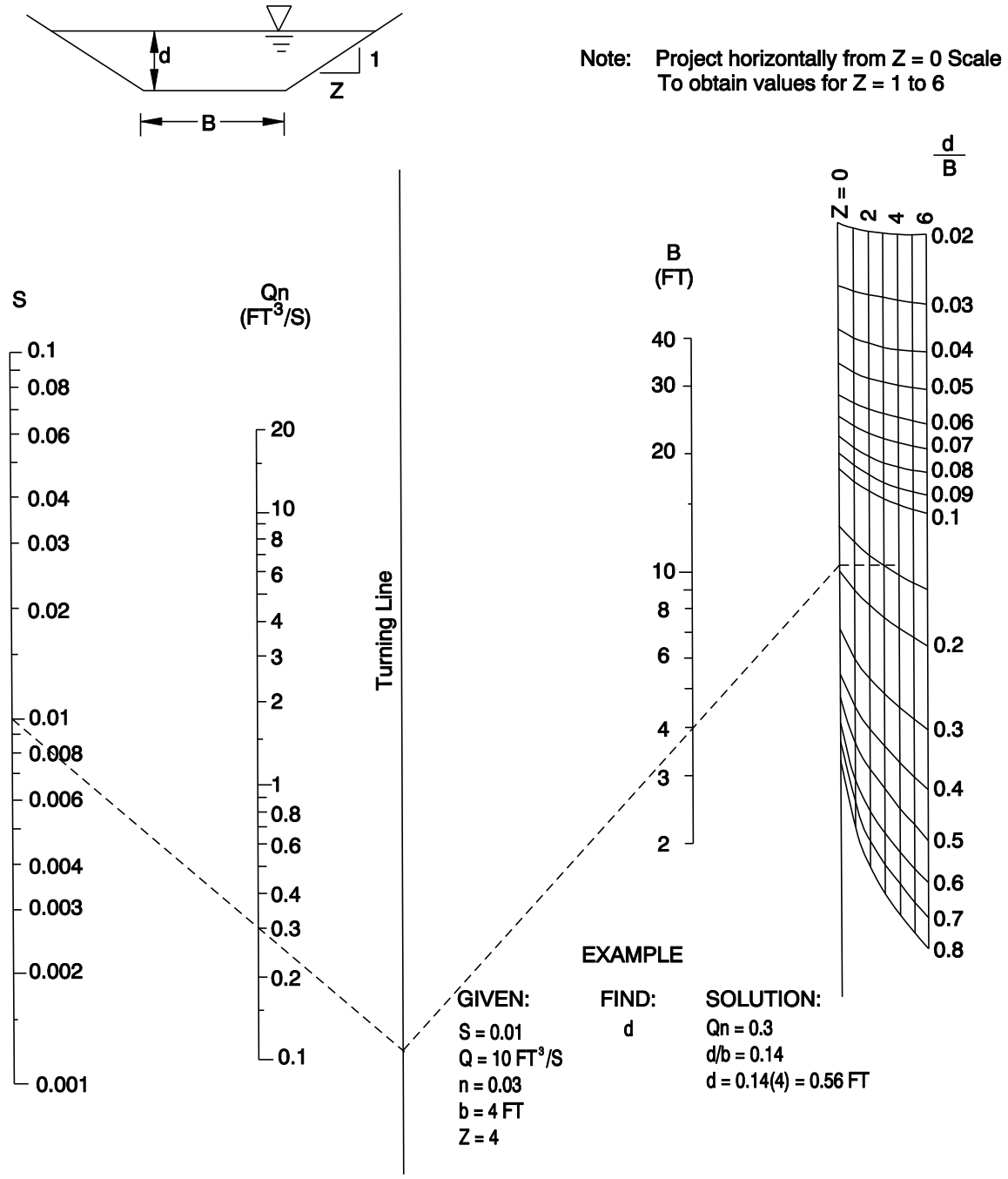



**Chart 12 - Solution to Manning's Equation for Channels of Various Side Slopes**

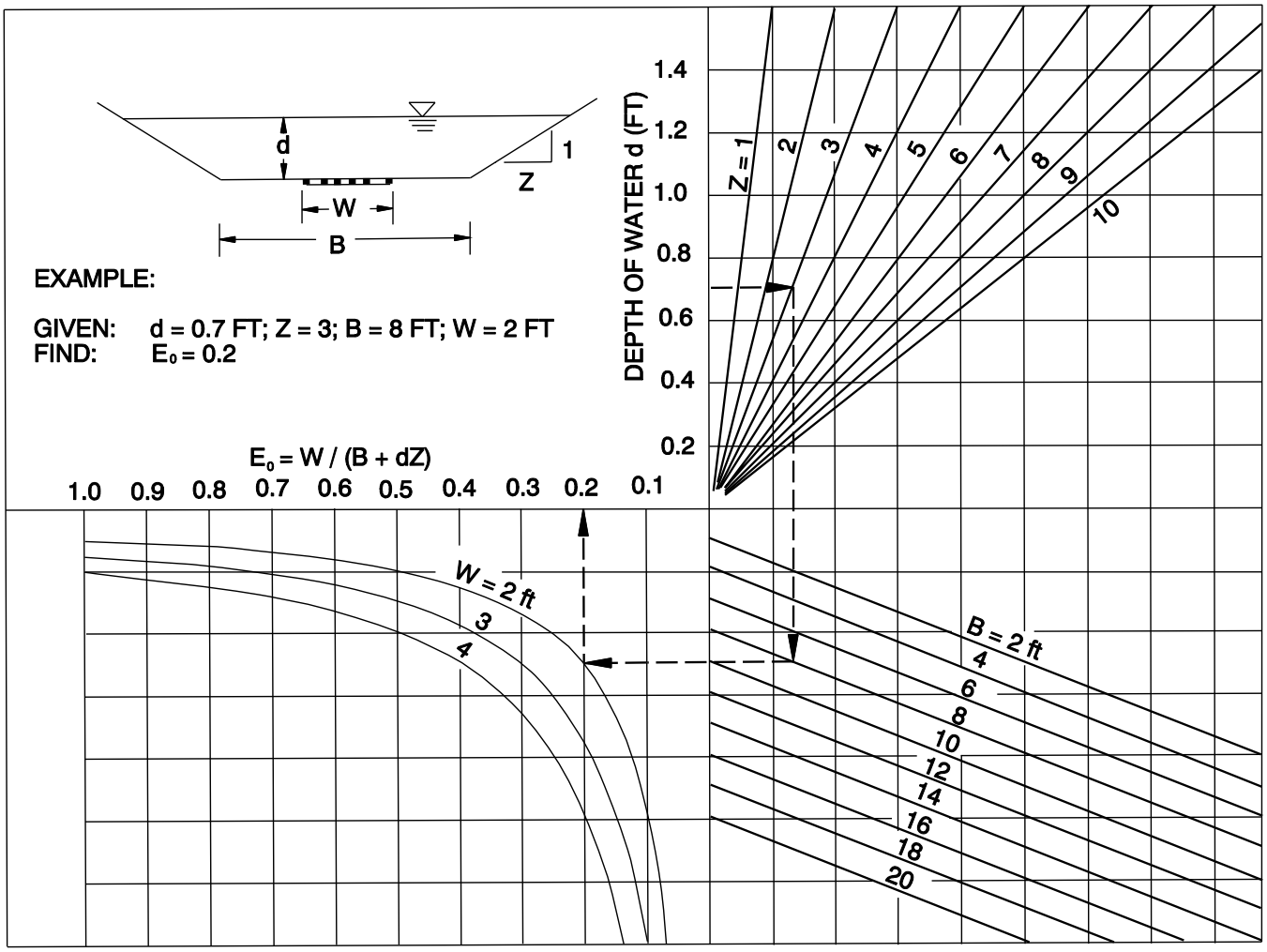



**Chart 13 - Ratio of Frontal Flow to Total Flow in a Trapezoidal Channel**

# CHAPTER 14

# WATER QUALITY

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## APPENDICES

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**Appendix B– Biofiltration Facilities**
**Appendix C – Media Filtration Facilities**
**Appendix D – Density Separation Facilities**
**Appendix E – Stormwater Treatment Facility Components**
**Appendix F – Oil Control Structures**

## 14.1 Introduction

This chapter provides minimum requirements and technical guidance for the planning and design of permanent stormwater control facilities that provide water quality treatment. The methodology is intended for those with an understanding of basic hydrologic and hydraulic methods and some experience in the design of stormwater structures. Projects that follow the stormwater best management practices contained in this chapter should achieve compliance with federal and state water quality regulations. While federal and state water quality requirements are subject to change, this chapter is based on the best practicable engineering approaches to stormwater management currently available for ODOT facilities.

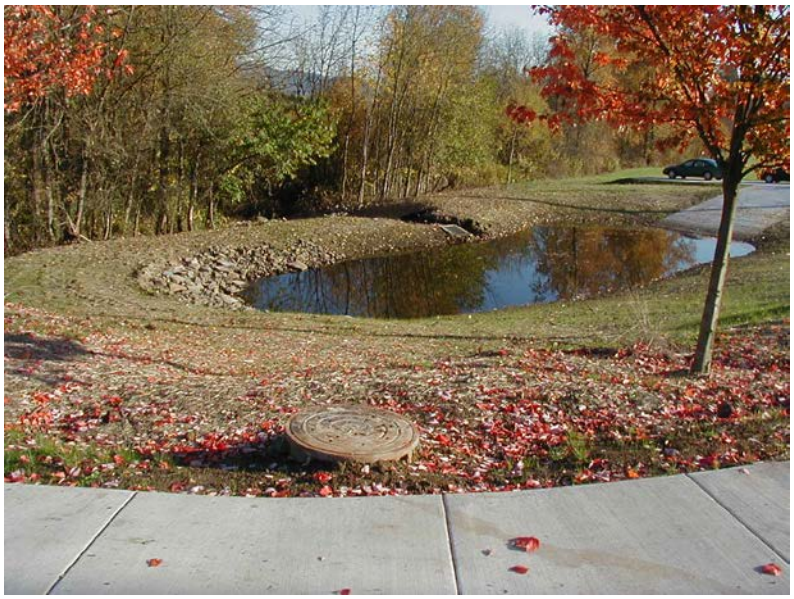

**Figure 14-1 Extended detention dry pond**

A water quality facility, also called a stormwater treatment facility or BMP (for Best Management Practice), is that portion of the highway drainage system whose function is to remove or reduce pollutants in roadway runoff prior to discharging to receiving waters. The facility receives surface water by way of sheet flow or through a conveyance system of inlets, pipes, and ditches. They range from unmodified or enhanced side slopes within the right-of-way to intensely engineered facilities.

Planning and designing the treatment of highway runoff needs to be done early in project development. Delay only limits options and leads to less effective and more expensive facilities, and can cause project deadlines to slip. Proposed treatment facility types, locations and area requirements should be determined before DAP so design, right-of-way acquisition and

environmental permitting can proceed on schedule. Choosing the best BMPs for a project is a collaborative effort; the water quality designer will need to coordinate with Roadway, Environmental, and Right-of-Way staff. Guidance on the selection and siting of stormwater Best Management Practices is provided in Section 14.9.

The stormwater BMPs described in this chapter, its appendices and current technical bulletins are approved for use on ODOT projects. Projects are not limited to these BMPs. If new, innovative BMPs or substantially modified approved BMPs provide the best option for a project, their use may be approved through the Hydraulic Design Deviation process. See **Chapter 3** for guidance on the Hydraulic Design Deviation process.

## 14.2 Organization of this Chapter

The intent of the main body of this chapter is to provide an overview of:

- Goals and objectives of water quality treatment,
- Project elements that trigger the requirement for treatment of highway runoff,
- Pollutants of concern from stormwater runoff (oil and grease, sediment, nitrogen, phosphorus, metals, and debris),
- Recommended water quality treatment approaches,
- Minimum requirements for design of water quality facilities, and
- Generalized procedures for treatment facility design.

The intent of the chapter appendices is to provide:

- Detailed design criteria for each permanent water quality treatment approach,
- Step-by-step procedures to aid the designer in understanding the analysis process, and
- Detailed design criteria for facility components such as pretreatment, inlet structures, piping conveyance, outlet control structures, and outfalls.

## 14.3 How to use this Chapter

Completing a water quality design requires the use of many other manual chapters.

The design of a stormwater treatment facility is generally a process that evolves as a project develops. The components of this process are listed below in a general order with references to their respective chapters.

- Collect data– **Chapter 6**
- Receive approval for new, innovative BMPs or substantial modifications to approved BMPs through the Hydraulic Design Deviation process –**Chapter 3**

- Prepare Preliminary Stormwater Recommendation – **Chapter 4**
- Evaluate hydrology – **Chapter 7**
- Analyze storm drainage – **Chapter 13**
- Design treatment facility – Chapter 14, Section 12
- Design and analyze stormwater treatment features:
  - Selection and Siting – Chapter 14, Section 9
  - Requirements and Design Elements – Chapter 14, Section 10
  - Treatment Facility Design – Chapter 14, Appendix A through D
    - **Appendix A** - Low Impact Development Approach
    - **Appendix B** - Biofiltration Facilities
    - **Appendix C** – Media Filtration Facilities
    - **Appendix D** – Density Separation Facilities
  - Facility Components Design – Chapter 14, **Appendix E**
    - Inlet Structures (Flow Splitter, weir type or orifice type)
    - Pretreatment Facilities or Structures
    - Flow Spreading Options
    - Outlet Control Structures
    - Auxiliary Outlets
    - Conveyance Piping (also see **Chapter 13**)
    - Outfalls (also see Chapters **8** and **11**)
  - Oil Control Structures - **Appendix F**
- Prepare Stormwater Design Report – **Chapter 4**
- Prepare Operation and Maintenance Manual – **Chapter 4**
- Prepare contract plans – ODOT Contract Plans Development Guide
- Prepare specifications and/or special provisions – ODOT Specifications Manual
- Stormwater Control Facilities Special Provisions – **ODOT Specification Website**

## 14.4 Definitions

Definitions and terms important to understanding water quality features and their design are listed in this section. Additional definitions are located in the manual glossary.

1. Basic Treatment – Intended to achieve a goal of 80% removal of total suspended solids as well as targeting petroleum hydrocarbon concentrations.

2. Best Management Practice (BMP) – a physical, structural, and/or managerial practice that when used individually or in combination prevents or reduces pollution of water.

3. Biochemical Oxygen Demand (BOD) - a measure of the amount of oxygen needed by aquatic organisms to break down solids and other readily degradable organic matter present in water.

4. Chemical Oxygen Demand (COD) - amount of oxygen required for the chemical oxidation or decomposition of compounds in water.

5. Complexing - bonding between a dissolved metal species and another chemical (ligand).

6. Contributing Impervious Area (CIA) – consists of all impervious surfaces within the strict project limits plus impervious surfaces outside the project limits which drain to the project via direct flow or discrete conveyance.

7. Dissolved Metals – the fraction of metals in stormwater that pass through a 0.45 micron filter. A portion of dissolved metals will be in ionic form, but most will be complexed or otherwise attached to very fine organic or inorganic particles.

8. Dissolved Oxygen - the amount of oxygen dissolved in water. This term also refers to a measure of the amount of oxygen available for biochemical activity in a waterbody, an indicator of the quality of that water.

9. Drainage Facility Identification (DFI) - a unique identifier assigned to each stormwater treatment and storage facility. It is used to associate or link the stormwater facility to an Operation and Maintenance Manual. The number is assigned by contacting the Geo-Environmental Section's Senior Hydraulics Engineer to obtain a unique "DFI". The Geo-Environmental Section will maintain a database of assigned Drainage Facility IDs.

10. Enhanced Treatment – Intended to achieve a higher level of treatment than basic treatment. Enhanced treatment is targeted at the removal of dissolved metals.

11. Hydraulic Conductivity – a measure of the rate of flow through soils. Also known as the coefficient of permeability. It depends on the inherent permeability of the material and on the degree of saturation.

12. Infiltration – the flow of a fluid into a substance through pores or small openings or the process by which water on the ground surface flows into the soil.

13. Infiltration Rate – the rate at which water enters the soil under a given condition. The rate is usually expressed in inches per hour

14. Low Impact Development (LID) - a stormwater management approach intended to mimic natural hydrology using vegetation and soil conditions that reduce the rate and quantity of runoff, filter out pollutants, and facilitate infiltration, detention, and evapotranspiration of stormwater.

15. Media Filtration – the flow of stormwater through compost, amended soil or other porous material to remove pollutants. Media filtration is differentiated from infiltration because the stormwater is discharged to surface water instead of into groundwater.

16. Natural Resources Conservation Service (NRCS) Hydrologic Soil Groups:

    - Type A – soils having high infiltration rates, even when thoroughly wetted. These soils consist of deep, well-drained to excessively-drained sands or gravels and have a high rate of transmission (rate at which water moves within the soil).

    - Type B – soils having moderate infiltration rates when thoroughly wetted. These soils consist of moderately-fine to moderately-coarse textures and have a moderate rate of water transmission.

    - Type C – soils having slow infiltration rates when thoroughly wetted. These soils consist of a layer that impedes downward movement of water or soils with moderately fine to fine textures. These soils have a slow rate of water transmission.

    - Type D – soils having very slow infiltration rates when thoroughly wetted. These soils consist of clay soils with a high swelling potential, soils with a permanent high water table, soils with a hardpan or clay layer at or near the surface, and shallow soils over bedrock or other nearly impervious material and have a very slow rate of water transmission.

17. Operation and Maintenance (O&M) Manual - provides information about a facility's maintenance and operation. It is prepared by the project hydraulics designer to assist personnel who maintain the facility. All engineered stormwater treatment and storage facilities are required to have an Operation and Maintenance Manual.

18. Phosphorous Treatment – Intended to achieve a goal of 50% total phosphorous removal from the influent.

19. Plug Flow - a flow regime where water flows as if in a full pipe, the unit that enters first, exits first and there is no mixing between different units of water, designing for this type of flow prevents "short circuiting".

20. Pretreatment – the initial component of a stormwater treatment facility component that is designed to extend the life and efficiency of the primary stormwater treatment facility by removing coarse sediment and debris. Effective pretreatment allows the primary facility to function as designed between inspection/maintenance cycles and minimizes damages associated with sediment loading, and clogging of orifices.

21. Short Circuiting - the passage of runoff through a BMP in less than the design treatment time.

22. Stormwater Control Facilities (Treatment and/or Storage) - A treatment or storage facility that requires engineering analysis to determine the hydrology, hydraulics, and design of the structure. Treatment facilities include dry and wet detention basins, swales, filter strips, treatment wetlands, bioslopes and proprietary systems. Storage facilities used for stormwater management include dry detention basins, vaults, and tanks. Design guidance for storage facilities is discussed in Chapter 12.

23. Stormwater Facility Field Marker - used to identify and locate ODOT stormwater facilities or alert maintenance crews of stormwater facility maintenance areas. Three types of markers are recommended for identifying, locating, or alerting.

    - Field Marker Type S1 – Marker for identifying the beginning and end of a stormwater control facility maintenance area using a non-reflective flexible plastic post.

    - Field Marker Type S2 – Marker for displaying the DFI for a stormwater control facility using a white background non-reflective aluminum paddle mounted onto one (1) steel post.

    - Field Marker Type S3 - Marker created by stamping the drainage facility ID onto the top of manhole, vault, and/or tank access covers.

24. Stormwater Facility Maintenance Area - the stormwater facility footprint which includes the area needed to accommodate the pretreatment structure, storm drain piping, treatment zone (basin or pond, swale, filter strip, etc.), primary and auxiliary outlets, outfalls, maintenance access road, and maintenance operational area. These maintenance areas are applicable to all above ground stormwater treatment and storage facilities. Markers are placed at the start and end of each facility's maintenance area to alert maintenance crews of special maintenance requirements as outlined in the applicable Operation and Maintenance Manual.

25. Total Dissolved Solids - the dissolved matter found in water, comprised of mineral salts and small amounts of other inorganic and organic substances.

26. Total Maximum Daily Load (TMDL) - the maximum amount of a pollutant that can be discharged into a water body from all sources (point and non-point) and still maintain water quality standards. Under Clean Water Act section 303(d), TMDLs must be developed for all water bodies that do not meet water quality standards.

27. Total Nitrogen - a measure of all forms of nitrogen (e.g., nitrate, nitrite, ammonia-N, and organic forms) that are found in a water sample.

28. Total Kjeldahl Nitrogen - the sum of organic nitrogen and ammonia in a water body, measured in milligrams per liter (mg/L). High measurements typically result from sewage and manure discharges to water bodies.

29. Total Phosphorous - the total concentration of phosphorus found in the water. Phosphorus is a nutrient and acts as a fertilizer, increasing the growth of plant life such as algae.

30. Total Suspended Solids (TSS) - solids suspended in water including a wide variety of material such as silt, decaying plant matter, industrial wastes and sewage.

31. Treatment Train or System - the combination of several treatment facilities with unique unit processes applied in a linear progression (also called "in series").

32. Underground Injection Control (UIC) - any system, structure, or activity created to place fluid below the ground or sub-surface. Common stormwater UICs or activities include, but are not limited to sumps, infiltration galleries, drywells, trench drains, and drill holes. Infiltration basins are generally not considered UICs.

33. Unit Process - the specific mechanism of pollutant removal (i.e., sedimentation or vegetative uptake).

34. Unit Operations - the treatment facilities in which the unit process occurs (i.e., wet pond or swale).

35. Vegetative (or Biological) Uptake - absorption or utilization of nutrients and other dissolved chemicals by plants or algae.

## 14.5 Stormwater Quality Design Objectives

The selection and design of stormwater treatment facilities can be a complex, time consuming, and costly process. It is important that overall management objectives be clearly defined early in the design process. These objectives must ensure compliance with water quality goals and support permitting of ODOT projects by regulatory agencies.

### 14.5.1 Stormwater Quality Treatment Goals and Triggers

ODOT's water quality goal is to design and implement highway projects in a manner that manages project runoff to protect the beneficial uses of the receiving surface and ground waters, and to manage project runoff quantities and flows to protect the receiving water's stream form, function, and stability**.** This chapter is concerned with water quality. Flow control is covered in **Chapter 12**.

Managing highway runoff for water quality is defined as, "Directing highway runoff through natural and artificial features that remove pollutants and reduce the volume and flow rate of the stormwater." The requirement to treat stormwater therefore does not automatically mean that consolidated, engineered facilities must be designed and built.

The water quality goal is considered to be met if the following design criterion is met:

- Treatment is provided for all of the runoff generated by the Water Quality Design Storm from the Contributing Impervious Area using Best Management Practices that utilize infiltration, media filtration or vegetative filtration.

It is not always possible to meet this criterion. The goal can still be met by taking alternative actions, including off-site mitigation, that provide a similar water quality benefit.
Projects that include the following "trigger" elements are required to provide treatment of highway runoff:

- Producing new impervious surface area. Does not include:
  - Minor actions such as constructing sign or signal post pads, etc., or
  - Non-pollutant generating areas such as detached bike paths and detached sidewalks
- Changing the total Contributing Impervious Area. In situations where the CIA is decreased, there may be a possibility for "banking" of treatment provided.
- Re-constructing a roadway from the subgrade. Does not include pavement overlays or inlays, nor spot reconstruction.
- Changing the type, location, direction, length or endpoint of the pre-project stormwater conveyance system.
- Replacing or widen a stream crossing structure including adding new bridge decks or retrofitting bridge deck drainage.
- Requiring a Clean Water Act Section 404 permit and actively involving modification of impervious surfaces.

**Figure 14-2 Combined Biofiltration Swale/Detention Pond**

### 14.5.2 Facility Maintenance Objectives

Maintenance is necessary to ensure:

- Long term capacity and treatment capability are not diminished,
- Removal of pollutants as designed,
- Prevention of damage to the adjacent highway, and
- Protection of adjacent property from damage.

Therefore, all designs must include recommendations for appropriate preventive maintenance that would ensure that stormwater treatment facilities operate properly. **Water quality facilities that require extensive or specialized maintenance activities or equipment are discouraged.**

Project designs should include features that will minimize maintenance tasks. It is possible to minimize maintenance requirements while still achieving removal goals. A few ways to minimize and make maintenance easier or efficient include:

- Facility Selection
  - Use above ground facilities whenever possible and appropriate. Above ground facilities are easier to access and maintain than underground facilities.
- Site Selection
  - Site access must be adequate to allow for necessary maintenance vehicles and equipment to get to the facility. Avoid restrictive access conditions.
  - Do not place underground facilities in a location that would require closing a traffic lane to access.
- Sediment control
  - Provide appropriate pretreatment as discussed in Section 14.10.3.
  - Provide adequate dead or sediment storage to allow for sediment accumulation between maintenance cycles as discussed in Appendix E.
- Inlet and Outlet structures
  - Screens or debris risers should be applied to orifices smaller than 6 inches because small diameter orifices can be susceptible to clogging from debris.
  - Inlet and outlet structure configuration and size should be selected to provide adequate access for maintenance or inspection.
  - Minimize depth of inlet and outlet structures to prevent the need for specialized equipment for maintenance or inspection.

### 14.5.3 Cost Effectiveness

The construction and maintenance costs for different stormwater treatment options vary substantially. The goal is to select a treatment facility that minimizes lifecycle costs while achieving treatment goals, overcoming site constraints, and ensuring public safety. In most cases there will be more than one BMP to choose from and this may make the selection process more difficult. When more than one type of facility can meet treatment goals, select the one with the lowest overall costs, including maintenance, construction costs, and right-of-way costs for the life of the facility.

## 14.6 Pollutants

The purpose of water quality treatment facilities is to remove pollutant loads contained in stormwater runoff. Pollutants of concern could include:

- Petroleum hydrocarbons, including oil and grease, and combustion by-products
- Sediment, including sand, silt and suspended solids
- Nutrients such as nitrogen and phosphorus,
- Oxygen demanding substances (Chemical oxygen demand and Biological oxygen demand)
- Metals, both total and dissolved, such as copper, zinc and lead
- Certain bacteria,

- Herbicides and pesticides, and
- Debris, litter, and other pollutants

The source of these pollutants may be highway related, or may be external to the highway and deposited by air fall or run on. Highway related sources include:

- Vehicle tires
- Brake linings
- Fluid leaks from engines or other vehicle parts
- Fuel combustion
- Wear of the pavement
- Litter
- Organic debris and other materials

External sources include:

- Industrial emissions
- Drift from agricultural or landscape spraying
- Run on from adjacent land uses, such as agriculture, landscaped areas and industrial land

Identifying these constituents, aids in determining which water quality treatment facilities can be effective in meeting receiving water quality goals. The following subsections provide a brief review of the pollutants that may be present in highway runoff.

### 14.6.1 Petroleum Hydrocarbons

Petroleum hydrocarbons include a wide range of organic compounds such as oil and grease and combustion byproducts such as polycyclic aromatic hydrocarbons (PAHs). Common sources of hydrocarbons in highway runoff are gas tank leaks, oil leaks, car wash-off during rain events, and heavy equipment activity near and on the roadway. PAHs result from the burning of gasoline and diesel fuels. Petroleum hydrocarbons are generally toxic. Hydrocarbons are not highly soluble in water, although some slight dissolution does occur. Oil and grease are lighter than water, and runoff transporting these compounds may produce sheen on the surface of receiving waters. Hydrocarbons have a strong tendency to attach to particulate matter; thus, oil and grease, as well as PAHs may attach to particles that ultimately settle and accumulate in bottom sediments of an aquatic environment, creating the potential for adverse impacts to aquatic life.

### 14.6.2 Sediment

Sediment is typically one of the principle pollutants in highway runoff. Sediment may include natural material such as sand, soil particles, and vegetative matter, as well as synthetic material such as bits of plastic and metal. Typically, suspended sediment concentration is measured as total suspended solids (TSS). This test measures the weight of dry solids retained after filtering a known volume of water containing the suspended matter. TSS measurements are not generally

used to determine the level of gross solids such as litter and gravel. Since TSS measurement detects both inorganic and organic suspended matter, TSS values can include particulate matter that originates from many possible sources, including soils, vegetation, vehicles, and atmospheric deposition. TSS measurements are given as concentrations, generally grams of TSS per liter of tested water.

Often sediments carry other pollutants with them, due to chemical bonding between the pollutant and the sediment particle. Effectively removing sediment from runoff is an excellent means of improving water quality.

Turbidity is another measure of suspended matter in water. Unlike the TSS test, however, it is not a weight measurement. Rather, turbidity is a measure of the light-scattering tendency of particulates dispersed in water and is a useful measure of the light penetrating properties of the dispersion. Unfortunately there is no simple correlation between TSS and turbidity. Turbidity measurements are given in Nephelometric Turbidity Units or NTU.

Suspended material can cause a variety of adverse impacts to aquatic habitat. Particulates can clog fish gills, interfere with filters of invertebrates, smother bottom-dwelling organisms, and impact spawning areas. Increased turbidity can interfere with light penetration, reducing prey capture for sight feeding predators, or conversely by increasing prey capture by allowing predator to sneak up on prey.

### 14.6.3 Nitrogen and Phosphorus

Nitrogen and phosphorus play a role in the aquatic habitat as valuable nutrients for plant and algal growth. The principle undesirable environmental effect associated with these nutrients is the stimulation of excess algae and plant growth, which subsequently increases oxygen demand, particularly in lakes and other water impoundments. Both nitrogen and phosphorus may be present as dissolved species or in particulate form. The distinction is important because treatment methods that reduce particulate forms differ from those that reduce dissolved forms.
Dissolved forms of nitrogen are typically inorganic and include ammonia, nitrite, and nitrate. Organic nitrogen (e.g., originating from vegetation) is frequently particulate. Often, nitrate and nitrite measurements are reported as the sum of both forms, since nitrite converts very rapidly to nitrate in the environment. Total nitrogen is the sum of all dissolved and particulate forms, inorganic and organic. The amount of organic nitrogen and ammonia nitrogen combined is given as Total Kjehldahl Nitrogen or TKN as a concentration, generally milligrams per liter.

Total phosphorus is the sum of both particulate and dissolved phosphorus. Phosphorus can exist in several chemical forms in the aquatic environment and is generally reported as a concentration in milligrams per liter.

### 14.6.4 Oxygen-Demanding Pollutants

Some organic compounds in highway runoff can be decomposed by aerobic microbes present in the aquatic environment. These aerobic microbes use oxygen during the decomposition process, and when this occurs dissolved oxygen in the water may be reduced. The Biochemical Oxygen Demand (BOD) test provides a rough measure of the concentration of organic material that can be readily oxidized. The Chemical Oxygen Demand (COD) test provides a means of measuring the organic "strength" of a waste stream or water body. The result is the total quantity of oxygen required to oxidize all the waste's carbon into carbon dioxide and water. These tests are generally used for wastewater treatment where the concentration of organics present is much higher than in typical stormwater. Some sources of organics in stormwater include grass clippings, fallen leaves, hydrocarbons, and human and animal waste from septic systems, yard runoff, or feedlots. BOD and COD are both given as concentrations, generally milligrams per liter of runoff.

Increases in water temperature and salinity both reduce the amount of oxygen that can be dissolved in water. Healthy streams can contain up to 7 milligrams per liter at 95 degrees Fahrenheit and nearly 15 milligrams per liter at 32 degrees Fahrenheit assuming standard atmospheric conditions (1 atmosphere and dry air containing 20.9 percent oxygen). The amount of oxygen that can be dissolved into a water body is dependent on the partial pressures and a relation called Henry's Law.

It is possible to deplete dissolved oxygen levels to nearly 0 milligrams per liter; this is often a result of high algal or plant growth and very warm, standing water. To help raise dissolved oxygen levels, adding shading, controlling plant and algae growth, and adding riffles or highly turbulent areas in the stream or facility (when appropriate) can increase entrained dissolved oxygen concentrations.

### 14.6.5 Metals

Metals found in highway runoff include copper, lead, zinc, iron, cadmium, chromium, nickel, and manganese. They may occur as particulate metals or as dissolved metals. Particulate metals behave like sediment, as a physical contaminant. Dissolved metals (often metal salts) can be much more difficult to remove. Biological uptake and complexing (chemical bonding) are the two major removal mechanisms for dissolved metals. Potential toxicity to aquatic life is the principle concern with the discharge of both particulate and dissolved metals. However, potential contamination of drinking water supplies can also be a concern.

### 14.6.6 Bacteria

Bacteria levels in highway runoff are typically measured using procedures that test for a specific group of bacteria — the coliform group. These bacteria are used as indicators of potential public health hazards. Measurements may be reported as either total coliform values or as fecal coliform values, depending on the specific procedure used and the intent of the measurement.

The intent of the fecal coliform test is to determine whether stormwater runoff has been exposed to fecal contamination and, therefore, has the potential to spread disease.

The major sources of bacterial contamination are failing septic systems and animal feedlots, and therefore is generally not a priority for treatment of highway runoff. If bacterial contamination is found to be an issue in highway runoff, then preventative measures are generally the most beneficial way to prevent bacterial from entering the system. These measures can include dog ordinances requiring owners to collect waste, buffers between livestock and septic system facilities, and other site-specific measures.

### 14.6.7 Herbicides and Pesticides

The most common sources of herbicides and pesticides in highway runoff are roadside herbicide spraying to kill encroaching vegetation and domestic pesticide use on private property discharging into public stormwater collection systems. Herbicides and pesticides (or "biocides") may be inorganic or organic and, if present, are generally found in very low concentrations. However, some biocides can accumulate to high levels in certain aquatic species, causing chronic effects. If present in high concentrations, (e.g., from a spill) acute effects to aquatic life could occur.

### 14.6.8 Debris, Litter, and Other Pollutants

Debris, litter, and a variety of chemicals such as household wastes, plasticizers, and wood preservatives may occasionally be present in highway runoff as a result of spills, illegal disposal, or other activities. The occurrence of these pollutants is generally sporadic and detected concentrations are typically very low. These pollutants can be a primary contributor to the clogging of drainage features and may interfere with the effectiveness of treatment when present in highway runoff.

## 14.7 Treatment Mechanisms

There are six primary treatment mechanisms considered most appropriate for stormwater. The following subsections provide a brief review of these six treatment mechanisms.

### 14.7.1 Hydrologic Attenuation

Hydrologic attenuation achieves pollutant reduction through runoff volume reduction. Infiltration is the primary means of hydrologic attenuation in the types of BMPs used in stormwater management. Evapotranspiration is also used by some BMPs. Attenuation reduces the pollutant load discharged to surface waters, but does not necessarily reduce pollutant concentrations. Infiltration allows for several different treatment mechanisms. Processes such as sorption, filtration, and microbial degradation occur as runoff infiltrates through the soil matrix.

### 14.7.2 Sedimentation/Density Separation

Density separation refers to the unit processes of sedimentation and flotation that are dependent on the density differences between the pollutant and the water to effect removal. Sedimentation is the gravitational settling of particles having a density greater than water. Flotation is used for pollutants that are lighter than water, and therefore float. Typically, floatable materials such as trash, debris, and hydrocarbons are removed through treatment processes that collect or skim these pollutants from the water surface.

### 14.7.3 Sorption

Sorption refers to the individual unit processes of both absorption and adsorption. Absorption is a physical process whereby a substance of one state is incorporated into another substance of a different state (e.g., liquids being absorbed by a solid or gases being absorbed by water). Adsorption is the physiochemical adherence or bonding of ions and molecules (ion exchange) onto the surface of another molecule. In stormwater treatment applications, the primary pollutant types targeted with absorption unit processes are petroleum hydrocarbons, while adsorption processes typically target dissolved metals, nutrients, and organic toxicants such as pesticides and polycyclic aromatic hydrocarbons (PAHs). Different types of filter media, including natural or amended soils, may provide either or both of these unit processes.

### 14.7.4 Filtration

Filtration can encompass a wide range of physical and chemical mechanisms, depending on the filtering media. Typical media filter materials include sand, natural soil, grassy vegetation, or mixes of chemically active ingredients such as perlite, zeolite, and granular activated carbon. Filtration is an important component of infiltration, media filtration and vegetative filtration treatment facilities. Filtration removes particulate matter either on the surface of the filter or within the pore space of the filter by physically trapping or capturing the particles in miniature hydraulic dead zones. Filtration such as a sand filter can provide the added benefit of removing stormwater constituents that may be attached to solids such as metals and bacteria. Filtration can also provide opportunities for sorption processes to occur, reducing dissolved and fine suspended constituents. Filtration can often be an effective preliminary treatment for stormwater, by increasing the longevity of downstream BMPs and reducing maintenance frequency.

### 14.7.5 Uptake/Storage

Uptake and storage refer to the removal of organic and inorganic constituents by plants and microbes through nutrient uptake and bioaccumulation. Nutrient uptake converts required micro- and macro-nutrients into living tissue. In addition to nutrients, various algae and wetland and terrestrial plants accumulate organic and inorganic constituents in excess of their immediate

needs (bioaccumulation). The ability of plants to accumulate and store metals varies greatly. Significant metals uptake by plants will not occur unless the appropriate species are selected.

### 14.7.6 Microbially Mediated Transformation

Microbial activity promotes or catalyzes redox reactions and transformations including degradation of organic and inorganic pollutants and immobilization of metals. Bacteria, algae, and fungi present in the soil or water column are primarily responsible for the transformations. Stormwater treatment that incorporates vegetation or permanent water pools usually has a diverse microbial population. These transformations can remove dissolved nitrogen species, metals, and simple and complex organic compounds. Soils may be inoculated with desirable microbes to promote specific reactions.

### 14.7.7 Effectiveness of Treatment Mechanisms

The treatment effectiveness of a BMP is essentially related to which processes are actually utilized by the BMP and the ability of the BMP to maximize the process(es). Table 14-1 summarizes the stormwater related pollutants of concern considered to be effectively removed by each treatment mechanism.

**Table 14-1 Treatment Mechanism – Target Pollutant**

| Treatment Mechanism - Target Pollutant Matrix | | Mechanism | | | | | |
|---|---|---|---|---|---|---|---|
| | | Hydrologic Attenuation (1) | Density Separation | Sorption (chemical activity) | Filtration | Uptake/Storage(2) | Microbial Transformation(3) |
| Target Pollutant | Sediment/Particulate (suspended solids) | ■ | ■ | | ■ | | |
| | Nutrients(4) | ■ | | ■ | | ■ | ■ |
| | Oil and Grease | ■ | ■ | ■ | ■ | | ■ |
| | Polycyclic Aromatic Hydrocarbons (PAH) | ■ | □ | ■ | □ | ■ | ■ |
| | Metals (particulate) | ■ | ■ | | ■ | | |
| | Metals (dissolved) | ■ | | ■ | □ | □ | □ |

■ *= Treatment mechanism effective for target pollutant removal*

□ *= Depending on chemical activity of filter media*

(1) *Refers to infiltration which is credited for overall pollutant mass load reduction of all target pollutants primarily through volume reduction; pollutant removal is also achived through filtering, sorption, and microbial transformation in the soil column.*

(2) *Dependent on plant species*

(3) *Dependent on types of microbes present (in soil or water column)*

(4) *May not be considered a highway target pollutant, but included for completeness*

## 14.8 Treatment Facilities

The following sub-sections provide an overview of the acceptable runoff treatment facilities used on ODOT projects. Specific design criteria for each BMP are provided in the chapter appendices.

### 14.8.1 Low Impact Development (LID) Techniques

Low Impact Development (LID) is an innovative stormwater approach that mimicks natural hydrology and utilizes vegetation and infiltration to reduce the rate and quantity of runoff, remove pollutants, and facilitate detention and evapotranspiration of stormwater. LID was pioneered in Prince Georges County, Maryland and has been applied successfully across the country. When effectively and broadly implemented, LID helps improve the quality of receiving waters and stabilize the hydrology of nearby streams.

LIDs are intended to:

- Be effective at treating stormwater runoff,
- Facilitate detention and infiltration opportunities,
- Promote groundwater recharge,
- Preserve existing flow paths,
- Minimize disturbance,
- Be less expensive to construct and maintain than traditional facilities,
- Eliminate or reduce the need for storm drain systems such as pipes, inlets, curbs, and large consolidated facilities such as ponds and swales, and
- Be applied to roadway re-construction, widening, and new alignment projects.

LID is not a significant departure from the current rural road design practices in which curb and gutter systems are not typically used. The major difference is that LID techniques are specifically designed not to concentrate flows while still providing effective means of water quality treatment close to the source of pollution. This reduces the need for long or complex pipe routing configurations. The LID approach utilizes the hydrologic characteristics of the existing roadside area, modifications of the roadside area, and distributed, small, hydraulically engineered BMPs that incorporate vegetation, infiltration and media filtration. For the latter techniques the difference between LID and non-LID is more a matter of scale and location than any qualitative distinction.

**The use of LIDs should be evaluated for *feasibility* on all transportation projects.** The feasibility depends on the physical characteristics of the site (including slope, soil infiltration, existing drainages, etc.), the adjacent development, the availability and cost of additional right-of-way if necessary, safety, maintenance, highway structural integrity, and local groundwater contamination. Note that LID methods will not always be feasible.
The recommendations and requirements for planning and designing LIDs are outlined in **Appendix A**.

### 14.8.2 Biofiltration Facilities

#### 14.8.2.1 Biofiltration Swales

Biofiltration swales are above ground open channels engineered to treat stormwater runoff by sedimentation and filtration as runoff is conveyed through the vegetated surface (usually grass)

and upper soil layer. Grassed swales are popular because of their low construction and maintenance costs and minimal design limitations. Biofiltration swales incorporate amended soils or compost to enhance infiltration and maximize the contact of stormwater with pollutant removing media.

Good locations for grassed swales include median strips, roadways shoulders, and parking lots.

Additional information regarding biofiltration swales can be found in **Appendix B**.

**Pollutant Removal Mechanisms**

- Filtration
- Sorption
- Density Separation
- Uptake/Storage
- Microbial Transformation
- Hydrologic attenuation

**Primary Targeted Pollutants**

- Sediments
- Metals (particulate)
- Metals (dissolved) (using amended soil)

**Secondary Targeted Pollutants**

- Nutrients
- Oil and Grease
- Polycyclic Aromatic Hydrocarbons (PAH)

**Facility Treatment Types**

The two application techniques for swales are:

- Basic biofiltration swale – designed to provide water quality treatment only.
- Combination biofiltration swale – designed to provide water quality treatment and convey high flows.

### 14.8.2.2 Filter Strips

Filter strips consist of the right-of-way parallel to the road, with a relatively flat cross slope to maintain sheet flow of stormwater runoff over the entire width of the strip and are designed to treat sheet flow from an adjacent impervious area. Treatment occurs as the stormwater runoff flows through the grass and soil surface. Filter strips are good for highway application because of their low maintenance requirements.

Additional information regarding filter strips can be found in **Appendix B**.

**Pollutant Removal Mechanisms**

- Filtration
- Sorption
- Hydrologic attenuation
- Density Separation
- Uptake/Storage
- Microbial Transformation

**Primary Targeted Pollutants**

- Sediment/Particulates
- Metals (particulate)
- Metals (dissolved) - may require using amended soil depending on the native soil type

**Secondary Targeted Pollutants**

- Nutrients
- Oil and Grease
- Polycyclic Aromatic Hydrocarbons (PAH)

### 14.8.3 Media Filtration Facilities

#### 14.8.3.1 Bioretention Facilities

A bioretention facility is an above ground basin or cell that is designed to capture stormwater runoff and infiltrate it through a water quality mix to remove pollutants through a variety of physical, biological and chemical treatment processes. They are good for highway applications because of their moderate construction and maintenance cost.

Opportunities for siting bioretention facilities include medians, interchanges, adjacent to ramps, parking lot islands, and along right-of-way adjacent to roads.

Additional information regarding bioretention facilities can be found in **Appendix C**.

**Pollutant Removal Mechanisms**

- Hydrologic Attenuation
- Sorption
- Filtration
- Density Separation
- Uptake/Storage
- Microbial Transformation

**Primary Targeted Pollutants**

- Sediment/Particulate
- Nutrients
- Polycyclic Aromatic Hydrocarbons (PAH)

- Metals (particulate and dissolved)

**Secondary Targeted Pollutants**

- Oil and Grease

#### 14.8.3.2 Bioslopes

Bioslopes are flow-through BMPs incorporated into roadside embankments and placed between pavement and a downstream conveyance system that use a variety of physical, biological and chemical treatment processes to provide stormwater treatment. Bioslopes are good for highway applications because of their minimal right-of-way requirements and maintenance schedule.

Additional information regarding bioslopes can be found in **Appendix C**.

**Pollutant Removal Mechanisms**

- Hydrologic Attenuation
- Sorption
- Filtration
- Density Separation
- Uptake/Storage
- Microbial Transformation

**Primary Targeted Pollutants**

- Sediment/Particulates
- Nutrients
- Polycyclic Aromatic Hydrocarbons (PAH)
- Metals (particulate and dissolved)

**Secondary Targeted Pollutant**

- Oil and Grease

### 14.8.4 Density Separation Facilities

#### 14.8.4.1 Extended Detention Dry Pond

An extended detention dry pond is an above ground basin that is designed to detain stormwater runoff for some minimum time to allow particles and attached pollutants to settle, thus providing water quality treatment. Extended dry ponds do not have a large permanent pool of water; instead, water that enters the pond is released over a period of time, longer than standard detention ponds. They typically have the following facility components:

- An extended detention outlet control structure, and
- An emergency spillway if the primary outlet control structure cannot safely pass the projected high flows.

Extended detention dry ponds are good for highway applications because of their moderate construction and maintenance cost.

Opportunities for siting extended detention dry ponds include medians, interchanges, adjacent to ramps and along right-of-way adjacent to roads.

Additional information regarding extended detention dry ponds can be found in **Appendix D**.

**Pollutant Removal Mechanisms**

- Density separation
- Sorption (secondary mechanism)
- Filtration (secondary mechanism)
- Uptake/Storage (secondary mechanism)
- Microbial transformation (secondary mechanism)

**Primary Targeted Pollutants**

- Sediments
- Particulate bound pollutants

**Secondary Targeted Pollutants**

- Nutrients
- Oil and Grease
- Polycyclic Aromatic Hydrocarbons (PAH)
- Particulate metals (not effective for dissolved metals)

### 14.8.5 Proprietary Structures

There is a wide range of proprietary structures that can be used for stormwater treatment but only a few have been approved for use on ODOT projects. (See the ODOT QPL) ODOT relies on the Washington Department of Ecology's Emerging Stormwater Treatment Technologies Evaluation Program Technology Assessment Protocol - Ecology (TAPE) to determine which products are added to the QPL. Structures obtaining General Use Level Designation (GULD) through the TAPE Program are placed on the QPL and are considered to be "highly" capable of removing the category or target pollutant.

Most proprietary products are more compact than other treatment systems and are installed underground. These features often make them a preferred method for treatment in space limited settings. Information should be obtained from the various manufacturers for more detailed discussion of the product types discussed below.

The general types of proprietary products include:

**Separation structures**

Separation structures utilize underground vaults and manholes to remove coarse sediment, pollutants absorbed to sediment, oil, and floatables from stormwater runoff. These units use separation inserts and either gravity, vortex, or centrifugal separation to provide treatment.

These units do not meet treatment standards as a stand-a-lone product and require additional facilities to meet current water quality standards.

**Pollutant Removal Mechanism**

- Density Separation

**Primary Targeted Pollutants**

- Sediment/Particulate
- Metals (particulate)
- Oil/Grease (separation)

**Filtration structures**

Filtration structures (or media filters) utilize special media mix in underground vaults, manholes, or catch basins to remove sediment, metals, and oil/grease from stormwater runoff. These structures will need to have GULD approval as providing "Enhanced Treatment" prior to be used as a stand-a-lone water quality facility.

**Pollutant Removal Mechanisms**

- Sorption
- Filtration

**Primary Targeted Pollutants**

- Sediment/Particulate
- Metals (particulate)

**Secondary Targeted Pollutants**

- Oil and Grease
- Metals (dissolved)

The following is a list of guidelines for the use of proprietary water quality structures.

- All water quality structures shall comply with applicable general requirements outlined in Section 14.10.
- There are several types of media used in filtration structures. Select a media that will remove the target pollutant.
- Select a structure that is on the Qualified Products List (QPL) in the "Stormwater Control Facilities" Category. There are five treatment categories established and structures can be placed under one or more treatment categories. The treatment categories include:
    - Sediment pretreatment,
    - Oil treatment,
    - Suspended solids treatment,

  - Dissolved metals treatment, and
  - Phosphorous treatment.

- Utilize Special Provision SP1010 for projects using proprietary structures. Include the following information in the project's special provision SP1010:
  - Requirement that the facility be selected from the QPL.
  - Drainage facility identification number
  - Hydraulic performance requirements such as the water quality design flow and peak conveyance flow rate
  - Specification of on-line or off-line application
  - Water quality structure definition – An underground self-activating structure with no moving mechanical parts or external power sources which removes pollutants from stormwater runoff and retains the pollutants in the structure. Pollutants to be removed and retained as specified include, but are not limited to, suspended solids, floatables, petroleum products, particulate and dissolved metals, and phosphorous.

### 14.8.6 Other Non-Proprietary BMPs

Other non-proprietary methods of treating stormwater runoff may be proposed but must be evaluated on a project-by-project basis to determine if the proposed treatment method is adequate. A hydraulic design deviation request must be submitted to the region hydraulics engineer for consideration. Concurrence from the region hydraulics engineer is required and is then submitted to the Geo-Environmental Senior Hydraulics Engineer for approval.

The hydraulic design deviation request form and additional discussion of the review process is discussed in Chapter 3. The design deviation request should include an explanation of the issues, a brief description of the project, a justification for the deviation, and adequate supporting documentation.

After a non-proprietary BMP is approved for use on a specific ODOT project it is placed on an evaluation list. This list is available at Geo-Environmental's website. The performance of the BMPs placed on the evaluation list will be monitored and approved stormwater management approaches will be added to future revisions to this chapter.

### 14.8.7 Emerging Proprietary Technology

An emerging proprietary technology must be approved prior to use on an ODOT project. The following steps outline the emerging proprietary technology evaluation process:

Step 1: Manufacturer submits required documentation to Washington State's Department of Ecology Emerging Stormwater Treatment Technologies Evaluation Program. Go to

step 2 if proprietary technology obtains General Use Level, Conditional Use Level, or Pilot Use Level designation through Department of Ecology's evaluation program.

Step 2: Manufacturer submits required documentation to ODOT's Structure Service Unit's Product Evaluation Coordinator. Contact information and submittal requirements can be obtained at the following website:

**http://www.oregon.gov/ODOT/HWY/CONSTRUCTION/QPL/QPIndex.shtml**

## 14.9 Selection and Siting

The selection of a BMP is based on many factors including treatment capability, site characteristics, maintenance requirements, and lifecycle costs. BMP site requirements, pollutant removal effectiveness, and basic lifecycle cost considerations are covered in this section.

The expectation is that a project will fully meet the stormwater treatment criteria (Section 14.5.1) unless site constraints make it impractical to do so. When site constraints prevent the criteria from being met, the project will provide as much on-site treatment as is practical, and will also arrange for off-project mitigation.

### 14.9.1 BMP Selection Prioritization

ODOT has established a hierarchy of stormwater treatment approaches and techniques that supports regulatory requirements and the agency's water quality goals and objectives. The preference for stormwater treatment approaches is to disperse treatment along the length of the project, using the characteristics of the right of way to provide treatment, using consolidated facilities only when that is not possible. The preferred treatment techniques are those that have high effectiveness at treating the broad range of highway pollutants, and use infiltration, media filtration, or vegetative filtration as a primary treatment mechanism (see Table 14-2).

A requirement to manage highway runoff for water quality does not necessarily mean that a treatment facility needs to be constructed. Often the properties of the right-of-way provide excellent stormwater treatment, or can do so with minimal enhancement.

The stormwater designer should evaluate treatment approaches and techniques in the following order:

1. Use of the adjacent unaltered right-of-way as a treatment filter strip
2. Modification of the right-of-way (slopes, soils and/or vegetation) to provide treatment
3. Use of small, distributed treatment facilities along the length of the project
4. Use of large, consolidated treatment facilities.

The first choice for a BMP should be one of those that are highly rated for multiple pollutants. If none of those are suitable, the next step is to try to assemble a treatment train that is, in aggregate, as effective overall as one of the preferred BMPs. Failing that, use a single BMP that

targets the pollutant of greatest concern in the receiving water, and anticipate compensatory actions. Off-site project mitigation may be necessary if treatment is not feasible at all.

More detail about the stormwater design process is found in Section 14.12.

### 14.9.2 Pollutant Removal Capabilities

The treatment capability of a BMP is usually quantified as either percentage removal of a pollutant, or by average effluent concentration. Both have limitations. Percentage removal is often affected by the influent concentration – the more polluted the stormwater is going in, the greater the percentage removal achieved. Effluent concentration varies greatly between storms, and is affected by design, local conditions, and maintenance, and can be hard to quantify. Instead of using either of these measures, this manual uses a qualitative rating based on unit processes and demonstrated ability to substantially reduce pollutant concentrations between inflow and outflow.

The mechanisms of pollutant removal are discussed in detail in Section 14.7. Detailed discussions of the different classes of BMPs and their unit processes are provided in Section 14.8.

Table 14-2 summarizes key pollutant removal capabilities of the water quality facilities described in this chapter (references 2, 3, 5, 6-10). Ranges shown are based on the following criteria:

-=Low capability to remove target pollutant
◦=Moderate capability to remove target pollutant
•=High capability to remove target pollutant

These ranges assume a properly designed, operated, and maintained treatment facility. Note also that in many cases removal is dependent on whether the pollutant is in the particulate or dissolved form.
Limited information exists on the capability of treatment facilities to remove pollutants other than those identified in Table 14-2. One source reports the following, based on limited information:

- Bacteria – can potentially be reduced by grassed swales, wet ponds, and treatment wetlands, but these treatment techniques may have no effect or could even increase bacterial concentration.
- Herbicides/Pesticides – can potentially be reduced by grassed swales, wet ponds, treatment wetlands, and media filters.

Additional guidance on pollutant removal capabilities for bacteria, herbicides/pesticides, etc. will be provided as it becomes available.

## Table 14-2 Key Pollutant Removal Capabilities for Water Quality Facilities

| Treatment Mechanism - BMP Matrix | | Best Management Practice | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Pre-treatment | | Infiltration | | | | Filtration | | | | Pool-Ponds | | | Space-constrained or Urban Application | | | |
| | | Oil Control Stuctures * or *** | Sediment Control * or *** | Infiltration facility ** | Bioretention facility * | Bioslope * | Porous Pavement (not stand-alone) ** | Biofiltration Swale (soil amended) * | Filter Strip (soil amended) * | Biofiltration Swale (no soil amendment) * | Filter Strip (no soil amendment) * | Constructed Wetlands ** | Extended Detention Dry Pond * | Wet Ponds ** | Wet Vaults ** | Media Filters (non-proprietary) ** | Proprietary Separation *** | Proprietary Filtration *** |
| Treatment Mechanism | Hydrologic Attenuation | | | ■ | ■ | ■ | ■ | □ | □ | □ | □ | □ | ■ | □ | | | | |
| | Density Separation (Sedimentation or Flotation) | ■ | ■ | □ | □ | □ | | □ | □ | □ | □ | ■ | ■ | ■ | ■ | | ■ | |
| | Sorption | □ | | □ | ■ | ■ | | ■ | ■ | □ | □ | ■ | □ | □ | | □ | □ | ■ |
| | Filtration | □ | □ | ■ | ■ | ■ | | ■ | ■ | ■ | ■ | □ | □ | □ | | ■ | □ | ■ |
| | Uptake/Storage(1) | | | □ | ■ | □ | | □ | □ | □ | □ | ■ | □ | □ | | | | |
| | Microbial Transformation(1) | | | □ | ■ | □ | | □ | □ | □ | □ | ■ | □ | □ | | □ | | □ |
| Target Pollutant | Sediment/Particulate (suspended solids) | ○ | ● | ● | ● | ● | ○ | ● | ● | ● | ● | ● | ● | ● | ○ | ● | ● | ● |
| | Nutrients | | | ● | ● | ● | ○ | ○ | ○ | ○ | ○ | ● | ○ | ○ | | - | | |
| | Oil and Grease | ● | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ○ | ● | ○ |
| | Polycyclic Aromatic Hydrocarbons (PAH) | | | ● | ● | ● | ○ | ○ | ○ | ○ | ○ | ● | ○ | ○ | | ○ | | |
| | Metals (particulate) | ○ | ○ | ● | ● | ● | ○ | ● | ● | ● | ● | ● | ● | ● | ○ | ● | ● | ● |
| | Metals (dissolved) | | | ● | ● | ● | - | ● | ● | ○ | ○ | ● | ○ | ○ | | ○ | | ● |

■ = *Primary treatment mechanism for BMP*

□ = *Secondary treatment mechanism for BMP; dependent on plant species/microbes present*

● = *High capability to remove target pollutant*

○ = *Moderate capability to remove target pollutant*

\- = *Low capability to remove target pollutant*

(1) *Dependent on types of plant species or microbes (in soil or water column) present*

* = *acceptable bmp with design guidance provided in the chapter appendices*

** = *acceptable bmp and design guidance under development or submit a Hydraulic Design Deviation with supporting documentation*

*** = *acceptable bmp is added to Qualified Product List (QPL). BMP added to conditional product list requires construction project manager approval. Target pollutant rating above applicable for proprietary structures added to the QPL list for "Stormwater Control Facilities". Assume "low" capability to remove target pollutant for proprietary structures added to the Conditional list for "Stormwater Control Facilities".*

### 14.9.3 Maintenance Requirements

**Maintenance requirements must be taken into account when selecting BMPs.** All other factors being equal, the BMP with the lowest maintenance costs should be selected. Aspects of maintenance that need to be considered include:

- Accessibility (see below)
- Frequency of maintenance
- Complexity of maintenance
  - Tasks required
  - Equipment required
- Materials required (such as proprietary filters, or replacement compost), with availability and cost
- District equipment resources

Every BMP installed must be accessible for the equipment required for maintenance. Both physical access and safety of the Maintenance crews must be addressed in the BMP design. Access roads must be wide enough for equipment to get in, maneuver and get out. No BMP should be installed in a location where it would be necessary to close a lane of traffic to perform routine maintenance.

**Always coordinate with Maintenance before making a final selection of BMPs**.

### 14.9.4 Right of Way and the Facility Footprint

The facility footprint includes the area needed to accommodate any pretreatment structure, storm drain piping, treatment zone (basin, swale, filter strip, etc.), primary and auxiliary outlets, outfalls, maintenance access road, and maintenance operational areas. Facility footprints will vary from spanning the entire length of the project to being localized to a small area to accommodate a facility.

The facility footprint should be evaluated for right-of-way requirements. If existing right-of-way is not adequate for the facility, additional property or easement will have to be acquired. The designer will need to coordinate with the Right-of-Way specialist, as well as the Project Leader and, at times, the Roadway Designer.

It is important to determine placement and sizing of BMPs early in project development, because right-of-way acquisition of real property and various types of easements is a lengthy process. The process begins around the design acceptance phase of project design and ends when all of the necessary right-of-way has been acquired and it is certified for the project bid letting. Note that it takes the same amount of time to process and negotiate easements as it does to acquire real property. **While minimization of right-of-way requirements is a factor in selecting a BMP, lack of time to acquire necessary right-of-way is not an acceptable reason for not choosing the most effective BMP.**

Note that design changes that require additional right-of-way after the right-of-way acquisition has started will likely delay a project. Design changes with minor right-of-way impacts could delay the acquisition process up to four months. Design changes with major right-of-way impacts could delay the acquisition process for seven months or more.

The ODOT Highway Design Manual provides additional discussion on time allowances for the acquisition process, and the different property rights such as fee title, permanent easements and temporary easements.

### 14.9.5 Site Requirements for BMPs

Each type of BMP has minimum requirements for size and slope, and the infiltration-based BMPs have soil characteristics criteria. Additional information regarding specific criteria is provided in the appendix for the individual BMPs.

**Table 14-3 Potential Restrictions for Applying Treatment Approaches**

| Treatment Facility | No Slope | Steep Site Slope | High Groundwater Table | Bedrock Close to Surface | Soils | Proximity to Structures/ Wells |
|---|---|---|---|---|---|---|
| **Extended Detention Dry Facility** | + | +++ | +++ | ++ | + | ++ |
| **Bioretention/ Infiltration Facility** | + | +++ | +++ | +++ | ++/ +++ | +++ |
| **Biofiltration Swale** | +++ | +++ | ++ | ++ | ++ | ++ |
| **Filter Strip** | ++ | ++ | ++ | ++ | ++ | ++ |
| **Bioslopes** | ++ | ++ | +++ | +++ | + | ++ |
| **Proprietary Structures** | **See mfg's literature** | **See mfg's literature** | **See mfg's literature** | **See mfg's literature** | **n/a** | **See mfg's literature** |

+++ Will or may preclude the application of this type of facility
++ Can generally be overcome with appropriate site design
+ Generally not a restriction

### 14.9.6 Site Characteristics

#### 14.9.6.1 Topography

Topography controls the size, shape, and type of BMP that can be placed in a given location. Some BMPs, such as detention and retention basins, require fairly level ground, while flow-through facilities need a minimum slope. Modification of the topography is sometimes feasible, but will likely increase construction costs. Topography can also affect Maintenance access, so the designer must look at the entire facility footprint (as described above) when selecting a BMP. Table 14-3 includes slope restrictions for the categories of BMPs.

#### 14.9.6.2 Soils

Site soils determine whether infiltration based BMPs are feasible or not. Hydrologic class A and B soils support infiltration BMPs, C soils will allow for minor infiltration as a component of a BMP, while D soils are not suitable for infiltration (see Definitions for descriptions of the soil classes).

The character of the soil also determines its ability to remove pollutants from stormwater. Organic material and fine particles support the removal of many dissolved pollutants. Hydrologic Class A soils, which are generally coarse grained, may have relatively low pollutant removal capability. Removal of the upper soil horizons can also reduce the soil's pollutant removal capability.

Keep in mind that the Hydrologic soil classes are based on the NRCS soil survey and may not accurately reflect the properties of the soils at the site. It is preferable to obtain site-specific soil information whenever possible.

Soil amendments (see **Appendix E**) can improve the pollutant removal characteristics of soils while maintaining acceptable permeability. They can also improve permeability in tight soils, but only in the layer with the amendment, so the amendment will support media filtration but not infiltration.

#### 14.9.6.3 Climate

Consider climate when selecting and designing water quality facilities. Arid conditions and temperature extremes can reduce the density and survivability of vegetation and lengthen the establishment period. Poor vegetation cover can reduce the effectiveness of pollutant removal. Biofiltration swales, wet ponds, and treatment wetlands are examples of facilities that need to be placed in the proper climatic environment to achieve projected performance expectations. In some cases, design modifications that account for climatic conditions can still allow for effective BMP performance.

### 14.9.6.4 Groundwater

A high groundwater table precludes the use of most BMPs that require deep excavation. The aquifer may be vulnerable to contamination if there is not a sufficient and good soil between the bed of a BMP and the groundwater. Table 14-3 includes a column showing the sensitivity of BMPs to high groundwater.

Groundwater contamination is a risk anywhere surface runoff from developed areas is infiltrated into the soil. BMP practices that allow for infiltration of polluted runoff can be of concern if sited incorrectly. BMP infiltration systems should not be implemented if there is a high potential for spills that may be conveyed directly to the facility. Best Management Practices designed to infiltrate stormwater are typically very effective at removing normal concentrations of pollutants and numerous studies have shown that they present only a minor risk of contaminating either groundwater or soil if sited in areas where there is a low potential for spills. These studies indicate that natural and amended surface soils are very effective at removing pollutants from urban runoff because concentrations are typically low and surface soils utilize a number of natural processes such as physical filtering, ion exchange, adsorption, biological processing, conversion, and uptake by plants.

Drywells, infiltration galleries and subsurface drainfields that release stormwater or other fluids directly below the land surface are considered Class V injection wells (often called underground injection control systems, or UICs) and are subject to regulation by DEQ and the U.S. EPA. A Class V injection well is defined as any bored, drilled, or driven shaft, or dug hole, that is deeper than its widest surface dimension. These types of facilities are considered stormwater disposal systems, not treatment systems, and have impacted groundwater quality in a number of communities across the nation. See Section 14.10.14 for detailed design criteria for these types of facilities.

### 14.9.6.5 Competing Resources

Water quality facilities should not be placed so they impinge on protected resources or other sensitive areas. Examples include wetlands, habitat for threatened and endangered species, riparian zones, archaeological sites, historic sites, and parks and public recreation areas. When there is a potential conflict between protected resources and the ability to implement water quality treatment, early coordination with Environmental staff is imperative.

In some circumstances it may be appropriate to impact a protected resource in order to provide stormwater treatment, but those circumstances will be rare and will require extensive coordination and mitigation for the impacted resource. Both the REC and the appropriate resource specialist must be included in the conversation.

If it is necessary to expand the right-of-way to install a BMP, the adjacent land uses will affect the cost of doing so, and may preclude the use of particular BMPs.

#### 14.9.6.6 Hazards

Hazards that may affect the selection and placement of BMPs include geotechnical constraints, floodplains, hazardous material contamination, safety and health issues. Coordination between the designer and geotechnical and hazmat staff is important. Following are some examples of potential hazards and BMPs:

- Infiltration BMPs are not acceptable on unstable slopes.
- Excavation for BMPs on unstable or steep slopes may trigger geotechnical instability.
- Hazardous material sites are unsuited for infiltration, since the stormwater could cause migration of the contamination.
- BMPs that include basins with steep side slopes are a potential drowing hazard in residential areas. If such BMPs are selected, fencing may be required.
- Vector control, specifically mosquito control, may be an issue if there will be standing water for four or more days. (Mosquitoes can go from egg to biting in four days.)

### 14.9.7 Cost

The cost of a BMP includes construction, right-of-way, and maintenance. Construction costs must cover any access roads, fencing, and signage, pretreatment structure or basin, flow splitter manhole, excavation and grading, water quality mix placement, porous pavers, underdrain system, outlet structure, energy dissipator, and storm drain piping when applicable. Table 14-4 is provided to assist with the screening and selecting of water quality treatment alternatives. It rates the right-of-way, capital, and maintenance costs of the treatment facilities discussed in this chapter.

The goal is to select a treatment facility that minimizes lifecycle costs while achieving treatment goals, overcoming site constraints, and ensuring public safety. In most cases there will be more than one BMP to choose from and this may make the selection process more difficult. In general, when more than one type of facility is appropriate, select the BMP with the lowest construction and maintenance costs.

**Table 14-4 Cost and Effective Life of BMP Options**

| Structural BMP | Right-of-Way Costs[1] | Capital Costs | O&M Costs | Effective Life[2] |
|---|---|---|---|---|
| Extended Detention Dry Facilities | Moderate to High | Moderate | Low to Moderate | 20 - 50 years |
| Bioretention Facilities | Moderate to High | Moderate | Low to Moderate | 5 – 20 years |
| Biofiltration Swales | Moderate | Low to Moderate | Low | 5 - 20 years |
| Filter Strips | Low to Moderate | Low | Low | 20 - 50 years |
| Bioslopes | Low to Moderate | Moderate | Low to Moderate | 5 – 20 years |
| Proprietary Structures | Low | Moderate to High | Moderate to High | See manufacturer's literature |

Adapted from Young et al. (1996); Claytor and Schueler (1996); USEPA (1993); and others
NA = Not Applicable or Not Available
[1] Rating could be reduced to "low" if adequate right-of-way is available with easement or acquisition.
[2] Assumes regular maintenance, occasional removal of accumulated materials, and removal of any clogged media.
Modified from Reference 10, Stormwater Best Management Practices in an Ultra-Urban Setting: Selection and Monitoring, FHWA

Cost is a valid factor in determining if treatment is feasible at any given location, but cannot be the sole determinant. There is no standard formula or percentage for apportioning costs between the project (construction and right-of-way) and maintenance and determining if treatment is too expensive. The decision about appropriate costs will be influenced by the sensitivity of the receiving water and the impact from the project; the more sensitive the receiving water or the bigger the impact, the greater the acceptable cost.

If on-project treatment is not practical, off-project compensatory mitigation will often be required. The designer will need to coordinate with the REC and resource specialists to develop mitigation, and additional time may be needed.

### 14.9.8 Siting Criteria

**Location**

- ODOT must have legal access to stormwater treatment facilities in order to perform inspection and maintenance. If necessary, additional right-of-way or a permanent easement must be acquired if a treatment facility footprint cannot fit within existing ODOT right-of-way
- Access for maintenance shall not require shutting travel lanes.
- Facilities treating only stormwater runoff from outside ODOT right-of-way must be placed outside the ODOT right-of-way.
- Facilities shall avoid impacts to protected environmental or cultural resources.
- Facilities must not be located on steep hillsides or other geologically unstable areas.
- If the facility includes a berm, the toe of the berm must be setback from property line by one half the berm height or 5 foot minimum.

- Maintain a minimum distance of 3 feet from the bottom or invert of a facility to bedrock or seasonally high water table.
- UIC horizontal and vertical setback requirements are given in Section 14.10.14
- Infiltration BMPs should be located at least 100 feet (in Type "B" soils) to 200 feet (in Type "A" soils) away from domestic water wells. Contact the municipal drinking water well owner to determine specific setback requirements. Groundwater protection areas and Wellhead Protection Zones may also have special requirements.
- Above-ground facilities shall be located at least 100 feet from septic tanks and drain fields.

**Geotechnical and Geological**

- Embankments must be adequately designed to safely impound stormwater runoff.
- The long-term permeability of the surrounding soil must be verified to be adequate when designing infiltration facilities.
- Retaining walls must be designed according to the ODOT Geotechnical Design Manual.

## 14.10 Requirements and Design Elements for Water Quality Facilities

This section provides the general requirements for stormwater treatment facilities

### 14.10.1 Contributing Impervious Area for Transportation Projects

The project's contributing impervious area (CIA) consists of all impervious surfaces within the strict project limits plus impervious surfaces outside the project limits which drain to the project via direct flow or discrete conveyance (See Figure 14-3). For ODOT, the CIA is limited to ODOT owned or operated right-of-way or facilities, and for Local Agency projects, the CIA covers facilities owned or under the control of the local jurisdiction.

**Treatment Requirements for Contributing Impervious Area for ODOT Transportation Projects:**

The entire ODOT CIA must be treated for most projects with stormwater treatment triggers. There are some situations when the area that must be treated is smaller than the CIA. These include projects where the only triggering element is the replacement of a stream culvert, and long paving projects with only limited segments with widening or other triggers. Coordinate with the project Environmental staff to determine the treatment requirements for those projects.

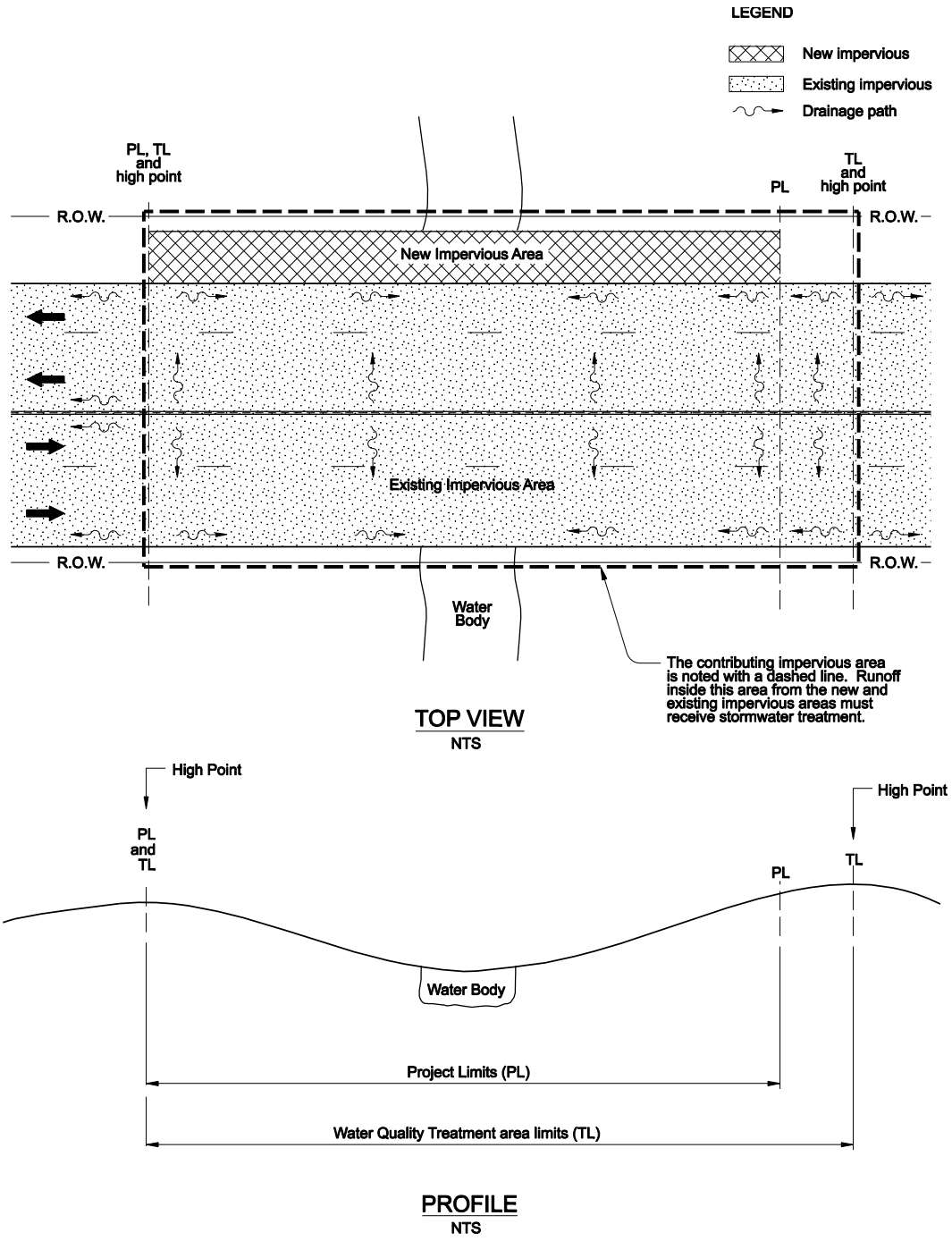


**Figure 14-3 Treatment Area Limits (ODOT Transportation project example)**

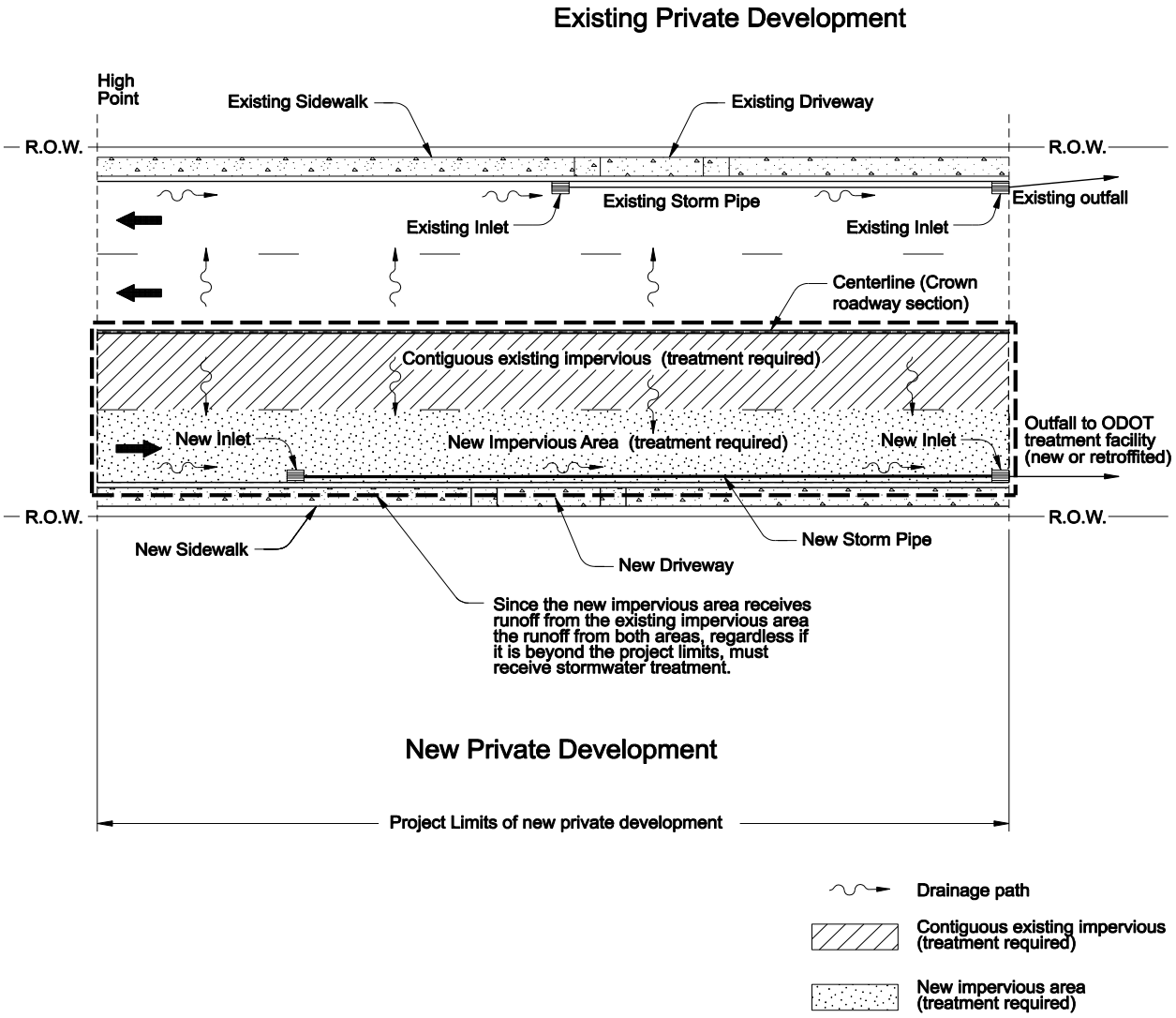


**Figure 14-4 Treatment Area Limits (private development example)**

## Treatment Requirments for Contributing Impervious Area for ODOT Mandated Frontage Improvements for Private Development Projects:

Private development projects adjacent to ODOT right-of-way maybe required to construct frontage improvements. Frontage improvements could include constructing sidewalks, driveway aprons, travel lanes, turn lanes, bike lanes, storm drain piping, manholes, catch basins, runoff treatment and detention facilities, etc. These frontage improvements are paid for with private funds and then ownership and maintenance obligations are transferred to ODOT.

These projects must treat all new ODOT impervious area and contiguous existing ODOT impervious area whose runoff flows over the new impervious surface (See Figure 14-4).

### 14.10.2 Water Quality Design Storm, Flow, and Volume

A stormwater treatment facility is sized based on a water quality design flow rate or water quality design volume. The hydrologic analysis needed to determine a design flow rate or volume is discussed in **Chapter 7**. The water quality design storm is discussed below.

#### Water Quality Design Storm

The water quality design storm is designated as a percentage of the 2-year 24-hour storm and is used to determine the water quality design flow rate or water quality design volume. The maximum design storm depth is 2.5 inches and the minimum water quality design storm depth is 0.7 inches.

The following steps outline how to select the design storm for a project:

Step 1: Determine the 2-year, 24-hour storm for the project. Use the precipitation maps to determine the project's 2-year, 24-hour storm or the GIS project created for use to view Oregon's precipitation data. See **Chapter 7** for more information.

Step 2: Determine the water quality design storm factor. Figure 14-5 outlines the storm factor to use for each climate zone in the state.

Step 3: Determine the water quality design storm. It is determined by multiplying the project's 2-year, 24-hour storm (step 1) times the design storm factor (step 2).

#### Water Quality Design Flow

The water quality design flow rate is the predicted peak discharge for the proposed conditions using the water quality design storm determined from the steps noted above. The design flow rate is calculated using hydrology guidance in **Chapter 7**. Flow-through stormwater quality facilities discussed in this chapter, such as swales and filter strips, are sized using this flow rate.

#### Water Quality Design Volume

The water quality design volume is the predicted volume of runoff for the proposed conditions using the water quality design storm determined from the steps noted above. The design volume is calculated using hydrology guidance in **Chapter 7**. Stormwater quality facilities discussed in this chapter that temporarily store runnoff, such as stormwater treatment wetlands, wet ponds, extended dry detention ponds, bioretention facilities, and infiltration facilities are sized using this design volume.

### 14.10.3 Pretreatment

The purpose of a pretreatment facility component is to extend the life and efficiency of the primary stormwater treatment facility. Pretreatment facilities are designed to target coarse sediment and debris transported by stormwater runoff. Installing a pretreatment facility should allow the primary facility to function as designed between inspection/maintenance cycles and should minimize damage associated with sediment loading, clogging of orifices, and erosion.

Although most BMPs benefit from pretreatment, it is especially important for:

- Facilities whose primary treatment mechanism is infiltration or media filtration.
- Facilities with a drainage area anticipated to produce heavy sediment loads (e.g., high frequency of winter sanding or agricultural activities).
- Facilities that include orifices.

The most common pretreatment facilities are:

- Sediment basin (such as a forebay or wet pool)
- Pollution control manhole
- Proprietary separation structure, or
- A vegetated filter such as a filter strip placed prior to a bioslope or a continuous swale parallel to roadway pavement

Additional discussion and design criteria are presented in **Appendix E**.

### 14.10.4 On-line and Off-line Facilities

Water quality treatment facilities can be designed to be “on-line” or “off-line”.

**On-line**

On-line BMPs are designed so that all flows pass through the facility. The goal of an on-line facility is:

- to provide treatment and storage capacity in a single facility, or
- to provide treatment and high flow conveyance in a single facility.

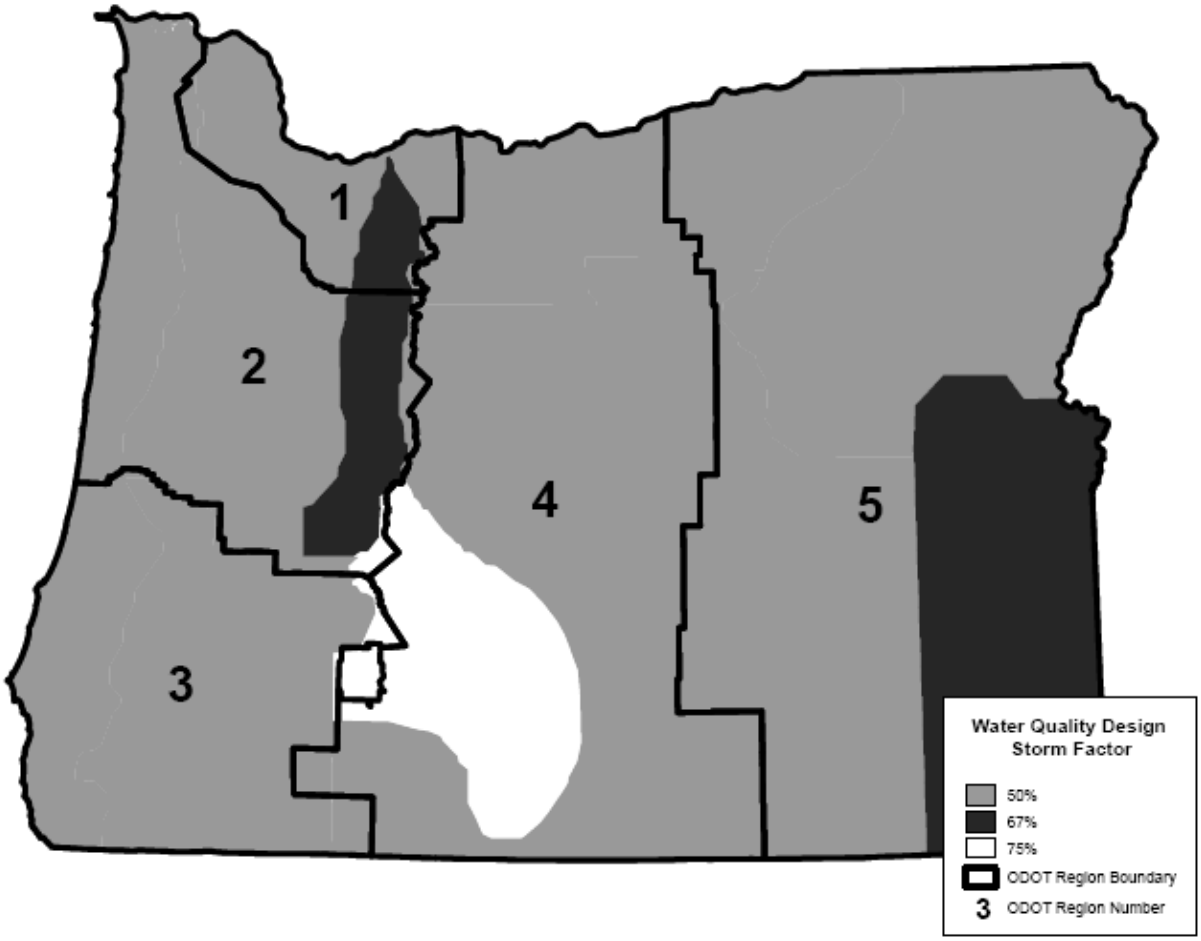


**Figure 14-5 Design Storm Factors**

Figure 14-6a illustrates an on-line treatment facility.

A single treatment and storage capacity facility is an option when both water quality and water quantity must be provided because of receiving water requirements. This application is considered to be an "on-line" facility and in many situations the most cost-effective stormwater management approach. Use the water quality design guidance in this chapter when designing combination facilities. Combination facilities are examples of units that can provide treatment and storage capacity in a single unit. Additional information on combination facilities is discussed in Section 14.10.7. Storage facility design guidance is discussed in **Chapter 12**.

A single treatment and high flow conveyance facility is an option when:

- Water quality must be provided because of receiving water requirements, and
- Regulating the quantity of stormwater is not required.

Swales and most proprietary facilities are examples of units that can provide treatment and convey high flows.

On-line facilities should be used when:

- Pollutants of concern can be removed and retained and
- Peak flows will not damage the facility.

**Off-line**

The goal of off-line facilities is to provide only water quality treatment. They are designed such that flows up to the water quality design storm flow rate are routed through the facility while higher flows bypass and continue downstream.

Figure 14-6b illustrates an off-line facility. An off-line facility must be preceded by a flow splitter manhole or other structure that allows flows only up to the water quality design storm flow rate to be conveyed into the treatment facility. Design criteria for flow splitters are discussed in Appendix L.

Off-line facilities should be used when:

- Pollutants of concern can only be removed and retained in an off-line facility, and
- Peak flows would damage an in-line facility.

Off-line water quality facilities are preferable because they prevent or minimize wash out of pollutants that have collected in the facility. All the design guidance discussed in this chapter is based on off-line facilities.

### 14.10.5 Sequence of Facilities

Water quality facilities should be installed upstream of detention facilities in order to prevent sediment build up in detention facilities. The build up of sediment in a detention facility would reduce storage volume over time, thereby resulting in a facility that does not function as designed. This sequencing also reduces the likelihood that a storage flow control structure will plug with sediment, litter, or debris because the water quality facility upstream controls and contains these pollutants within the water quality facility.

### 14.10.6 Multiple Treatment Facilities

Multiple treatment facilities in series (also known as a “treatment train” or “treatment system”) may be needed if a single treatment approach is not sufficient to meet water quality objectives. There are many combinations of standard structural BMPs that can form effective treatment trains. The effective combination of BMPs occurs when a treatment system incorporates two or more unit operations that each use different unit processes. This type of system is recommended

when it is not practical to use a single BMP with multiple unit processes that treat all of the pollutants of concern.

Be aware of the following limitations when planning treatment trains:

- Unit operations utilizing the same unit processes should not be placed in series. This is not a treatment train.
- Removal efficiencies cannot be added together to obtain the total removal rate. For example, an extended dry pond could remove 50 percent of phosphorus. The goal is to remove 90 percent of the total phosphorus. If two extended dry ponds were proposed in series, then it would be incorrect to assume that this application would remove 100 percent of the phosphorus (50 + 50 = 100). The actual removal rate would only be slightly greater than 50 percent because the first pond should have removed almost all that could be removed using the settling technique. There would be minimal benefit to using a second extended dry pond. The recommended approach is to use a different unit process that has a high capability of removing the target pollutant before or after the extended dry pond to achieve the desired removal rate.

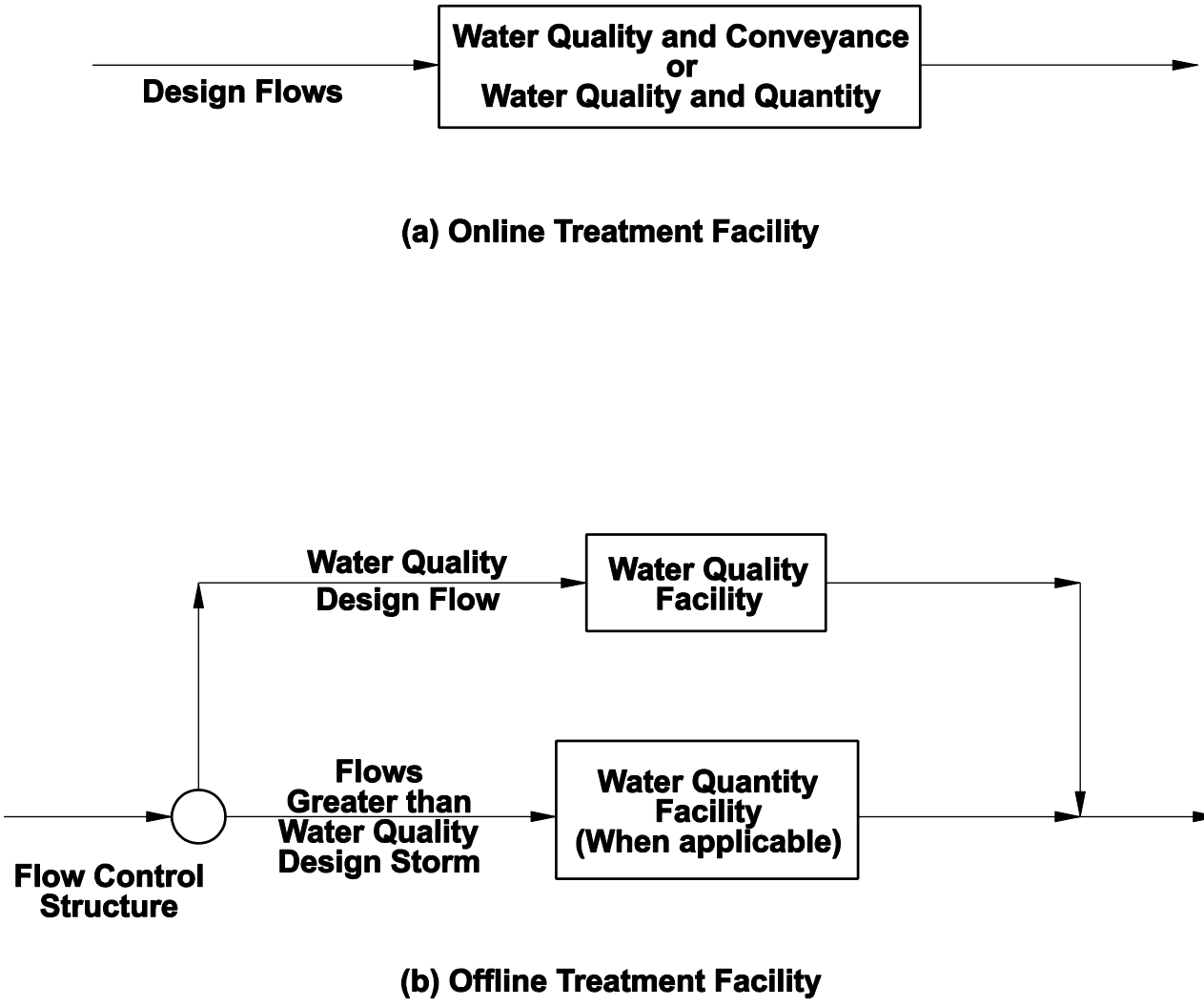


**Figure 14-6 Online and Offline Treatment Configurations**

### 14.10.7 Combined Facilities

Combination facilities are designed to treat stormwater and provide detention storage for peak flow reduction. Extended dry detention ponds, wet ponds, wet vaults, stormwater treatment wetlands, and infiltration facilities can be designed as combination facilities. The typical design technique for combined facilities is to dedicate the lower volume for water quality treatment and the upper volume is for detention storage.

A combination facility design is a two step process:

Step 1: Select a treatment method, design the stormwater quality component, and then design the stormwater facility components such as pretreatment and outlet control structure.

Water quality design guidance in this chapter should be used when designing combination facilities.

Step 2: Design the detention component, and then design the facility components such as the flow control structure, piping for conveyance, and outfall improvements when applicable. Storage facility design guidance is discussed in **Chapter 12**.

*Note: The pond volume required to provide water quality treatment should be assumed to be full and unavailable for peak flow storage when designing a pond as a combination extended detention dry water quality facility and peak flow detention facility.*

**Figure 14-7 Combined Facility (Hwy 213)**

### 14.10.8 Bypass Requirements

When a facility is placed off-line, stormwater flows must be directed or split in two directions. The water quality design flow is routed into the treatment facility. The higher flows are bypassed around the facility. This flow splitting is always located upstream of the treatment unit. The bypass flow is typically directed toward the downstream drainage system, or into a storage pond when necessary for flow control.

**Pre-facility Bypass**

General requirements include:

- High flow bypass is required for all water-quality-only (off-line) treatment facilities.

- The bypass structure and features must be designed to the same design storm as the upstream conveyance system.

**Facility Bypass (Overflow)**

The outlet structure releases the design peak flow rate or volume. The overflow structure must be designed to pass high flows in the event that the flow splitter fails or the flow control orifice is inadvertently removed or clogged by debris or sediment.

General requirements include:

- The outlet structure's high flow bypass feature or auxiliary outlet must be designed to convey the same design storm as the upstream conveyance system.
- Spillway weirs and storm drain pipes that outfall onto embankments and channels must be armored to prevent erosion. See Sections 14.10.12 and 14.10.13.

### 14.10.9 Inlet Structure (Flow Splitter)

The inlet structure, or flow splitter, is installed to control flow into the stormwater treatment facility. In most cases the inlet structure consists of a manhole, a flow splitter feature, and a high flow bypass feature. See **Appendix E** for more information.

**Flow splitter feature (orifice or weir)**

- Design using the water quality peak discharge.
- Orifices less than six inches in diameter require removable screening to prevent plugging.

### 14.10.10 Storm Drain Piping or Open Channel Conveyance

Storm drain piping and open channels are used to route stormwater into and out of a stormwater treatment facility.

General storm drain piping requirements include:

- All water quality facility storm drain piping must be designed to the same design storm as the upstream conveyance system.
- Minimum pipe size is 12-inches in diameter.
- Piping conveyance is analyzed using guidance in **Chapter 13**.

General open channel requirements are summarized in Section 14.10.12

### 14.10.11 Outlet Control Structure

The outlet control structure is installed to control flow out of the stormwater treatment facility. These structures are used in treatment ponds and combination facilities. In most cases the outlet

structure consists of a manhole, inlets, an outflow control device such as orifices or weirs, and a high flow bypass feature.

General requirements include:

- Overflow or bypass capability must be included in case of excess inflow or plugging of the outflow control device.
- Design using the same design storm as the upstream conveyance system.
- Orifices less than six inches in diameter require removable screening to prevent plugging.

### 14.10.12 Outfalls

The location where a stormwater treatment facility releases runoff is known as the outfall. The outfall can be along an embankment or into a natural channel or existing storm drain system.

General requirements include:

- Outfall channels stability and hydraulic capacity must be designed to the same design criteria as the upstream conveyance system. Channel improvements are analyzed using guidance in **Chapter 8**.
- Outfall channels located in a waterway's floodplain must be designed to conform to the local floodplain regulations.
- Outfalls into a channel should be oriented to prevent outflow from causing erosion on the channels opposite bank.
- Outfalls must be armor protected to prevent erosion as discussed in Section 14.10.13.

### 14.10.13 Energy Dissipation

Energy dissipators are used to reduce the velocity and, consequently, the erosion potential of flowing water. Protection is usually required at the outlet of storm drain systems that discharge onto embankments or into natural or unlined channels, swales, and ponds.

**Inlet dissipator**

- Inlet dissipators must be designed to the same design storm as the upstream conveyance system. Dissipator design is sized using guidance in **Chapter 11**.

**Outlet dissipator**

- Outlet dissipators must be designed to the same design storm as the upstream conveyance system.
- Outlet dissipators located in a waterway's floodplain must be designed to conform to the local floodplain regulations.

### 14.10.14 Underground Injection Control (UIC) Requirements

A UIC is a system, structure, or activity that is created to place fluid below the ground or sub-surface. Common stormwater UICs or activities include, but are not limited to sumps, infiltration galleries, drywells, trench drains, and drill holes. However, not all infiltration facilities are classified as UICs. The distinction between UICs and non-UIC infiltration facilities is often subtle. If there is any doubt, DEQ should be consulted for clarification.

UICs are not stand-alone treatment BMPs. Treatment of stormwater is required before discharge to a UIC.

Before deciding to use a UIC for stormwater management:

- Consult with DEQ to verify the selected method qualifies as a UIC.
- Consider all other treatment options before selecting a method that qualifies as a UIC.
- Determine DEQ's current UIC permit requirements prior to beginning design.

General requirements include:

- Provide treatment prior to discharging stormwater into a UIC. In most cases sediment removal is sufficient; sedimentation manholes are generally acceptable in urban areas.
- Comply with DEQ requirements for setback from domestic, public or irrigation water wells. The basic requirements are:
    - 500 feet from any domestic water well, any irrigation water well, and any public water well that does not have an Oregon Department of Human Services approved 2-year time of travel, and
    - Outside the Department of Human Services approved 2-year time of travel for a public water supply well, or
    - Demonstrate that the water wells are protected from pollutants entrained in stormwater discharged into the underground injection systems within these setback areas.
- Comply with DEQ requirements for vertical separation between the bottom of a UIC and the seasonal high groundwater level. The basic requirements are:
    - Underground injection systems that are more than 5 feet deep shall have a minimum vertical separation distance of 10 feet between the bottom of the underground injection system and the seasonal high water table.
    - Underground injection systems less than or equal to 5 feet deep shall have a minimum separation distance of 5 feet between the bottom of the injection system and the seasonal high water.

### 14.10.15 Operation and Maintenance

The proper operation, performance, structural integrity, and aesthetics of a stormwater treatment facility are dependent on routine inspection and adequate maintenance. Inspection schedule and maintenance guidelines for each facility are summarized in an Operation and Maintenance Manual prepared to assist personnel who maintain the facility.

General requirements include:

- **Discuss proposed stormwater treatment facilities with the responsible Maintenance District before selection and design. Maintenance input can help in selecting and developing BMPs that are maintainable.**
- All stormwater treatment facilities require an Operation and Maintenance Manual. Prepare an operation and maintenance manual as outlined in **Chapter 4**.
- Distribute all prepared manuals to the appropriate district maintenance office and Geo-Environmental's Senior Hydraulics Engineer. An inventory of prepared manuals can be viewed at the following website:

  **Operation & Maintenance Manuals Website**

- All facilities need to be assigned a drainage facility identification number (see 14.10.17 below). Guidance on obtaining a drainage facility identification number is outlined in **Chapter 17**.
- All stormwater treatment facility structures should be accessible by foot and necessary equipment (e.g., vactor truck or mowers) for inspection and maintenance. Access road design criteria for pretreatment and primary treatment facilities are discussed in Appendices A, B, C, and D.
- Implement with Maintenance District concurreance: Manhole lids located in non-traffic areas outside or beyond the clear zone such as grassed areas or behind guardrail must be set 1 foot above finish ground so that manhole location is visible for locating and for maintenance. Lid elevations must match proposed finish grade in traffic areas.

### 14.10.16 Field Marking

Field markers are used to locate and identify ODOT stormwater facilities or alert maintenance crews of the location of a stormwater facility's maintenance area. There are three stormwater markers recommended for identifying, locating, or alerting. Two of these markers are used for marking above ground facilities and there is one marker applicable to underground stormwater facilities. ODOT's field marking process is outlined in **Chapter 17**.

### 14.10.17 Drainage Facility Identification Number

A drainage facility identification number (DFI) is a unique identifier assigned to each stormwater treatment and storage facility. It is used to associate or link the stormwater facility to an Operation and Maintenance Manual. The number is assigned by contacting the Geo-Environmental Section's Senior Hydraulics Engineer to obtain a unique "DFI". The Geo-Environmental Section will maintain a database of assigned Drainage Facility IDs. Guidance on obtaining a drainage facility identification number is outlined in **Chapter 17**.

**Figure 14-8 Stormwater Facility Field Marker (Type S2)**

## 14.11 Documentation

The following documents are required for most stormwater design projects. **Chapter 4** provides documentation guidance, outlines when the following reports are prepared, and what is to be included in each of these reports.

**ODOT Projects**

- Preliminary Stormwater Recommendations,
- Stormwater Design Report,

- Operation and Maintenance Manual, and
- A Stormwater Management Plan. This plan is required for Clean Water Act (CWA) Section 401 Water Quality Certification for projects needing CWA 404 permits. A template is available that provides guidance on what stormwater design information is required for a stormwater management plan. The template sections highlighted in green are to be completed by the project stormwater designer. This template can be viewed using the link below:

  **Stormwater Management Plan Template**

Distribute a copy of the Operations and Maintenance Manual to the appropriate district maintenance office. Distribute a copy of the Stormwater Design Report and Operation & Maintenance Manual in Adobe Acrobat portable document format (pdf) and in word format, and the operational plan Microstation file to Geo-Environmental's Senior Hydraulics Engineer by completing the project report submittal form located at the following website:

**http://www.oregon.gov/ODOT/HWY/GEOENVIRONMENTAL/hyd_data_resources.shtml**

## 14.12 Treatment Facility Design Procedure

Water quality planning and design begins as soon as a project has been determined to include elements that trigger the requirement for stormwater treatment (see section 14.5.1). The criteria for stormwater treatment are the same for all projects where treatment is required, but the approach and complexity will depend on the specifics of the project and its location. In some situations, the right-of-way can provide adequate treatment with little or no modification, while in other situations intensively engineered facilities may be needed. The first step in design is early scoping to gather information needed to select and locate treatment facilities, including opportunities for Low Impact Development (LID) techniques. Coordination with environmental specialists is vital during scoping and through DAP. Based on information gathered during scoping and subsequent project design information, treatment options are evaluated, with final recommendations ready by DAP. The design then advances into calculating site hydrology (sub-basin delineation, land use types, runoff coefficients, time of concentration, and peak discharges or volume) and facility component design (facility sizing, pretreatment, inlet structure, piping conveyance, outlet control structure, and outfall improvements, when applicable).

A generalized procedure for the selection of a permanent stormwater treatment facility or facilities follows. Although many projects may require different tasks, many of the following steps will be applicable.

Step 1 - Determine if stormwater treatment is needed. The Prospectus Part 3 will indicate if the project elements trigger stormwater treatment requirements. This information can be supplied by the Region Environmental Coordinator or the project Water Resources Specialist. Special treatment requirements are identified by the Water Resources Specialist or other Environmental staff.

Step 2 - Attend scoping trip with project team. Collect data as needed to develop the Preliminary Stormwater Recommendation (see **Chapter 4**). An additional site visit may be needed to collect more data or to address specific concerns. See **Chapter 6** for data collection guidance. Information required for water quality purposes includes the extent of the Contributing Impervious Area, characteristics of the roadside area (topography, soils, vegetation, protected resources etc.), and identification of receiving waters and location of outfalls. Coordinate with Environmental staff to identify conflicting resources that would preclude the placement of BMPs.

Step 3 - Evaluate stormwater treatment techniques and strategies in the following priority order:

1. Utilize existing site characteristics to treat stormwater runoff by:

   - Preserving or minimizing disturbance of permeable native soils and vegetation to maintain its function of storing and infiltrating stormwater runoff

   - Using existing dispersion site conditions within existing right-of-way to provide water quality treatment and limited flow control. Dispersion conditions occur along roadway sections in which curb and gutter systems are not used to collect and convey stormwater runoff from highway pavement. See Appendix **A** and **B**.

   Coordinate with Maintenance during the evaluation process to identify potential issues.

   Consider constructability costs, maintenance requirements and costs, site constraints such as steep slopes, high groundwater table, bedrock location, and proximity to existing or proposed structures and wells when performing this determination.

2. Improve site characteristics to treat stormwater runoff by:

   - Acquiring additional right-of-way to provide sufficient dispersion area needed for treatment
   - Modifying slopes or using water quality mix (see **Appendix E**) to improve treatment opportunities

3. Treat the remaining contributing impervious area by distributing treatment facilities throughout the project or consolidating runoff and routing to a treatment facility. Select BMPs from Table 14-2 (Engineered BMPs). Consider BMP removal capabilities; constructability; costs; and maintenance, site constraints such as steep slopes, high groundwater table, bedrock location, and proximity to existing or proposed structures and wells when performing this determination.

The goal is to consider the best treatment alternatives. Consider treatment alternatives in the following order:

1. "high" capability of removing the target pollutants.
2. "moderate" capability of removing the target pollutants.
3. "low" capability of removing the target pollutants

If high capability BMPs are not appropriate for the site, assemble a treatment train of other BMPs to achieve similar pollutant removal capability.

New, innovative BMPs or substantially modified approved BMPs that provide the best option for a project may be approved through the Hydraulic Design Deviation process. See **Chapter 3** for guidance

Step 4 - Eliminate from consideration the BMPs that are obviously impractical (including due to maintenance issues) or ineffective.

Step 5 - Coordinate design with other agencies – this is a very important element to the design process for stormwater treatment facilities. Coordinating ODOT work with cities and counties would:

- Inform the designer of local drainage or water quality ordinances that ODOT projects must satisfy.
- Allow local agencies an opportunity for watershed-wide collaboration and cost sharing

Step 6 - Prepare the Preliminary Stormwater Recommendation. This recommendation does not contain facility designs. See **Chapter 4** for guidance.

Step 7 - Distribute the Preliminary Stormwater Recommendation to the project team and Maintenance prior to DAP. Review comments and concerns from project team members should be considered when designing the engineered stormwater treatment facility or facilities.

Step 8 - Verify if adequate right-of-way is available for the approved facility or facilities. No easements or additional right-of-way are necessary if adequate right-of-way is available for placement and maintenance access. If adequate right-of-way is not available, then coordinate easements or additional right-of-way with the project leader, roadway designer, and right-of-way specialist.

Step 9 - Assemble additional data required for design of the selected BMPs. See **Chapter 6** for data collection guidance.

Step 10 - Verify survey. The type, source, and complexity of data for a stormwater treatment facility design will vary depending on the location and type of construction activity. If necessary, obtain additional survey information. See **Chapter 6** for data collection guidance.

Step 11 - If additional data collection was necessary and has been conducted, verify accuracy and completeness. See **Chapter 6** for guidance.

Step 12 - Design the approved facility or facilities based on the design criteria provided in the appropriate appendix or appendices. Design tasks include:

- Determine the water quality storm.
- Determine hydrology. See **Chapter 7**.
- Size facility or facilities to convey or store the water quality design storm

Step 13 - Design the appropriate facility components such as:

- Inlet structure
- Pretreatment
- Piping conveyance
- Outlet control structure
- Outfalls, and
- Energy dissipation

The facility components should be based on the design criteria provided in **Appendix E**.

Step 14 - Evaluate and address hydraulic grade line effects through the entire system to assure proper operation under design conditions. See **Chapter 13** for guidance.

Step 15 - Obtain a drainage facility identification number for each proposed facility. See **Chapter 17** for guidance.

Step 16 - Prepare the Stormwater Design Report. See **Chapter 4** for guidance.

Step 17 - Prepare the Operation and Maintenance Manual. An O&M manual is required for every new facility. The is a Microsoft Word template document available for water quality facilities that must be used to prepare these manuals. See Chapter 4 for guidance.

Step 18 - Complete the Stormwater Management Plan sections required by the project stormwater designer when applicable. See Section 14.11.

Step 19 - Distribute the Stormwater Design Report. Forward an Adobe Acrobat portable document format (pdf) copy and word format copy to Geo-Environmental's Senior Hydraulics Engineer by completing the project report submittal form located at the following website:

**http://www.oregon.gov/ODOT/HWY/GEOENVIRONMENTAL/hyd_data_resources.shtml**

File project stormwater design report and background information. See **Chapter 4** for guidance.

Step 20 - Utilize the appropriate stormwater control special provisions. Special provisions are provided for:

- SP01010 – Water Quality Structures
- SP01011 – Ponds
- SP01012 – Water Quality Biofiltration Swales
- SP01013 – Water Quality Bioslopes

- SP01014 – Water Quality Filter Strips

These provisions can be viewed and downloaded at the following website:

http://www.oregon.gov/ODOT/HWY/SPECS/2008_special_provisions.shtml

Step 21 - Coordinate installation of the field markers. Marking guidance is provided in **Chapter 17**.

Step 22 - Distribute a copy of the Operations and Maintenance Manual to the appropriate district maintenance office. Forward an Adobe Acrobat portable document format (pdf) copy and word format copy and the operational plan Microstation file to Geo-Environmental's Senior Hydraulics Engineer by completing the project report submittal form located at the following website:

**http://www.oregon.gov/ODOT/HWY/GEOENVIRONMENTAL/pages/omm.aspx**

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# APPENDIX A
# LOW IMPACT DEVELOPMENT APPROACH

## 1.0 Introduction

Low impact development (LID) is a comprehensive approach to managing stormwater. It involves designing projects to minimize the amount of runoff generated, and then uses management techniques that utilize natural hydrologic processes to treat the stormwater and moderate the changes in stormwater volume and flow. Specifically, LID techniques enhance infiltration and rely heavily on filtration through vegetation and natural or amended soil for treatment. As much as possible these functions are spread across the landscape, with minimal concentration of the stormwater. The LID approach targets both water quality and hydrology (flow control).

For highway projects, LID is defined as stormwater management techniques within the linear right-of-way that incorporate infiltration, filtration through soil or amended soil, and vegetation. Proprietary and other devices that are confined and depend on mechanical separation or installed filters are not considered LID BMPs, though they may be the best or most appropriate treatment techniques for the situation.

## 2.0 Advantages and Disadvantages

LID BMPs are generally effective at treating major highway runoff pollutants. The size of conveyance facilities may be able to be smaller than if conventional treatment facilities were used because they are frequently able to promote considerable infiltration of stormwater. Routine maintenance activities are usually fairly simple since LID BMPs do not involve confined spaces, frequent replacement of filters or material, nor orifices, or other devices that can clog or fail. On the other hand, the distributed nature of LID BMPs means that routine maintenance may involve multiple stops, which can be more time consuming than servicing a single, large BMP. Also, long term maintenance requirements of some LID BMPs are not yet entirely understood.

The footprint for some LID BMPs can be large relative to conventional, consolidated stormwater management facilities. This may be counterbalanced by lower construction costs.

Conventional and LID BMPs are not mutually exclusive, and can be components of a treatment train to remove pollutants.

## 3.0 Elements of LID

The LID approach consists of the following elements:

***Minimization:*** Keeping the amount of impervious area to a minimum reduces the amount of stormwater runoff generated, and therefore reduces the size and extent of management facilities. For

highways, minimization is difficult to achieve to any substantial extent. Safety related design standards take precedence over reductions in runoff generation. Minimization can be achieved incidentally when realignment shortens the length of the roadway. Removal of abandoned pavement is also a minimization technique. Pervious pavement is acceptable in certain situations, but generally not for highways. See current ODOT guidance on pervious pavement.

***Conservation:*** Unpaved ground provides water quality and hydrologic functions and can be used to manage stormwater runoff. The capability of the unpaved area depends on multiple factors including soil properties, vegetation cover, and topography. Some features, such as wetlands and forests, are very effective, while previously compacted and modified surfaces may be only slightly better than pavement. Retaining natural conditions is the important first step in LID stormwater management.

***Enhancement:*** The existing right-of-way will not always be well suited for stormwater management due to previous manipulation, poor soils or topography, or inadequate space. Modifications such as soil amendments, reshaping, regrading and plantings can improve conditions enough to meet the stormwater management goals. These enhancements are intended to increase permeability, increase pollutant removal capability, reduce the velocity of flow of stormwater, and provide for increased filtration by vegetation.

***Dispersion:*** Keeping stormwater spread out once it leaves the pavement minimizes flow depth and flow velocity, and increases the amount of infiltration and contact between the water and soil/organic material. Conservation and enhancement are intended to support dispersion as a stormwater management strategy. Ideally, highway runoff will sheetflow from the edge of pavement across the right-of-way and on to its natural course. In some situations, such as at the end of bridges, concentrated flow must be dispersed as much as possible.

***Stormwater Control Facilities:*** Local conditions may preclude the use of dispersion for stormwater management, so engineered BMPs become necessary. LID engineered BMPs emphasize infiltration, filtration through media (amended soils) and, where climate allows, vegetation. Standard LID BMPs include Bioretention facilities and soil-amended vegetated swales. (biofiltration swales) These are differentiated from similar conventional BMPs mostly by size and placing. LID BMPs serve relatively short segments of highway, so they are smaller than conventional BMPs, and there may be several instead of a single large one.

## 4.0 LID in Project Development

Stormwater management must be a part of project development from project initiation. If it is not, a project may find itself locked into expensive to construct or maintain facilities that, in the worse case, may not meet stormwater management requirements. The project may then face delays and additional expense.

The following sequential steps are recommended for determining how stormwater will be managed on a project:

1. Incorporate project design elements that would reduce total runoff (Minimization).
2. Evaluate the ability of the post construction right-of-way to manage project runoff. If the right-of-way is not capable of completely providing adequate stormwater management, then
3. Identify enhancements of right-of-way. If the right-of-way is still not capable of completely providing adequate stormwater management, then
4. Evaluate the use of LID Structural BMPs. If additional stormwater management is still necessary, then
5. Design conventional BMPs

Projects may use LID BMPs on some segments, conventional BMPs on others, and any combination as needed to treat stormwater.

## 5.0 Evaluating a Project for LID Opportunities

The type, design, and extent of LID practices that can be incorporated into a project depend on both highway design elements and the characteristics of the existing and prospective right-of-way. The following are the primary considerations:

***Highway Features:***

- Width of roadway draining to the side of the highway: Filter strips depend on maintaining sheet flow, so the depth of stormwater as it enters the filter strip should not be deep enough to initiate concentrated flow. The ability to infiltrate a design storm also depends on the amount of water flowing across a filter strip, along with the permeability of the soil. Guidance on the ratio between road width and filter strip width, and maximum road width is discussed in Appendix B.
- Curbs and existing/proposed storm drain system: Filter strips are ruled out as a treatment option if curbing is installed to intercept and direct stormwater runoff into drainage features such as inlets and storm drain piping. Instead, localized facilities such as bioretention basins should be considered, along with off-site dispersion, swale, or proprietary facilities.

***Physical Characteristics of the Right-of-way:***

- Topography:
    - Steepness of the highway embankment and adjacent land: Filter strips, dispersion areas and bioswales all have optimal ranges of slope for pollutant removal performance. Excessive steepness can be compensated for to a certain extent by greater length of the facility and other modifications. Guidance on appropriate slope ranges is discussed in each BMP appendix.
    - Distance to constraining features: This is anything that limits the size and location of a treatment BMP, and includes cliffs, protected natural resources, protected cultural resources, hazmat sites, and property that is not feasible to acquire. Modification of

BMPs can, to a certain extent, compensate for inadequate space. Guidance on length or width requirements is discussed in each BMP appendix.

- Location of surface waters, including streams, lakes, and wetlands: These are a subset of constraining features. If close, they increase the value of on-site stormwater management while possibly constraining what can be done. On the other hand, lakes, ponds, and wetlands can eliminate the need for flow control BMPs (treatment before discharge is still required).

- Soils:

  - Permeability: The area required to infiltrate a design storm is dependent on soil permeability. Hydrologic class A and B soils are well suited for infiltration. C soils are poorly suited and D soils are unsuited for infiltration. For guidance in infiltration and sizing of filter strips and infiltration facilities, see Appendix B and C. Soil amendments increase the ability of the amended layer to capture and hold runoff, but increase infiltration only by providing more time for it to occur.
  - Organic and fines content: The pollutant removal capability of the soil is positively related to the amount of organic material and fines. Sandy soils, particularly hydrologic class A, offer high permeability, but may be lacking in the ability to remove pollutants. Soil amendments can adequately compensate for poor organic content. See Appendix E for guidance on amending or importing a water quality mix.

- Vegetation:

  - Density and type of shrub and overstory: Shrubs and trees, particularly evergreens, can substantially reduce the amount of stormwater runoff compared to even permeable ground covered only by grasses. Their preservation outside of the clear zone is therefore important, particularly when their part of the right-of-way drains to the storm drainage system.
  - Density and type of ground cover: Dense herbaceous ground cover slows overland flow, maintaining sheet flow and allowing for increased infiltration, and traps pollutants. Removal of pollutants seems to be most effective when the ground has 80 percent or greater cover.

- Groundwater:

  - The seasonal high groundwater table should be at least 3 feet below the bed of infiltration BMPs.
  - Infiltration BMPs should be located at least 100 feet (in Type “B” soils) to 200 feet (in Type “A” soils) away from domestic water wells. Contact the municipal drinking water well owner to determine the setback requirements. Groundwater protection areas and Wellhead Protection Zones may have special requirements.

- Geotechnical and geological hazards:
    - Unstable slopes should not receive additional water, therefore BMPs that support infiltration should not be used near these locations
    - Flood prone areas are not well suited for stormwater BMPs because the accumulated pollutants in the BMPs may be washed out, and besides the BMPs may be flooded by the very waters they are intended to protect. If there is no option other than treatment within the floodplain, vegetated BMPs should be used to stabilize accumulated sediment.

## 6.0 Low Impact Development Techniques

The following sub-sections provide an overview of low impact development approaches used on ODOT projects. These techniques can be implemented as a stand-alone treatment option or combined with other BMPs to meet treatment goals.

## 6.1 Dispersion

### Description

Dispersion relies on maintaining sheet flow across vegetated and permeable ground, which maximizes stormwater contact with soil and vegetation. Only a narrow width of contiguous vegetation is needed for effective attenuation and treatment as long as runoff enters the dispersion area as sheet flow. Concentrated flow may be dispersed with a flow spreader to achieve sheet flow conditions.

Dispersion uses existing vegetation, soils, soil amendment, and topography to provide stormwater runoff treatment and some level of flow control. The pollutant-removal processes include infiltration, sorption by soils and organic material, and filtration by vegetation. Moderation and reduction of storm flows results from infiltration or retention in the soil, and to a lesser extent evapotranspiration. Dispersion areas can consist of preserved native habitat and can provide visual buffering of the roadway.

The filter strip, where runoff sheet flows from the highway pavement over vegetated shoulders, side slopes and right-of-way, is the most common form of dispersion. In cases where conditions do not allow dispersion adjacent to the highway, stormwater can be collected and conveyed to a dispersion area. Energy dissipation and flow spreaders are required at the outfall into the dispersion area.

While dense vegetation is desirable in dispersion areas, in some climates this is not possible. In those locations it is important to ensure that the soils, amended or natural, are protected from both wind and water erosion.

### Applications

- Ideal for highways with wide linear, undeveloped right of way.
- Is applicable for non-roadway projects (e.g. parking lots, maintenance yards), as long as sheet flow conditions can be achieved, i.e. impervious and pervious areas graded to avoid concentrating flows.

### Design Criteria

Utilize the design criteria for filter strips outlined in Appendix B to meet requirements for:

- Siting of facility
- Groundwater separation
- Soil suitability
- Facility geometry (slope, width, length)
- Water quality mix
- Planting requirements, and
- Field markers

## 6.2 Low Impact Development Bioretention

### Description

LID bioretention areas are landscaped depressions or cells with water quality mix, grass, and/or plants adapted to the local climate. Its function is to mimic natural conditions where the soils and plants store, treat, infiltrate and slow stormwater runoff. It also has been referred to as a "rain garden" in other stormwater management design manuals.

### Applications

- Treat and infiltrate stormwater runoff from building rooftops, driveways, pedestrian/bike paths, loop roads, cul-de-sacs, parking lots, and maintenance yards.
- Retrofit landscaped area or impervious areas to treat stormwater runoff.
- Can be implemented along sites with lower infiltration rates by installing an underdrain system to bypass water not able to infiltrate.

### Design Criteria

Utilize the design criteria for bioretention facilities outlined in Appendix C to meet requirements for:

- Siting of facility
- Groundwater separation
- Soil suitability
- Pretreatment

- Facility geometry (side slopes, depth, volume, and freeboard)
- Berm embankments
- Maintenance access
- A bottom marker
- Water quality mix
- Planting requirements
- Field markers
- Sub surface drainage, and
- Protective treatment

### 6.3 Soil Amending to Establish a Water Quality Mix

Water Quality Mix is used to retain stormwater to allow for infiltration into the underlying soils and to improve pollutant removal capabilities. It is comprised of a mix of inorganic materials and organic matter.

Utilize water quality mix in dispersion areas (filter strips), biofiltration swales, bioretention facilities, and extended dry detention facilities.

See the Water Quality Mix Section in Appendix E for additional discussion and design guidance.

## 7.0 Design Guidelines

### 7.1 Adaptation for Climate

LID BMPs have traditionally incorporated vegetation as an integral component. Dry or cold climates present challenges to rapid or dense establishment or sustainability of vegetation, so BMPs dependent on dense growth may fail. The pollutant removal capability of vegetation can be replaced by the use of amended soil or compost, but poor vegetative cover will leave the soil vulnerable to erosion. Protection of the soil is therefore important. In climates where vegetation establishment is feasible but slow, temporary ground cover may be adequate. Arid region BMPs may need permanent cover, such as the placement of coarse drain rock on top of amended soil. Irrigation to ensure dense plant growth is recommended only where the vegetation will survive on its own after the establishment period.

Innovative and modified LID BMPs to meet local conditions are encouraged, but are to follow the hydraulic design deviation process (Chapter 3) so they can be tracked and evaluated for future formal design guidance.

## 7.2 LID in Urban Situations

Many LID approaches to stormwater management developed by other agencies have been focused on the urban and suburban environment. However, popular LID BMPs such as rain gardens and porous pavement, that are appropriate and useful on low volume and low speed roads need modification or are not realistic for high volume or high speed highways.

Space is the biggest challenge for LID techniques for urban highways. Some freeways have wide, unpaved rights-of-way, but urban highways that function as city streets are frequently bordered by buildings and pavement on all sides. Bioretention rain gardens and some proprietary systems do support the LID approach, but urban constraints will affect their design and placement. Infiltration may be precluded by the presence of adjacent buildings with basements that could be damaged by water seepage. Pedestrian movement should not be obstructed by the placement of BMPs. Bulb-outs for traffic calming can house BMPs, but they are not acceptable on high speed roads.

# APPENDIX B
# BIOFILTRATION FACILITIES

## 1.0 Introduction

This appendix provides information for the design of biofiltration facilities such as swales and filter strips. Biofiltration facilities are intended to maximize the amount of stormwater that flows through dense vegetation, compost or soil, and to increase the potential for infiltration as compared to standard conveyance systems.

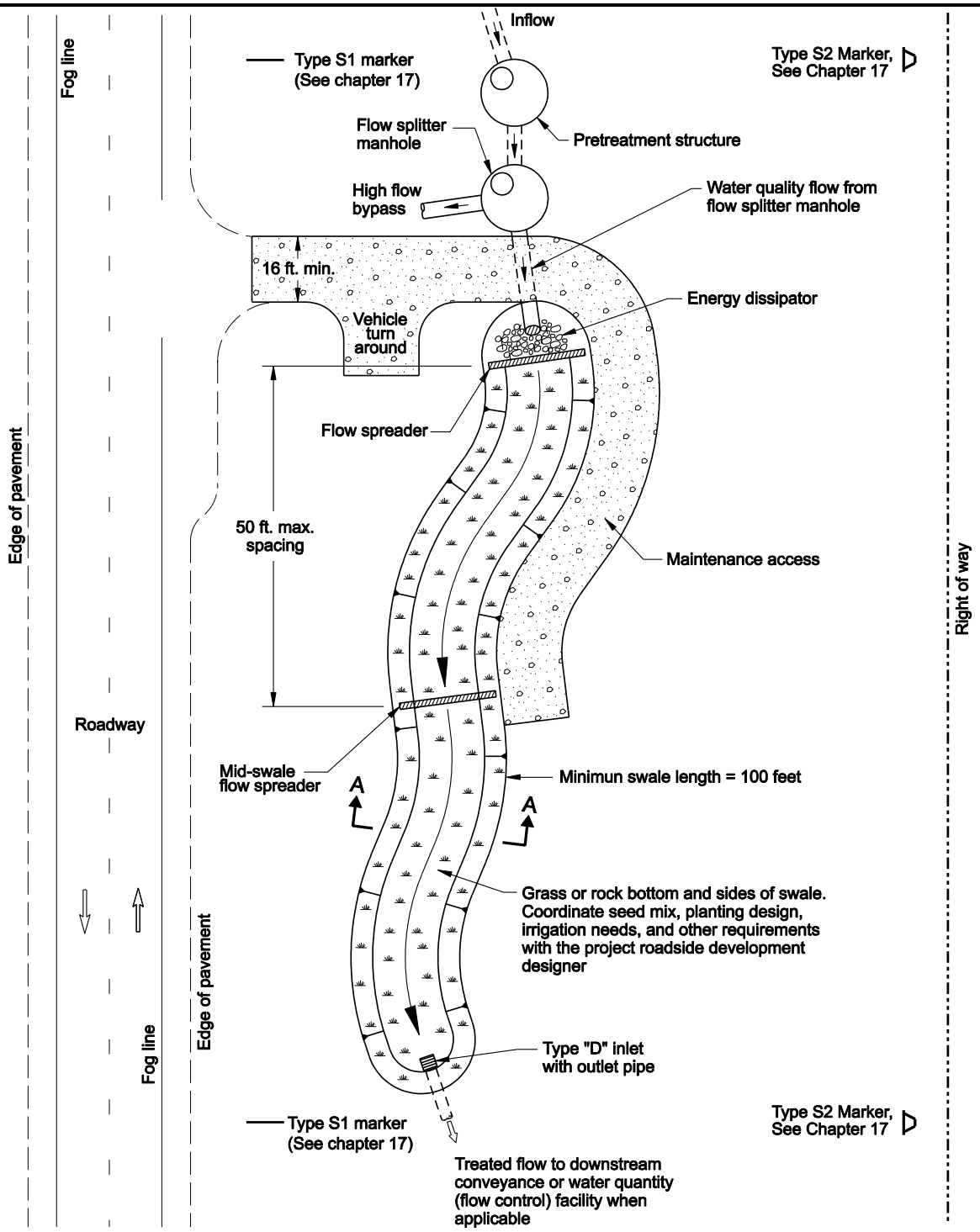
## 2.0 Biofiltration Swales

Biofiltration swales are open channels engineered to treat stormwater. Figures 1A and 1B are general configurations of a biofiltration swale. They are designed with gentle slopes, shallow flows, and lined with grass. Biofiltration swales are very popular because of their low construction and maintenance cost, few design limitations, and ability to be located in median strips, along the shoulders of roadways, and parking lots.

### Applications

- Retrofit existing drainage channel characteristics such as the slope and shape or incorporate a soil amendment to maintain or improve treatment opportunities (low impact approach)
- Place prior to the outfall when runoff is consolidated using curbing, inlets, and storm drain piping
- Place along median strips, roadway shoulders, and parking lots

Trapezoidal channel cross-sections are used for biofiltration swales because they are easy to construct and are hydraulically efficient cross-sectional shapes. These flat-bottomed open channels minimize flow depth and therefore maximize the amount of runoff flowing through the vegetation. This is turn increases pollutant removal by trapping and sedimentation. Also, the wide and flat-bottom cross section increases the amount of water that comes in contact with soil/organics which increases the removal of dissolves and increases the potential for infiltration.




PLAN VIEW
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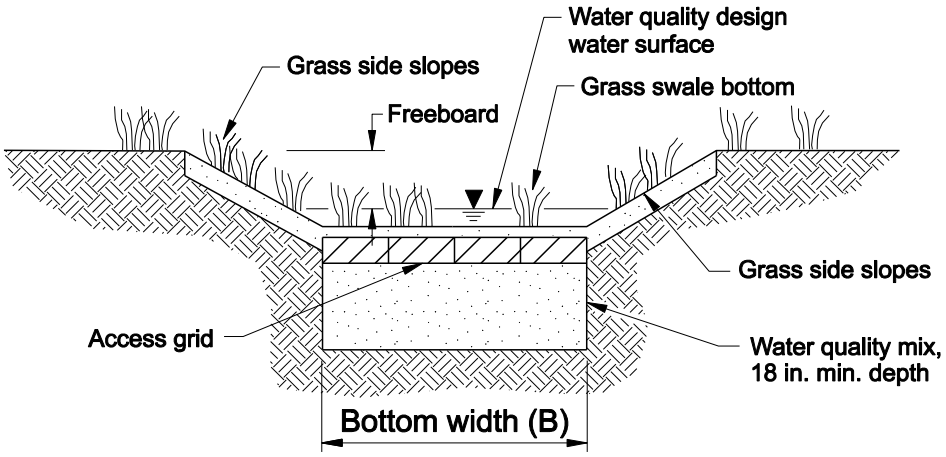
**Figure 1A Biofiltration Swale – off-line (Plan View)**




Cross Section A-A
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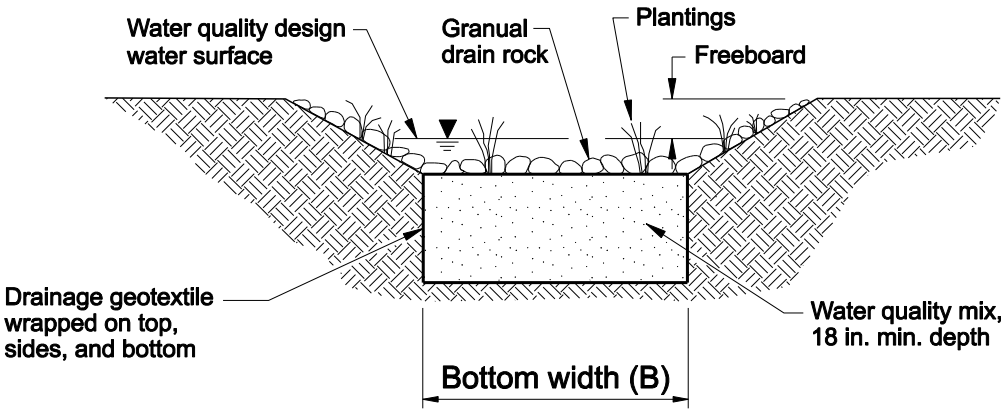
(Typical cross-section in Oregon climate zones 1, 2, 3, and 4)




Cross Section A-A
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(Typical cross-section in Oregon climate zones 5, 6, 7, 8, and 9)

**Figure 1B Biofiltration Swale (Cross Sections)**

Biofiltration swales can be designed to provide water quality control only. These types of facilities are:

- Constructed offline, thereby requiring a flow splitter structure upstream.
- Constructed channels with regular geometric cross-sections.
- Dry between storms otherwise vegetation would die off.
- Designed for storm drain pipe to direct roadway runoff into the upstream end of the swale. The runoff drains along the swale and outlets into an appropriate outfall.

Combination biofiltration swales are designed to provide water quality control and high flow conveyance. These types of facilities are:

- Constructed online, thereby not requiring an upstream flow splitter structure.
- Constructed channels with regular geometric cross-sections.

Swale with an access grid using porous pavers

**Figure 2 Biofiltration Swale with an access grid using porous pavers**

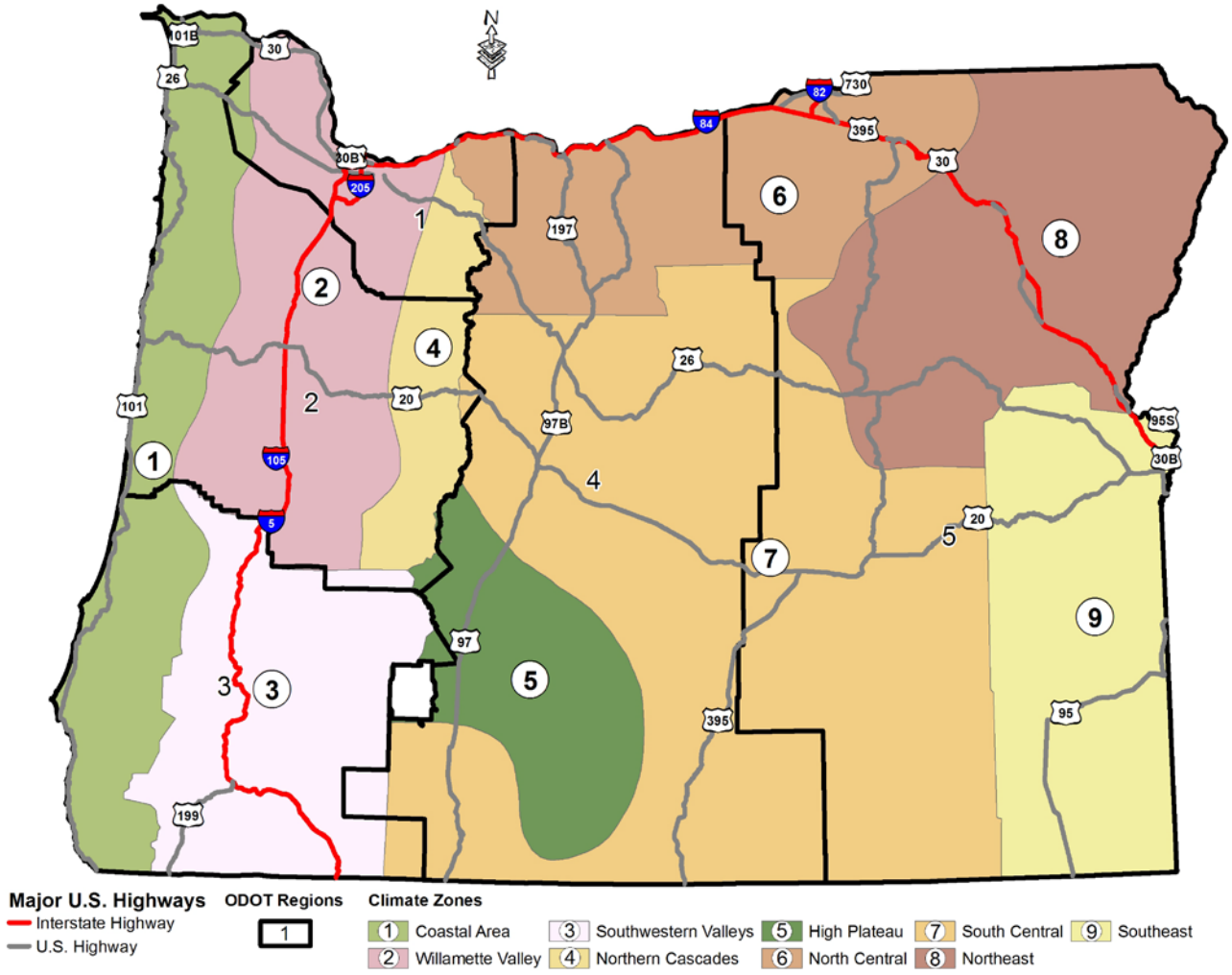



**Figure 3 Oregon Climate Zones**

## 2.1 Design Criteria

The design goal is to provide a uniform shallow flow depth over and through densely grassed area and long hydraulic residence time because this provides the most favorable opportunity for pollutant removal. The design criteria for grassed swales that achieve this goal are summarized below. Also apply the general requirements discussed in Section 14.10.

**Site Selection**

1. General siting requirements are discussed in Section 14.9. Additional siting criteria that apply specifically to swales include:

   a) The site must be of sufficient size to accommodate the swale and maintenance access.
   b) Do not place swales in shady areas. Daily sunlight is needed to maintain adequate vegetation cover.
   c) Climate conditions that affect the condition of grass and plantings as discussed in Section 14.9.

**Groundwater**

1. Maintain a minimum distance of 3 feet from the bottom or invert of a facility to bedrock or seasonally high water table.

**Pretreatment**

1. A pretreatment facility component is required to be installed upstream of the proposed swale. Design a pretreatment facility component according to guidance provided in Appendix E. Swales that are adjacent and parallel to the highway may receive pretreatment by sheet flow along vegetated side slopes. The minimum width of the vegetated side slope is 3 feet.

**Swale Geometry**

1. The **minimum bottom width** is 4 feet. The final width must be wide enough to convey the peak water quality design flow (see Design Water Depth below), and allow for maintenance. Therefore, the final design requires concurrence by the Maintenance District responsible for maintaining the facility.

2. The **longitudinal slope** (along the direction of flow) of the bottom of these facilities must be sloped toward the outlet.

   a) The minimum bottom grade is 0.5 percent.
   b) The maximum bottom grade is 6 percent.

3. The swale shall have a flat cross section (perpendicular to the flow direction) to allow for even flow across the entire width of the swale.

4. The swale cross-section shall be trapezoidal with 1V:4H maximum side slopes. Side slopes may be steeper if approved by the project design team and the maintenance district.

5. The swale **depth** must be adequate to convey the peak water quality design flow for off-line designs. The swale **depth** must be adequate to convey the peak water quality design and high flow for on-line designs.

6. The swale **freeboard** depth is 1 foot minimum measured between the design storm water elevation to the top of side slope.

7. The flow **length** between the swale inlet and outlet must be equal to or greater than 100 feet. There is no maximum length guideline. The total treatment length does not include the energy dissipator footprint. For example, assume a treatment length of 125 feet is required and a 3-foot-long by 4-foot-wide inlet energy dissipator is proposed. Therefore, the total bottom treatment length should be 128 feet long (125 feet + 3 feet).

8. Swales with horizontal curves are encouraged, but the curves must be mild to prevent bottom and side slope erosion and allow for equipment access to perform maintenance activities.

*Note: Locate the treatment section along the downstream end of combination swales or online designs. Provide a minimum 5 foot wide filter strip between the combination swale and roadway pavement along the treatment section.*

**Water Quality Design Water Depth**

1. A swale that will have a gradient of 0.5 percent to 4 percent shall have a maximum depth of 0.33 feet (4 inches) for the water quality design storm

2. A swale that will have a gradient of greater than 4 percent to 6 percent shall have a maximum depth of 0.25 feet (3 inches) for the water quality design storm.

**Manning Coefficient**

1. The flow resistance coefficient (Manning's n) is 0.24.

**Hydraulic Residence Time**

1. The **minimum residence time** is 9 minutes to allow the opportunity for pollutant removal from stormwater entering the swale.

**Flow Velocity, Flow Spreading, Energy Dissipation**

1. The **maximum flow velocity** is 3 feet per second during the 25-year design flow.

2. A **flow spreader** must be used at the inlet of a swale (off-line designs) to dissipate energy and evenly spread runoff as sheet flow over the swale bottom. An inlet flow spreader is not

required for swales that are parallel and adjacent to the road (on-line designs). Provide additional flow spreaders at 50-foot intervals. Design flow spreaders according to guidance provided in Appendix E.

3. An **energy dissipator** must be provided when the swale outlet pipe discharges into an outfall channel or sloped bank. Energy dissipator design guidance is provided in Appendix E.

**Sub Surface Drain**

A typical sub surface drain pipe system is shown in Figure 5. A sub surface drain pipe system is required to prevent standing water conditions when the subsoil classification is Natural Resources Conservation Service (NRCS) Hydrologic soil groups C or D and the bottom slopes less than 1.5 percent. A sub surface drain pipe may not be needed for NRCS Hydrologic subsoil groups A and B.

Subsurface drains must meet the following criteria:

1. The sub surface drain pipe must be a perforated pipe, laid parallel to the swale bottom, centered beneath the swale, and backfilled and bedded as shown in Figure 5.

2. The sub surface drain pipe must be 6 inches or greater in diameter.

3. The granular drain backfill material must be wrapped with drainage geotextile.

4. The sub surface drain pipe must drain freely to an existing discharge point or outfall.

**Bottom Marker**

1. A **bottom marker** made of porous pavers, are installed along the swale bottom to indicate the bottom elevation. Pavers are required in Oregon climate zones 1, 2, 3, and 4 (See Figure 3). Pavers are not required in Oregon climate zones 5, 6, 7, 8, and 9 (See Figure 3). Select a porous paver from the Qualified Products List. The porous paver must provide a minimum 80 percent bottom area opening for grass growth. Spaced solid paver blocks are not allowed. Pavers can also function as the access grid to support maintenance equipment. See maintenance access section below.

2. Note the following in the facility's operation and maintenance manual:

   - the use of porous pavers to mark the bottom elevation of the swale
   - use sediment removal techniques that will not damage porous pavers

**Maintenance Access**

3. An **access road** shall be provided along one side of the swale when constructed away from the highway. The proposed access road must be able to support heavy equipment such as a vactor truck, dump truck, track hoe, or large mower.

4. Access road must be 16 feet in **width**.

5. The access road **maximum longitudinal slope** must be:

   a) 2 percent (edge of pavement to a longitudinal distance of 20 feet)
   b) 10 percent (20 feet from edge of pavement to end of access road)

6. The access road **maximum cross slope** is 4 percent.

7. An **access grid** made of porous pavers must be installed along the swale bottom for maintenance vehicle and mowing equipment access. Select a porous paver from the Qualified Products List. The porous paver must provide a minimum 80 percent bottom area opening for grass growth. Spaced solid paver blocks are not allowed. Pavers are required in Oregon climate zones 1, 2, 3, and 4 (See Figure 3). Pavers are not required in Oregon climate zones 5, 6, 7, 8, and 9 (See Figure 3).

8. **Manhole lids** for access to inlet and outflow pipe that are located in non-traffic areas such as grassed areas or behind guardrail must be set 1 foot above finish ground so that manhole location is visible for locating and for maintenance. This should be coordinated with the maintenance districts, lids may be placed flush with the finished grade at the request of the serving maintenance district. Lid elevations must match proposed finish grade in traffic areas. No manholes should be placed in biofiltration swales.

**Retaining Walls**

1. Retaining walls are not to be located within the active treatment channel.

**Water Quality Mix**

There are three design options to establish a "Water Quality Mix" that meets criteria for organic content, long term hydraulic conductivity and other soil characteristics. See Appendix E.

**Planting Requirements**

1. Grass shall be established along the sides and bottom of swale prior to facility operation.

2. Permanent seeding is best performed as follows:

- West of the Cascades – March 1 through May 15 and September 1 through October 31 if grass areas are watered regularly during the establishment period.
- East of the Cascades – October 1 through February 1 or March 1 through October 1 if grass areas are watered regularly during the establishment period.

3. Sod can be used if the sod is grown from a seed mix suitable for the wet conditions of a swale.

4. Grass seeding is not required, but should be considered for Oregon climate zones 5, 6, 7, 8, and 9 (See Figure 4). Coordinate herbaceous plants and shrubs planting plan with the project roadside development designer or landscape architect when grass is not an appropriate option.

**Field Markers**

1. Field Markers are required to be installed at the start and end of a facility's maintenance area. Marking guidance is provided in Chapter 17.

**Figure 4 Biofiltration Swale (Southbound I-205 ramp onto Southbound I-5)**

## 2.2 Design Procedure

The procedure for designing an off-line or on-line swale is presented below.

**Step 1** – **New or relocated designs:** Identify facility locations according to the site suitability requirements noted in Section 2.1.

**Existing vegetated roadside drainage channels**: Identify existing channels with the following characteristics:

- receives sheet flow from adjacent roadside pavement
- channel length is equal to the adjacent roadside pavement length
- has a maximum longitudinal slope of 6 percent
- is within the project limits and will not be eliminated or impacted by proposed improvements, or
- has a trapezoidal shape or can be modified to the desired shape within existing or purchased right-of-way limits

*Note: Additional right-of-way may be required for construction of new, relocated, or modified swales.*

**Step 2** – Determine water quality design storm. Highway runoff from impervious areas needs to address the most stringent standards or reference ODOT's requirements summarized in Section 14.10.2.

**Step 2a** – Determine the Contributing Impervious Area for the facility. See Section 14.10.1.

**Step 3** – Determine the water quality peak flow. Use hydrology guidance in Chapter 7 and the design recurrence interval from step 2.

**Step 4** – Select what appears to be the best longitudinal slope based on site-specific conditions.

**Step 5** – Assume an initial water quality depth. The maximum treatment depth allowed is 3 inches or 4 inches. See design water depth section (Section 2.1).

*Note: The removal rate of stormwater pollutants through a swale should increase as the water depth decreases.*

**Step 6 –** Calculate swale bottom width (B).

B = Bottom width of the trapezoidal swale (feet)

$$B = \frac{nQ}{1.49y^{1.67}s^{0.5}}$$

Where:

n = Manning's Roughness Coefficient = 0.24 (dimensionless)
Q = peak water quality design flow (cubic feet per second)
y = design water depth (feet)
s = swale slope (feet per feet)
Z = side slope (dimensionless); for example, 1V:4H slope; Z = 4

**Step 7 –** Calculate the cross-sectional area (A).

A = Cross sectional area of the swale (square feet)

$$A = By + Zy^2$$

*Note: Variables are defined in Step 5.*

**Step 8 –** Calculate the average velocity (V).

V = Average velocity (feet per second)

$$V = \frac{Q}{A}$$

*Note: Variables are defined in Steps 5 and 6.*

**Step 9 –** Calculate the swale length (L).

L = Swale flow path length (feet)

L = Vt (60 seconds per minute)

Where:

V = average velocity (feet per second) from Step 6
t = hydraulic residence time in swale (minutes)

The minimum hydraulic residence time is 9 minutes. Substituting 9 minutes into the above equation results in the following:

L = 540V

*Note: Length of swale is never less than 100 feet long. Use (L) calculated in this step or 100 feet; whichever is greater.*

**Step 10 –** Calculate the swale top width (T) using the water depth in Step 3.

T = Swale top width at the water quality depth (feet)

$$T = B + 2yZ$$

*Note: Variables are defined in Step 5.*

**Step 11– Off-line designs** (new installations or modifying an existing channel/swale)**:** Adjust swale cross-section design to include freeboard using the bottom and top width calculated in Steps 6 and 10.

**OR**

**On-line designs** (new installations or modifying an existing channel/swale): Adjust swale cross-section design to include freeboard and high flow conveyance using the bottom and top width calculated in Steps 6 and 10. Follow these steps to design for high flow conveyance:

11.1 Compute the 25-year design discharge using hydrology guidelines in Chapter 7.

11.2 Calculate the average velocity using the design discharge calculated in step 11.1. Check adequacy of lining if average velocity is greater than 3 feet per second during the 25 year design discharge using guidance on shear stresses on channel linings that are presented in Chapter 8. Compare maximum to permissible shear stress. Reduce maximum shear stress by one or more of the following if maximum shear stress is greater than allowed:

- widening the channel, or
- Decreasing the channel slope
- Flattening the bank side slope, or
- Increasing the radius of curvature if the swale channel has bends.

11.3 Re-check water quality control design.

**Step 12 –** Coordinate the following field testing with the project geologist for the areas of interest:

- Determine the soil type(s). Take at least three samples (one at each end and mid-point of the channel.
- Determine the organic matter content

**Step 13** – Evaluate the soils tests for gradation and organic matter content for each facility site. Go to Appendix E, Section 2 regarding Water Quality Mix. One of the following options is met to achieve treatment performance:

- the existing soil gradation and organic matter content criteria is met, or
- add compost to the topsoil or subsoil in-place, or
- excavate and place imported or stockpiled soil that meets the gradation and organic matter content criteria

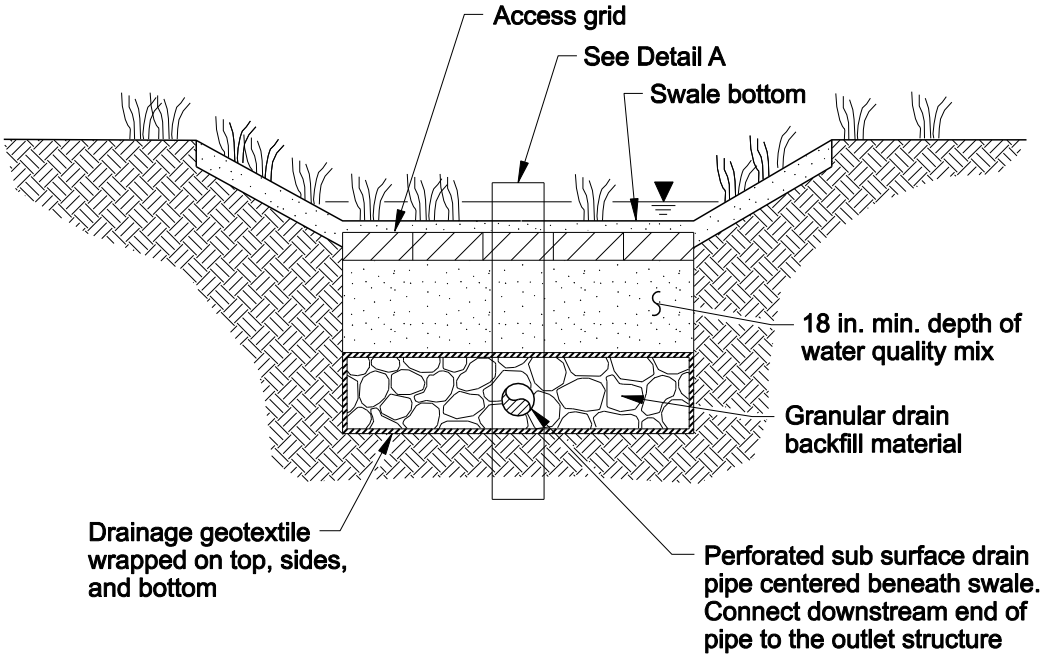
**Step 14–** Design a sub surface drain pipe system for bottom slopes less than 1.5 percent or when the subsoil classification is Natural Resources Conservation Service Hydrologic soil groups C or D. See Sub Surface Drain Section (Section 2.1).

**Step 15–** Design or coordinate the following facility components using the guidelines included in Appendix E:

- Pretreatment
- Flow splitter manhole
- Inlet energy dissipation
- Flow spreaders
- Storm drain piping
- Outfall
- Energy dissipation
- Coordinate soil preparation, seed mix, planting requirements, irrigation needs, and other requirements with the project roadside development designer.
- Coordinate temporary and/or permanent erosion control measures with the project erosion control designer

**Step 16–** Prepare the Stormwater Design Report and Operations and Maintenance Manual as discussed in Section 14.10.15 and 14.11.

**Step 17** – Coordinate the installation field markers at the start and end of a facility's maintenance area. Marking guidance is provided in Chapter 17.




Cross Section A-A
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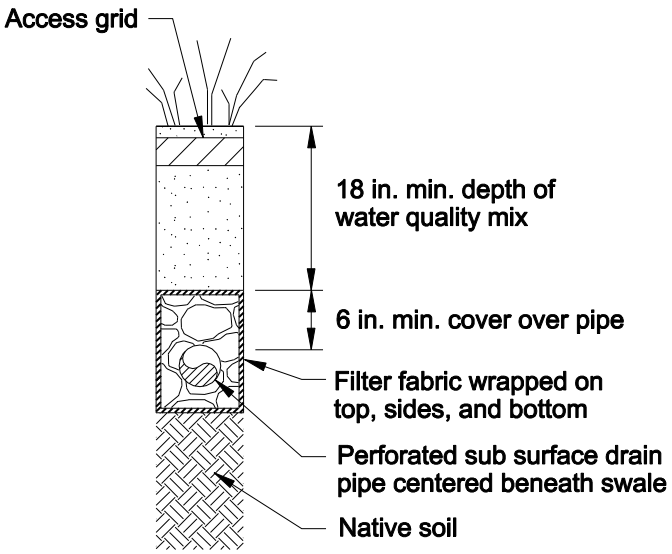


DETAIL A
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**Figure 5 Biofiltration Swale with Sub Surface Drain**

## 3.0 Filter Strips (Dispersion)

Dispersion is a simple and common method of treating stormwater runoff. It relies on maintaining sheet flow across vegetated and permeable ground which maximizes stormwater contact with soil and vegetation. In arid areas, aggregate may be used instead of vegetation where the soil supports infiltration.

Filter strips are the most common form of dispersion for highways, and can be used as either the sole BMP or as part of a treatment train. They consist of the right-of-way parallel to the road, with a relatively flat cross slope to maintain sheet flow of stormwater runoff over the entire width of the strip. Dispersion areas away from the highway receive collected runoff and use flow spreaders to create shallow, dispersed flow over vegetated slopes. The discussion here will focus on filter strips. Elements particular to other dispersion areas will be specifically called out.

A filter strip removes pollutants from pavement runoff by means of filtration through vegetation, media filtration and infiltration. Treatment mechanisms include physical trapping of particles, density separation (settling) in hydraulic dead zones and absorption, and to a lesser extent biological uptake and decomposition. Factors affecting the ability of filter strips to treat stormwater include vegetation density, slope and soil characteristics.

A filter strip (Figures 6 and 7) is a grassed sloped area located or placed between pavement and a downslope conveyance system. In cases where site conditions are not appropriate for a filter strip, stormwater can be collected and conveyed to a dispersion area.

The low impact approach is to preserve or enhance existing filter strip characteristics by modifying the side slope or incorporating a soil amendment to maintain or improve infiltration or media filtration.

Filter strips may be appropriate where:

- The road is elevated above the landscape on at least one side
- Impervious drainage area longitudinal slope is 4 percent or less
- Lateral slope of the highway (impervious surface) is 5 percent or less
- At least 6 feet of width from the edge of the shoulder is available
- Slope of the filter strip would be 15 percent or less (6:1 or flatter)

Sites that do not meet all of these criteria may still be used as filter strips. Modifications such as soil amendments may compensate for some shortfalls, or the strip may be part of a treatment train. For example, a too narrow filter strip may function as pre-treatment for a biofiltration swale.

**Figure 6 Filter Strip**

Filter strips would not be effective and should not be considered when:

- Sheet flow cannot be maintained
- Steep slopes are proposed
- Impervious drainage area longitudinal slope is steeper than 4 percent, or
- Longitudinal slope of filter strip area is greater than 2 percent.
- Impervious drainage area lateral slope is steeper than 5 percent.
- Climate conditions adversely affect the condition of grass and plantings as discussed in Section 14.9.6.3.
- Site conditions affect the condition of grass such as heavily shaded areas. Filter strips require sunlight exposure and moisture to ensure vigorous grass growth

Figure 7 is a typical grassed filter strip configuration. Filter strip width is measured perpendicular to the pavement and filter strip length is measured parallel to the pavement. In addition, the figure defines the longitudinal and lateral slopes.

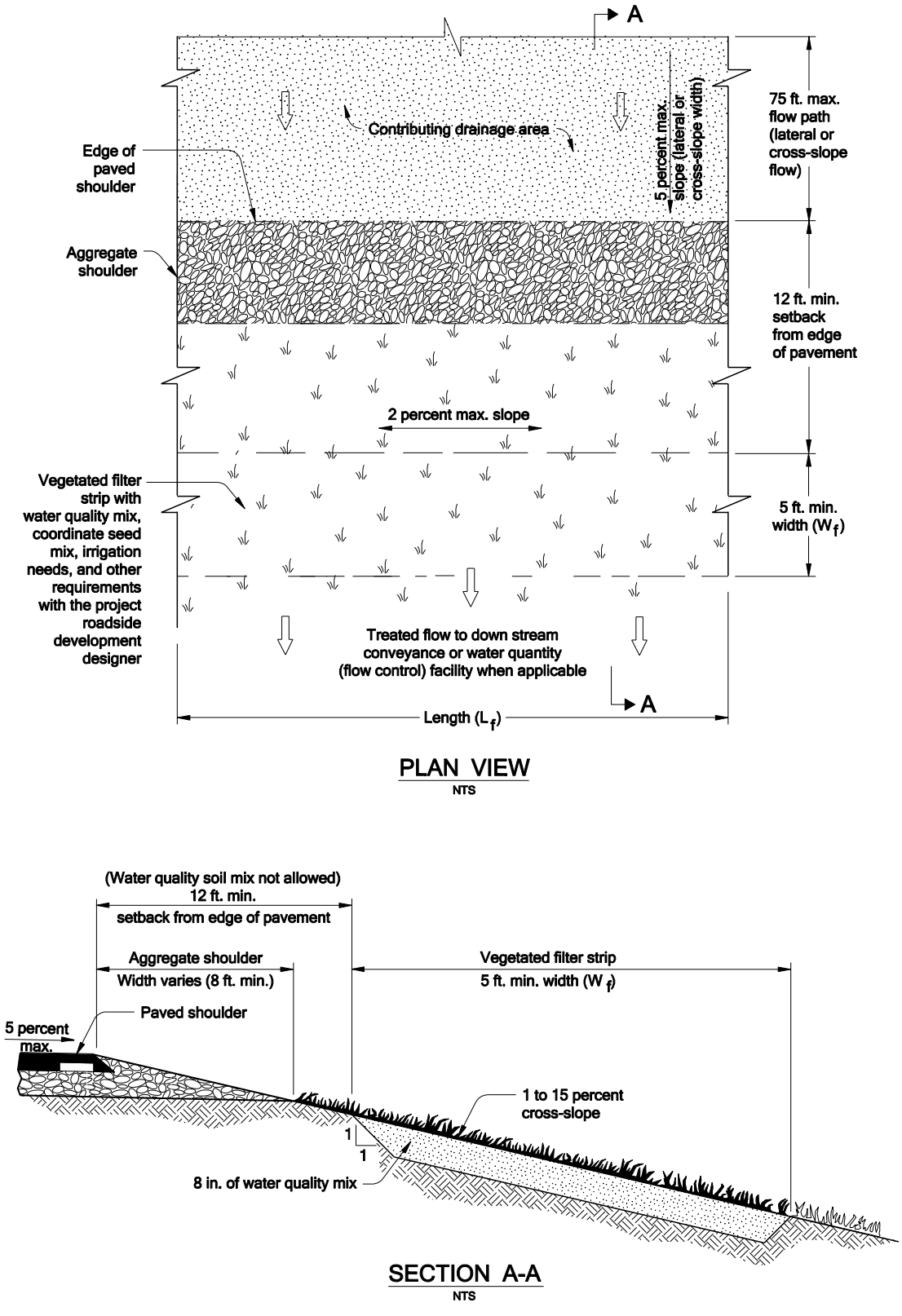



**Figure 7 Grassed Filter Strip**

## 3.1 Design Criteria

The design criteria for vegetated filter strips are presented in this section. Also apply the general requirements discussed in Section 14.10.

### Site Selection

1. General siting requirements are discussed in Section 14.9. Additional siting criteria that apply specifically to filter strips include:

    a) The site must be of sufficient size to accommodate filter strips.
    b) Do not place a filter strip in shady areas. Daily sunlight is needed to maintain adequate vegetation cover.
    c) Climate conditions that affect the condition of grass and plantings as discussed in Section 14.9.6.3.

### Contributing Impervious Area Restrictions

1. The **maximum flow path** across the contributing impervious area to the filter strip must not exceed 75 feet.

2. The **lateral slope of the contributing impervious area** shall be 5 percent or less.

3. The **longitudinal slope of the contributing impervious area** shall be 4 percent or less.

### Groundwater

1. Maintain a minimum distance of 3 feet from lowest point of the filter strip to bedrock or seasonally high water table.

### Filter Strip Geometry

1. The **flow width** of the filter strip must be equal to or greater than 5 feet.

2. The **length** of filter strips placed parallel to the road must be equal to the length of the contributing impervious or pavement area. The **length** of dispersion areas away from the highway must be the length needed to create a dispersed flow condition equal to the design water depth noted below

3. The **lateral or cross-section** of the filter strip must be equal to or greater than 1 percent and to not exceed 15 percent.

4. The **maximum longitudinal slope** of the filter strip is 2 percent.

5. The flow resistance coefficient is 0.24.

**Design Water Depth**

1. Shallow non-concentrated flow is the goal. The maximum water depth is 1-inch.

**Sizing**

1. The **flow width or filter strip width** must be determined using the ratios or table provided below:

   - 2% sloped filter strip to treat 4 feet of pavement for every 1 foot of filter strip
   - 5% sloped filter strip to treat 3 feet of pavement for every 1 foot of filter strip
   - 10% sloped filter strip to treat 2 feet of pavement for every 1 foot of filter strip
   - 15% sloped filter strip to treat 1.5 feet of pavement for every 1 foot of filter strip

| filter strip slope (%) | filter strip width for 20 ft pavement width | filter strip width for 30 ft pavement width | filter strip width for 40 ft pavement width | filter strip width for 50 ft pavement width | filter strip width for 60 ft pavement width |
|---|---|---|---|---|---|
| 2 | 5 | 8 | 10 | 13 | 15 |
| 5 | 7 | 10 | 14 | 17 | 20 |
| 10 | 10 | 15 | 20 | 25 | 30 |
| 15 | 14 | 20 | 27 | 33 | 40 |

**Table 1 Filter Strip Sizing**

**Flow Spreader**

A **flow spreader** must be used between the roadway pavement and filter strip to ensure runoff is evenly distributed across the filter strip. This function is usually performed by the gravel shoulder.

A **flow spreader** must be used to create a dispersed flow condition equal to the design water depth at the inlet of dispersion areas placed away from the highway.

**Water Quality Mix**

There are three design options to establish a "Water Quality Mix" that meets criteria for organic content, long term hydraulic conductivity and other soil characteristics. See Appendix E.

**Planting Requirements**

1. Grass shall be established along the entire treatment area of the filter strip. In arid areas, aggregate may be used instead of vegetation where the soil supports infiltration.

2. Permanent seeding is best performed as follows:

   - West of the Cascades – March 1 through May 15 and September 1 through October 31 if grass areas are watered regularly during the establishment period.
   - East of the Cascades – October 1 through February 1 or March 1 through October 1 if grass areas are watered regularly during the establishment period.

**Field Markers**

1. Field Markers are required to be installed at the start and end of a facility's maintenance area. Marking guidance is provided in Chapter 17.

## 3.2 Design Procedure (low impact development approach or new installation)

The following design procedure is for new installation or for determining if an existing vegetated area meets dispersion requirements to treat stormwater runoff.

**Step 1 –** Identify areas within the project limits that will not be paved or gravelled. Areas of interest are vegetated areas or areas that can be modified with vegetation, slopes less than 15 percent, and minimum flow path widths of 5 feet (see site criteria in Section 3.1).

**Step 2 –** Determine the lateral or cross-sectional width of the impervious surface.

**Step 3 –** Determine the average lateral or cross-sectional slope. Use 2 percent for sizing treatment area if the slope is less than 2 percent.

**Step 4 –** Using the sizing table provided in Section 3.1 determine the minimum filter strip width using the information obtained in Steps 2 and 3. Coordinate with the Project Leader and Right-of-Way if additional right-of-way is necessary.

**Step 5 –** Coordinate the following field testing with the project geologist for the areas of interest identified in Step 1:

- Determine the soil type(s). Take at least three samples (one at each end and mid-point of the dispersion area(s).
- Determine the depth of the seasonally high water table and bedrock is at least 3 feet below existing ground for the entire limits of the dispersion area(s).

**Step 6 –** Evaluate the soils tests for gradation and organic matter content for each vegetated area identified in Step 1. Go to Appendix E, Section 2 regarding Water Quality Mix. A vegetated area(s) can be utilized as a dispersion area when the soil gradation and organic matter content is met. Alternate option for areas with soils meeting gradation requirements but not meeting the necessary percentage of organic matter is to add compost.

**Step 7 -** Coordinate seed mix, seed establishment irrigation needs, and other requirements with the project roadside development designer or landscape architect. Coordinate temporary and/or permanent erosion control measures with the project erosion control designer.

**Step 8 –** Prepare the Stormwater Design Report and Operations and Maintenance Manual as discussed in Section 14.10.15 and 14.11.

**Step 9** – Coordinate the installation field markers at the start and end of a facility's maintenance area. Marking guidance is provided in Chapter 17.

# APPENDIX C
# MEDIA FILTRATION FACILITIES

## 1.0 Introduction

This appendix provides information for the design of media filtration facilities such as bioretention ponds or cells, and bioslopes.

Media filtration, where stormwater flows through soil, amended soil, compost or a special mix of materials, allows for maximum absorption of dissolved pollutants, as well as physical trapping of particles. It is a major component of treatment by infiltration. Media filtration BMPs may infiltrate into the soil all, some or none of the stormwater they treat.

## 2.0 Bioretention Facility

A bioretention facility is a roadside depression, cell, basin, or pond constructed with amended soils, plantings, and may either include an underdrain system or dispose of stormwater entirely through infiltration. They are designed to capture the water quality design volume and filter out the pollutants by filtering the runoff through the water quality mix. The filtration process removes a variety of pollutants through physical, biological and chemical treatment mechanisms. The water in the facility exits through an under drain pipe below the water quality mix or infiltration into the soils underneath. The appropriate discharge method, infiltration, underdrains, or a combination of the two, is based on site infiltration rates determined from infiltration testing at the proposed facility location.

Bioretention is a BMP that can be utilized as a LID or conventional treatment facility. It is categorized as an LID when several small facilities are placed at relatively short intervals along the highway, and as a conventional BMP when the stormwater is collected and conveyed to a single, large facility.

A typical bioretention facility has the following facility features and components:

- **Storage** – The provided facility volume necessary to temporarily store the runoff from the water quality design storm until it filters through the Treatment Zone.
- **Treatment Zone using water quality mix** – The layer of the water quality mix covering the bed of the facility. Stormwater runoff flows down through the treatment zone and either infiltrates into underlying soils or is collected into the under drain pipe.
- **Underdrain Pipe** – The perforated pipe placed beneath the Treatments Zone to collect the treated stormwater and discharge it into a conveyance system. This is put in place when the underlying soils are not adequate or suitable for infiltration.
- **Auxiliary Outlet** – Provided to bypass high flows.

Figure 1 is a general configuration of a bioretention facility. The actual configuration will vary depending on site constraints and applicable design criteria.

Bioretention ponds or cells can be placed:

- adjacent to roadways
- in medians,
- within interchanges,
- adjacent to ramps, and
- away from the highway

## 2.1 Design Criteria

This section describes the features of bioretention facilities and the design criteria that apply specifically to these installations. Also apply the general requirements discussed in Section 14.10.

### 2.1.1 Site Selection

**Area Requirement**

1. The site must be of sufficient size to accommodate the facility and also to provide adequate setback distances. The proper setback distances are important to ensure slope stability, and maintenance access. General siting requirements are discussed in Section 14.9.

**Slope**

1. A Bioretention facility may sit on flat to moderately sloping ground. Slopes will require the use of an embankment or wall on at least one side, which will increase the cost and footprint of the facility (see Embankments below).

**Groundwater**

1. A minimum distance of 3 feet from the pond bottom or drain rock bed bottom to bedrock or seasonally high water table is required.

**Infiltration Rate of the Existing Soil**

1. Soils in NRCS hydrologic soil groups A and B are appropriate for Bioretention facilities using only infiltration for stormwater disposal. Underdrains are required for hydrologic soil groups C and D. Final determination of the need for underdrains requires establishing the existing soil's long term infiltration rate.
2. The long term infiltration rate of the native soil can be established with field testing or estimated as outlined in Design Procedure Section 2.2, Step 5.

3. Design or long term infiltration rate of 9 inches per hour or less is acceptable outside of groundwater protection areas and is acceptable at providing water quality treatment for groundwater protection. See Appendix E, Section 2 regarding organic matter content requirement. Place water quality mix over soils with a design or long term infiltration rate greater than 9 inches per hour. Determine any special requirements for groundwater protection areas and wellhead protection zones. Also see Water Quality Mix section for guidance on organic content requirements.

**Inappropriate Site Conditions**

1 Geotechnical stability. Bioretention facilities should not be placed on potentially unstable slopes
2 Hazardous materials. Bioretention facilities should not be placed in areas with hazardous material contamination of the soil or groundwater.

### 2.1.2 Facility Components

**Pretreatment**

1. A pretreatment facility component is required to be installed upstream of the proposed facility. Design a pretreatment facility component according to guidance provided in Appendix E.

**Pond (basin) or Cell Geometry**

1. The **facility bottom** must be flat.

2. The facility **storage volume** is designed to temporarily store the water quality volume. This is the volume between the bottom of the pond up to the start of the freeboard volume. The water quality volume is the predicted volume of runoff for the proposed conditions using the appropriate water quality design storm. Freeboard volume is in addition to the storage volume of the water quality volume.

3. The maximum water quality depth is 3 feet (not including freeboard). Other considerations when setting the depth include public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, shading, maintenance requirements, and freeboard. Aesthetically pleasing features are also important in urbanized areas.

4. The **freeboard** required is 1 foot minimum between the water quality design storm water surface elevation to the top of embankment. Freeboard is the vertical distance between the design water surface and the top of embankment, as shown in Figures 1.

5. The **minimum bottom width** for ponds is 10 feet to provide the needed storage and allow for maintenance. For bioretention cells (LID application) the minimum width is 4 feet.

6. **Interior side slopes** should not be steeper than 1V:4H unless approved by the project design team and the maintenance district.

7. **Facility walls** may be retaining walls designed in accordance with the ODOT Geotechnical Design Manual. A fence is typically provided along the top of the wall.

**Embankments**

Pond or cell berm embankments are often needed to obtain sufficient storage volume. Facility berm embankments must meet the following criteria:

1. Vegetated berm embankments must be less than 20 feet in height and have exterior side slopes no steeper than 1V:3H and interior side slopes no steeper than 1V:4H. Riprap-protected embankments should be no steeper than 1V:4H.

2. Facility berm embankments higher that 6 feet shall be **designed by a geotechnical enginee**r.

3. The **minimum top width** shall be 6 feet for facility berms 6 feet high or less, or as recommended by a geotechnical engineer for higher berms.

**Maintenance Access**

***Note: Heavy equipment is not allowed along the bottom of these facilities. Note this limitation in the facility's operation and maintenance manual.***

1. An **access road** shall be provided to the primary and auxiliary outlet control structures of bioretention ponds. The proposed access road must be able to support heavy equipment such as a vactor truck, dump truck, track how, or large mower. Bioretention cells may not need an access road if the Maintenance District concurs site conditions are acceptable for access.

2. Access road must be 16 feet in **width**.

3. The access road **maximum longitudinal slope** must be:

   a) 2 percent (edge of pavement to a longitudinal distance of 20 feet)
   b) 10 percent (20 feet from edge of pavement to end of access road)

4. The access road **maximum cross slope** is 4 percent.

5. An access ramp is required for mowing, repairs, and sediment removal. The ramp must extend to the pond bottom.
6. **Maximum grade** of an access road or ramp shall be 10 percent.

7. **Manhole lids** located in non-traffic areas such as grassed areas or behind guardrail must be set 1 foot above finish ground so that manhole location is visible for locating and for maintenance. This should be coordinated with the maintenance districts, lids may be placed flush with the finished grade at the request of the serving maintenance district. Lid elevations must match proposed finish grade in traffic areas. No manholes should be placed in ponds or cells.

**Bottom Marker**

1. A **bottom marker** is required to indicate the bottom elevation of the facility. The marker must be a minimum 1.5 foot by 1.5 foot concrete pad placed along the bottom of pond. The elevation of the top of concrete pad must be equal to the facility bottom elevation.

2. Porous pavers are not allowed in bioretention facilities.

3. Note the following in the facility's operation and maintenance manual:

   - the location of the bottom marker
   - check and restore bottom elevation referencing bottom marker after performing sediment removal work

**Water Quality Mix**

There are three design options to establish a "Water Quality Mix" that meets criteria for organic content, long term hydraulic conductivity and other soil characteristics. See Appendix E.

**Planting Requirements**

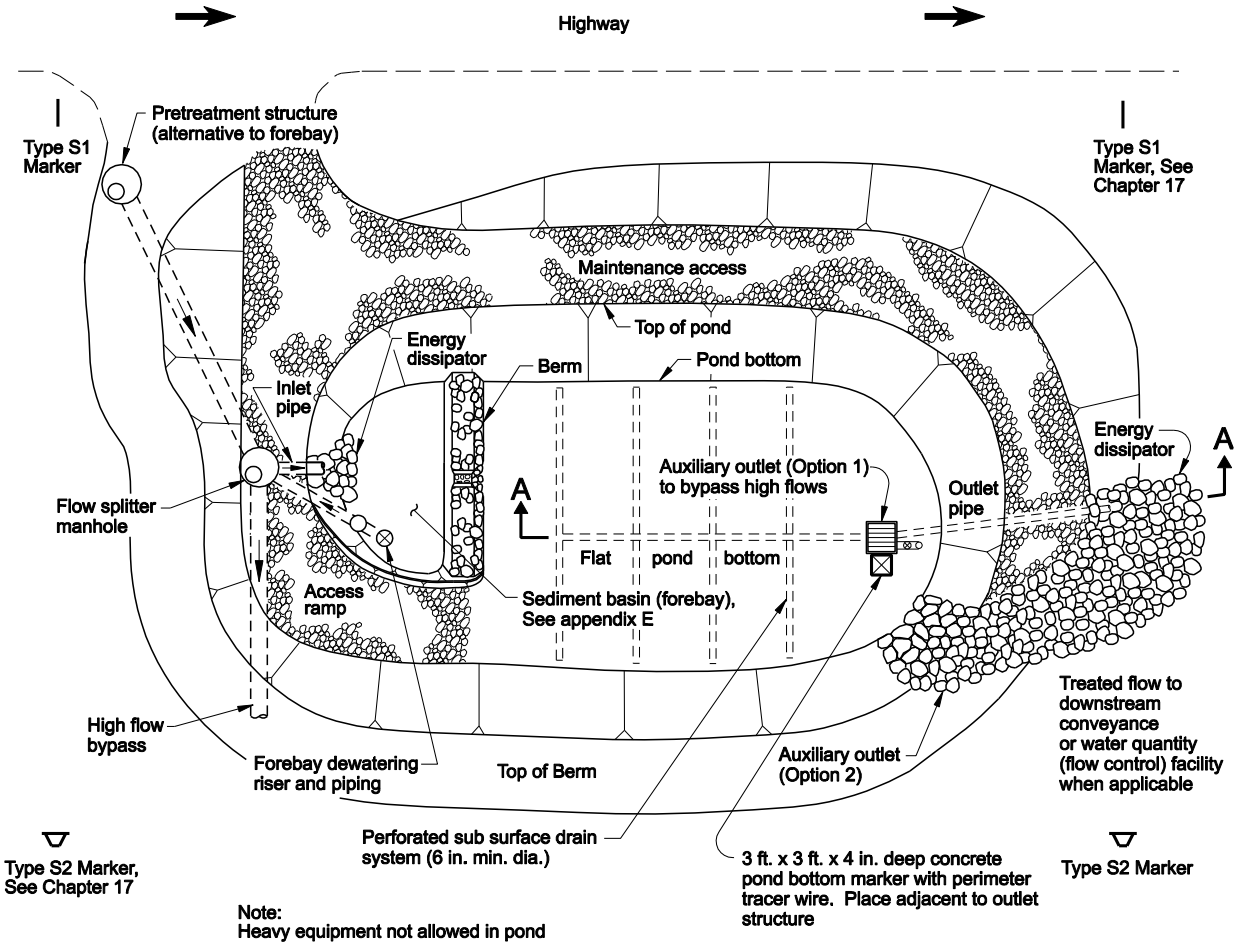
1. Grass shall be established along the sides and bottom of facility unless the local climate is unsuitable. Where grass is not appropriate, side slopes should be protected against erosion by other means. Note that urban jurisdictions may require plants and shrubs instead of grass.

2. Permanent seeding is best performed as follows:

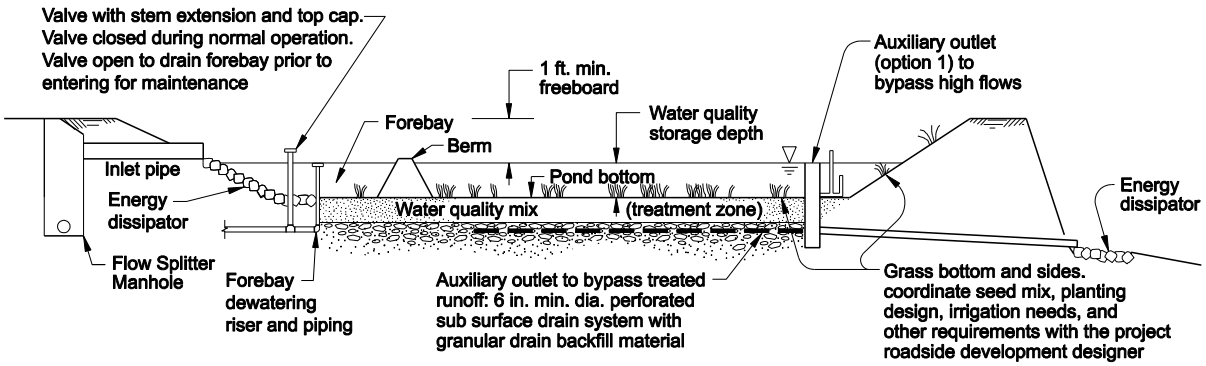
   - West of the Cascades – March 1 through May 15 and September 1 through October 31 if grass areas are watered regularly during the establishment period.
   - East of the Cascades – October 1 through February 1 or March 1 through October 1 if grass areas are watered regularly during the establishment period

**Field Markers**

1. Field Markers are required to be installed at the start and end of a facility's maintenance area. Marking guidance is provided in Chapter 17.

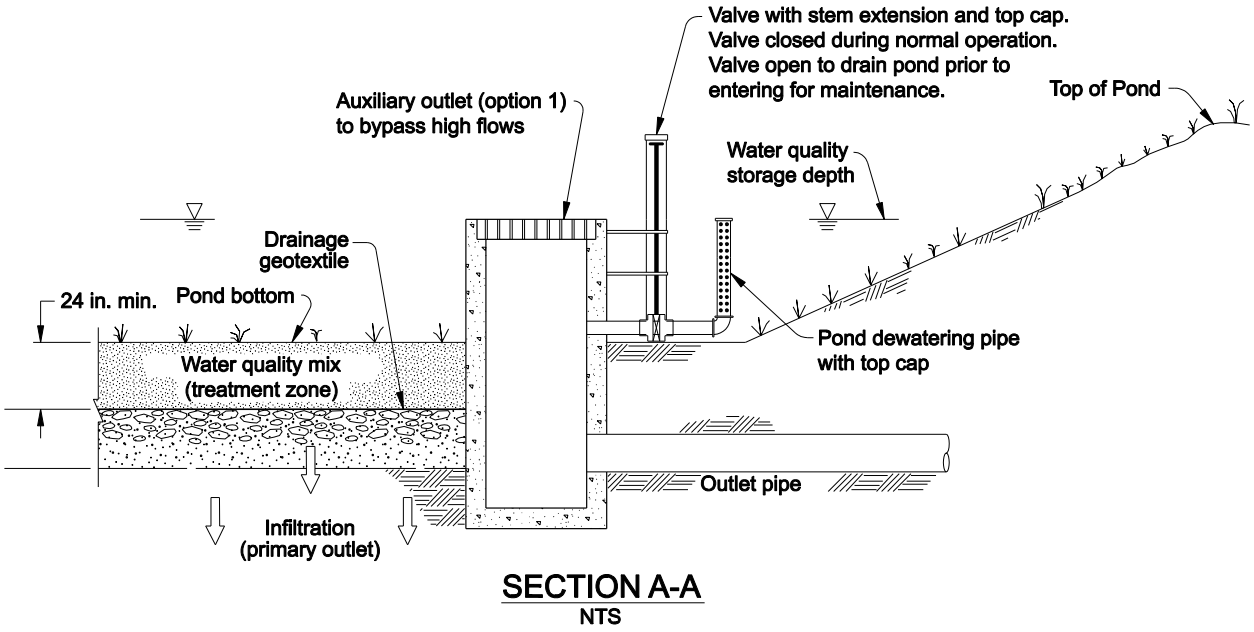



TOP VIEW
NTS

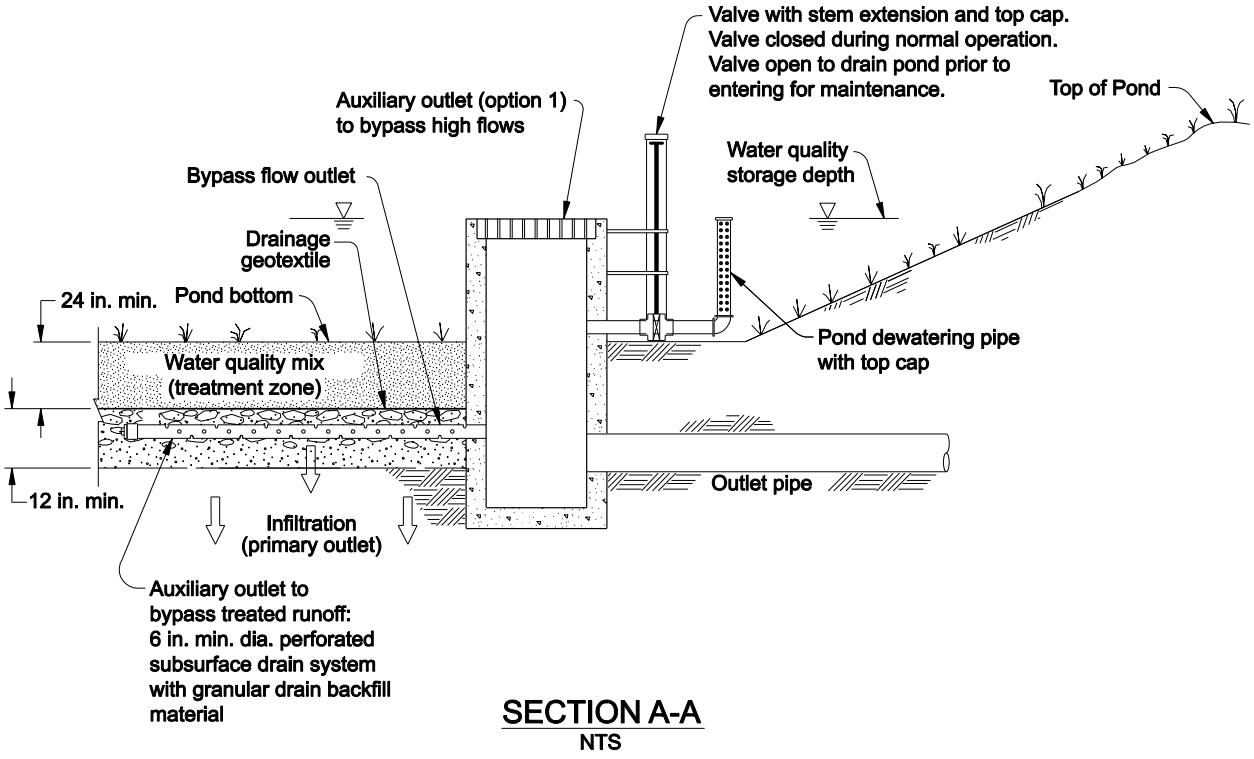



ELEVATION VIEW
NTS

**Figure 1 Bioretention Facility**




**(Cross-Section without a perforated subsurface drain system)**




**(Cross-Section using a perforated subsurface drain system)**

**Figure 2 Bioretention Facility Cross-Sections**

**Sub Surface Drain**

A sub surface drain pipe system is required to prevent standing water conditions when the subsoil infiltration rate is less than 0.5 inches/hour. Subsurface drains must meet the following criteria:

1. The sub surface drain pipe must be perforated pipe, laid in a grid pattern, and backfilled and bedded with granular drain backfill material

2. The sub surface drain pipe must be 6 inches or greater in diameter.

3. The sub surface drain pipe must drain freely to an acceptable discharge point.

**Safety Features**

1. Exclusionary measures may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences are recommended for basin areas where one or more of the following conditions exist:

   - areas where small children are present, particularly residential areas, and near playgrounds and schools.
   - areas where rapid water level increases would make escape difficult,
   - side slopes steeper than 1V:3H and have water depths greater than 3 feet for more than 24 hours or are permanently wet.

## 2.2 Design Procedure

The procedure for designing a bioretention pond is presented below.

**Step 1 –** Establish the location of the facility according to the site suitability requirements

**Step 1a – Determine any special requirements for groundwater protection areas and wellhead protection zones.**

**Step 2 –** Determine water quality design storm. See Section 14.10.2.

**Step 2a -** Determine the Contributing Impervious Area for the facility. See Section 14.10.1.

**Step 3 –** Determine the water quality volume ($V_{wq}$). Use hydrology guidance in Chapter 7 and the design recurrence interval from step 2.

**Step 4 –** Estimate the dimensions of the pond that will contain the required storage. The method of determining the dimensions of a basin is presented in subsection 12.8.2.

**Step 5 –** Coordinate with the project geologist to perform a subsurface soil characterization

analysis at the location(s) determined in Step 1 and estimate the Long Term Infiltration Rate of the site's soils. Determine:

- The soil gradation characteristics at the proposed location(s) of the facility. One grain-size analysis per soil stratum in each test hole must be conducted within 2.5 times the maximum design water depth, but not less than 6 feet.
- The stratification of the soil/rock below the facility
- The depth of the seasonally high water table and bedrock. Monitor groundwater during at least one wet season. **Note that at least 3 feet vertical separation is required between the bottom of the facility and annual high groundwater.**
- Determine the long term infiltration rate using one of the following options:

**Note: Design or long term infiltration rate of 9 inches per hour or less is acceptable at providing water quality treatment for groundwater protection. See Appendix E, Section 2 regarding organic matter content requirement. Place water quality mix over soils with a design or long term infiltration rate greater than 9 inches per hour. This criteria is applicable outside of groundwater protection areas. Determine any special requirements for groundwater protection areas and wellhead protection zones. This applies to option 1 or 2 below.**

## Option 1

A Pilot Infiltration Test (PIT). **This is required for facilities with a contributing area greater than 15,000 ft²**. Coordinate test with the project geologist. The infiltration rate determined from this test must be adjusted using a safety factor to obtain the long term infiltration rate. The long term infiltration rate is used to design the facility. See below for safety factor guidance.

**Table 1** Safety Factors for PIT Infiltration Rate Adjustment

| Consideration | Partial Correction Factor |
|---|---|
| Confidence in site variability and number of test locations | CF = 1.5 to 6 |
| Availability of maintenance to limit siltation and biofouling | CF = 2 to 6 |
| Degree of influent control available to limit siltation and biofouling | CF = 2 to 6 |

To calculate the safety factor, use the following equation.

Safety Factor = Σ Correction Factors

The following equation applies the safety factor to reduce the tested infiltration rate to the "adjusted" or long-term rate:

Design or long term Infiltration Rate = PIT field infiltration rate ÷ Safety Factor

Example:

PIT field infiltration rate = 10 inches per hour

Safety Factor Calculation, using engineering judgment:

- This site has low potential for bio-fouling as vegetation does not exist nearby to clutter with leaves and it is exposed to sun reducing moss buildup (CF = 2).
- The proposed facility has a vegetated swale to remove suspended solids before runoff enters infiltration facility (CF = 2).
- This site has low variability in soil types, groundwater depths and soil stratification. A safety CF of 1.5 is applied.

Safety Factor = Σ CF = 2 + 2 + 1.5 = 5.5

Calculate design or long term infiltration rate.

Design or long term Infiltration Rate = PIT field infiltration rate ÷ Safety Factor

10 inches/hour ÷ 5.5 = 1.8 in/hr (design or long term infiltration rate)

**OR**

**Option 2**

Use soil gradation results and Table 2 or Figure 3 to **determine the long term infiltration rate for facilities draining an impervious area of 15,000 square feet or less.** Do not use a reduction factor or factor of safety for the values noted in Table 2 or Figure 3.

Use Table 2 to estimate the long term infiltration rate unless the site has variable soil types and characteristics, low maintenance, high chance of biofouling, and no pretreatment. Biofouling is the accumulation of bacteria, algal, or other organics with the pore spaces of the soil matrix.

Table 2 was derived from a study conducted by Massmann and Butchart (2000) that compared infiltration measurements from infiltration facilities to soil gradation data using the ASTM D422 procedure. Massmann and Butchart used data from several counties (King, Clark, Thurston) in Washington (Wiltsie 1998).

| **$D_{10}$ Size from ASTM D422 Soil Gradation Test (mm)** | **Estimated Long-Term (Design) Infiltration Rate (in/hr)** |
|---|---|
| ≥0.4 | 9 |
| 0.3 | 6.5 |
| 0.2 | 3.5 |
| 0.1 | 2.0 |
| 0.05 | 0.8 |

**Table 2**

Estimated Infiltration Rates Based on ASTM Gradation Testing (Massmann and Butchart 2000)

Figure 3 can also be used for soils with a $D_{10}$ of less than 0.05 mm. It plots the relationship between the infiltration rates and the $D_{10}$ values noted in Table 2. Table 2 only provides estimated infiltration rates to a $D_{10}$ of 0.05 mm.

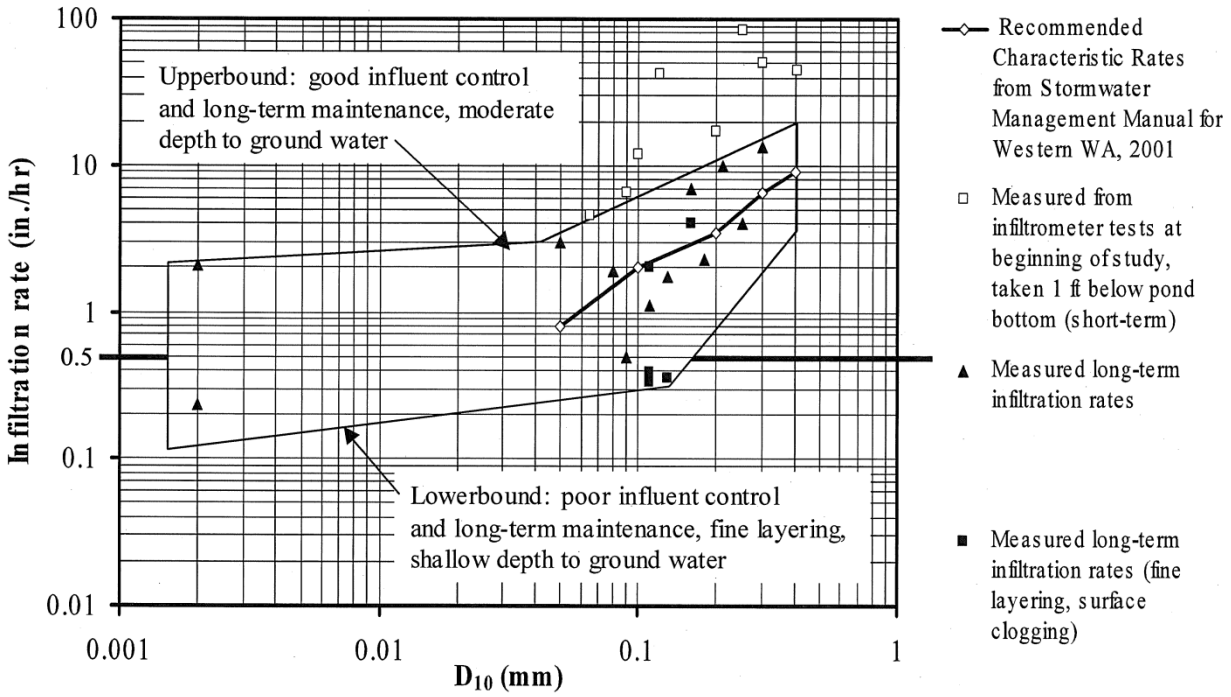



**Figure 3.** Infiltration Rate as a Function of the $D_{10}$ Soil Size (Massmann, et al. 2003)

Use the lowerbound infiltration rate associated to the $D_{10}$ of interest if the site has variable soil types and characteristics, low maintenance, high chance of biofouling, shallow groundwater and no pretreatment. The soil type with the lowest infiltration rate should be used to design the facility. Apply this criteria to a depth of 2.5 times the maximum design water depth, but not less than 6 feet.

The upperbound infiltration rates are not applicable for ODOT facilities.

**Step 6 –** Skip to step 10 if the infiltration rate is less than 0.5 inches per hour otherwise go to Step 7.

**Step 7 –** Determine the **infiltration flow rate Q** by developing a stage/discharge curve from the infiltration rate determined in Step 5. At a given stage the discharge may be computed using the pervious surface area through which infiltration will occur (which varies with stage) multiplied by the infiltration rate determined in Step 5 or the water

quality mix infiltration rate estimated at 3 inches per hour. Use the smaller infiltration rate.

**Step 8 –** Determine the amount of time it takes the facility to empty. The facility must infiltrate the water quality design storm volume within 36 hours.

T = The time required to infiltrate the water quality volume ($V_{wq}$)

$$T = \frac{V_{WQ}}{Q}$$

Where:

$V_{wq}$ = Water quality volume in cubic feet
$Q$ = Infiltration flow rate in cubic feet per second

**Step 9 –** Adjust pond cross-section to include freeboard.

**Step 10 –** Design a subsurface drain when the infiltration rate is less than 0.5 inches per hour. See subsurface drain section (Section 2.1).

**Step 11 –** Design the following facility components using the guidelines included in Appendix E:

- Pretreatment (sediment basin, pollution control manhole, or proprietary structure)
- Flow splitter manhole
- Auxiliary outlet
- Storm drain piping
- Outfall
- Energy dissipation at the outfall
- Coordinate soil preparation, seed mix, irrigation needs, and other requirements with the project roadside development designer
- Coordinate temporary and/or permanent erosion control measures with the project erosion control designer

**Step 12 –** Note in the Operation and Maintenance Manual that heavy equipment is not allowed within the facility.

**Step 13–** Prepare the Stormwater Design Report and Operations and Maintenance Manual as discussed in Section 14.10.15 and 14.11.

**Step 14 –** Coordinate the installation field markers at the start and end of a facility's maintenance area. Marking guidance is provided in Chapter 17.

## 3.0 Bioslope

This section provides information for the design of bioslopes. Figures 4, 5, and 6 are general configurations of a bioslope. Other names for bioslope that have been used include ecology embankment and media filter drain.

Bioslopes are flow-through stormwater treatment facilities incorporated into roadside embankments and placed between pavement and a downstream conveyance system. These facilities utilize physical straining or filtration, sorption, carbonate precipitation, vegetative uptake and microbial degradation to provide stormwater treatment. Bioslopes are recommended for highway application because of their minimal right-of-way requirements and maintenance schedule.

Bioslopes are designed to treat sheet flow from an adjacent impervious surface. A typical bioslope has the following facility features and components:

- **Vegetated filter strip** – It is provided upstream of the bioslope to evenly distribute flow into the treatment zone, reduce the runoff velocity, and provide pretreatment.
- **Treatment Zone using Ecology mix** – It is provided to remove pollutants as stormwater runoff drains through this zone. The ecology mix is a mixture of aggregate, dolomite, gypsum, and perlite.
- **Sub surface drain** – it is provided to allow positive outflow for runoff at the toe of the bioslope.

Bioslopes can be placed:

- Along existing roadway embankments, and
- Urban and suburban highway with adequate right-of-way
- Rural areas

Bioslopes would not be effective and should not be considered when:

- Sheet flow can not be maintained
- Steep slopes are proposed (side slopes will be greater than 4:1)
- Unstable slopes are present,
- Shallow groundwater is present

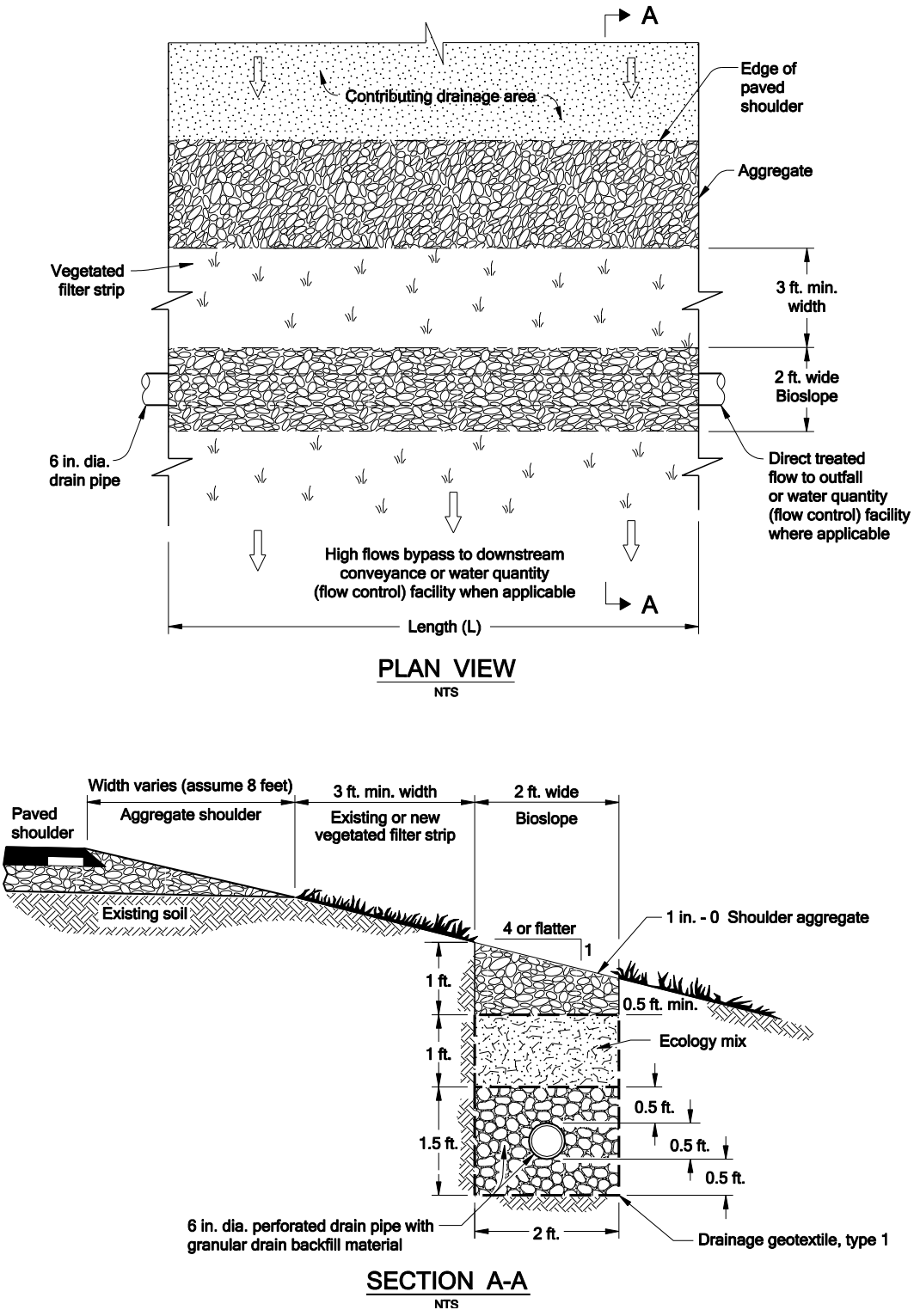
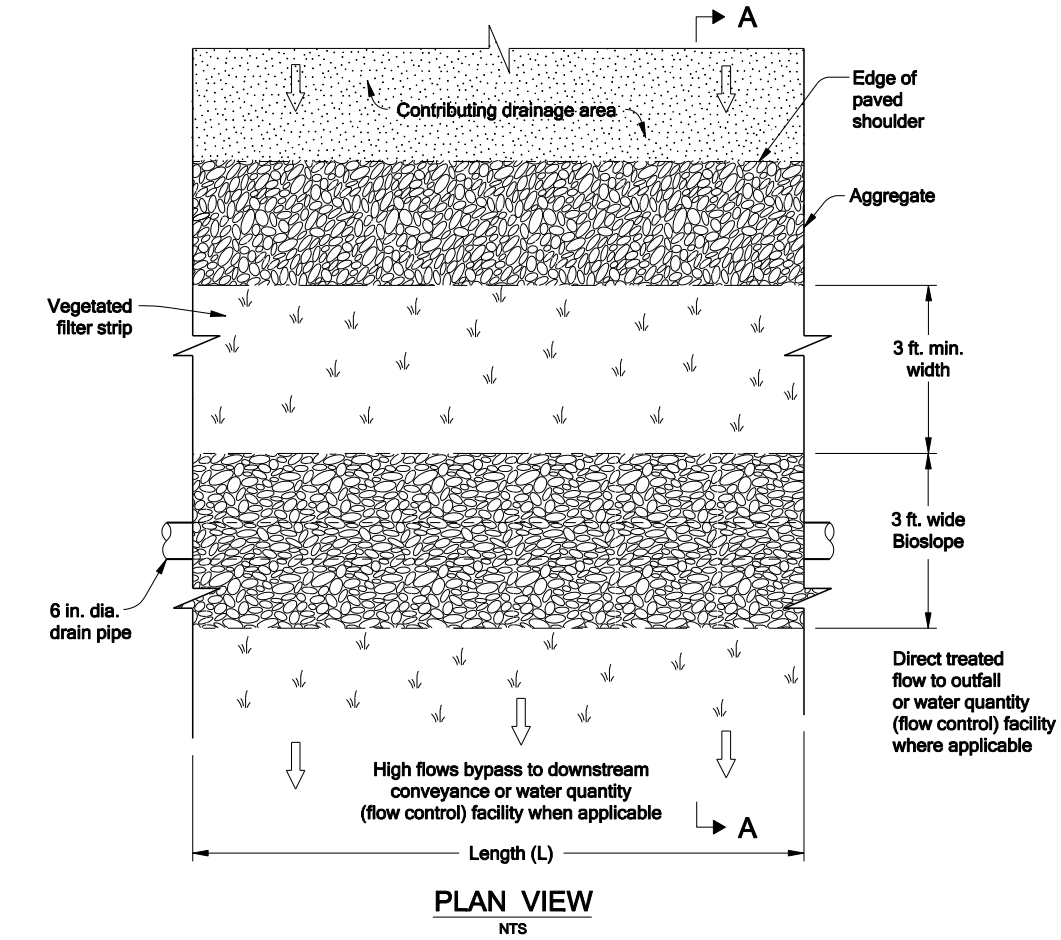



**Figure 4 Bioslope (2 foot wide)**



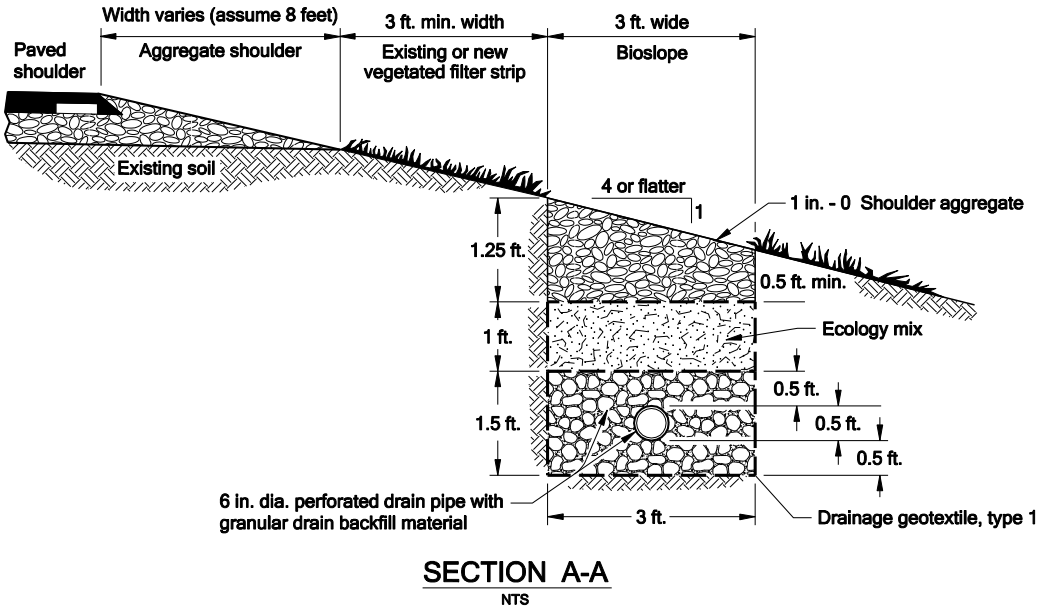




**Figure 5 Bioslope (3 foot wide)**

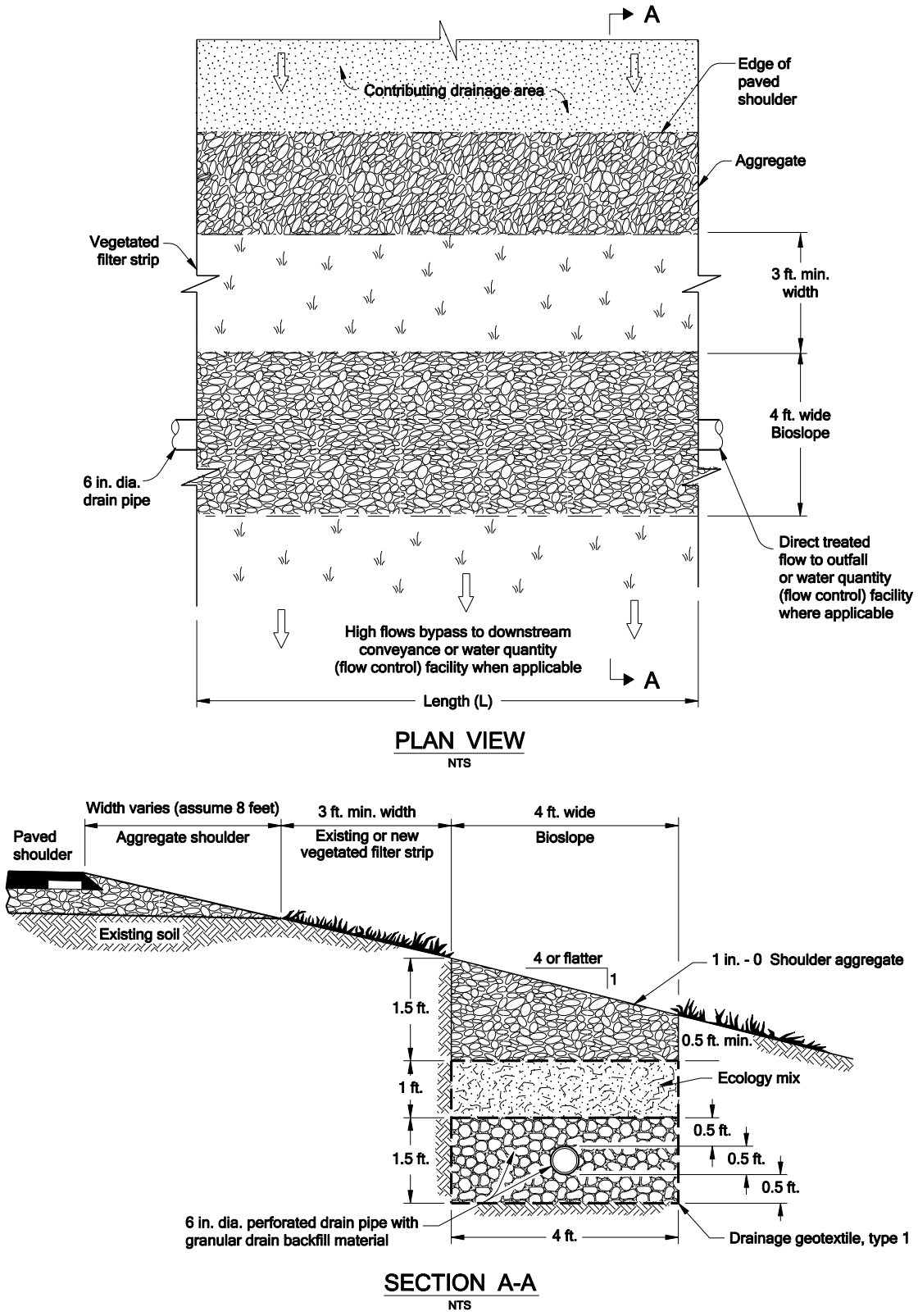



**Figure 6 Bioslope (4 foot wide)**

## 3.1 Design Criteria

This section describes the features of bioslopes and the design criteria that apply specifically to these installations. Also apply the general requirements discussed in Section 14.10.

**Site Selection**

1. General siting requirements are discussed in Section 14.9. Additional siting criteria that apply specifically to bioslopes include:

    a) Side slopes are 4:1 or less
    b) Side slopes are geotechnically stable
    c) Seasonal high groundwater table is 3 feet or more below the bottom of the bioslope granual drain backfill

**Bioslope Geometry**

1. The **minimum width** of the bioslope ecology mix bed is equal to or greater than 2 feet.

2. The **lateral slope of the roadside shoulder** along the placement of the bioslope should not be steeper that 1V:4H. Less than 1V:4H lateral slope is preferred.

3. The **longitudinal slope of the roadside shoulder** along the placement of the bioslope should not be steeper than 5 percent.

4. The **length** of the bioslope must be equal to the length of the contributing impervious or pavement area.

**Vegetated Filter Strip**

1. The **minimum width** of the grass filter strip is 3 feet. The grass filter strip must be located between the contributing impervious area and the bioslope. The vegetated filter strip will assist in maintaining uniform flow across the bioslope section and provide pretreatment.

**Sub Surface Drain**

A typical sub surface drain pipe system is shown in Figure 4. A sub surface drain pipe system is required to allow positive outflow for runoff at the toe of the bioslope. A sub surface drain may not be needed for Natural Resources Conservation Service (NRCS) Hydrologic soil groups A and B. A sub surface drain should be installed when the subsoil classification is C or D.

A subsurfance drain must meet the following criteria:

1. The sub surface drain pipe must be a perforated pipe, laid parallel to the bioslope length, and backfilled and bedded as shown in Figure 4.
2. The sub surface drain pipe diameter must be 6 inches or greater.
3. The granular drain backfill material must be wrapped with drainage geotextile.
4. The sub surface drain pipe must infiltrate or drain freely to an acceptable discharge point.
5. The minimum sub surface drain trench width is 2 feet.

**Ecology Mix**

Ecology mix is provided to remove pollutants as stormwater runoff drains through this media mix. General design criteria includes:

1. The ecology mix has an estimated long term filtration rate of 20 inches per hour due to siltation. Therefore, use a safety factor of two (2) or an infiltration rate of 10 inches per hour.

2. The long term infiltration capacity ($Q_{infiltration}$) must be greater than the water quality peak flow

3. The minimum mix thickness is 12 inches

The ecology mix consists of the following materials:

| Amendment | Description | Quantity |
|---|---|---|
| **Aggregates** | 3/8" – No. 8 aggregate meeting the requirements of ODOT Standard Specifications Section 00680 | 3 cubic yards (c.y.) |
| **Perlite** | • Horticultural grade, free of any toxic materials. (Course Gradation)<br>• 0 – 29% passing No. 18 sieve<br>• 0 – 10% passing No. 30 sieve | 1 cubic yard per 3 cubic yards of aggregate |
| **Dolomite** | • Calcium magnesium carbonate – $CaMg(CO_3)_2$ (Ground)<br>• Agricultural grade, free of any toxic materials.<br>• 95 - 100% passing No. 8 sieve<br>• 0 -5% passing No. 16 sieve | 10 pounds per 1 c.y. of perlite |
| **Gypsum** | • Noncalcined, agricultural gypsum – $CaSO_4$-2($H_2O$) (hydrated calcium sulfate). (Ground or Peletized)<br>• Agricultural grade, free of any toxic materials<br>• 95 - 100% passing No. 8 sieve<br>• 0 – 5% passing No. 16 sieve | 1.5 pounds per 1 c.y. of perlite |

Aggregate

perlite

Dolomite

gypsum

**Figure 6 Materials blended to produce ecology mix**

## 3.2 Design Procedure

The procedure for designing a bioslope is presented below.

**Step 1** – Determine water quality design storm. Highway runoff from impervious areas needs to address the most stringent standards or reference requirements summarized in Section 14.10.2.

**Step 2 -** Determine Contributing Impervious Area. See Section 14.10.1

**Step 3** – Determine the water quality peak flow. Use hydrology guidance in Chapter 7 and the design recurrence interval from step 1.

**Step 4**– Calculate the long term infiltration capacity of the bioslope ($Q_{infiltration}$). Assume an initial bioslope ecology mix bed width and use a bioslope ecology mix bed length equal to the length of the contributing pavement area.

$Q_{infiltration}$ = long term infiltration capacity (cubic feet per second)

$$Q_{infiltration} = \frac{(LTIR_{EM})(L_{BIO})(W_{BIO})}{(C)(SF)}$$

Where:

| | | |
|---|---|---|
| $LTIR_{EM}$ | = | Long term infiltration rate of ecology mix (use 10 inches per hour) |
| $L_{BIO}$ | = | Length of the bioslope ecology mix bed, parallel to roadway (feet) |
| $W_{BIO}$ | = | Width of the bioslope ecology mix bed (feet) |
| C | = | Conversion factor of 43200 [(in/hr)/(ft/sec)] |
| SF | = | Safety Factor (equal to 1.0, unless unusually heavy sediment loading is anticipated) |

*Note: The long term infiltration capacity must be greater than the water quality peak flow. Increase width of bioslope ecology mix bed until the above condition is satisfied. The minimum media filter strip ecology mix bed is 2 feet.*

**Step 5 –** Design or coordinate the following facility components using the guidelines included in Appendix E:

- Storm drain piping
- Outfall
- Energy dissipation
- Coordinate soil preparation, seed mix, planting requirements, irrigation needs, and other requirements with the project roadside development designer.
- Coordinate temporary and/or permanent erosion control measures with the project erosion control designer

**Step 6–** Prepare the Stormwater Design Report and Operations and Maintenance Manual as discussed in Section 14.10.15 and 14.11.

**Step 7 –** Coordinate the installation field markers at the start and end of a facility's maintenance area. Marking guidance is provided in Chapter 17.

## 4. **Proprietary Media Filter Systems**

Several media filter treatment systems developed by private firms are on ODOT's Qualified Products List and are approved for use on ODOT facilities. They are generally have a relatively small footprint, and are best suited for urban situations with limited space available for stormwater BMPs. Proprietary media filter BMPs do not support infiltration into the soil, but rather discharge treated stormwater into the main storm sewer system.

The cost of purchasing and maintaining a proprietary media filter system may be higher than for the generic media filter systems discussed earlier. On the other hand, the extra expense may be counterbalanced by reduced right of way requirements and the ability to provide required levels of treatment at all.

It is vital that designers discuss maintenance requirements with Maintenance before selecting a particular system. Of particular importance is making sure that the responsible Maintenance district has the equipment required to maintain the facility.

Design criteria for proprietary units will be specific to the particular system. Designers should follow the manufacturer's guidance.

If a proprietary system does not include a pre-treatment element, the designer should place a pre-treatment BMP upstream if at all possible.

# APPENDIX D
# DENSITY SEPARATION FACILITIES

## 1.0 Introduction

This appendix provides information for the design of density separation facilities such as the extended detention dry pond.

## 2.0 Extended Detention Dry Pond

An extended detention dry pond (Figure 1) is a basin that has been designed to detain stormwater for a sufficient time to allow particles and attached pollutants to settle. Infiltration is incidental, and is not taken into account in the pond's design Extended detention dry ponds do not have a large permanent pool of water; the entire amount of water that enters the pond is released over a period of time.

Extended detention dry ponds effectively remove sediment and sediment bound pollutants, but not dissolved pollutants. They should not be used as stand-alone BMPs in urban areas or on highways with more than very low traffic volumes. Dry ponds are useful as pre-treatment for biofiltration and media filter BMPs.

A typical extended detention dry pond has the following facility features and components:

- **Storage** –The provided pond volume necessary to temporarily store the water quality volume.
- **Outlet control structure** – The outlet designed for a controlled release rate so the water quality design storm volume is detained for a specific time to allow for settling of pollutants, and to bypass high flows. The recommended primary outlet structure for these types of ponds is shown in Figures 2 and 3.
- **Auxiliary Outlet** – It is provided as a safety factor in the event the primary outlet control structure can not safely pass extreme high flows.

**Figure 1 Extended detention dry pond**

Figure 2 and 3 are general configurations of an extended detention dry pond. The actual configuration could vary depending on site constraints and applicable design criteria.

Extended detention dry ponds can be placed along:

- medians,
- interchanges,
- adjacent to ramps,
- and adjacent to roadways
- Away form the highway

## 2.1 Design Criteria

This section describes the features of extended detention dry ponds and the design criteria that apply specifically to these installations. Also apply the general requirements discussed in Section 14.10.

### 2.1.1 Site Selection

**Area Requirements**

1. The site must be of sufficient size to accommodate the pond and also to provide adequate setback distances, include. The proper setback distances are important to ensure slope stability, and maintenance access. General siting requirements are discussed in Section 14.9.

**Groundwater**

1. Maintain a minimum distance of 3 feet from the bottom or invert of a facility to bedrock or seasonally high water table.

**Soil Suitability**

1. Extended detention dry ponds are applicable in NRCS hydrologic soil groups B, C, and D.

**Pretreatment**

1. A pretreatment facility component is required to be installed upstream of the proposed pond. Design a pretreatment facility component according to guidance provided in Appendix E.

**Pond Geometry**

1. The **pond bottom area** must be sloped toward the outlet to prevent standing water conditions. A minimum bottom grade of 1 percent is recommended.

2. The pond must have adequate **depth** to provide the needed storage. Other considerations when setting depths include public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, shading, maintenance requirements, and freeboard. Aesthetically pleasing features are also important in urbanized areas.

3. The **minimum bottom width** is 10 feet to provide the needed storage and allow for maintenance.

4. The **flow path** between the pond inlet and outlet must be maximized to ensure sufficient time to allow for sedimentation of pollutants. A pond length-to-width ratio of 3:1 or greater is recommended.

5. The pond **storage volume** is designed to temporarily store the water quality volume. This is the volume between the bottom of the pond up to the start of the freeboard volume. Freeboard volume is in addition to the storage volume needed for treatment.

   The water quality volume is the predicted volume of runoff for the proposed conditions using the appropriate water quality design storm.

6. **Interior side slopes** should not be steeper than 1V:4H.

7. **Pond walls** may be retaining walls designed in accordance with the ODOT Geotechnical Design Manual. A fence is typically provided along the top of the wall.

8. The **freeboard** criteria of 1 foot minimum is required between the water quality design storm water elevation to the top of embankment. Freeboard is the vertical distance between the design water surface and the top of embankment, as shown in Figures 4.

**Bottom Marker**

1. A **bottom marker** made of porous pavers must be installed along the pond bottom to indicate the bottom elevation. Select a porous paver from the Qualified Products List. The porous paver must provide a minimum 80 percent bottom area opening for grass growth. Spaced solid paver blocks are not allowed. Pavers can also function as the access grid to support maintenance equipment. See maintenance access section below.

2. Note the following in the facility's operation and maintenance manual:

   - the use of porous pavers to mark the bottom elevation of the pond
   - use sediment removal techniques that will not damage porous pavers

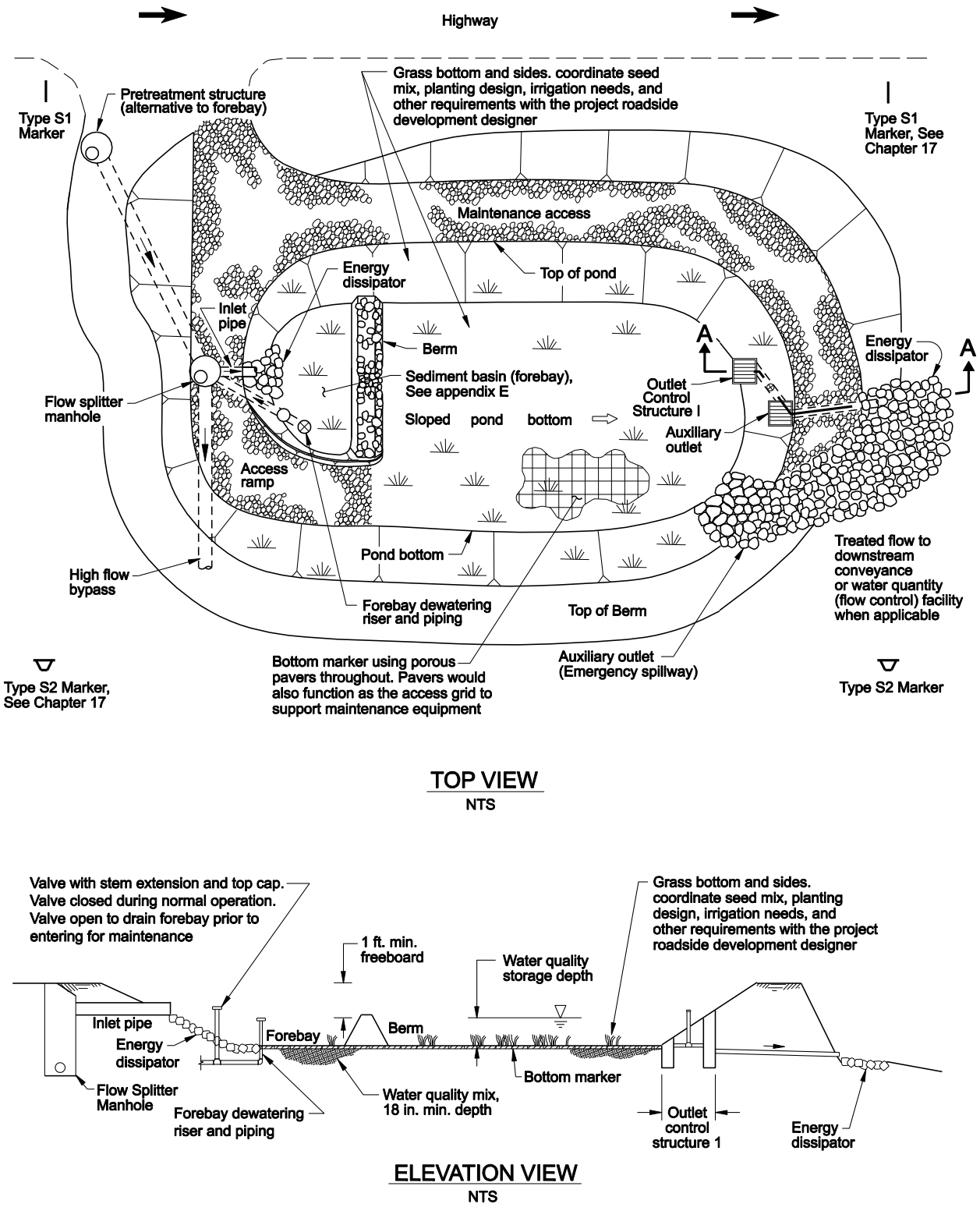



**Figure 2 Extended Detention Dry Pond**

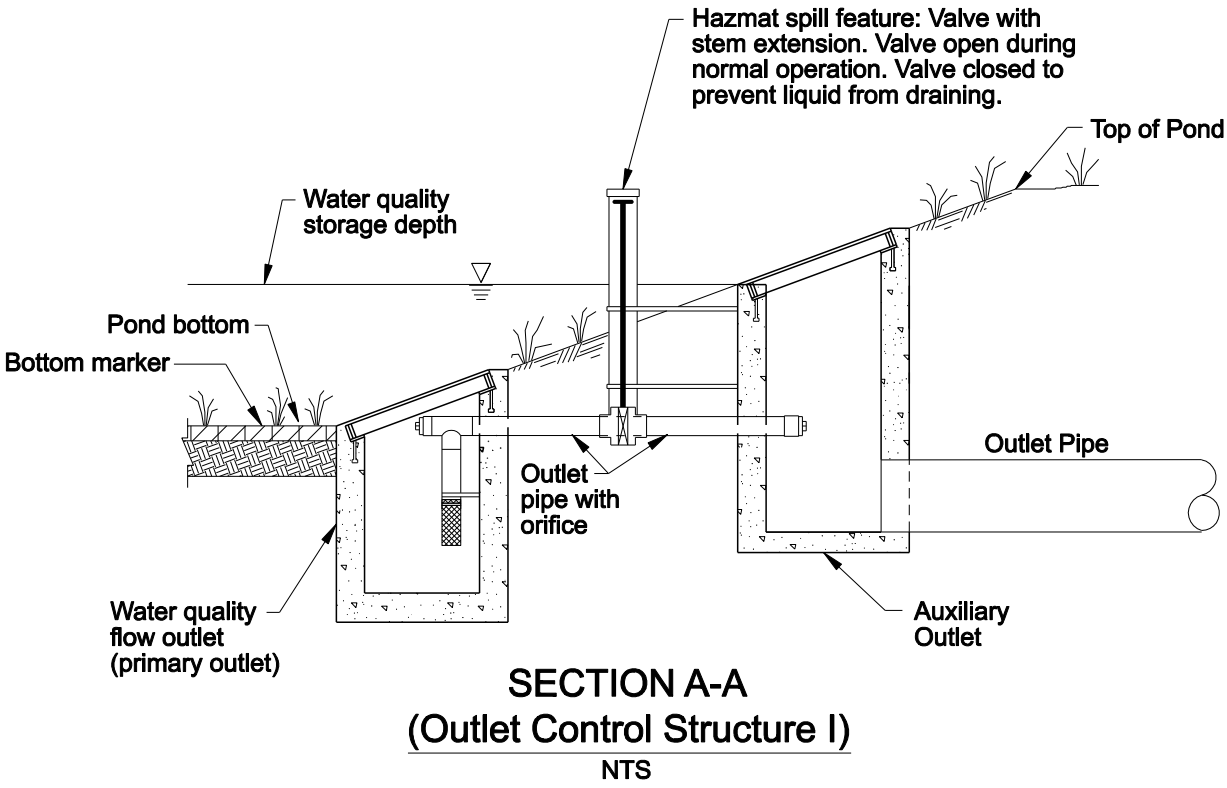



**Figure 3 Extended Detention Dry Pond Outlet Control Structure**

## Embankments

Pond berm embankments are often needed for dry ponds to obtain sufficient storage volume. Pond berm embankments must meet the following criteria:

1. Vegetated pond berm embankments must be less than 20 feet in height and have exterior side slopes no steeper than 1V:3H and interior side slopes no steeper than 1V:4H.

2. Pond berm embankments higher that 6 feet shall be **designed by a geotechnical enginee**r.

3. The **minimum top width** shall be 6 feet for pond berms 6 feet or less, or as recommended by a geotechnical engineer for higher berms.

**Maintenance Access**

1. An **access road** shall be provided to the primary and auxiliary outlet control structures. The proposed access road must be able to support heavy equipment such as a vactor truck, dump truck, track how, or large mower.

2. Access road must be 16 feet in **width**.

3. The access road **maximum longitudinal slope** must be:

   a) 2 percent (edge of pavement to a longitudinal distance of 20 feet)
   b) 10 percent (20 feet from edge of pavement to end of access road)

3. An **access grid** made of porous pavers must be installed along the pond bottom for maintenance vehicle and mowing equipment access. Select a porous paver from the Qualified Products List. The porous paver must provide a minimum 80 percent bottom area opening for grass growth. Spaced solid paver blocks are not allowed. Pavers would also function as the bottom marker.

4. The access road **maximum cross slope** is 4 percent.

5. An access ramp is required for mowing, repairs, and sediment removal. The ramp must extend to the pond bottom.

6. **Maximum grade** of an access road or ramp shall be 10 percent

7. **Manhole lids** located in non-traffic areas such as grassed areas or behind guardrail must be set 1 foot above finish ground so that manhole location is visible for locating and for maintenance. This should be coordinated with the maintenance districts, lids may be placed flush with the finished grade at the request of the serving maintenance district. Lid elevations must match proposed finish grade in traffic areas.

**Treatment Detention Time**

1. The **minimum required treatment detention time** is 48 hours. The detention time is the time duration to release the entire water quality volume.

**Safety Features**

1. Exclusionary measures may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences are recommended for detention areas where one or more of the following conditions exist:

- areas where small children are present, particularly in residential areas and close to schools and playgrounds.

- areas where rapid water level increases would make escape practically impossible,
- side slopes steeper than 1V:3H and have water depths greater than 3 feet for more than 24 hours or are permanently wet and have side slopes steeper than 1V:3H, or

2. Guardrail may be needed if the extended dry detention pond is placed next to a highway

**Water Quality Mix**

There are three design options to establish a "Water Quality Mix" that meets criteria for organic content, long term hydraulic conductivity and other soil characteristics. See Appendix E.

**Planting Requirements**

1. Grass shall be established along the sides and bottom of pond, unless the local climate is unsuitable. Where grass is not appropriate, side slopes should be protected against erosion by other means. Note that urban jurisdictions may require plants and shrubs instead of grass.

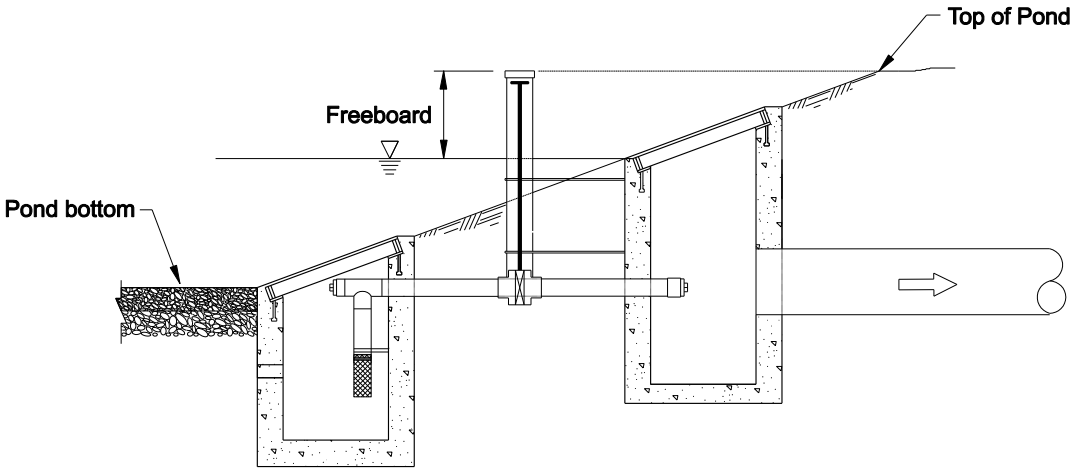
2. Permanent seeding is best performed as follows:

   - West of the Cascades – March 1 through May 15 and September 1 through October 31 if grass areas are watered regularly during the establishment period.
   - East of the Cascades – October 1 through February 1 or March 1 through October 1 if grass areas are water regularly during the establishment period.

**Field Markers**

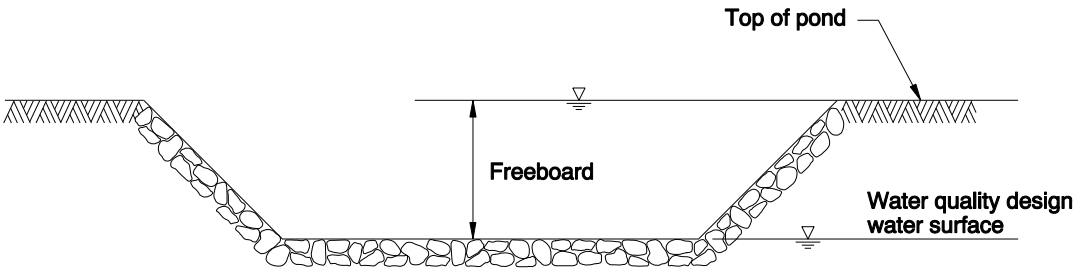
1. Field Markers are required to be installed at the start and end of a facility's maintenance area. Marking guidance is provided in **Chapter 17**.

The figure below illustrates freeboard when the two-type D inlet structure is used:




OUTLET CONTROL STRUCTURE I
NTS

The figure below illustrates freeboard when an emergency overflow is used:




EMERGENCY SPILLWAY
NTS

**Figure 4 Freeboard**

## 2.2 Design Procedure

The procedure for designing an extended detention dry pond is presented below.

**Step 1 –** Determine water quality design storm. Highway runoff from impervious areas needs to address the most stringent standards or reference ODOT's requirements summarized in Section 14.10.2.

**Step 2 -** Determine contributing impervious area. See Section 14.10.1

**Step 2 –** Determine the water quality volume ($V_{wq}$). Use hydrology guidance in **Chapter 7** and the design recurrence interval from step 1.

**Step 3 –** Determine the dimensions of the pond that will contain the required storage. The method of determining the dimensions of a basin is presented in subsection 12.8.2.

**Step 4 –** Adjust pond cross-section to include freeboard.

**Step 5 –** Design the following facility components using the guidelines included in Appendix E:

- Pretreatment (sediment basin, pollution control manhole, or proprietary structure)
- Water quality mix
- Flow splitter manhole
- Primary outlet structure
- Auxiliary outlet when applicable
- Storm drain piping
- Outfall
- Energy dissipation
- Coordinate soil preparation, seed mix, irrigation needs, and other requirements with the project roadside development designer
- Coordinate temporary and/or permanent erosion control measures with the project erosion control designer

**Step 6–** Prepare the Stormwater Design Report and Operations and Maintenance Manual as discussed in Section 14.10.15 and 14.11.

**Step 7 –** Coordinate the installation field markers at the start and end of a facility's maintenance area. Marking guidance is provided in **Chapter 17**.

# APPENDIX E
# STORMWATER TREATMENT FACILITY COMPONENTS

## 1.0 Introduction

This appendix provides information for the design of stormwater treatment facility components.

Facility components are the features that when combined with the treatment facility design assist with treatment, conveyance, erosion countermeasures, and energy dissipation. Facility components mentioned in this appendix include:

- Water Quality Mix – Section 2
- Inlet Structures – Section 3
- Pretreatment Facilities or Structures – Section 4
- Flow Spreading Options – Section 5
- Outlet Control Structures – Section 6
- Auxiliary Outlets – Section 7
- Storm Drain Piping – Section 8
- Outfalls – Section 9

Facility components are designed after a BMP is selected and sized from guidance provided in this chapter or other method approved through the hydraulic design deviation process.

## 2.0 Water Quality Mix

The goal is to create a soil "Water Quality Mix", that meets criteria for organic content, infiltration rate (long-term hydraulic conductivity), and other soil characteristics described below, by combining soil amendments (such as compost) along with the appropriate in-situ, stockpiled or imported soil.

Soil amendments include inorganic materials and organic matter that helps restore the fertility and condition of the soil, supports infiltration and improves pollutant removal capabilities. Soil organic matter can be increased by adding materials such as compost, composted woody material, biosolids, and forest product residuals. Soil amendment is used to enhance the efficiency of filter strips (dispersion areas), biofiltration swales, bioretention facilities, and extended dry detention facilities. In those facilities, it assists in the retention of stormwater to allow for infiltration down into the underlying soil, as well as providing treatment before the stormwater enters the groundwater or conveyance system.

## Applications

- Can be incorporated into native or imported soils to improve infiltration, retention capacity and pollutant removal capability

- Can be used in most locations where highway runoff will flow over or be detained on soil and vegetated ground.

- Is a standard component in new facilities such as swales, filter strips, detention and retention ponds, and roadside ditches to enhance pollutant removal and promote infiltration.

- Can be installed in existing facilities such as swales, filter strips, detention and retention ponds, and roadside ditches to enhance pollutant removal and promote infiltration.

## Mixing or Blending Methods

There are three methods to achieve a water quality mix:

1. on-site soils are suitable
2. on-site soils can be amended in-place, or
3. special materials must be imported

More than one method can be used on different portions of the same site. The method selected must be based upon laboratory testing and analysis of the existing soil, imported soil, and organic soil amendment.

**Method 1: Soil Amendment is not necessary if the surface native soil meets the following properties. This guidance is for providing water quality treatment outside of groundwater protection areas. Determine any special requirements for groundwater protection areas and wellhead protection zones.**

**Applicable for filter strips, planting beds, dispersion areas, and swales:**

- Organic matter content (dry weight) – 8% to 10% for planting beds and landscaped areas; 5% or greater for filter strips, dispersion areas, swales, or area supporting foot traffic during wet months (per ASTM Designation D 2974).
- The existing soil meets the gradation requirements in Method 3.

**Applicable for cells and ponds:**

- Organic matter content (dry weight) – 5% or greater for cells and ponds (per ASTM Designation D 2974).

- The design or long term infiltration rate of 9 inches per hour or less outside of groundwater protection areas or the soil is logged as one of the classes from the USDA Textural Triangle provided below excluding sand, loamy sand, and clay.

Soil texture classes other than sand and loamy sand may be assumed to have an infiltration rate of 9 inches per hour or less. Place a layer of water quality mix over soils with a design or long term infiltration rate greater than 9 inches per hour. See Method 3.

The minimum thickness of the qualifying soil to meet Method 1 requirements for the type of facility is:

**A. Filter Strips**

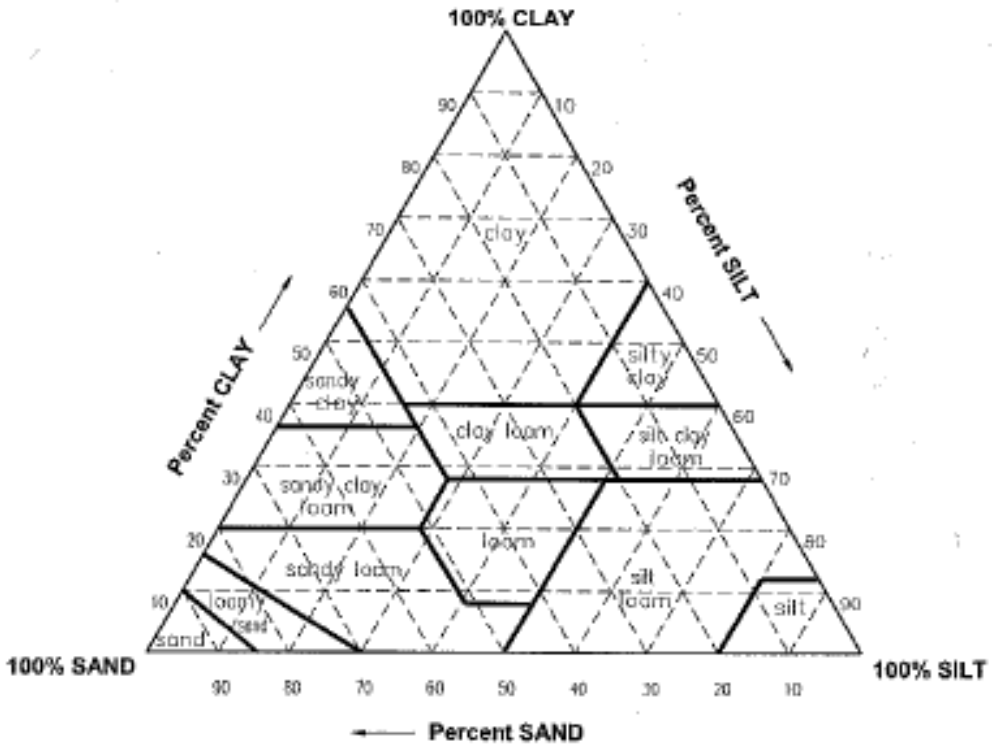
- 8 inches

**B. Planting Beds, dispersion areas, and swales**

- 18 inches

**C. Cells and ponds**

- 18 inches or 24 inches (reference applicable design guidelines)




**USDA Textural Triangle**

**Method 2: Soil amendement <u>is</u> necessary when the existing soil meets the criteria noted in Method 1 but the organic matter content does not meet the following requirement:**

- Organic matter content (dry weight) – 8% to 10% for planting beds and landscaped areas; 5% or greater for filter strips, dispersion areas, swales, cells, ponds or area supporting foot traffic during wet months (per ASTM Designation D 2974).

Increase the organic matter content by amending the soil as noted below. Scarify or till to a 12 inch depth. Entire surface should be disturbed by scarification, except do not scarify within drip line of existing trees that will be retained. **Coordinate this work with the project roadside Development designer.**

**A. Filter Strips**

- DEFAULT RATE: Place 2-inches of composted material (see water quality mix section below) and roto-till into 6 inches of soil
- CALCULATED RATE: Place calculated amount of composted material or approved organic material and roto-till into depth of soil needed to achieve 8 inches of settled soil at 5% or greater organic content.

Use a manually operated landscape water filled roller to compact soil. Rake to level, and remove surface woody debris and rocks larger than 1 inch diameter. In arid climates a cover of larger rocks is acceptable if needed to prevent erosion of the amended soil.

**B. Planting Beds**

- DEFAULT RATE: Place 6-inches of composted material and roto-till into 12 inches of soil (total 18 inches of amended soil)
- CALCULATED RATE: Place calculated amount of composted material or approved organic material and roto-till into depth of soil needed to achieve 18 inches of settled soil at 8-10 percent organic content.

Rake beds to smooth and remove surface rocks larger than 2 inches diameter. Mulch planting beds with 2 inches of organic mulch.

**C. Dispersion Areas, Swales, Cells, and Ponds**

- DEFAULT RATE: Place 4-inches of composted material and rototill into 14 inches of soil (total 18 inches of amended soil.) for dispersion areas and swales. Place 5-inches of compost material and rototill into 19 inches of soil (total 24 inches of amended soil.) for cells and ponds.

- CALCULATED RATE: Place calculated amount of composted material or approved organic material and rototill into depth of soil needed to achieve 18 or 24 inches of settled soil at 5% or greater organic content.

Use a manually operated landscape water filled roller to compact soil. Rake to level, and remove surface woody debris and rocks larger than 1 inch diameter. In arid climates a cover of larger rocks is acceptable if needed to prevent erosion of the amended soil.

**Method 3: Existing soil does not meet the requirements noted in Method 1. Place or excavate and replace with the water quality mix appropriate for the specific application.**

Scarify or till to a 12-inch depth. Entire surface should be disturbed by scarification, except do not scarify within drip line of existing trees to be retained.

**A. Filter Strips**

- Use imported or stockpiled Mix containing 20%-25% compost with 75%-80% water quality soil media.
- Place 8 inches of Mix.

Use a manually operated landscape water filled roller to compact soil. Rake to level, and remove surface rocks larger than 1-inch diameter. In arid climates a cover of larger rocks is acceptable if needed to prevent erosion of the amended soil.

**B. Planting Beds**

- Use imported or stockpiled Mix containing 30%-35% compost with 65%-70% water quality soil media.
- Place 18 inches of Mix.

Rake beds to level, and remove surface rocks and woody debris over 1 inch diameter. Mulch planting beds with 2 inches of organic mulch.

**C. Dispersion Areas, Swales, Cells, and Ponds**

- Use imported or stockpiled Mix containing 20%-25% compost with 75%-80% water quality soil media.
- Place 24 inches of Mix in ponds or cells otherwise use 18 inches.

Use a manually operated landscape water filled roller to compact soil. Rake to level, and remove surface rocks larger than 1-inch diameter. In arid climates a cover of larger rocks is acceptable if needed to prevent erosion of the amended soil.

## Water Quality Mix

### Soil Properties

1. The existing, stockpiled or imported soil must meet the following gradation requirements:

| Sieve Size | Percent Passing (By Weight) |
|---|---|
| No. 4 | 100 |
| No. 10 | 95 – 100 |
| No. 40 | 40 – 60 |
| No. 100 | 10 – 25 |
| No. 200 | 5 – 10 |

Water Quality Soil Media

- Sampling must meet the requirements of AASHTO T2.
- Sieve analysis must meet the requirements of AASHTO T27 and AASHTO T11.
- Soil pH – 5.5 to 8.0

### Compost Properties

1. The organic content shall be met using medium type compost according to special provision 3020. Special provisions can be viewed using the following link:

   http://www.oregon.gov/ODOT/HWY/SPECS/special_provisions.shtml

### Mix or Blended Properties

1. Mix shall be uniform and free of stones, stumps, roots or other similar material larger than 1 inch.
2. Mix medium type compost with soil media using the default or calculated rate:
   - Default Rate:
     - For Filter Strips: 20%-25% compost with 75%-80% water quality soil media or 2-inches of compost for 6-inches water quality soil media
     - For Planting Beds: 30%-35% compost with 65%-70% water quality soil media or minimum of 6-inches of compost added to every 12-inches of water quality soil media

- For Dispersion Areas and Swales: 20%-25% compost with 75%-80% water quality soil media or minimum of 4-inches of compost for 14-inches water quality soil media
- For Cells and Ponds: 20%-25% compost with 75%-80% water quality soil media or minimum of 5-inches of compost for 19-inches water quality soil media

- Calculated Rate:
  - Use the following equation to calculate compost application rates to achieve a target final soil organic matter content (FOM, which should be 10% for landscape beds or 5% for turf areas) for a soil with a given bulk density (SBD) and initial soil organic matter (SOM).

$$CR = D \times [SBD \times (SOM\% - FOM\%)] / [SBD \times (SOM\% - FOM\%) – CBD \times (CBD \times (COM\% - FOM\%)]$$

Where:

- *CR = compost application rate (inches) calculated to achieve the target final organic matter (FOM)*
- *D = Depth of finished incorporation (inches)*
- *SBD = Soil bulk density (lb/cubic yard dry weight)**
- *SOM % = Initial soil organic matter(%)****
- *FOM% = Final target soil organic matter(%)****
- *CBD = Compost bulk density (lb/cubic yard dry weight)***
- *COM% = Compost organic matter (%)****

*Assumption: This equation calculates compost rate using an additive approach. For example, a 3-inch compost rate incorporated to an 8-inch depth will be a final mix containing 3/8 compost and 5/8 soil by volume.*
** SBD To convert Soil Bulk Density in g/cm3 units to lb/cubic yard, multiply by 1697.*
*** CBD To convert Compost Bulk Density from lb/cubic yard "as is" to lb/cubic yard dry weight, multiply by solids content.*
**** All Organic Matter measurements are based on the commonly used "loss-on-combustion" method*

## 3.0 Inlet Structures

An inlet structure is needed to direct flow into the stormwater treatment facility. They either discharge all collected runoff into the facility (used for in-line or combined BMPs), or only up to the design flow (off-line BMPs). The latter type is called "flow splitter".

## 3.1 Flow Splitters

In most cases the inlet structure for off-line BMPs consists of a manhole, a flow splitter and a high flow bypass feature. Its primary functions are:

- To only allow runoff flows up to the water quality design flow to enter the water quality facility.
- To bypass or re-direct the flows greater than the water quality design flow around the stormwater treatment facility.

The following design guidance and criteria are for the two most common types of flow splitters used in stormwater treatment facility design. Other flow splitting methods may be proposed but must perform the primary functions of a stormwater treatment facility inlet structure.

The two most common flow splitters are the:

- Weir type
- Orifice type

### 3.1.1 Flow Splitter – Weir Type

A typical weir type flow splitter configuration is illustrated in Figure 1. Weir type flow splitters need two compartments to function. The two compartments are created by placing a barrier or also known as a weir across the center of the manhole. The upstream compartment contains the upstream conveyance system inlet pipe and the conveyance pipe that drains into the stormwater treatment facility. All flows up to the water quality peak flow exit the flow splitter manhole by way of the stormwater treatment facility pipe and into the downstream treatment facility. The downstream compartment contains the bypass pipe. Flow depths greater than the height of the weir overtops the weir and is known as bypass flow. These flows are routed into the downstream conveyance system or detention facility.

#### 3.1.1.1 Design Criteria for Weir Type Flow Splitter

This section describes the features of a weir type flow splitter and the design criteria that apply specifically to these installations.

**Site Selection**

1. General siting requirements are discussed in Section 14.9. Additional siting criteria that apply specifically to flow splitters include:

    a) The site must be of sufficient size to accommodate the inlet structure and maintenance access

b) The site must be located upstream of offline facilities.

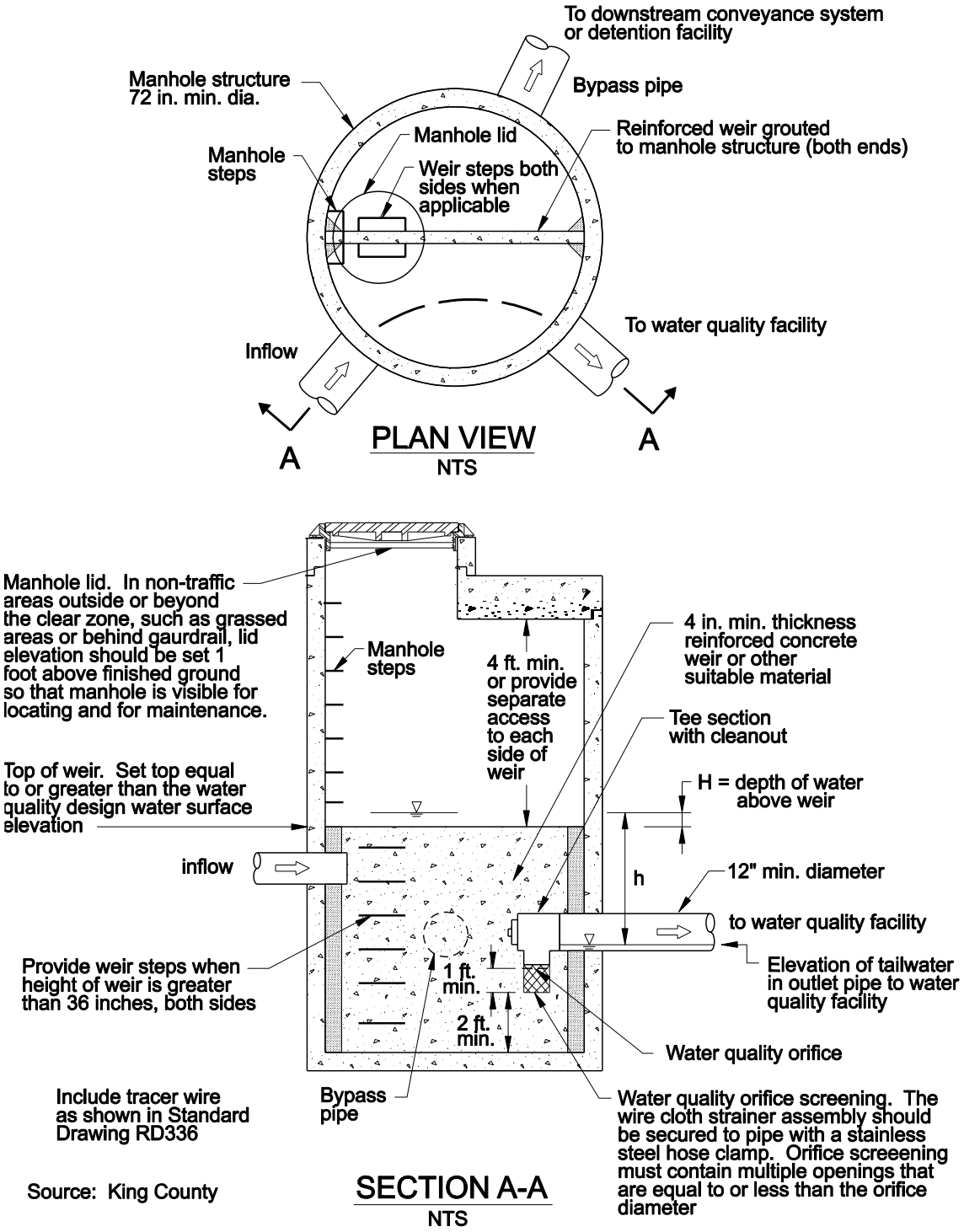



**Figure 1 Weir Type Flow Splitter**

**Weir Type Flow Splitter manhole Dimensions**

1. The **minimum manhole diameter** is 72 inches.

2. The **minimum depth** between top of weir and bottom of manhole cover is 4 feet (see figure 1). Avoid Inlet structure depths greater than 20 feet due to the limitations of vactor trucks. Adjacent access to the inlet structure is needed for the vactor to operate to this maximum depth. Therefore, verify access is appropriate by coordinating the design with the Maintenance District.

**Weir Type Flow Splitter Features**

*1.* The **flow splitting weir** should be centered within the manhole. The top of weir is set equal to or greater than the stormwater treatment facility design water surface elevation. Verify the weir can bypass the design high flow without adversely impacting the upstream conveyance system. The design high flow is calculated using the same design storm as the upstream conveyance system. Modify design as needed. *Note: The goal is to establish a splitter configuration that achieves the largest outlet pipe and minimizes the weir height.*

2. Provide **water quality orifice screening** to protect orifice from plugging for all orifices 6 inches or less. Orifice screen must contain multiple openings that are equal to or less than the orifice diameter.

3. **Bypass piping** shall be designed using the same design criteria as the upstream conveyance system. The minimum pipe size is 12 inches.

4. **Water quality piping** shall be designed using the water quality peak discharge. The minimum pipe size is 12 inches.

**Sediment Control**

1. Provide a sump with a minimum depth of 2 feet.

**Maintenance and Inspection Access**

1. Provide access to each side of weir if the height between the top of the weir and the bottom of the manhole cover is less than 4 feet.

2. Provide a ladder on each side of weir if height of weir is greater than 3 feet.

3. Provide two manhole lids. Locate one on each side of the weir.

4. Implement with Maintenance District concurrence: **Manhole lids** located in non-traffic areas

outside or beyond the clear zone such as grassed areas or behind guardrail must be set 1 foot above finish ground so that manhole location is visible for locating and for maintenance. This should be coordinated with the maintenance districts, lids may be placed flush with the finished grade at the request of the serving maintenance district. Lid elevations must match proposed finish grade in traffic areas.

5. An access road to this structure is required for inspection and maintenance. See the primary facility's design criteria section for access road design guidance.

### 3.1.1.2 Design Procedure for Weir Type Flow Splitter

The procedure for designing a weir type flow splitter is presented below. It assumes the treatment facility's dimensions such as slopes, grades, depth, width, length, etc. are known.

**Step 1** – Determine top of weir elevation ($E_{Top\ of\ Weir}$). It should be equal to or greater than the water quality design water surface elevation projected in the facility.

**Step 2** – Determine the hydraulic head over the bypass weir using the rectangular sharp-crested weir equation.

$$Q_W = \frac{2}{3} C_d (2g)^{0.5} L H^{1.5}$$

Where:

$Q_W$ = conveyance peak flow in cubic feet per second
$C_d$ = sharp crested weir coefficient of discharge = 0.60
L = length of weir in feet
H = depth of water above weir in feet

Rearrange to solve for H:

$$H = \left( \frac{3Q_W}{2C_d (2g)^{0.5} L} \right)^{2/3}$$

**Step 3** – Determine water elevation ($E_{Weir}$) at weir using calculated depth of water above the weir in Step 2.

$$E_{Weir} = E_{Top\ of\ Weir} + H$$

Where:

$E_{Topf\ of\ Weir}$ = top of weir elevation as determined in Step 1
H = depth of water above weir in feet as determined in Step 2

**Step 4** – Determine invert elevation for pipe draining into treatment facility.

**Step 5** – Determine tailwater elevation ($E_{TW}$) in pipe that drains into treatment facility using the water quality peak flow.

**Step 6** – Size the water quality orifice using the following equation.

$$Q_{WQ} = CA(2gh)^{0.5}$$

Where:

$Q_{WQ}$ = water quality peak flow in cubic feet per second
C = orifice loss coefficient = 0.60 (see Chapter 12)
A = area of circular orifice opening in square feet = $0.25\pi(\text{diameter})^2$
g = gravitational constant = 32.2 feet per second squared
h = effective head ($E_{Weir} - E_{TW}$) in feet

Rearrange equation to solve for orifice diameter (d):

$$d = \left(\frac{4Q_{WQ}}{C\pi(2gh)^{0.5}}\right)^{0.5}$$

**Step 7** – Provide water quality orifice screening to protect orifice from plugging for all orifices 6 inches or less. Orifice screen must contain multiple openings that are equal to or less than the orifice diameter.

**Step 8** – Evaluate capacity of bypass pipe and water quality outlet pipe as discussed in Sections 7 and 8 of this appendix.

**Step 9** – Evaluate and address the backwater effects. Check the hydraulic grade line (HGL) upstream. Refer to **Chapter 13**.

**Step 10** – Summarize flow splitter dimensions determined in this analysis.

### 3.1.2 Flow Splitter – Orifice Type

A typical orifice type flow splitter is illustrated in Figure 2. An orifice and riser pipe is used. The orifice is attached to the conveyance pipe that drains into the stormwater treatment facility. The orifice is sized to allow all flows up to the water quality peak flow to exit the flow splitter manhole by way of the stormwater treatment facility pipe and into the downstream treatment facility. The riser pipe is used to convey flows greater than the water quality peak flow around the stormwater treatment facility. This is known as bypass flow and is either routed into the downstream conveyance system or detention facility.

#### 3.1.2.1 Design Criteria for Orifice Type Flow Splitter

This section describes the features of an orifice type flow splitter and the design criteria that apply specifically to these installations.

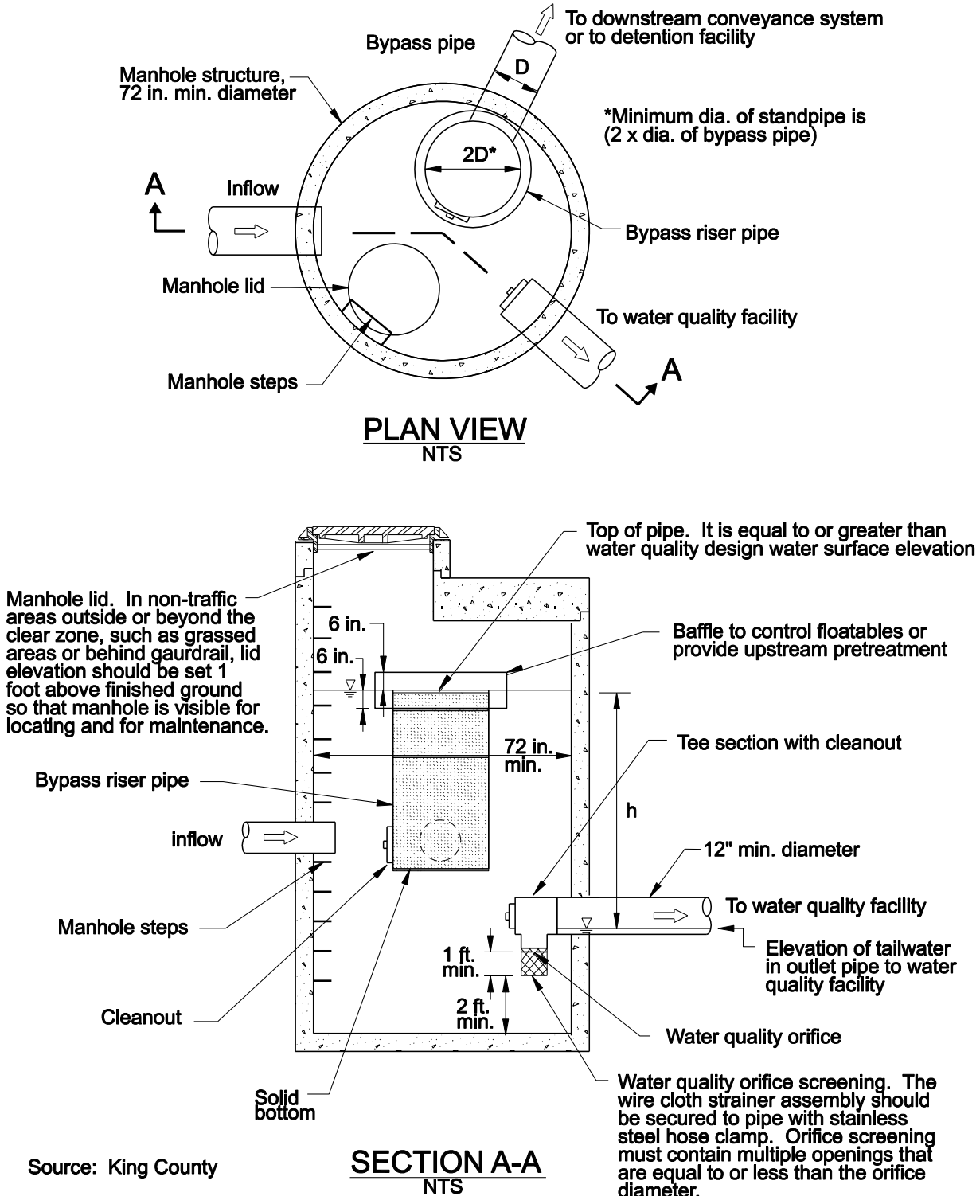



**Figure 2 Orifice Type Flow Splitter**

**Site Selection**

1. General siting requirements are discussed in Section 14.9. Additional siting criteria that apply specifically to flow splitters include:

   a) The site must be of sufficient size to accommodate the inlet structure and maintenance access
   b) The site must be located upstream of the BMP

**Orifice Type Flow Splitter Manhole Dimensions**

1. The minimum manhole diameter is 72 inches.

2. Avoid Inlet structure depths greater than 20 feet due to the limitations of vactor trucks. Adjacent access to the inlet structure is needed for the vactor to operate to this maximum depth. Therefore, verify access is appropriate by coordinating the design with the Maintenance District.

**Orifice Type Flow Splitter Features**

1. Provide **water quality orifice screening** to protect orifice from plugging for all orifices 6 inches or less. Orifice screen must contain multiple openings that are equal to or less than the orifice diameter.

2. **Bypass riser pipe** shall be designed using the same design criteria as the upstream conveyance system. The riser pipe must be designed so it operates as a weir rather than an orifice (See Section 12.9.4.2). The minimum riser pipe size is two times the diameter of the bypass outlet pipe.

3. **Bypass piping** is connected to the bypass riser pipe that drains high flows into the downstream conveyance system or detention facility. Design using the same design criteria as the upstream conveyance system. The minimum pipe size is 12 inches.

4. **Water quality piping** shall be designed using the water quality peak discharge. The minimum pipe size is 12 inches.

**Sediment Control**

1. Provide a sump with a minimum depth of 2 feet.

**Maintenance and Inspection Access**

1. Provide a ladder for inspection access and maintenance.

2. Provide two manhole lids.

3. Manhole lids located in non-traffic areas outside or beyond the clear zone such as grassed areas or behind guardrail must be set 1 foot above finish ground so that manhole location is visible for locating and for maintenance. This should be coordinated with the maintenance districts, lids may be placed flush with the finished grade at the request of the serving maintenance district. Lid elevations must match proposed finish grade in traffic areas.

4. An access road to this structure is required for inspection and maintenance. See the primary facility's design criteria section for access road design guidance.

### 3.1.2.2 Design Procedure - Orifice Type Flow Splitter

The procedure for designing a orifice type flow splitter is presented below. It assumes the treatment facility's dimensions such as slopes, grades, depth, width, length, etc. are known.

**Step 1** – Determine top of riser pipe elevation ($E_{Top}$). It should be equal to or greater than the water quality design water surface elevation projected in the facility.

**Step 2** – Determine invert elevation for pipe draining into treatment facility.

**Step 3** – Determine tailwater elevation ($E_{TW}$) in pipe that drains into treatment facility using the water quality peak flow.

**Step 4** – Size the water quality orifice using the following equation.

$$d = \left( \frac{4Q_{WQ}}{C\pi(2gh)^{0.5}} \right)^{0.5}$$

Where:

| | | |
|---|---|---|
| $Q_{WQ}$ | = | water quality flow rate in cubic feet per second |
| C | = | orifice loss coefficient = 0.60 |
| d | = | diameter of circular orifice opening, in feet |
| g | = | gravitational constant = 32.2 feet per second squared |
| h | = | effective head ($E_{Top}$ - $E_{TW}$) in feet |

**Step 5** – Provide water quality orifice screening to protect orifice from plugging for all orifices 6 inches or less. Orifice screen must contain multiple openings that are equal to or less than the orifice diameter.

**Step 6** – Size the bypass riser pipe. The riser pipe must be designed so it operates as a weir rather than an orifice. Design guidance is provided in Chapter 12. The minimum riser pipe size is two times the diameter of the bypass outlet pipe. Design using the same design criteria as the upstream conveyance system.

**Step 7** – Evaluate capacity of bypass pipe and water quality outlet pipe as discussed in Sections 7 and 8 of this appendix.

**Step 8** – Evaluate and address the backwater effects.

**Step 9** – Summarize flow splitter dimensions determined in this analysis.

## 4.0 Pretreatment Facilities or Structures

The goal of a pretreatment facility is to extend the life and efficiency of the primary stormwater treatment facility by removing coarse sediment and debris transported by stormwater runoff. Pretreatment can be integrated within the primary treatment facility or provided as a stand alone facility.

Runoff from pretreatment facilities or structures can not discharge directly into a receiving water body. It must always drain into a primary stormwater treatment facility for additional treatment.

The most common pretreatment facilities are:

- sediment basin (such as a forebay, wet pool, or extended dry detention pond)
- pollution control manhole, and
- proprietary structures

### 4.1 Sediment Basin Pretreatment

A sediment basin is a pretreatment facility that is intended to slow the flow of stormwater enough to allow coarse sediment and debris to settle out before entering the primary treatment facility. They are often integrated within the primary treatment facility, but can be placed separately upstream. Sediment basins are also commonly known as forebays and wet pools. It is always the initial or first cell that stormwater runoff drains into as illustrated in Figures 3, 3a, 4, and 4a. Extended dry ponds, bioretention ponds, wet vaults, stormwater treatment wetlands, and sand filter vaults can be designed with a forebay. Biofiltration swales may also benefit from a pretreatment sediment basin, particularly in areas with heavy winter sanding.

### 4.1.1 Design Criteria for a Sediment Basin

This section describes the features of a sediment basin and the design criteria that apply specifically to these installations.

**Site Selection**

1. General siting requirements are discussed in Section 14.9. Additional siting criteria that apply specifically to sediment basins include:

    a) The site must be of sufficient size to accommodate the inlet facility and maintenance vehicle access.
    b) The site must be located upstream of the BMP.

**Sediment Basin Geometry**

1. The **sediment basin** bottom area must be flat.

2. **Interior side slopes** should not be steeper than 1V:4H

3. The sediment basin must have adequate **depth** to provide the needed storage noted below for pretreatment and sediment storage (sediment storage is also known as dead storage).

4. The **minimum bottom width** is 4 feet to provide the needed storage and allow for maintenance.

5. The sediment basin **storage volume** is designed to temporarily store 5 to 10 percent of the primary treatment facility volume. This is in addition to the sediment or dead storage volume. Use 5 percent for limited space applications.

6. **Pond walls** may be retaining walls designed in accordance with the ODOT Geotechnical Design Manual. A fence is typically provided along the top of the wall.

7. The basin must be designed to provide a minimum depth of storage of 6 inches for sediment accumulation.

**Maintenance Access**

1. An access road to the sediment basin is required for inspection and maintenance. See the primary facility's design criteria section for access road design guidance.

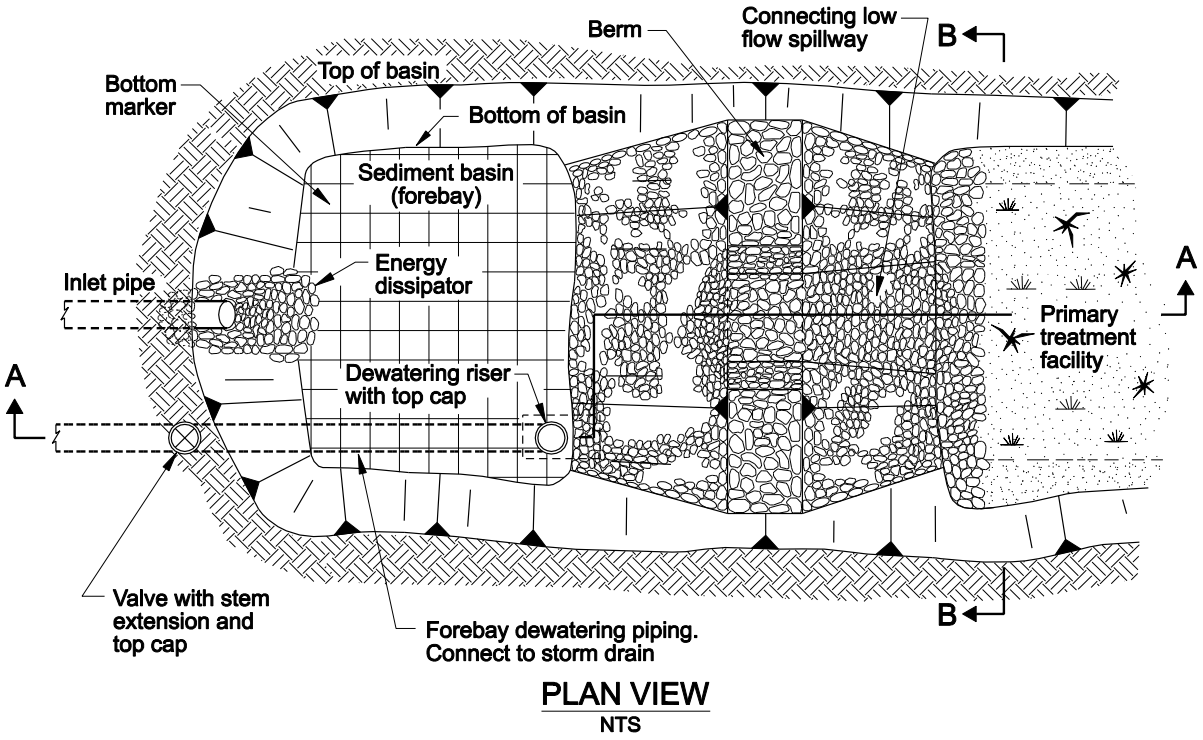



**Figure 3 Sediment Basin (Option 1) Plan View**

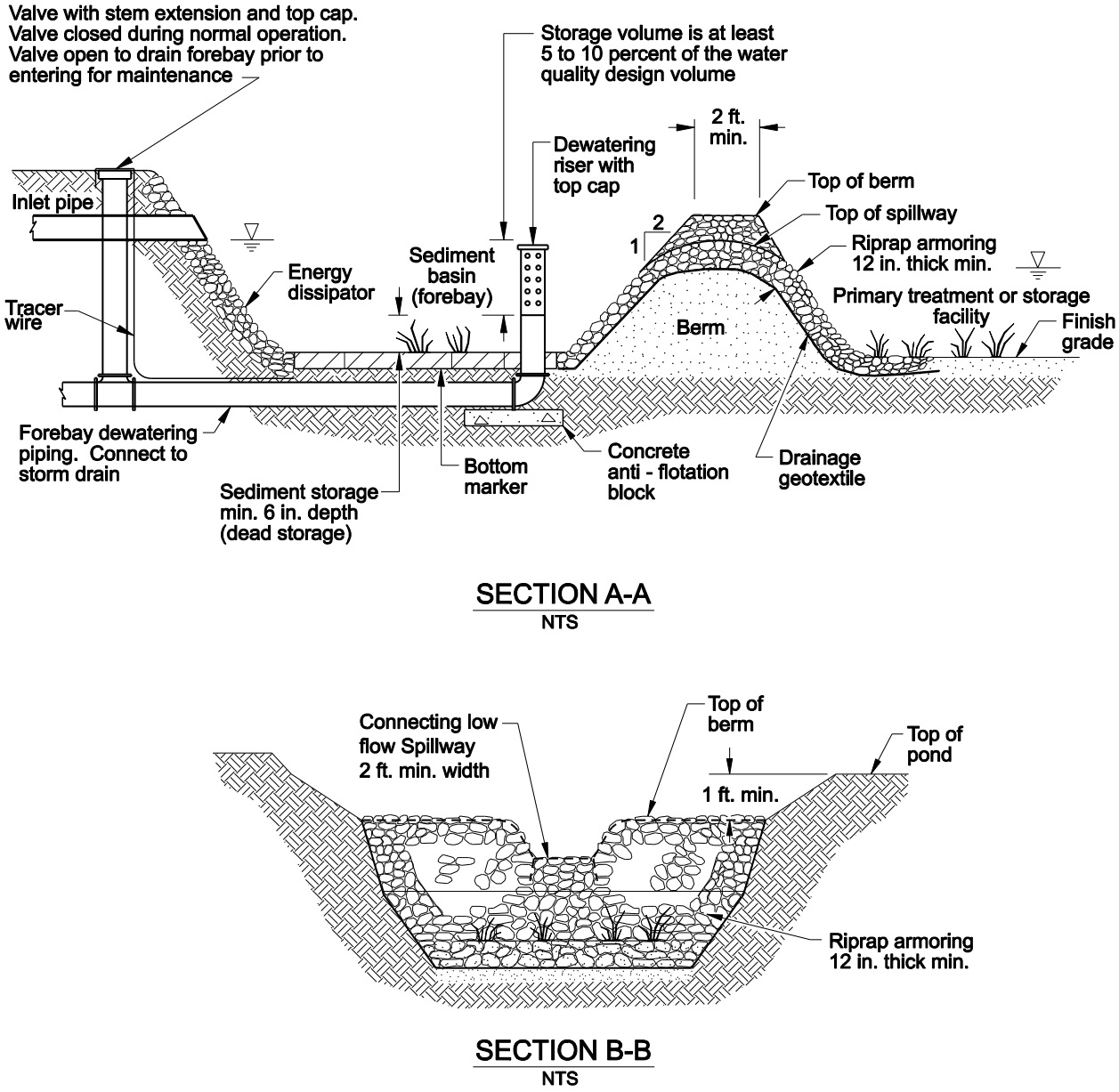



**Figure 3A Sediment Basin (Option 1) Cross-Sections**

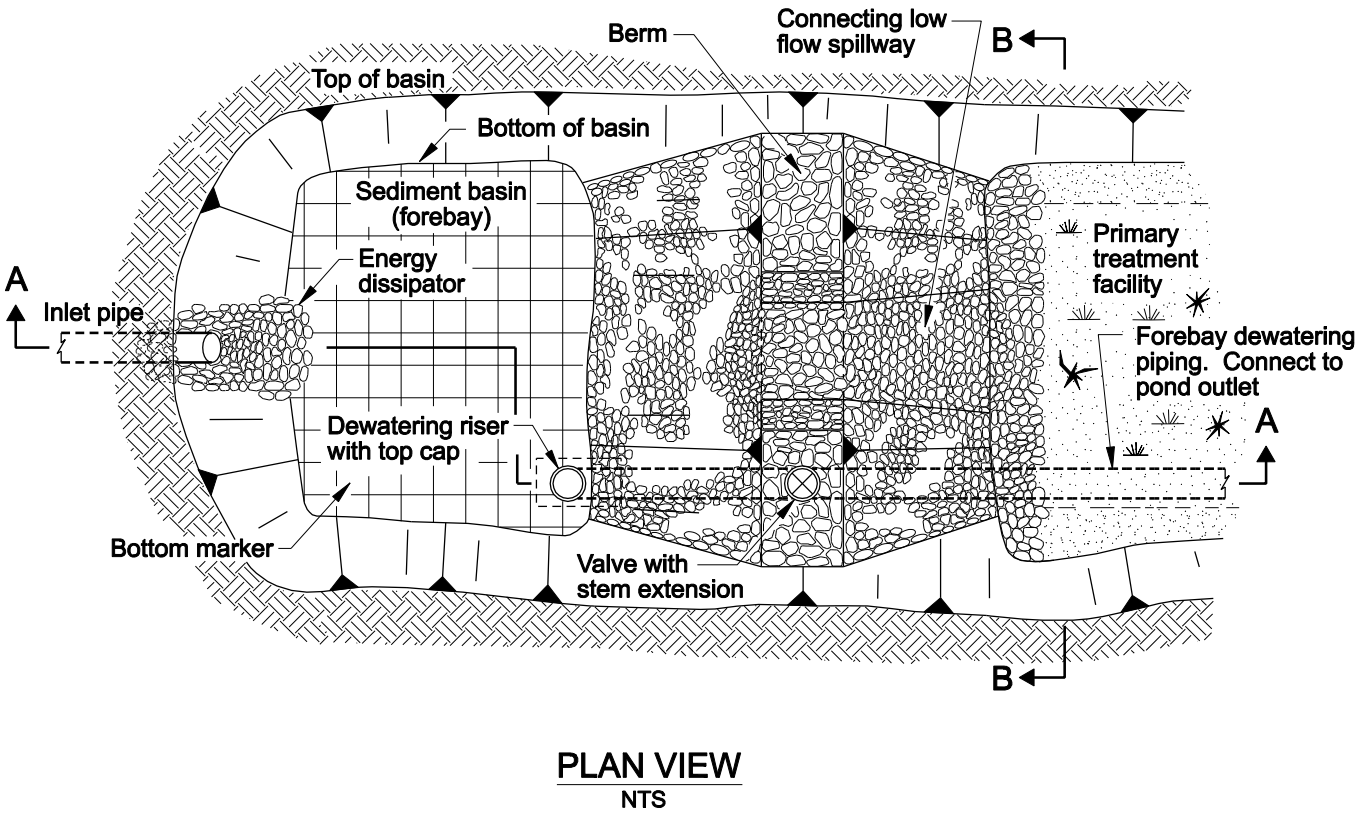



**Figure 4 Sediment Basin (Option 2) Plan View**

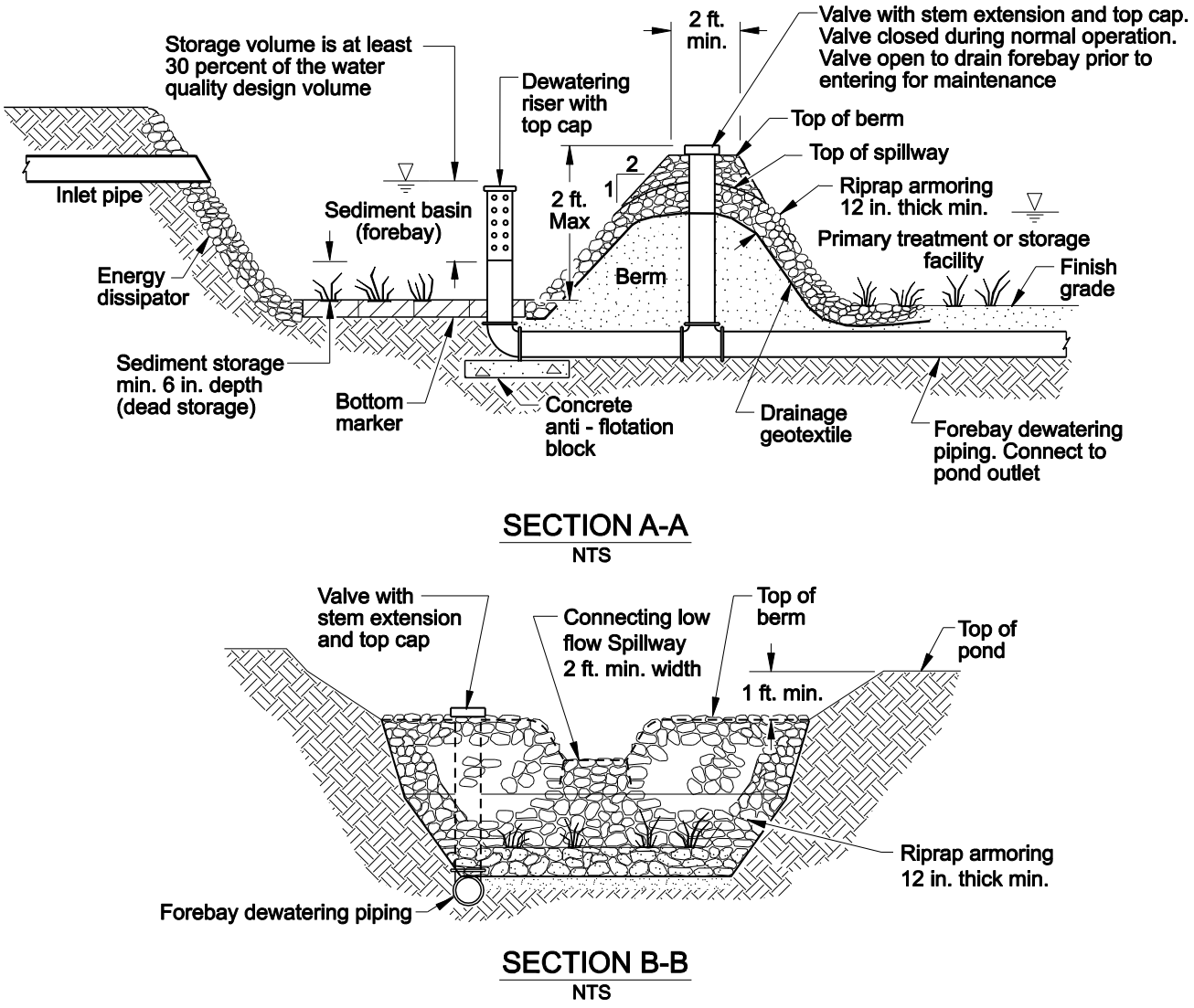



**Figure 4A Sediment Basin (Option 2) Cross-Sections**

**Bottom Marker**

1. A **bottom marker** made of porous pavers must be installed along the basin bottom for maintenance vehicle and mowing equipment access. Select a porous paver from the Qualified Products List. The porous paver must provide a minimum 80 percent bottom area opening for grass growth. Spaced solid paver blocks are not allowed.

**Energy Dissipator**

1. The side slope around the inlet pipe shall be **armored with riprap**. Armor using riprap designed to the same design storm as the upstream conveyance system. The minimum depth of riprap is 12-inches.

**Dewatering Riser**

1. A dewatering riser is optional, and is only opened to remove water from the sediment basin to allow for maintenance. The riser assembly has three components:
    1. Riser pipe with top cap. The pipe is perforated from the top down to just above the dead (sediment) storage elevation.
    2. Forebay dewatering pipe, which connects the riser pipe to the primary BMP outfall or other downstream conveyance system.
    3. Valve with stem extension and top cap. The valve is located downstream of the riser pipe, and should be placed for easy access.

**Spillway Berm**

1. The **top, downstream side slope, and toe** of the spillway berm shall be **armored with riprap**. Armor using riprap designed to the same design criteria as the upstream conveyance system. The minimum depth of riprap is 12-inches.

2. The **side slopes** should not be steeper that 1V:4H

3. The low flow spillway **minimum width** is 2 feet.

4. The **low flow spillway elevation** should not be greater than the water quality design elevation.

**Safety Features**

Exclusionary measures may be required to prevent entry into facilities that present a hazard to children and, to a lesser extent, all persons. Fences are recommended around these facilities where one or more of the following conditions exist:

- areas where small children are present, particularly in residential areas and close to schools and playgrounds
- areas where rapid water level increases would make escape practically impossible,
- water depths exceed 3 feet for more than 24 hours,
- basin bottom is permanently wet and has side slopes steeper than 1V:3H, or

## 4.2 Pollution Control Manhole

A pollution control manhole is used to capture sediment and trap floatable debris/oils. They consist of a large diameter manhole and an outlet cross tee. Details of this pretreatment structure are shown on Standard Drawing RD340. They are suitable for built-up areas where space is at a premium.

Extended dry ponds, bioretention facilities, media filter BMPs, wet vaults, stormwater treatment wetlands, sand filter vaults and sand filters can use pollution control manholes for pretreatment.

### 4.2.1 Design Criteria for a Pollution Control Manhole

**Site Selection**

1. General siting requirements are discussed in Section 14.9. Additional siting criteria that apply specifically to pollution control manholes include:

   a) The site must be of sufficient size to accommodate the structure and maintenance vehicle access
   b) The site must be located upstream of a BMP.

**Pollution Control Manhole Geometry**

1. The minimum manhole diameter is 72 inches.

2. Avoid structure depths greater than 20 feet due to the limitations of ODOT vactor trucks.

**Maintenance and Inspection Access**

1. Provide a ladder for inspection access and maintenance.

2. **Manhole lids** located in non-traffic areas such as grassed areas or behind guardrail must be set 1 foot above finish ground so that manhole location is visible for locating and for maintenance. This should be coordinated with the maintenance districts, lids may be placed flush with the finished grade at the request of the serving maintenance district Lid elevations must match proposed finish grade in traffic areas.

3. An access road to this structure is required for inspection and maintenance. See the primary facility's design criteria section for access road design guidance.

### 4.3 Proprietary Structures

Several proprietary structures are available for stormwater pretreatment. These structures are briefly discussed in Sections 14.8.5 and 14.8.6. The reader is referred to the various manufacturers for more detailed information on the products discussed.

## 5.0 Flow Spreading Options

Several stormwater treatment facilities require uniform sheet flow as runoff enters and flows along the intended treatment path. Swales, dispersion areas and sand filters are treatment facilities that require flow spreaders to maintain uniform sheet flow during the treatment process.

The most common flow spreaders are the:

- Concrete basin
- Rock basin
- Anchored board

### 5.1 Concrete Basin Flow Spreader

This flow spreader technique utilizes a box with sump to assist with maintaining sheet flow. It is typically installed downstream of the stormwater treatment facility inlet pipe. The goal is to allow the concentrated runoff from the inlet pipe to outfall into the box spreader. Runoff leaves the box spreader by spilling over the level downstream wall of the box spreader as sheet flow.

A concrete basin flow spreader should be used with swales and sand filters. See Figure 5.

#### 5.1.1 Design Criteria for Concrete Basin Flow Spreader

This section describes the features of a concrete basin flow spreader and the design criteria that apply specifically to these installations.

**Concrete Basin Flow Spreader Geometry**

1. Provide a minimum **sump depth of** 8-inches. The sump depth is the distance between the top of the downstream basin wall and the bottom of the concrete basin.

**2.** Provide a **width** that extends 6 inches (both ends) beyond the width of the treatment path or swale bottom width. The width is shown and noted in Figure 5.

3. Provide a **minimum length** of 24-inches. The length is shown and noted in Figure 5.

4. **Wingwalls** must be 4 inches higher than the bottom of the facility.

5. The portion of the **downstream wall** that will serve as the weir to spread flows must be two inches higher than the bottom of the facility.

6. The **Upstream wall** must be 2 inches higher than the downstream wall.

## 5.2 Rock Basin Flow Spreader

This flow spreader technique utilizes a basin lined with rock to assist with maintaining sheet flow. It is typically installed downstream of the stormwater treatment facility inlet pipe. The concentrated runoff from the inlet pipe outfalls into the rock spreader. Runoff leaves the basin by spilling over the level downstream portion of the basin spreader as sheet flow.

### 5.2.1 Design Criteria for Rock basin Flow Spreader

This section describes the features of a rock basin flow spreader and the design criteria that apply specifically to these installations. See Figure 6.

**Rock basin Flow Spreader Geometry**

1. Provide a minimum sump depth of 8-inches. The sump depth is the distance between the top of the downstream rock and the bottom of the rock basin.

2. Provide a **width** that equals the width of the treatment path or swale bottom width. The width is shown and noted in Figure 6.

3. Provide a **minimum length** of 24-inches. The length is shown and noted in Figure 8.

## 5.3 Anchored Board Flow Spreader

This flow spreader technique utilizes a plate or board to assist with maintaining sheet flow. See Figure 7.

Anchored board flow spreaders are typically installed every 50 feet along swales to maintain sheet flow.

The goal is to allow the concentrated runoff from the inlet pipe to drain up to the plate and forcing the flowing water to spread along the upstream face of the board. Runoff spills over the board as sheet flow when flow depths exceed the board height.

An anchored board flow spreader should be used with swales and sand filters.

### 5.3.1 Design Criteria for Anchored Board Flow Spreader

This section describes the features of an anchored board flow spreader and the design criteria that apply specifically to these installations.

**Anchored Board Flow Spreader Geometry**

1. Provide a width that extends 2 feet (both ends) beyond the width of the treatment path or swale bottom width. The width is shown and noted in Figure 7.

2. Provide a maximum **exposure height** of 1 inch measured at the upstream face of the anchored board. This is the height between the top of anchored board and the bottom of the facility.

**Anchor Board Pad**

1. Provide a 4-inch wide by 4-inch long anchor board pad along each end of board as shown in figure 7.

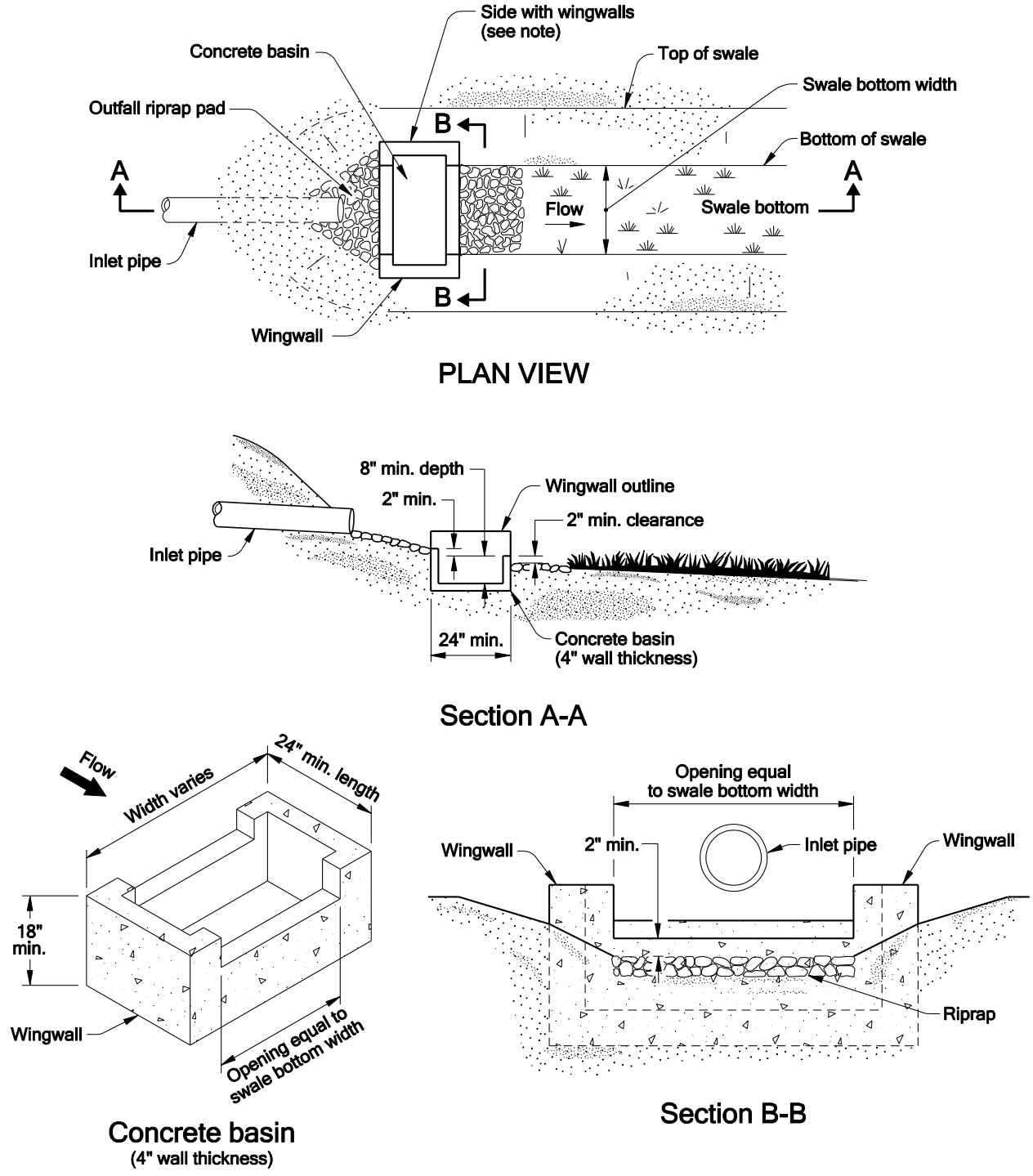



**Figure 5 Concrete Basin Flow Spreader**

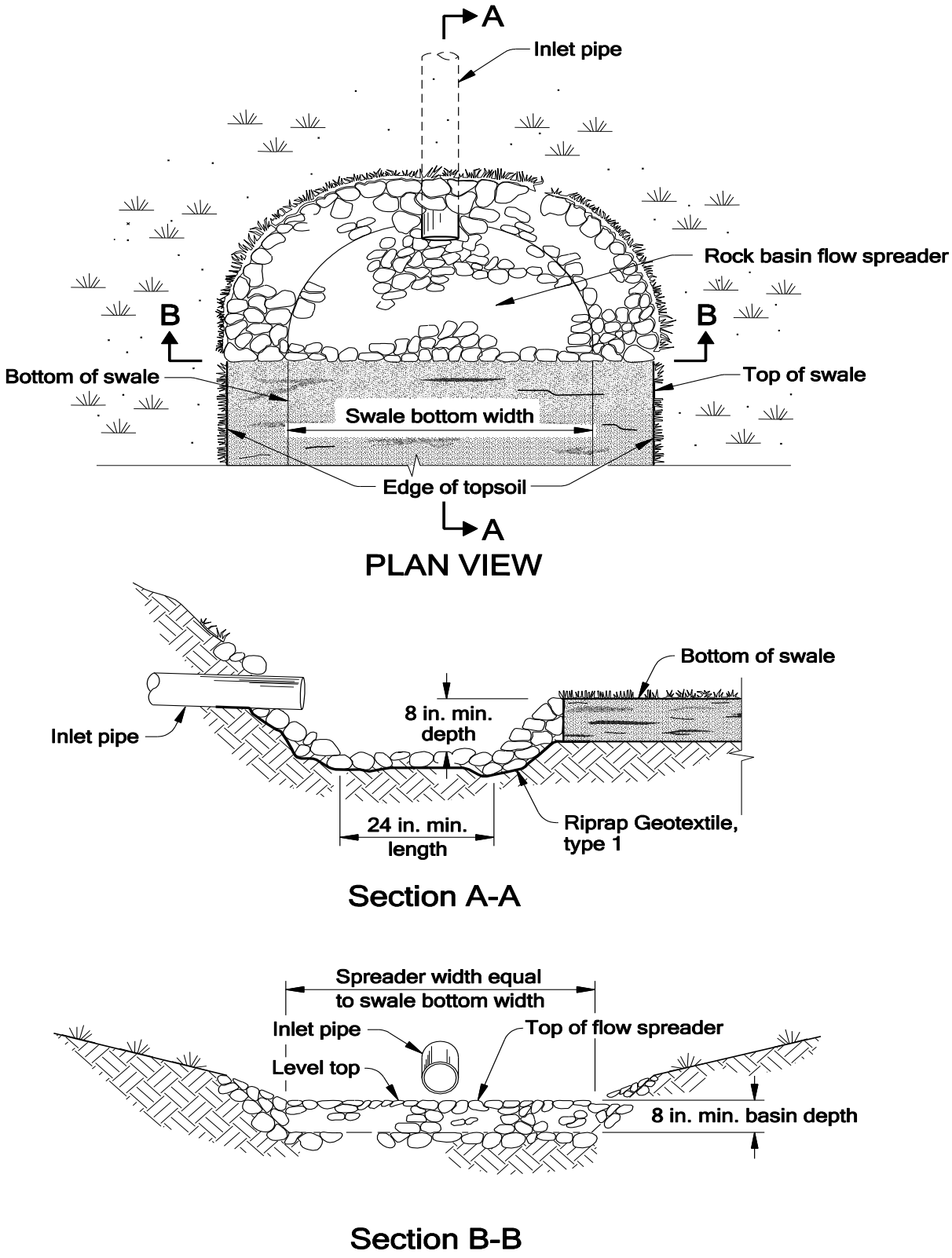



**Figure 6 Rock Basin Flow Spreader**

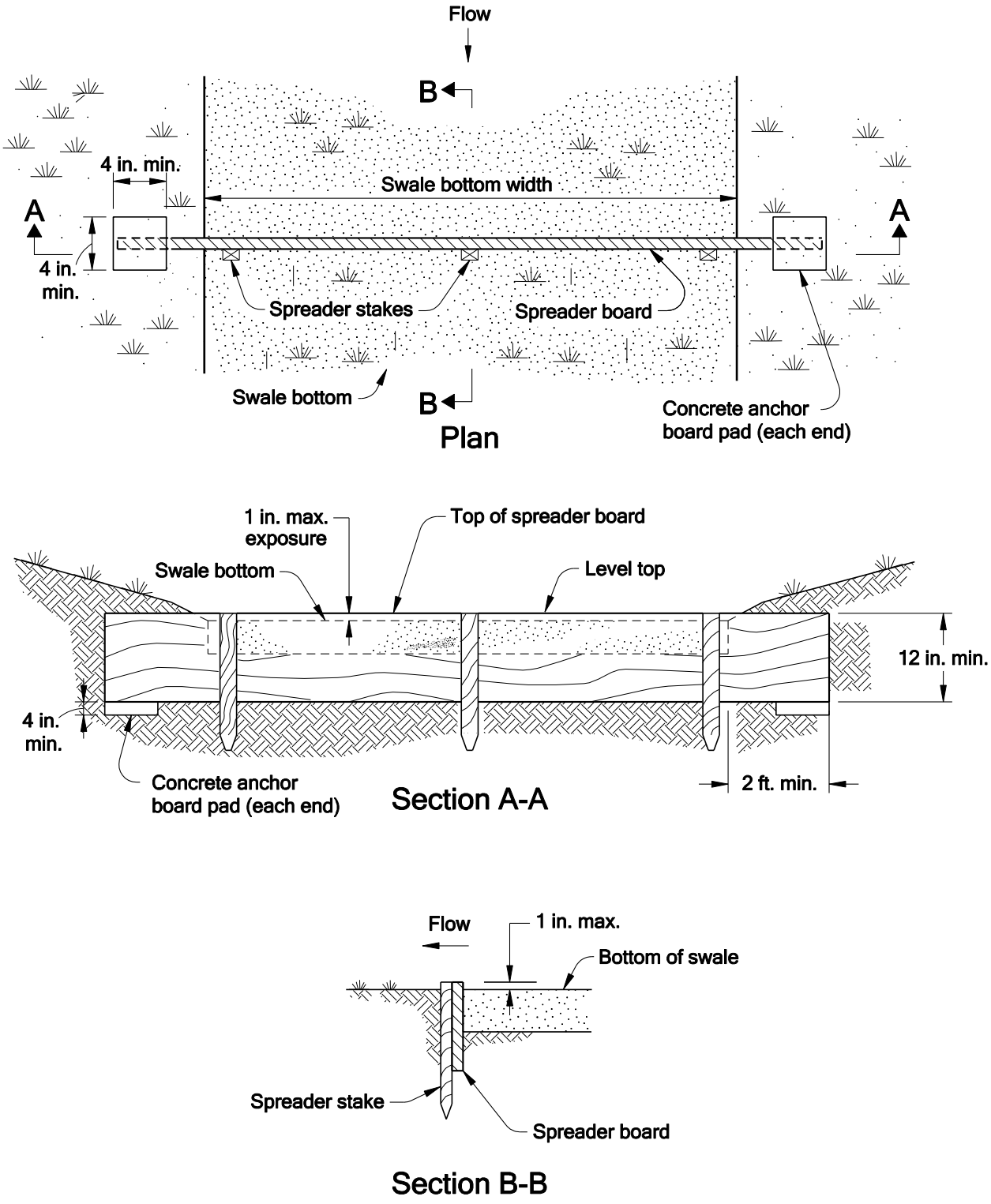



**Figure 7 Anchored Board Flow Spreader**

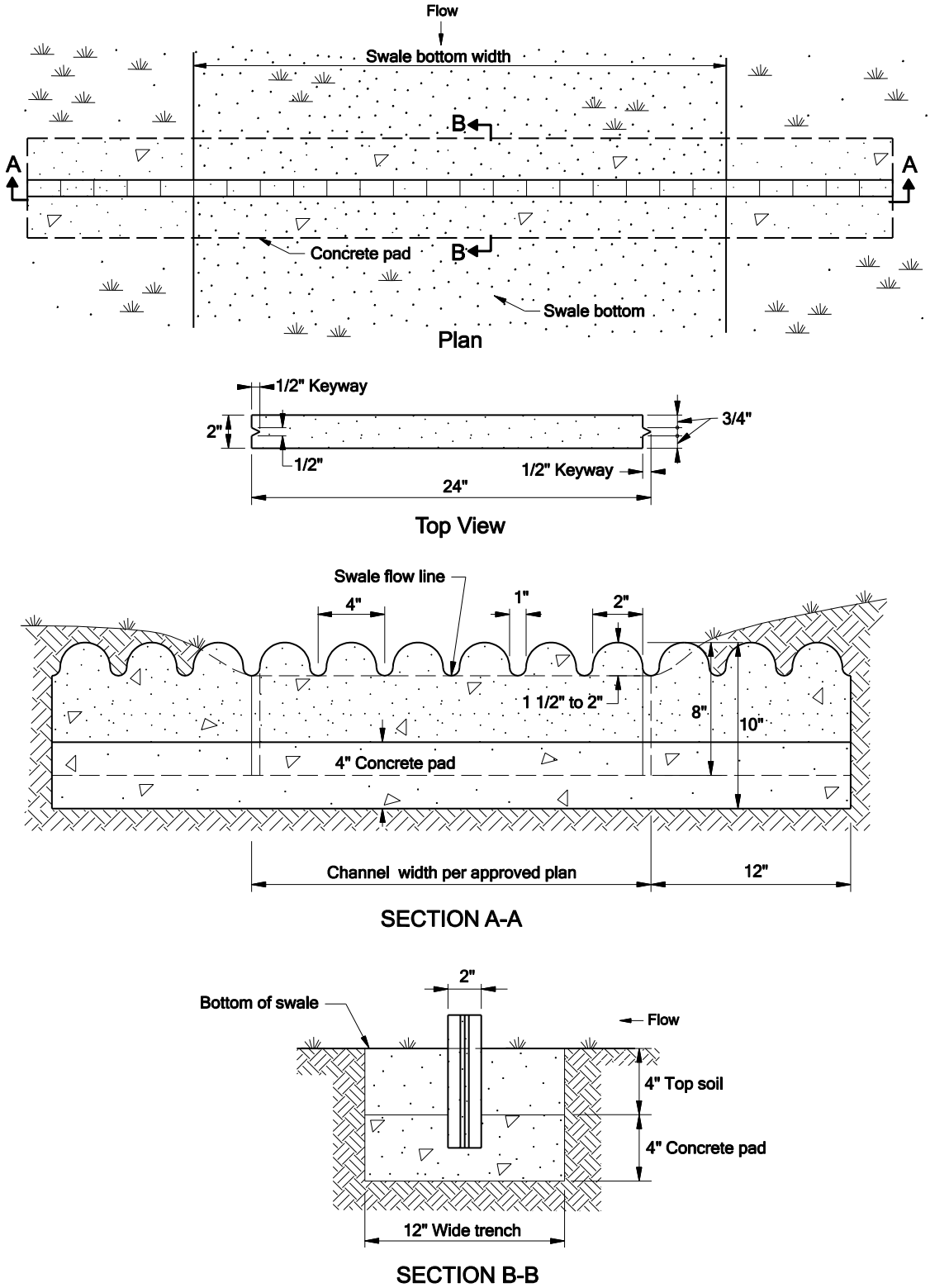



**Figure 8 Anchored Concrete Flow Spreader**

## 5.4 Anchored Concrete Flow Spreader

This flow spreader technique utilizes a formed concrete strip to assist with maintaining sheet flow. See Figure 8.

Anchored concrete flow spreaders are typically installed every 50 feet along swales to maintain sheet flow.

The goal is to force flowing water in the swale to spread along the upstream face of the flow spreader and drain through the spaced gaps.

An anchored concrete flow spreader is used in swales.

### 5.4.1 Design Criteria for Anchored Concrete Flow Spreader

This section describes the features of an anchored concrete flow spreader and the design criteria that apply specifically to these installations.

**Anchored Concrete Flow Spreader Geometry**

1. Provide a width that extends 12 inches (both ends) beyond the width of the treatment path or swale bottom width. The width is shown and noted in Figure 8.

2. Provide an **exposure height** of 1-1/2 to 2 inches measured at the upstream face of the anchored spreader. This is the height between the top of anchored spreader and the bottom of the facility.

**Anchor Pad**

1. Provide a 4-inch high by 12-inch wide anchor pad along the entire length as shown in figure 8.

## 6.0 Outlet Control Structures

An outlet control structure is needed to control flow leaving the stormwater treatment facility. In most cases the outlet structure consists of a catch basin or manhole, water quality flow control feature and a high flow bypass feature. Its primary functions are:

- To only allow runoff flows up to the water quality design flow to exit the water quality facility.
- To bypass high flows through the stormwater treatment facility. This is common when combined facilities are proposed for a project.

The following design guidance and criteria is for the most common types of outlet control structures used in stormwater treatment facility design. Other outlet control methods may be proposed but must perform the primary functions of a stormwater treatment facility outlet structure (see above).

### 6.1 Outlet Control Structure I

Outlet Control Structure I is most commonly used with extended detention dry ponds. It is illustrated in Figure 9. It is designed to release the water quality event over a 48 hour period and to pass high flows.

Outlet Control Structure I uses two type D inlets and an orifice to function. The lower inlet controls the flow by using an attached orifice that drains the water quality volume over a period of 48 hours and the high flows drain into the upper inlet that routes these flows into the downstream conveyance system or detention facility.

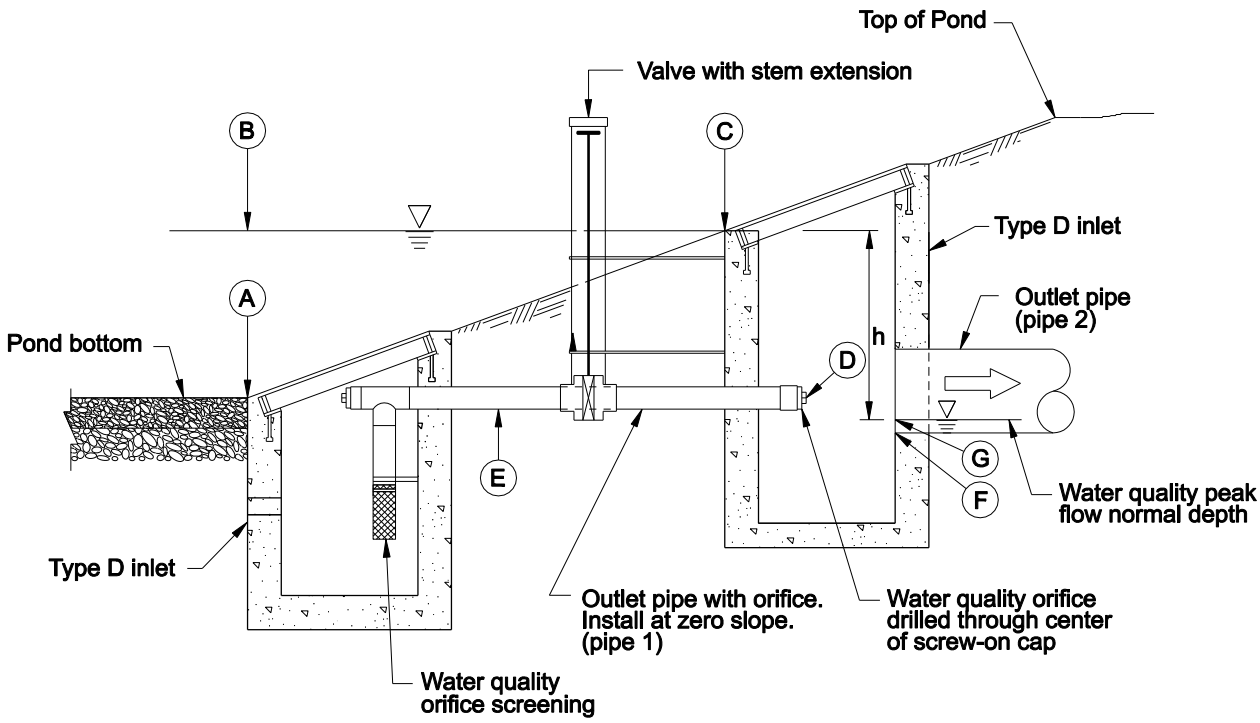



**Figure 9 Outlet Control Structure I**

### 6.1.1 Design Criteria for Outlet Control Structure I

This section describes the features of Outlet Control Structure I and the design criteria that apply specifically to these installations.

1. Provide an **outlet structure** using two type D inlets. Detail of a Type D inlet is shown on Standard Drawing RD370.

2. Provide **water quality orifice screening** to protect orifice from plugging for all orifices 6 inches or less. Orifice screen must contain multiple openings that are equal to or less than the orifice diameter.

3. The lower inlet **grate capacity** must be designed to convey the water quality peak flow. The upper inlet grate must be designed to convey the same peak flow as the upstream conveyance

system. An emergency spillway or auxiliary outlet is needed if the upper inlet grate could not convey the appropriate peak flow. Auxiliary outlet design guidance is provided in Section 7 of this appendix.

4. Provide **valve with stem extension** that is connected to the outlet pipe with orifice to provide an emergency shut-off mechanism in-case of a hazmat spill. Stem extension should extend above the water quality design surface elevation.

### 6.1.2 Design Procedure for Outlet Control Structure I

The procedure for designing Outlet Control Structure I is presented below. It assumes the treatment facility's dimensions such as slopes, grades, depth, width, length, etc. are known.

**Step 1** – Determine the lip elevation ($E_{Lower}$, point A, Figure 9) of the lower inlet grate. It should be equal to the pond bottom elevation.

**Step 2** – Determine the water quality design water surface elevation ($E_{WS}$, Point B, Figure 9). The depth (D) of the water quality volume is determined in design procedure step 3 in Appendix D. Therefore,

$$E_{WS} = E_{Lower} + D$$

**Step 3** – Determine the lip elevation ($E_{Upper}$, point C, Figure 9) of the upper inlet grate. It should be equal to the water quality design water surface elevation ($E_{WS}$).

**Step 4** – Determine the orifice centroid elevation ($E_{Orifice}$, point D, Figure 9). It should be equal to the pond bottom elevation.

**Step 5** – Determine the outlet pipe with orifice (Pipe1) flow line elevation ($E_{Pipe1}$, point E, Figure 9). It should be equal to the pond bottom elevation minus one-half the diameter of Pipe1.

**Step 6** – Determine the outlet pipe (Pipe2) invert elevation ($E_{Pipe2}$, point F, Figure 9). It should always be lower than the orifice centroid elevation to allow for positive drainage.

**Step 7** – Size Pipe2 for high flow. See design guidance in Section 8 of this appendix.

**Step 8** – Determine the water quality peak flow using the water quality design storm determined in Step 1 in Appendix D.

**Step 9** – Determine the normal depth ($y_n$) of flow in Pipe2 using the water quality peak flow from step 8. The normal depth can be calculated using the Manning Equation. See

Chapter 8 for design guidance.

**Step 10 –** Determine the water quality flow elevation in Pipe2 ($E_{WQ}$, point G, Figure 9). Therefore,

$$E_{WQ} = E_{Pipe2} + y_n$$

**Step 11 –** Calculate the water quality outflow rate using the following equation. The required detention time is 48 hours.

$$Q_o = \frac{V_{wq}}{[(48\,\text{hour})(3600\,\text{second per hour})]}$$

$$= \frac{V_{wq}}{172{,}800}$$

Where:

$Q_o$ = Water quality outflow rate in cubic feet per second.

$V_{wq}$ = The water quality volume in cubic feet.

**Step 12 –** Calculate the water quality orifice diameter using the following equation.

$$d = \left[\frac{4Q_o}{\pi C(2gh)^{0.5}}\right]^{0.5}$$

Where:

d = Orifice diameter in feet.

$Q_o$ = Water quality outflow rate in cubic feet per second.

C = Orifice coefficient (usually 0.60).

g = Gravity constant = 32.2 feet per second squared.

h = effective head ($E_{WS} - E_{WQ}$) in feet

**Step 13 –** Provide water quality orifice screening to protect orifice from plugging for all orifices 6 inches or less. Orifice screen must contain multiple openings that are equal to or less than the orifice diameter.

**Step 14 –** Evaluate grate capacity of lower and upper inlets. The lower inlet grate must be designed to convey the water quality peak flow. The upper inlet grate must be designed to convey the same peak flow as the upstream conveyance system. An auxiliary outlet would be needed if the upper inlet grate could not convey the appropriate peak flow. Auxiliary outlets design guidance is discussed in Section 7.0 of this appendix.

**Step 15 –** Evaluate and address hydraulic grade line effects to assure proper operation under design conditions.

## 7.0 Auxiliary Outlets

Auxiliary Outlets are provided if the primary outlet control structure can not safely pass the projected high flows. Broad-crested spillway weirs and overflow risers are the two most common auxiliary outlets used in stormwater treatment facility design. Additional discussion on auxiliary outlets is presented in Chapter 12.

### 7.1 Design Criteria for Auxiliary Outlets

The design criteria discussed in Chapter 12 are recommended for designing stormwater treatment facility auxiliary outlets.

#### 7.1.1 Design Procedure for Auxiliary Outlets

The following steps outline how to size auxiliary outlets for stormwater treatment facilities:

**Step 1 –** Determine the design storm used to analyze the upstream conveyance system. The auxiliary outlet would be sized using the same storm. For example, the design recurrence interval for the conveyance system upstream of a proposed extended dry pond is the 10-year, 24 hour storm then the treatment facility piping would be designed using the same storm event.

**Step 2 –** The design discharge is calculated using hydrology guidance in **Chapter 7**. Use the storm drainage design recurrence interval obtained from step 1.

**Step 3 –** Size the auxiliary outlet using the design discharge calculated in step 2. Designing auxiliary outlets is discussed in **Chapter 12**.

*Note:*
*Evaluate grate capacity when a catch basin is used as the auxiliary outlet. The inlet grate must be designed to convey the same peak flow as the upstream conveyance system.*

## 8.0 Storm Drain Piping

Storm drain piping is required to drain runoff into and out of the stormwater treatment facility.

### 8.1 Design Criteria for Storm Drain Piping

The design criteria discussed in **Chapter 13** are recommended for sizing stormwater treatment facility storm drain piping.

#### 8.1.1 Design Procedure for Storm Drain Piping

The following steps outline how to size storm drain piping for stormwater treatment facilities:

**Step 1 –** Determine the design storm used to analyze the upstream storm drain system. All stormwater treatment facility piping would be sized using the same storm. For example, the design recurrence interval for the conveyance system upstream of a proposed extended dry pond is the 10-year, 24 hour storm then the treatment facility piping would be designed using the same storm event.

**Step 2 –** The design discharge is calculated using hydrology guidance in **Chapter 7**. The rational method should be used to determine the design discharge using the storm drainage design recurrence interval obtained from step 1.

**Step 3 –** Determine the appropriate pipe size using the design discharge calculated in step 2. The minimum pipe size is 12-inches in diameter. The methods discussed in **Chapter 13** are recommended for sizing storm drain piping.

**Step 4 –** Evaluate and address hydraulic grade line effects to assure proper operation under design conditions.

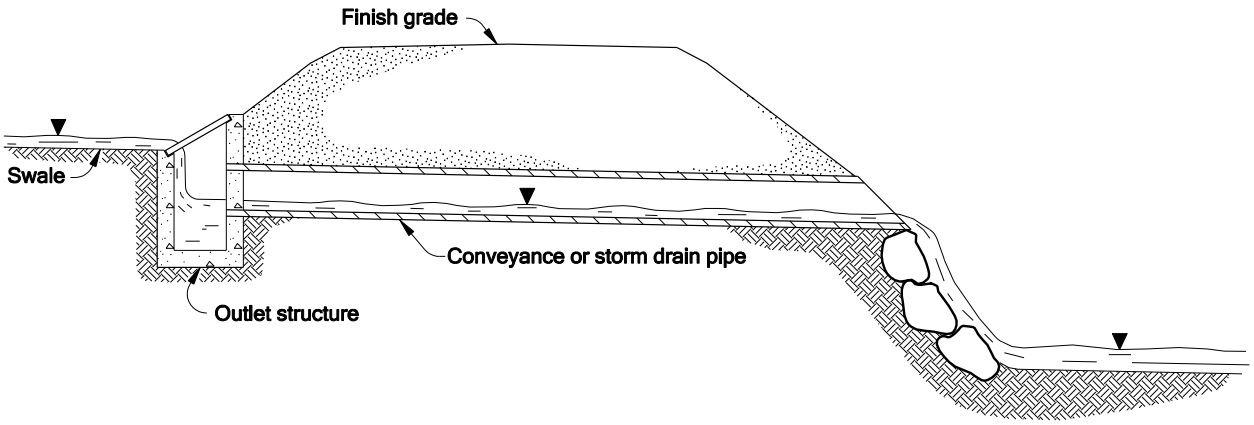



**Figure 10 Storm Drain Piping**

## 9.0 Outfalls

All flow from a stormwater treatment facility is discharged to an outfall. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the stomwater away from the highway.

The two most common outfalls are:

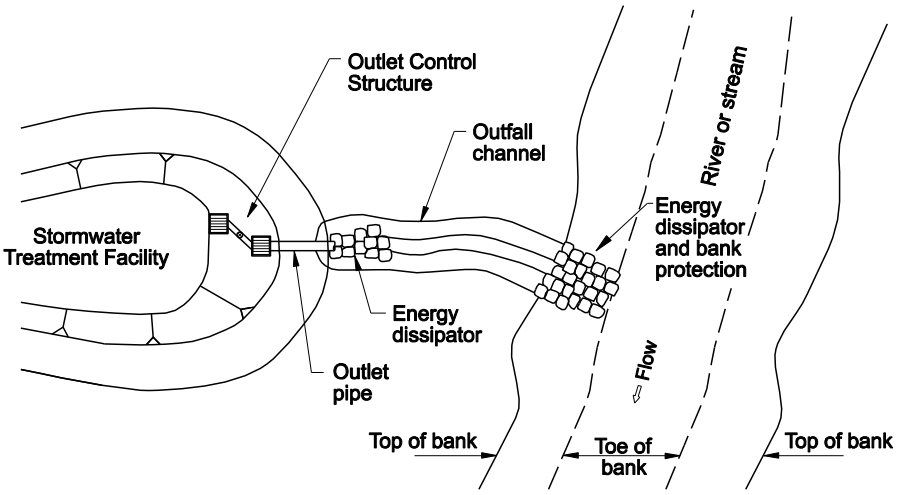
- Outfall channel/armored bank configuration
- Outlet pipe/armored bank configuration

### 9.1 Design Criteria

This section describes the features of an outfall and the design criteria that apply specifically to these installations.

#### Slopes and Grades

1. The flow line or invert elevation of the proposed outlet should be equal to or higher than the flow line of the outfall. If this is not the case, there may be a need to pump the water to the elevation of the outfall. Positive gravity flow is the desired method of discharge. All non-pumping options should be evaluated such as relocating the discharge point to an area that would allow positive gravity flow of stormwater before recommending the construction of a pumping system. Relocation of an outfall should not alter the existing drainage pattern without the written permission of the affected property owner.




OUTFALL CHANNEL/ARMORED BANK CONFIGURATION

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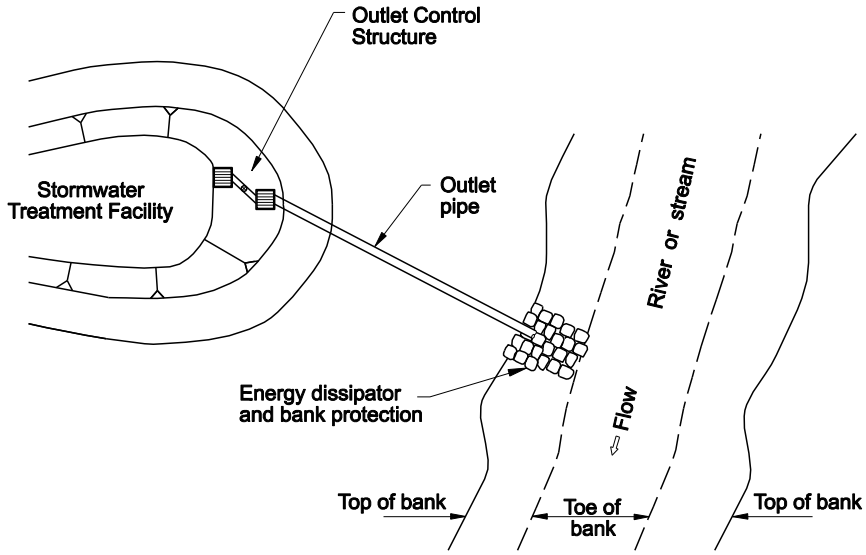


OUTLET PIPE/ARMORED BANK CONFIGURATION

NTS

**Figure 11 Typical Outfalls**

## Open Channel Capacity

1. Outfall channels convey flow from a stormwater treatment facility to a natural waterbody. They are typically composed of prismatic shapes where nearly uniform flow occurs.

   A capacity evaluation is required to assure the channel can convey the design discharge. The design criteria discussed in Section 14.10.12 and **Chapter 8** are recommended for sizing stormwater treatment facility outfall channels.

## Open Channel Stability

1. Outfall channels are typically lined with grass, riprap, or other protective linings. These linings must be able to withstand the shear stresses from hydrodynamic forces created by the expected peak flows in the channel.

   A channel lining stability evaluation is required to assure the channel will be stable. The evaluation consists of verifying the maximum shear stress caused by the design flow must be less than the maximum shear stress the lining can resist without particle movement. The design criteria discussed in Section 14.10.12 and **Chapter 8** are recommended for evaluating stormwater treatment facility outfall channels.

## Energy Dissipation

1. Energy dissipators are used to reduce the velocity and erosion potential of flowing water. Protection is usually required at the outlet of stormwater treatment facilities that discharge onto embankments and into natural or unlined channels.

   The design criteria discussed in Section 14.10.13 and **Chapter 11** are recommended for sizing stormwater treatment facility outfall energy dissipation.

## Bank Protection

1. Outfalls located along the banks of rivers and streams need to function as energy dissipators and bank protection to prevent erosion. If erosion of the embankment is to be prevented, bank protection must be anticipated and the proper type and amount of protection must be provided in the right locations.

   The design criteria discussed in **Chapter 8** should be used for stream and river discharges less than 50 cubic feet per second. The design criteria discussed in **Chapter 15** should be used for channel bank protection and linings on streams and rivers having design discharges greater than 50 cubic feet per second.

### 9.1.1 Design Procedure for Stormwater Treatment Facility Outfalls

The following are general steps to design the outfalls shown in Figure 11:

**Step 1 –** Determine the design storm used to analyze the upstream conveyance system. All stormwater treatment facility piping would be sized using the same design storm. For example, the design recurrence interval for the conveyance system upstream of a proposed extended dry pond is the 10-year, 24 hour storm then the treatment facility outfall features would be designed using the same storm event. Outfalls located in a waterway's floodplain must be designed to conform to the local floodplain regulations.

**Step 2 –** The design discharge is calculated using hydrology guidance in **Chapter 7**. Determine the design discharge using the storm drainage design recurrence interval obtained from step 1.

**Outfall channel/armored bank**

The following steps outline how to design the outfall shown in Figure 11 (outfall channel/ armored bank):

**Step 1 –** Establish typical channel cross-section. Provide adequate drainage depth and side slopes considering safety, maintenance and access.

**Step 2 –** Determine channel grade(s). Provide a minimum grade of 0.3 percent to minimize ponding and sediment accumulation.

**Step 3 –** Determine channel liner type and roughness. See **Chapter 8**.

**Step 4 –** Calculate the flow depth using the design discharge calculated above. The Manning's equation or nomograph is recommended to calculate the flow depth. These methods are discussed in **Chapter 8**.

**Step 5 –** One or more of these changes may be needed if flow is deeper than maximum allowable flow depth including freeboard:

- increase bottom width

- make channel side slopes flatter
- make channel slope steeper, or
- provide smoother channel lining

**Step 6 –** Calculate the maximum shear stress on lining. See **Chapter 8**. Compare maximum to permissible shear stress for all linings except riprap. Go to step 6.2 for riprap lining.

**Step 6.1 –** Reduce maximum shear stress by one or more of the following if maximum shear stress is greater than allowable:

- enlarging channel
- reducing the channel slope, or
- flattening the bank side slopes

or

Increase the allowable shear stress by changing the lining material.

**Step 6.2 –** Calculate $D_{50}$ needed to resist displacement for riprap lining. Compare $D_{50}$ needed to $D_{50}$ provided. Reduce the $D_{50}$ needed by:

- enlarging channel
- changing the channel slope, or
- flattening the bank side slopes

or

Increase the $D_{50}$ of the liner.

**Step 7 –** Verify that the depth of flow in the ditch is within allowable limits if liner roughness or channel cross-section is changed.

**Step 8 –** Design energy dissipation. See **Chapter 11**.

**Step 9 –** Design bank protection. Use the design criteria discussed in **Chapter 8** for stream and river discharges less than 50 cubic feet per second. Use the design criteria discussed in **Chapter 15** for channel bank protection and linings on streams and rivers having design discharges greater than 50 cubic feet per second.

**Outlet pipe/armored bank**

The following steps outline how to design the outfall shown in Figure 11 (outlet pipe/armored bank):

**Step 1 –** Design energy dissipation. See **Chapter 11**.

**Step 2 –** Design bank protection. Use the design criteria discussed in **Chapter 8** for stream or river discharges less than 50 cubic feet per second. Use the design criteria discussed in **Chapter 15** for channel bank protection and linings on streams and rivers having design discharges greater than 50 cubic feet per second.

# APPENDIX F
# OIL CONTROL STRUCTURES

## 1.0 Introduction

This appendix provides information for the design of oil control structures.

## 2.0 Oil Control Structures

Oil control Structures are used to remove free, dispersed and sorbed oil. This type of treatment is most effective in situations where relatively high concentrations of oil and grease occur in storm runoff such as in parking lots, high traffic areas and spill response activities. They do not function well at low concentrations of oil, therefore their use along roads is limited to high traffic highways. Oil control facilities are common pretreatment devices to prevent oil from impairing the function of the primary downstream BMP.

### 2.1 Oil/Water Separators

Oil/water separators are usually underground vaults that take advantage of density differences to separate light oil and dense sediment from stormwater. Oil/water separators are used generally as a pretreatment device to prevent oil from impairing the function of a downstream best management practice (BMP).

Oil/water separators should be used:

1. For vehicle storage and maintenance yards, fueling areas, and high-use parking and rest areas.
2. Upstream of the primary stormwater treatment facility.
3. Off-line to ensure that only the water quality design storm is treated and to prevent flushing by high flow events.
4. Only for treating oil contaminated runoff from impervious areas.

Oil/water separators have several limitations:

1. They should not be used for removal of dissolved or emulsified oils such as coolants and soluble lubricants.
2. They should not be used in situations where detergents are used to cleanse paved areas because such cleansers chemically stabilize oil and prevent flotation.

### 2.1.1 Baffle Type Oil/Water Separators

Baffle type oil/water separators use watertight vaults that have multiple compartments (bays) separated by baffles extending down from the top of the vault. The baffles prevent oil that floats to the surface from flowing out of the separator. Oil that collects at the surface is typically removed by floating skimmers, although more sophisticated mechanical skimmers can be used. Baffles are also installed at the bottom of the vault to trap oily settled material that accumulates.

Key features of a baffle type oil/water separator are shown in Figure 1. The forebay traps and collects sediments, encourages plug flow, and reduces turbulence prior to the oil separation bay. The oil separation bay traps oil as it floats to the surface and also collects any carryover sediment. The treated runoff exiting the oil separation bay enters the afterbay for discharge into the downstream conveyance system.

Flow spreading, sediment-retaining, and oil-retaining baffles are provided; these must be removable unless additional access ports are provided for maintenance.

#### Pollutant Removal Efficiency

Baffle type oil/water separators can meet a performance goal of 10 to 15 milligrams per liter in the treated runoff if designed to remove oil droplets 60 microns and larger.

#### Design Criteria for Baffle Type Oil/Water Separators

This subsection describes the features of baffle type oil/water separators and the design criteria that apply specifically to these installations.

#### Vaults must meet the following criteria:

a) Water depth (D) = 3 to 8 feet to minimize turbulence
b) Width (W) of vault = 6 to 20 feet
c) Depth to width ratio (r) = 0.3 to 0.5
d) Length to width ratio (L/W) = 5:1 minimum
e) Forebay length ($L_f$ ) = 1/3 to 1/2 of vault length
f) Forebay horizontal area must be greater than 20 square feet per 10,000 square feet of impervious drainage area
g) Afterbay length ($L_a$) = 8 feet minimum

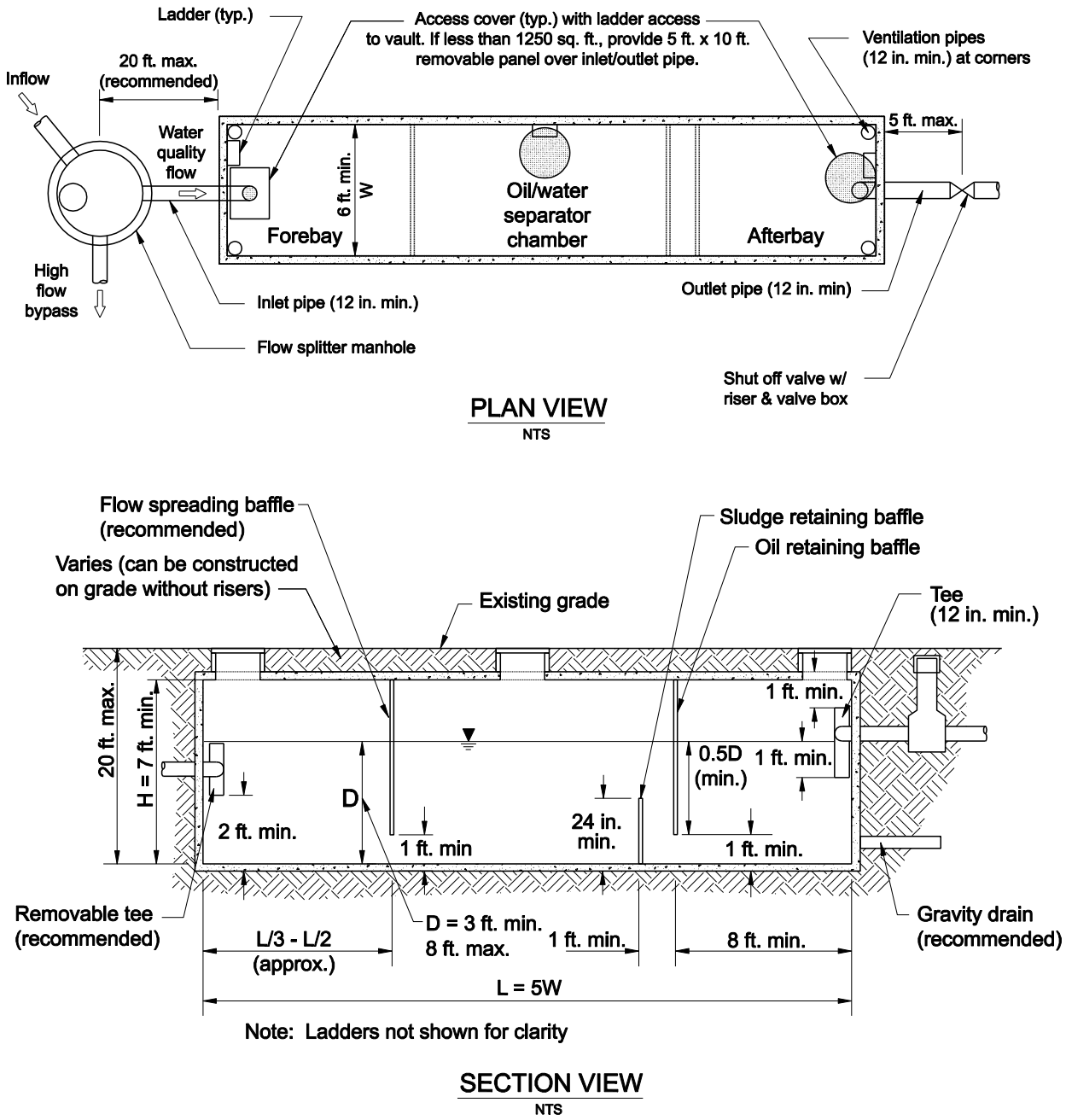


**Figure 1 Baffle Type Oil/Water Separator**

**Vault features**

1. **Oil retaining baffles** (top baffles) must be located at least ¼ of the total separator length from the outlet. They also must extend down at least 50 percent of the design water height and at least 1 foot from the separator bottom.

2. **Ventilation pipes** (minimum 12-inches in diameter or equivalent) must be provided in all four corners of vaults to allow for ventilation for maintenance personnel. Ventilation must also be provided to assure that pressure or vacuum does not occur within the vault due to fluctuations in the water surface elevation. Often this ventilation is provided by the holes in a standard manhole cover or the openings in a grate cover.

3. **Absorbent booms and/or skimmers** must be installed in the afterbay when needed.

4. **Shut-off valve** must be included along the outlet pipe.

5. Provide a **gravity drain** if grade allows for maintenance. The drain invert must be placed 6-inches above the bottom of vault to prevent sediment laden water from escaping when the vault is drained.

**Design Procedure for Baffle Type Oil/Water Separators**

Design steps for sizing baffle type separators are as follows:

**Step 1** – Determine water quality design storm. See Section 14.10.2.

**Step 2 -** Determine Contributing Area. Flow to oil/water separators should come from an area that is almost entirely impervious.

**Step 2a** – Determine the water quality peak flow. Use hydrology guidance in Chapter 7 and the design recurrence interval from step 1.

**Step 3 –** Establish design oil rise rate.

Use 0.033 feet per minute unless an alternative site-specific value is calculated using Stokes law.

**Step 4 –** Determine design horizontal velocity.

The design horizontal velocity is assumed to be uniform within the separator bay. The design horizontal velocity is calculated from the following equation:

$$V_h = 15V_o$$

Where:

$V_h$ = design horizontal velocity in feet per minute
$V_o$ = design oil rise rate in feet per minute

The value (15) is a dimensionless ratio based on American Petroleum Institute (API) criteria. For a design oil rise rate of 0.033-feet per minute, the design horizontal velocity is thus 0.495-feet per minute (0.008-feet per second).

**Step 5 -** Calculate the minimum vertical cross sectional area of the separator using the following equation:

$$A_c = \frac{Q}{V_h}$$

Where:

$A_c$ = minimum cross sectional area in square feet
$Q$ = water quality design flow in cubic feet per second
$V_h$ = design horizontal velocity in feet per second

**Step 6 –** Calculate the water depth using the following equation:

$$D = (rA_c)^{0.5}$$

Where:

$D$ = Water depth in feet
$r$ = Depth-to-width ratio, dimensionless (must be 0.3 to 0.5)

and $A_c$ is determined from Step 5.

**Step 7 –** Calculate the width of the separator vault using the following equation:

$$W = \frac{A_c}{D}$$

Where:

D = Water depth in feet
W = Vault width in feet

and $A_c$ is determined from Step 5.

**Step 8 –** Calculate the minimum length of the separator vault:

$$L_{min} = FD \frac{V_h}{V_o}$$

Where:

$L_{min}$ = minimum vault length in feet
F = turbulence and short circuiting factor, dimensionless

and other variables are defined in Steps 5 and 7.

The turbulence factor is a function of $V_h/V_o$ and varies from approximately 1.30 to 1.75. For a $V_h/V_o$ of 15, F has a value of 1.65. See reference 4 for additional information. Thus, the above equation reduces to the following:

$$L_{min} = 24.75D$$

**Step 9 –** Check the vault length-to-width ratio.

In order to minimize hydraulic disturbances from the forebay and afterbay, the vault length must be at least five times its width (W).

**Step 10 –** Select the length of the forebay and afterbay.

The length of the forebay should meet the following condition:

$$L_f = \frac{L}{3} \text{ to } \frac{L}{2}$$

Where:

$L_f$ = length of forebay in feet

L = length of vault in feet (L is greater than or equal to $L_{min}$)

The length of the afterbay should meet the following condition:

$$L_a = \frac{L}{4}$$

Where:

$L_a$ = length of the afterbay in feet

The minimum afterbay length ($L_a$) is 8 feet.

**Step 11 –** Calculate the horizontal surface area ($WL_f$) of the vault forebay.

The forebay horizontal area must be greater than 20 square feet per 10,000 square feet of impervious drainage area. The length of the forebay may be increased without having to increase the overall length of the vault.

**Step 12 –** Confirm that all dimensions meet the required criteria.

### 2.1.2 Coalescing Plate Oil/Water Separators

Coalescing plate oil/water separators use watertight vaults that have multiple compartments or bays. These units are commercially available from several manufacturers. The reader is referred to the various manufacturers for more detailed information on coalescing plate oil/water separators.

The key features of a coalescing plate oil/water separator are shown in Figure 2. The forebay traps and collects sediments, encourages plug flow, and reduces turbulence prior to the oil separation bay. The oil separation bay contains a series of inclined plates (plate pack) to separate oil from the stormwater runoff. These plates are typically inclined from the horizontal 45 to 60 degrees. The treated runoff exiting the oil separation bay enters the afterbay for discharge into the downstream conveyance system.

Oil removal occurs as stormwater flows horizontally through a coalescing plate separator and encounters the plate pack. As oil rises toward the surface, it contacts the underside of one of the inclined plates and accumulates on the surface. Eventually, the accumulation becomes thick enough to migrate along the surface of the plate. When it reaches the edge of the plate, it breaks free and rises rapidly as a large droplet to the surface of the unit. This coalescing of small oil droplets into larger ones increases treatment effectiveness; thus, vault size is smaller than that of

a baffle type separator.

Oil removal is a function of the sum of the horizontal projection of all the plates in the pack (total plan area of the plates). Design of a coalescing plate separator is based on determining the total plan area of the plate pack needed to remove oil of a specified droplet size for a specific design flow. Because plate packs are manufactured items, vault sizing depends on the specific design features of the plate pack.

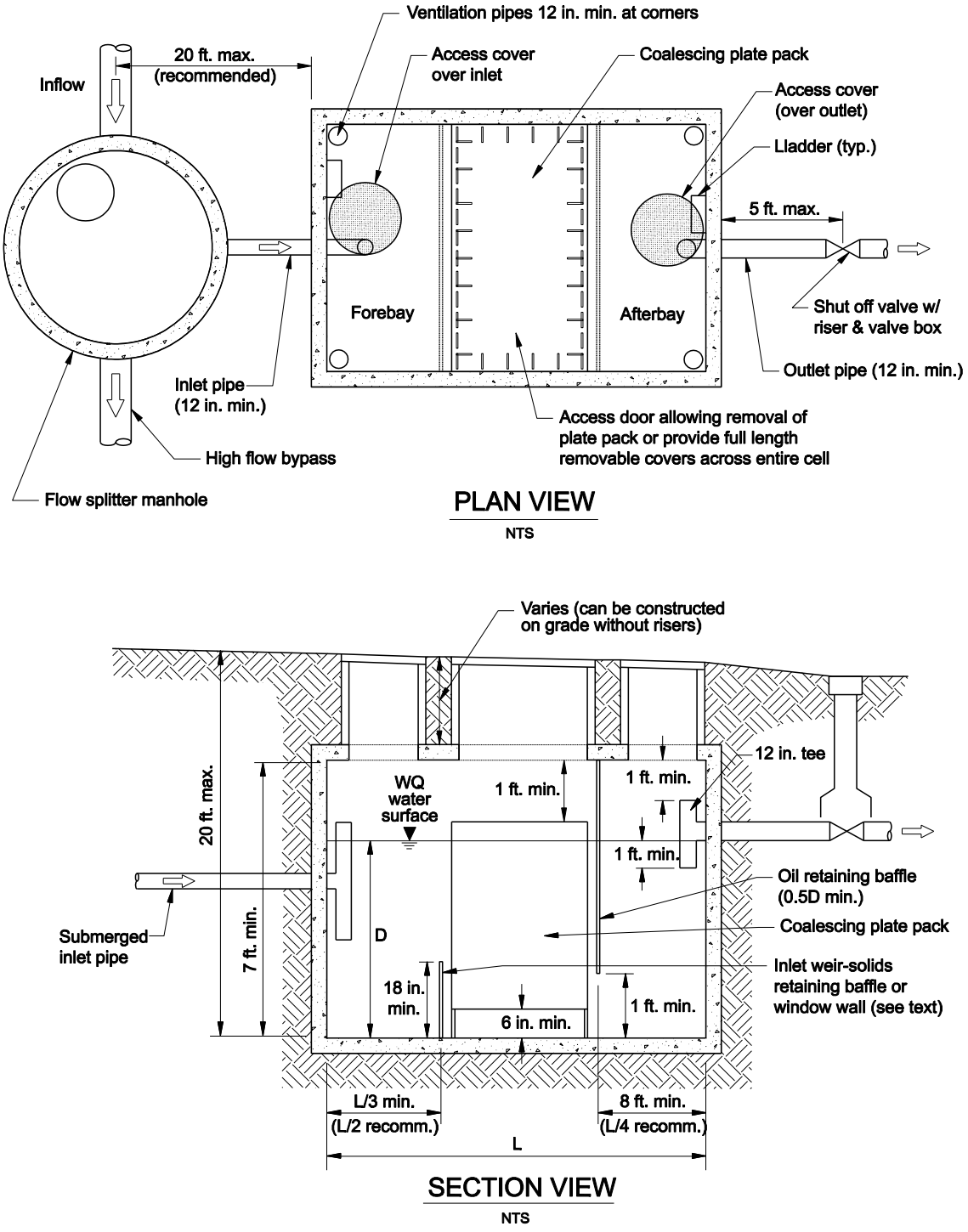


**Figure 2 Coalescing Plate Oil/Water Separator**

**Pollutant Removal Capabilities**

Typically, oil/water separators can meet a performance goal of 10 to 15 milligrams per liter in the treated runoff if designed to remove oil droplets 60 microns and larger.

**Design Criteria for Coalescing Type Oil/Water Separators**

This subsection describes the features of coalescing type oil/water separators and the design criteria that apply specifically to these installations.

1. Vaults must meet the following criteria:

    a) Forebay length ($L_f$ ) = 1/3 to 1/2 of vault length
    b) Forebay horizontal area must be greater than 20 square feet per 10,000 square feet of impervious drainage area
    c) The geometry and dimensions of the plate pack will depend on manufacturer's specifications.
    d) Afterbay length ($L_a$) = 8 feet minimum

**Vault features**

1. **Plate pack** shall be made of fiberglass, stainless steel or polypropylene. The spacing between the plate pack and vault sidewalls must be packed with a light weight removable material such as plastic or polyethylene foam to minimize short circuiting. Rubber flaps are not acceptable.

2. **Ventilation pipes** (minimum 12-inches in diameter or equivalent) must be provided in all four corners of vaults to allow for ventilation for maintenance personnel. Ventilation must also be provided to assure that pressure or vacuum does not occur within the vault due to fluctuations in the water surface elevation. Often this ventilation is provided by the holes in a standard manhole cover or the openings in a grate cover.

3. **Absorbent booms and/or skimmers** must be installed in the afterbay when needed

4. **Shut-off valve** must be included along the outlet pipe.

**Design Procedure for Coalescing Type Oil/Water Separators**

Basic steps are as follows:

**Step 1** – Determine water quality design recurrence interval. Runoff collected by the upstream storm drain system needs to address the most stringent standards or reference ODOT's requirements summarized in Section 14.10.2.

**Step 2 -** Determine contributing area.

**Step 2a –** Determine hydrology. The water quality peak flow is needed to design an oil/water separator. Use hydrology guidance in Chapter 7 and the design recurrence interval from step 1.

**Step 3 –** Coordinate the plate pack geometry and dimensions with the various manufacturers of this feature.

**Step 4 –** Determine the length of the forebay and afterbay.

The length of the forebay can be calculated using:

$$L_f = \frac{L}{3} \text{ to } \frac{L}{2}$$

Where:

$L_f$ = length of forebay in feet
$L$ = length of vault in feet

The length of the afterbay can be calculated using:

$$L_a = \frac{L}{4}$$

Where:

$L_a$ = length of the afterbay in feet

The minimum afterbay length ($L_a$) is 8 feet.

**Step 5 –** Calculate the horizontal surface area ($WL_f$) of the vault forebay.

The forebay horizontal area must be greater than 20 square feet per 10,000 square feet of impervious drainage area. The length of the forebay may be increased without having to increase the overall length of the vault.

# CHAPTER 15

# BANK PROTECTION

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## 15.1 Introduction

One of the hazards of placing a highway near a river or other water body is the potential for erosion of the highway embankment by moving water. If erosion of the highway embankment is to be prevented, bank protection must be anticipated and the proper type and amount of protection must be provided in the right locations. Embankment stabilization may be designed and constructed during the initial roadway project where the need is obvious or the risk of damage is high. There will be other locations where economic considerations, the availability of materials, and the probability of damage are such that construction of embankment stabilization measures can be delayed until a problem actually develops.

Five methods of protecting a highway embankment from bank erosion are available to the designer. These are:

- locate the highway away from the stream or water body initially,
- relocate the highway away from the stream or water body,
- move the river or stream channel away from the highway (channel change),
- change the direction of the current with training works, or
- protect the highway embankment from erosion.

This chapter provides procedures for the design of revetments to be used as channel bank protection and channel linings on larger streams and rivers (i.e., having design discharges generally greater than 50 cubic feet per second). In addition to revetments, design procedures are also presented for stream barbs, impermeable deflector spurs, and riprap protection at bridge piers and abutments. This chapter also provides guidance on biotechnical bank protection. For discharges less than 50 cubic feet per second, procedures presented in **Chapter 8** should be used.

## 15.2 Policy

Bank protection should satisfy the policies that are listed in **Chapter 3**.

## 15.3 Erosion Potential

Channel and bank stabilization is essential to the design of any highway affected by the water environment. The identification of the potential for bank erosion, and the subsequent need for channel stabilization or highway embankment protection, is best accomplished through observation. Observations are usually based on current site conditions supplemented with historic information. Documentation of channel movement or bank instability can be obtained from aerial photographs, old maps, surveying notes, bridge maintenance files, river survey data that are available at State and Federal agencies, gage station records and interviews of long-time residents. Additional discussion about data collection is presented in **Chapter 6**.

Even when historic information indicates that a bank has been relatively stable in the past, local conditions may indicate more recent instabilities. Local site conditions that are indicative of instabilities may include:

- tipping and falling of vegetation along the bank,
- cracks along the bank surface,
- the presence of slumped material due to an instability condition at or near the bank surface that causes transverse movement of a soil mass down the stream bank,
- fresh vegetation laying in the channel near the channel banks,
- deflection of channel flows in the direction of the bank due to some recently deposited obstruction or channel course change,
- fresh vertical face cuts along the bank,
- locally high velocities along the bank,
- new bar formation downstream from an eroding bank,
- local headcuts (refer to **Chapter 9** for headcut description),
- landslides,
- toe erosion, and
- pending or recent cutoffs. A cutoff is a channel formed naturally or artificially that shortens the length of a stream; typically occurs across the neck of a meander loop or across a point bar.

It is also important to recognize that the presence of any one of these conditions does not in itself indicate an erosion problem; some bank erosion is common in all channels even when the channel is stable.

## 15.4 Design Considerations

This section presents factors that need to be considered for the bank protection techniques presented in this chapter.

### 15.4.1 Stream Stability

Stream stability can affect the stability of the method chosen to provide bank protection. Therefore, it is important to evaluate the stream's stability as part of the design process for bank protection. A stable channel or graded stream exists when a stream has achieved a bed slope and cross-section that allows its channel to transport water and sediment delivered from the upstream watershed without aggradation, degradation, or bank erosion.

Factors that affect stream stability can be classified as geomorphic factors and hydraulic factors. Geomorphic factors include stream size, flow habit (i.e., ephemeral or perennial), composition and erodibility of bed and bank materials, geologic controls, floodplain characteristics (i.e., narrow or wide), planform characteristics (i.e., straight, meandering, braided, or anabranched), and other features such as natural levies, channel incision, and riparian vegetation. Hydraulic factors include flow resistance, bank and bed shear stresses, flow velocities, flow depths, magnitude and frequency of floods, and flow classification (e.g., unsteady, nonuniform, turbulent, supercritical or subcritical).

Geomorphic characteristics of particular interest are the alignment, geometry, and form of the stream channel. The behavior of the stream at the site of bank instability depends not only on the local conditions at the site, but also on the behavior of the overall stream system. Upstream and downstream changes may affect future stability at the site. These changes can be reflected in aggradation, degradation, or lateral migration of the stream channel. Changes can occur in streams in response to human activities in the watershed and/or natural disturbances of the fluvial system. For example human-induced changes in the watershed such as changes in land use (i.e., increase in impervious area), gravel mining, streamside levees and dikes, reservoirs, and channel straightening can have major effects on the hydrology and sediment yield and transport, which can effect the channel geometry and planform characteristics. Natural disturbances such as floods, drought, landslides, forest fires, etc., may also result in large changes in sediment load in a stream and major changes in the stream channel.

Either a qualitative or quantitative analysis can be used to evaluate stream stability. As an example, a qualitative study would describe critical areas and the problems encountered while a quantitative study would provide specific numeric data such as rate of erosion, etc. As a minimum, a qualitative analysis should be performed. The qualitative analysis should include a

site visit and evaluation of historic information described in Section 15.3. The conclusions of the qualitative analysis, along with the risk to highway safety associated with the stream instability, will determine whether a more detailed quantitative analysis is necessary. For a more detailed discussion of stream stability and analysis techniques, the reader is referred to References 1 and 2.

The effect of the proposed bank protection on the stream's stability also needs to be evaluated and the design should be modified as necessary to minimize the negative impacts. For example spurs that project too far into the channel may deflect the flow to the opposite bank and cause the bank to erode, or a design that constricts the channel may cause excessive scour that can destroy spawning beds or result in aggradation downstream.

The outcome of the stream stability analysis is that it will allow the designer to select a design that is the most compatible with overall stream instability and that provides the required bank protection. The goal of the project will be to protect the highway facility and will usually not include a solution to the identified stream instability; however, in some cases the stream instability may need to be addressed in order for the bank protection to be successful. The project should be monitored after construction to ensure that the design is performing as intended and also to identify any unexpected impacts caused by the project. The project limits will usually be confined to highway right-of-way.

### 15.4.2 Fish Habitat and Mitigation

Impact on fish habitat is an important consideration in the design of bank protection. The first priority of natural resource agencies in reviewing a stream bank project is to avoid habitat impacts. The second priority is minimizing habitat impacts. If negative impacts to habitat cannot be avoided, then the third priority is mitigation of impacts. Therefore, prior to the design, the Region Environmental Coordinator should be contacted. They will name the environmental personnel working on the project. The environmental personnel will contact the appropriate natural resource agencies, which are usually Oregon Department of Fish and Wildlife and the National Oceanic and Atmospheric Administration Fisheries Service. These agencies may require special design features, such as burying logs with rootwads in the toe of riprap revetments, minimizing the use of riprap, or maximizing the use of vegetation in the design.

### 15.4.3 Design Event Discharge

The appropriate recurrence intervals discussed in **Chapter 3** should be used for design. There may be instances where a lower discharge than the design event discharge might produce hydraulically worse conditions with respect to bank stability. Therefore, it is suggested that the

design be evaluated for a range of event discharges to ensure that the proposed design will provide adequate protection.

### 15.4.4 Scour in Channel Bends and Straight Sections

A large percentage of bank erosion problems occur at channel bends because they are naturally unstable. Flow conditions in channel bends are complicated by the distortion of flow patterns that develop in the vicinity of the bend. In long, relatively straight channels, flow conditions are uniform and symmetrical about the centerline of the channel. In channel bends the centrifugal forces and secondary currents produced cause non-uniform and non-symmetrical flow conditions.

The increased velocities and shear stresses that are generated as a result of non-uniform flow conditions in bends are difficult to assess quantitatively, but need to be considered when designing bank protection. When designing riprap using the equations developed from tractive force theory presented in Subsection 15.5.1.7.2, an increased riprap stability factor is used to account for the increased shear stress in a bend. For the velocity based riprap design procedure presented in Subsection 15.5.1.7.3, the velocity is increased 33 percent to account for the uncertainties associated with channel bends. **Chapter 8** presents a method to calculate the shear stress in straight channels as well as channel bends. It is the method used to estimate the shear stress when selecting an erosion control fabric for biotechnical stabilization discussed in Subsection 15.5.7.3.

Superelevation of flow in channel bends can sometimes be an important consideration in the design of revetments. Although the magnitude of superelevation is generally small, it should be considered when establishing freeboard limits for bank protection designs on sharp bends, particularly when the design event is contained within the channel. **Chapter 8** presents an equation for estimating superelevation for subcritical flow in channel bends. Superelevation does not need to be considered at sites where design event exceeds the top of bank elevation.

Scour that occurs at the toe of the outer bank of a channel bend also needs to be considered in the design of revetments. Estimates of the potential scour depth are needed to determine how deep to extend the toe protection to prevent undermining. Experience is usually the most reliable means of estimating scour depth when designing a bank protection project for a particular stream. Lacking experience on a particular stream, scour depths may be estimated using physically based analytical models or empirical methods. Empirical methods generally provide better agreement with observed data.

Maynord developed an empirical method for estimating scour depth in a channel bend. The following equation is adopted from his procedure. The reader is referred to Reference 3.

$$\mathbf{D_{mxb}} = \{\ \mathbf{1.8} - \mathbf{0.051}(A) + \mathbf{0.0084}(B)\ \}\mathbf{2D_{mnc}} \qquad \textbf{(Equation 15-1)}$$

Where:

$D_{mxb}$ = Maximum computed water depth in the bend in feet. See Figure 15-1.
$D_{mnc}$ = Average water depth in feet in the waterway upstream from the bend, as shown in Figure 15-1.
A = $(R_c/W)$ if $1.5 \leq (R_c/W) \leq 10$
A = 1.5 if $(R_c/W) < 1.5$,
B = $(W/D_{mnc})$ if $20 \leq (W/D_{mnc}) \leq 125$
B = 20 if $(W/D_{mnc}) < 20$
$R_c$ = Bend radius in feet, measured from radius center to channel centerline
W = Channel width in bend in feet, as shown in Figure 15-1

Do not use Equation 15-1 if $(R_c/W) > 10$, $(W/D_{mnc}) > 125$, or overbank depth is greater than 20 percent of main channel depth.

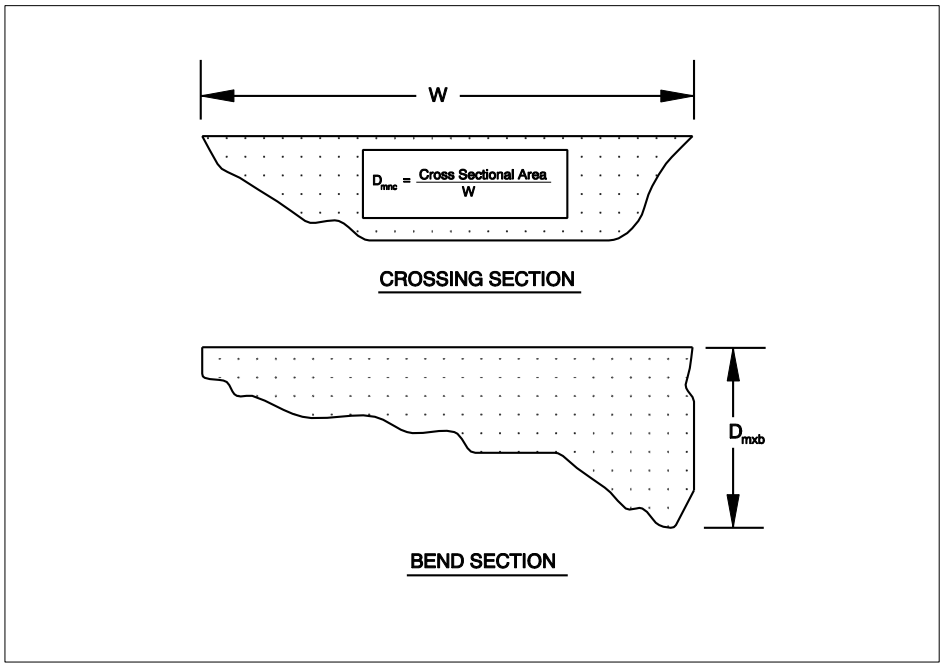



**Figure 15-1 Channel Bend Scour**

The equation is valid for sand and gravel streams with naturally developed widths and depths. It is not valid for channels with artificially confined widths less than a natural channel. The equation can be applied to overbank conditions, where the overbank depth is less than 20 percent of the main channel depth. The main channel depth is defined as the average depth within the bed and banks that confine the flow of surface water during the design event. The overbank depth is defined as the average depth of the overbank area beyond the top of the main channel bank during the design event.

An alternate method of calculating scour in mild bends (i.e., curve radius/channel width > 30) and straight channels has been developed by Blodgett[4] and the following equations are adopted from his procedure. The reader is referred to reference 4 for more information.

$$\mathbf{d_{s(max)} = 6.5D_{50}^{-0.115} \quad for\ D_{50} > 0.005\ feet} \qquad \textbf{(Equation 15-2)}$$

Where:

$d_{s(max)}$ = estimated probable maximum scour depth in feet
$D_{50}$ = median diameter of bed material in feet
If $D_{50}$ is less than 0.005 feet, $d_s$ equals 12 feet.

The $D_{50}$ used in Equation 15-2 is the gradation of the underlying materials at scour depth. It is not the $D_{50}$ for the coarser armoring layer. Use geological exploration as needed to look at deeper materials below armor layer.

Blodgett[4] developed Equation 15-3 for estimating the mean scour depth, $d_{s(mean)}$. The following equation is adopted from his procedure.

$$\mathbf{d_{s(mean)} = 1.42D_{50}^{-0.115}} \qquad \textbf{(Equation 15-3)}$$

Equations 15-2 and 15-3 calculate the scour depth below the lowest surveyed point in the cross-section. It should be assumed that the low point in the cross-section might eventually migrate adjacent to the bank protection, even if this is not the case in the current channel survey. The depth of scour predicted by equations 15-2 and 15-3 must be added to the magnitude of predicted degradation and local scour, if any, to arrive at the total scour depth. When equations 15-2 and 15-3 are used to design the toe protection, the toe protection should at least be designed to tolerate a scour depth that corresponds to $d_{s(mean)}$. Equation 15-2 represents an enveloping curve for the upper limit of observed scour and consequently may over predict the scour depth, therefore, judgment should be used to determine if the toe protection should be designed to tolerate a scour depth that corresponds to $d_{s(max)}$.

### 15.4.5 Extent of Protection

Extent of protection refers to the longitudinal and vertical extent of protection required to adequately protect the channel bank. It is also important for the stability of the selected bank protection scheme.

#### 15.4.5.1 Longitudinal Extent

The longitudinal extent of protection required for a particular bank protection scheme is highly dependent on local site conditions. In general, the bank protection scheme should be able to protect the bank for a distance greater than the length that is impacted by channel-flow forces severe enough to cause dislodging and/or transport of bank material. Investigators of field installations of bank protection have found that bank protection commonly extends farther upstream than necessary and not far enough downstream.

Figure 15-2 was developed from U.S. Army Corps of Engineers studies of the extent of protection required at meander bends, and it can be used as a guide when determining the length of bank to be protected. The minimum length of bank that should be protected is the length of bank that is observed to be actively eroding. A longer bank length may need to be protected if it is judged that additional bank is vulnerable to erosion if left unprotected based on local site conditions, particularly on the downstream side of the active erosion. The limit of protection on the downstream end should depend on where the flow crosses to the opposite bank, and should consider future bar building on the opposite bank, resulting in channel constriction and increased velocities. A review of aerial photographs along with the site inspection can provide some insight on where the crossover flow occurs. Geomorphic studies to determine the limits of protection should be considered. Environmental concerns may play a role in determining the limits of protection, particularly when the design proposes disturbing portions of the bank that are beyond the limits of the observed active erosion. Wherever possible, the bank protection should tie into stable anchorage points, such as bridge abutments, rock outcrops, or well-vegetated, stable channel sections. When a stable feature does not exist, flank protection as described in Subsection 15.5.1.6 should be considered.

*Note: Eroding streambanks frequently affect both the agency and neighboring landowners. ODOT funds are used only to protect ODOT facilities. Often the training structures or embankment protection needed to do this extends, or is built, on neighboring properties. This is done after obtaining the properties, drainage easements, cooperative agreements, or other suitable means.*

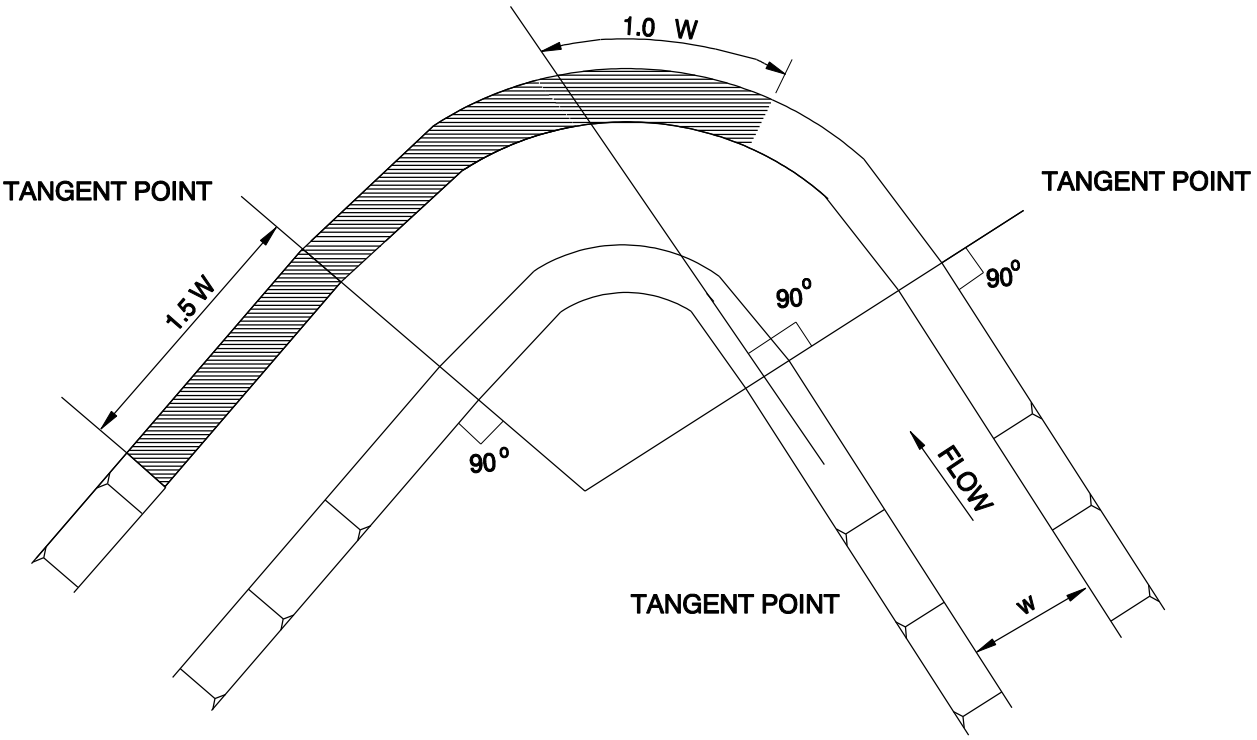



**Figure 15-2 Extent of Protection at a Channel Bend**

### 15.4.5.2 Vertical Extent

Embankment protection extends from the bottom of the toe upward to the top of the protected surface. These elevations are often based on the design and check floods listed in **Chapter 3**. Design practices for various embankment types follow. These are general statements. Specific guidance is in the ODOT structural and foundation manuals. Bridge abutment scour protection is discussed in **Chapter 10**.

#### 15.4.5.2.1 Lower Limit

The lower limits for various embankment protection methods follow.

**Pile supported walls, drill shaft supported walls, sheet pile walls, and other walls where lateral stability depends on vertical embedment into the underlaying soil and rock -** These

walls are at a minimum, designed to withstand hydraulic forces and scour that occurs during the design event, and they are verified to be stable during the check flood. Other stability issues also need to be addressed, such as seismic loading.

The pile or shaft tips may extend down well below the check flood elevation to provide stability. The lower edges of the wall faces extend at least down to the check flood scour elevation. This protects the embankment toe during large floods.

Riprap, articulated concrete block mats, jacks, or other suitable means can be used to reduce scour depths. The protection methods are designed using the design event and checked for stability during the check flood.

**Gabion walls and other gravity walls, footing supported walls, and reinforced earth walls –** These walls, at a minimum, are designed to withstand the hydraulic forces and scour depth associated with the design event, and it is verified they will be stable during the check flood. The bases of the wall or the bottoms of the footings are at or below the check flood elevation. The tops of the footings, if footings are used, are at or below the design event elevation. At a minimum, the bottoms of the footings or wall base are to be at least 6 feet below the channel bottom elevation unless they are keyed into non-erodible rock.

Potential scour depths can be reduced by use of riprap, blocks, or other means as mentioned for pile supported walls.

**Riprap revetment and riprap toes for other wall types –** The lower limits of riprap embankment protection are discussed in Subsection 15.5.1.5.

#### 15.4.5.2.2 Upper Limit

The upper limit for embankment protection is typically the design event elevation plus 1 foot for freeboard. Freeboard should be increased at sites subject to wave action generated from boat traffic or wind. Freeboard should also include the superelevation in channel bends. Superelevation can be estimated by methods in **Chapter 8**. Bank protection should extend to the top of the bank at sites where the design event elevation exceeds the top of bank elevation.

The embankment protection design should also be analyzed using the check flood. A higher top of protection elevation may be warranted in critical areas where damage cannot be tolerated.

## 15.5 Design Guidelines

The following sections present design guidelines for various techniques that are used for bank protection.

### 15.5.1 Rock Riprap

This subsection contains design guidelines for the design of rock riprap revetment embankment protection, and bridge abutment and pier protection. Guidelines are provided for bank slope, rock size, rock gradation, durability, riprap layer thickness, filter design, and edge treatment. In addition, typical construction details are illustrated and examples are provided. Additional information on structure revetment, including details, is in **Chapter 10**, Bridges.

#### 15.5.1.1 Bank Slope

A primary consideration in the design of stable riprap bank protection is the slope of the channel bank. Two items need to be considered. First is the stability of the riprap face. The second is the stability of the embankment behind the riprap.

Normally, the maximum recommended riprap face slope on an engineered embankment is 1V: 1½H. Engineered embankments are typically bridge abutments and roadway embankments made from materials complying with ODOT requirements and constructed according to practices in ODOT specifications. This includes placing the fill in layers, compaction in layers, etc.

Slopes made of natural materials placed by geologic processes are a different situation. Examples are hillsides and streambanks. The steepest slopes these materials may withstand may be greater or less than 1V: 1½H. It is recommended that an engineer or other qualified person familiar with slope stability determine the stability of non-engineered slopes. This is especially important if the slope supports a structure or roadway.

It is important to determine the cause of the bank failure prior to design, and to address the problem in the design. Experience has shown that streamside embankment failures are often caused by factors other than riverine scour. An example is a failure of a cross-culvert under the roadway. Water percolating into the embankment from a plugged or perforated culvert can weaken the embankment and cause failure.

The stability of the riprap itself limits the maximum recommended embankment face slope to 1V: 1½H. Although not recommended, riprap slopes as steep as 1V: 1H have been successfully constructed. These were done on maintenance repairs, and these steep slopes are strongly

discouraged on new designs. The successful riprap installations on overly steep slopes almost always:

- used larger rocks and greater layer thicknesses than an embankments with flatter slopes would need to withstand identical wave attacks or lateral velocities,
- were founded on a deep and stable toes (this is especially critical),
- were placed by methods other than dumping, such as carefully fitting the individual rocks into the revetment layer, one-at-a-time, with an excavator, and
- used extremely angular rock such as basalt blasted from a quarry face.

### 15.5.1.2 Rock Properties

The properties of the riprap rock are important if the installation is to perform as designed. The riprap must be stable when subjected to hydraulic forces. Riprap particle stability is a function of its density, its size expressed either in terms of its weight or equivalent spherical diameter, and its shape. The rock must be durable. The requirements in the *ODOT Standard Specifications for Construction* are adequate for most installations. Occasionally a special specification will be needed, and it is included in the project supplemental specifications.

**Density –** Rock to be used for revetment must be dense enough to resist displacement due to hydraulic forces. The most commonly used stone in Oregon is crushed basalt from quarries. Occasionally other rocks are used. The specific gravities for various rock types are listed in Table 15-1.

A density of 165 pounds per cubic foot and a specific gravity of 2.65 are almost always used in design. These values represent the typical riprap rock. The actual rock density should be used in the design if it is known and it is significantly different than 2.65. Most of the revetment design procedures have adjustment methods to accommodate different rock densities. The minimum allowable specific gravity for riprap is 2.50.

**Equivalent Spherical Diameter -** Revetment riprap stones are rarely round in shape. The properties of a hypothetical round stone are used to simplify calculations. The diameter of these rocks is called the "equivalent spherical diameter." The following equation can be used to calculate the stone weight for rock that weights 165 pounds per cubic foot:

$$\mathbf{W = 691.15\left(\frac{D}{2}\right)^3} \qquad \textbf{(Equation 15-4)}$$

**Table 15-1 Specific Gravities of Typical Oregon Riprap Stone**
(Based on a 62.4 pound per cubic foot water density.)

| Rock Type | Specific Gravity Range of Typical Rock |
|---|---|
| Andesite | 2.5 – 2.8 |
| Basalt/Traprock | 2.8 – 3.0 |
| Diabase | 2.6 – 3.0 |
| Diorite | 2.8 – 3.0 |
| Gneiss | 2.6 – 2.9 |
| Granite | 2.6 – 2.7 |
| Limestone | 2.3 – 2.7 |
| Marble | 2.4 – 2.7 |
| Quartzite | 2.6 – 2.8 |
| Rhyolite | 2.4 – 2.6 |
| Sandstone | 2.2 – 2.8 |

Lighter extrusive igneous rocks such as tuff often lack sufficient density. Dolomite, mica, schist, shale, and slate have undesirable properties besides density, and they are not used for riprap.

Where:

W = stone weight in pounds, and
D = equivalent spherical diameter of stone in feet.

Equation 15-4 can be rearranged to solve for the equivalent spherical diameter of the stone as follows:

$$\mathbf{D = 2\left(\frac{W}{691.15}\right)^{1/3}} \qquad \textbf{(Equation 15-5)}$$

Riprap revetment is composed of individual rock particles that vary in size. The gradation is the distribution of the particle sizes, either by size or weight. Table 15-2 lists the gradations in pounds for ODOT standard riprap classes. Other riprap specifications can be found in the ODOT *Standard Specifications for Highway Construction.*

**Table 15-2 Gradation of Standard Riprap Classes**

| Percent by Weight | Class 50 | Class 100 | Class 200 | Class 700 | Class 2000 |
|---|---|---|---|---|---|
| 20 | 50 – 30 | 100 – 60 | 200 – 140 | 700 –500 | 2000 – 1400 |
| 30 | 30 – 15 | 60 – 25 | 140 – 70 | 500 – 200 | 1400 – 700 |
| 40 | 15 – 2 | 25 – 2 | 70 – 7 | 200 – 20 | 700 – 40 |
| 10 – 0 | 2 – 0 | 2 – 0 | 7 – 0 | 20 – 0 | 40 –0 |

The properties of the median stone size are frequently used in design. This is the size of rock where 50 percent of the total weight of the riprap is smaller than the median stone size, and the other 50 percent are larger. Both the equivalent spherical diameter, $D_{50}$, and the weight, $W_{50}$, are used. Table 15-3 lists the $D_{50}$, $W_{50}$, $D_{100}$, and $W_{100}$ for ODOT standard riprap classes. $D_{100}$ and $W_{100}$ are listed in the table. All stones within the respective classes are equal to, or less, than these equivalent diameters or weights.

**Table 15-3 $D_{50}$, $W_{50}$, $D_{100}$, and $W_{100}$ for Standard Riprap Classes**

| Standard Riprap Class | $D_{50}$ (feet) | $W_{50}$ (lbs) | $D_{100}$ (feet) | $W_{100}$ (lbs) |
|---|---|---|---|---|
| Class 50 | 0.56 | 15 | 0.83 | 50 |
| Class 100 | 0.66 | 25 | 1.05 | 100 |
| Class 200 | 0.93 | 70 | 1.32 | 200 |
| Class 700 | 1.32 | 200 | 2.01 | 700 |
| Class 2000 | 2.01 | 700 | 2.85 | 2000 |

Occasionally riprap may be needed of a non-standard gradation class. The class designation is based on the largest stone size. As an example, the largest stone in Class 3000 riprap would

weigh 3000 pounds. Table 15-4 can be used to develop the gradation limits of any riprap class. $D_{50}$ and $W_{50}$ are the equivalent spherical diameter and weight of the median stone size.

**Table 15-4 Riprap Gradation Limits**

| Stone Size (feet) | Stone Weight (lbs) | Percent of Gradation Smaller Than |
|---|---|---|
| $1.44D_{50}$ to $1.71D_{50}$ | $3W_{50}$ to $5W_{50}$ | 100 |
| $1.26D_{50}$ to $1.40D_{50}$ | $2W_{50}$ to $2.75W_{50}$ | 80 |
| $1.00D_{50}$ to $1.14D_{50}$ | $W_{50}$ to $1.5\ W_{50}$ | 50 |
| $0.46D_{50}$ to $0.58D_{50}$ | $0.1W_{50}$ to $0.2W_{50}$ | 10 |

**Shape –** Riprap rock should be angular and blocky in shape. This helps the rocks to interlock with each other and resist displacement from gravitational and hydraulic forces. The ratio of the maximum to minimum diameters is an important aspect of shape. The maximum diameter is the longest dimension of the rock, and it is often termed the "length." The minimum diameter is the shortest dimension, and it is often called the thickness. ODOT riprap specifications require that the thickness be not less than one third of the rock length.

Large rounded rocks are often readily available at project sites. These are usually stones removed during roadbed excavation and leveling. These rocks are often used for habitat enhancement. They are placed along the bank and in the stream to provide shelter for fish. Sometimes they are placed at the toe of riprap slopes to provide habitat. When this is done, the habitat rocks should be much larger than the size of riprap used in the embankment. This larger size will help keep the habitat rocks from rolling downstream during flood flows. Rounded rocks should not be used for revetment unless authorized by the engineer.

**Durability -** The riprap rock must be durable. The rock types listed in Table 15-1 will often meet durability requirements. A significant exception is marine basalt, and it should not be used for riprap.

Marine basalt is an igneous rock extruded from the earth's crust into clays laying on the ocean floor. It has more frequent internal seams than basalt extruded onto land. Often the seams are filled with clay. The clay expands when it contacts water and this fractures the rock. Often the

**a) Strata**

**b) Seams and pillow structure**

**Figure 15-3 Marine Basalt**

rock fractures into pieces of only a few inches in size. The damaged rock no longer has the desired gradation and it is susceptible to displacement by hydraulic forces.

Marine basalts occur more frequently in western Oregon than the rest of the state. They can be identified by the many clay filled seams within the rock. Occasionally the "pillow" structure is evident from the underwater extrusion. Figure 15-3a shows marine basalt strata. Figure 15-3b shows a closer view. The pillow structure and seams are evident.

### 15.5.1.3 Layer Thickness "T"

The riprap layer thickness "T" should not be less than the spherical diameter of the $D_{100}$ stone or less than 1.5 times the equivalent spherical diameter of the $D_{50}$ stone, whichever results in the greater thickness. Table 15-5 lists the layer thickness for each ODOT standard riprap class. The riprap thickness should be increased 50 percent when the riprap is placed underwater to account for uncertainties associated with submerged placement.

**Table 15-5 Riprap Layer Thickness "T" for Standard Riprap Classes**

| Standard Riprap Class | Layer thickness "T" (feet) |
|---|---|
| Class 50 | 1.0 |
| Class 100 | 1.5 |
| Class 200 | 2.0 |
| Class 700 | 3.0 |
| Class 2000 | 4.0 |

### 15.5.1.4 Riprap Backing

Riprap backing is either a riprap geotextile or a granular filter blanket placed between the riprap and underlying soil. The riprap backing acts as a filter and prevents the migration of fine soil particles through voids in the riprap. Table 15-6 lists the riprap backing requirements for each ODOT standard riprap class. Riprap backing is not required if the underlying soil meets the gradation requirements of the granular filter blanket.

**Table 15-6 Riprap Backing for Standard Riprap Classes**

| Standard Riprap Class | Granular Filter Blanket | Riprap Geotextile |
|---|---|---|
| Class 50 | Not required | Not required[1] |
| Class 100 | Not required | Not required[1] |
| Class 200 | 6 inch layer of well graded 4″ – 0 stone embankment | Type 2 |
| Class 700 | 9 inch layer of well graded 6″ – 0 stone embankment | Type 2 |
| Class 2000 | 16 inch layer of Class 50 riprap | Type 2 |

[1]A Type 2 riprap geotextile is recommended when the riprap is placed on a fine-grained, cohesionless soil such as sand.

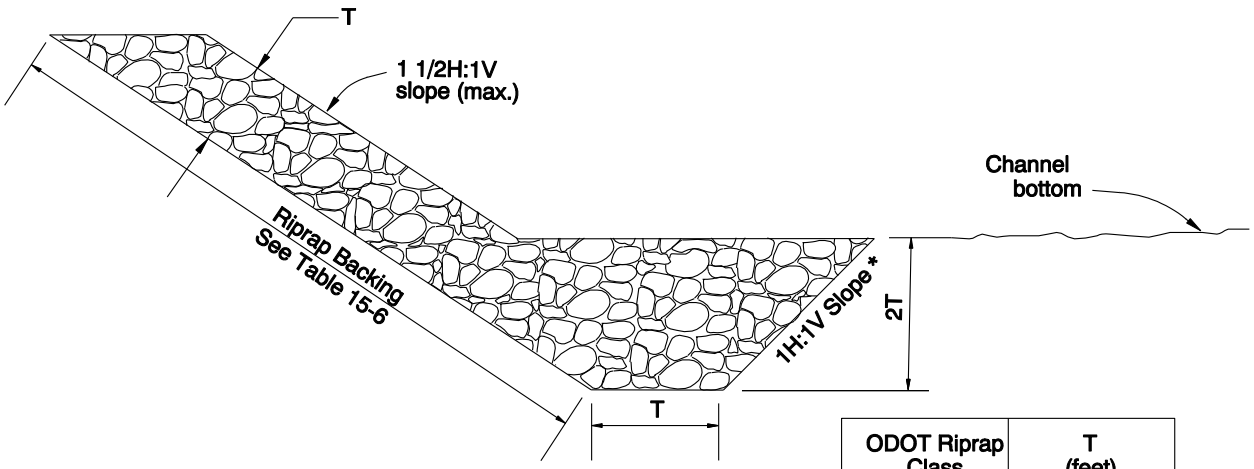
Stone embankment material that is used as a granular filter blanket shall be an unweathered, hard, durable, free draining rock. Specifications for riprap backing, stone embankment material, and riprap geotextiles are in the *ODOT Standard Specifications for Construction*.

### 15.5.1.5 Standard and Modified Riprap Sections

The majority of the riprap used in stream embankment and spillthrough bridge opening protection is placed according to the standard riprap section, as shown in Figure 15-4. The standard section includes rock filled toe trench. The toe provides a stable foundation for the revetment on the embankment face.

Undermining of toe protection has been identified as one of the primary mechanisms of riprap revetment failure. This is not only true for riprap, but also for a wide variety of protection techniques. Four methods are presented for providing toe protection. One is to use the standard toe trench. The other methods are modifications of the standard section.

*Note: Methods 2, 3, and 5 should be considered when the designer has enough confidence in the estimated scour depth to believe the standard toe trench will not provide adequate protection.*




| ODOT Riprap Class | T (feet) |
|---|---|
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |

* A flatter slope may be used in materials that will not have a stable excavated face at a 1:1 slope, such as wet sands and gravels.

**Figure 15-4 Standard Riprap Section**

*ODOT estimation practices often underestimate scour in bridge openings. Bridge abutment scour is not calculated. Scour due to debris or ice in the bridge opening is also not included.*

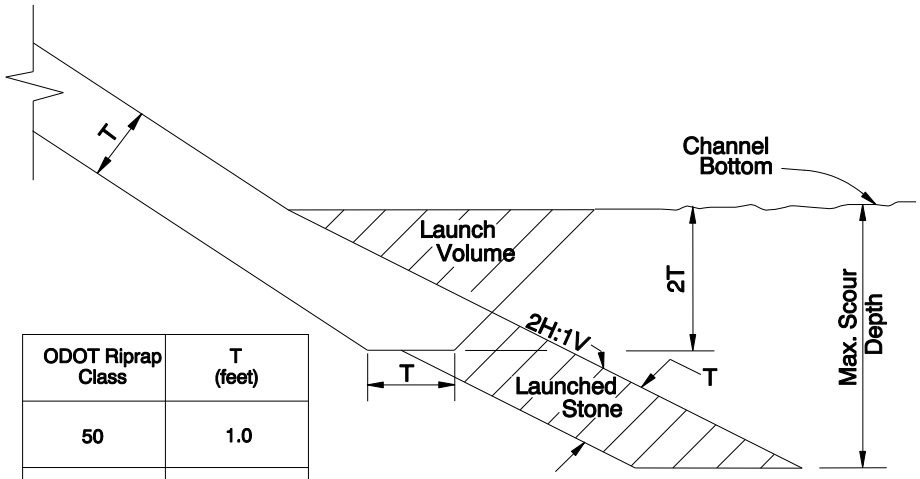
*ODOT uses riprap abutment protection with a rock filled toe trench in lieu of including these scour types in the estimates. As a result, Methods 3 and 5, which do not have rock filled toe trenches, should not be used to protect bridge abutments.*

Method 1: Construct a standard toe trench with the dimensions shown on Figure 15-4. The bottom of the toe trench is at or below the estimated maximum scour depth. In this case the maximum scour depth should not be greater than 2T, where T is the riprap blanket thickness. This is the preferred design for embankment protection, and it is also recommended for bridge abutments. As a minimum, the standard toe trench should be used for toe protection.

*Note: New facilities should be designed to have scour elevations equal to or higher than the bottom of the toe, if practicable.*

Method 2: This method is an alternative design that can be used where the estimated scour depth is below 2T. It is to provide a toe trench having a reservoir of rock that can roll into the scour hole. This displacement of riprap from the toe is called "launching." The launched riprap protects the toe from undermining. Observation of the performance of this toe design indicates the riprap will launch to a final slope of 1V: 2H on non-cohesive material. The stone will launch to a thickness equal to the design thickness, T. The final launch configuration of the riprap is shown in Figure 15-5.

*Note: Launching does not perform well on cohesive channel beds where scour occurs in the form of slumps with steep slip faces.*




| ODOT Riprap Class | T (feet) |
|---|---|
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |

**Figure 15-5 Launching of Riprap Toe Stone**

To use this method, a rock filled toe trench is constructed with the bottom elevation above the estimated scour depth. The dimensions of the toe trench are selected to provide a volume of rock necessary to extend the riprap blanket to the estimated scour depth. The volume necessary to extend the blanket to the estimated scour depth should be increased 50 percent to account for stone loss during launching. The scour depths that the standard toe trench can tolerate for each class of riprap are listed in Table 15-7. The Blodgett equations discussed in Subsection 15.4.4 were used to calculate the scour depths listed in the table. The 1V: 2H bank slope can tolerate deeper scour depths because there is a greater volume of rock available for launching from the rock filled toe trench.

**Table 15-7 Allowable Scour Depths for Standard Toe Trench**

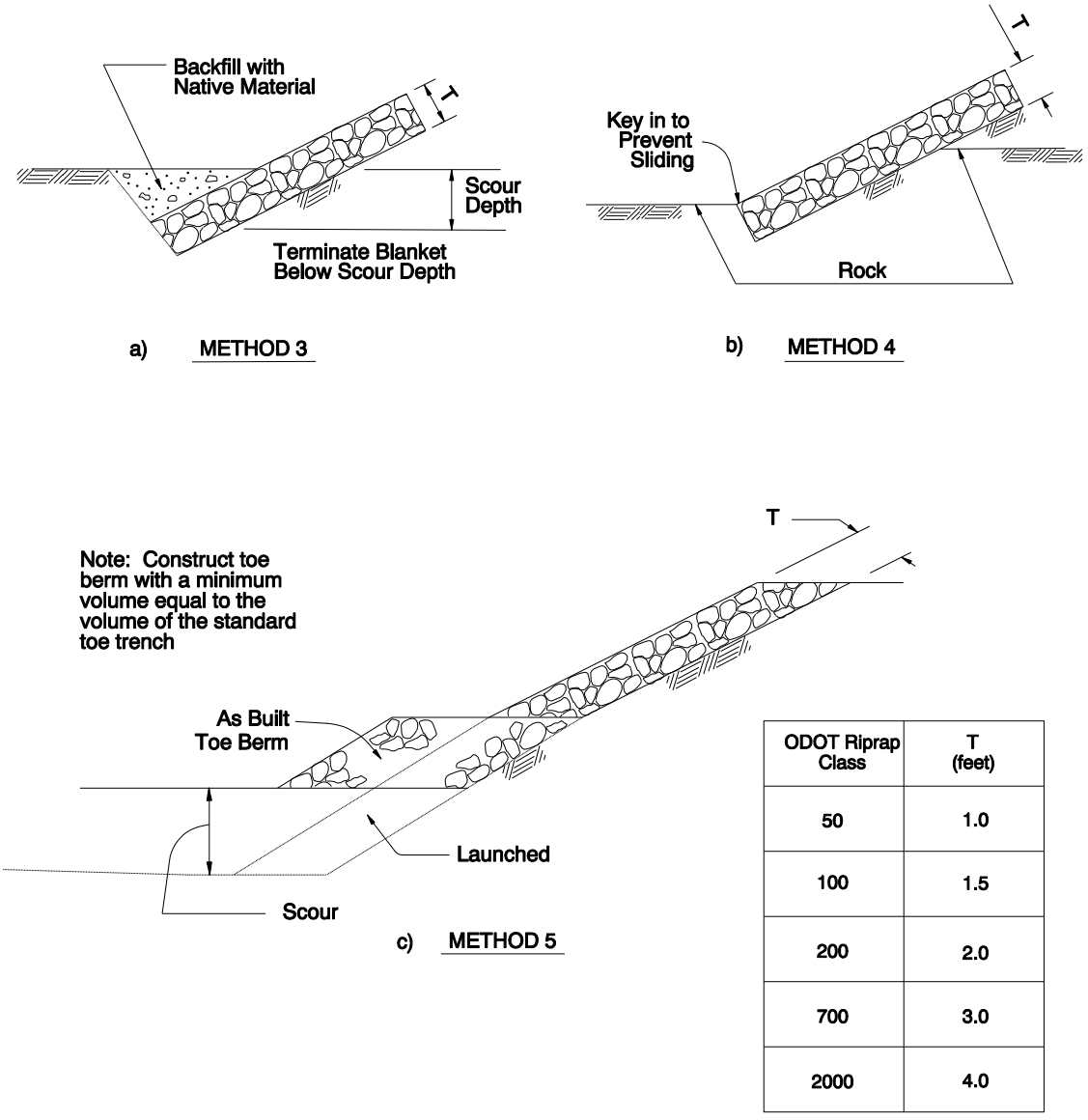
| | Maximum Scour Depth (feet) | | |
|---|---|---|---|
| Riprap Class | Bank Slope* (All) | Bank Slope** 1V: 1½H | Bank Slope** 1V: 2H |
| Class 50 | 2.0 | 2.7 | 3.1 |
| Class 100 | 3.0 | 4.1 | 4.6 |
| Class 200 | 4.0 | 5.4 | 6.1 |
| Class 700 | 6.0 | 8.1 | 9.2 |
| Class 2000 | 8.0 | 10.9 | 12.2 |

*Without toe launching
**With toe launching

Method 3: This method is to not provide a toe trench, to terminate the riprap blanket below scour depth, and to backfill over the toe with native streambed material. It is shown in Figure 15-6a.

Method 4: This method can be used where there is solid rock at the revetment toe. It is recommended for both channels and bridge openings where rock is present at a suitable depth. It is shown in Figure 15-6b.

Method 5: This procedure is to build an exposed toe with a volume of rock great enough to fill in the predicted scour hole. It is a modification of the launching toe previously discussed. It is shown in Figure 15-6c.




| ODOT Riprap Class | T (feet) |
|---|---|
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |

**Figure 15-6 Alternate Toe Protection Details**

### 15.5.1.6 Flank Treatment

Flank treatment includes special design features to prevent flanking of the ends of the revetment. Wherever possible, the bank protection should tie into stable anchorage points, such as bridge abutments, rock outcrops, or well-vegetated, stable channel sections. When a stable feature does not exist, flank protection as shown in Figure 15-7 should be considered.

### 15.5.1.7 Calculating Riprap Size

The following subsections present methods to calculate the riprap size required to resist streamflow and wave hydraulic forces. All applicable methods are to be used at each site. The selected rock size is equal to or larger than the most appropriate calculated value, unless:

- historical performance of riprap at or near the site, or the size of bed material moved by the stream indicates a different size rock is needed, or
- a suitable hydraulic model cannot be developed to calculate riprap size. In this case, the size is based on parol evidence, the history of the site, and a site investigation.

Riprap size is calculated using a design and a check discharge. It is sized to be stable during the design discharge. The size is checked to verify it will be stable during the check discharge. Occasionally the riprap size adequate for the design discharge will need to be increased to be stable during the check discharge. Design and check discharges are listed in **Chapter 3**.

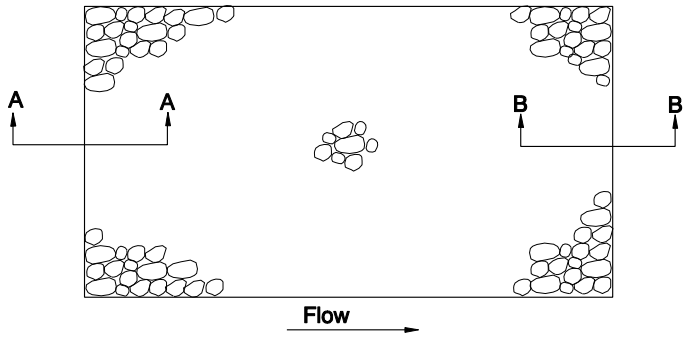
These methods can also be used to size rock for lining large channels (discharge is greater than 50 cubic feet per second). Riprap sizing methods for smaller channels are in **Chapter 8**.

#### 15.5.1.7.1 Parol Evidence, History & Site Investigation Method

The parol evidence, history, and site investigation method consists of a thorough evaluation of historic riprap stability at the site and similar nearby sites. This method is recommended for:

- all locations where data is available.

The results of this method should not be used at all sites, although its results should be considered. The results should be compared to the calculated values from the other procedures if they are used. This method is often the only practicable procedure at locations with complex interactions of oceanic, riverine, and estuarine hydraulics, or sites with complex river/floodplain hydraulic relationships. It is often very difficult at these locations to construct the numerical hydraulic models needed for the other procedures.

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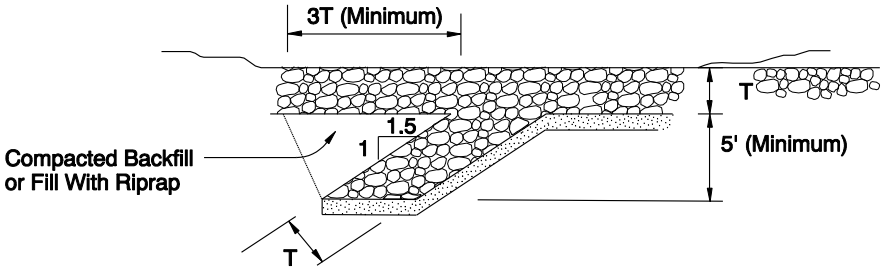
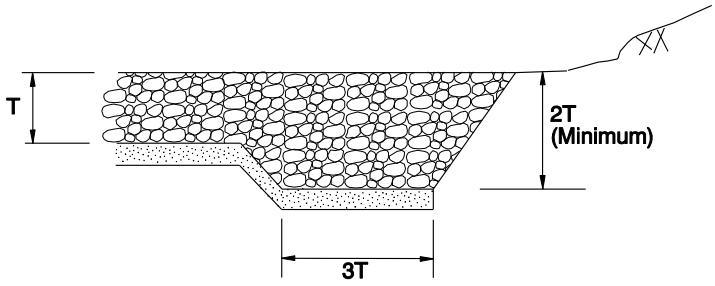



SECTION A - A (Upstream Flank or Leading Edge)




SECTION B - B (Downstream Flank or Trailing Edge)

| ODOT Riprap Class | T (feet) |
|---|---|
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |

**Figure 15-7 Typical Flank Protection Details**

The primary sources of information this procedure are:

- records of historic revetment installation and repairs,
- eyewitness accounts by people familiar with the structure, and
- field inspection.

Historic documentation often mentions the size of riprap installed in the past. Records of historic repairs and scour problems for structures are documented in the bridge inspection and bridge maintenance files. Copies are available from the ODOT Bridge Section. Records of many repairs to both structures and roadways are documented in the ODOT vertical "V" files. Copies are available from the ODOT Roadway Section. Scour problems and repairs are also documented in the ODOT hydraulics files. Copies are available from the ODOT Geo-Environmental Section. Additional guidance on information collection is in **Chapter 6**.

Eyewitness accounts from people familiar with the structure can also be helpful when sizing riprap. Structure and roadway maintenance personnel are good sources. Nearby residents can also be helpful. Photographs are also valuable. They can be used to estimate riprap size.

Field inspection often gives insight on adequate riprap size. Particular attention should be given to the size of the riprap that is in place at or near the site. The designer should also note the predominant hydraulic forces at the site: i.e. tractive forces due to streamflow, tidal action, wind driven waves, boat wakes, etc.

Careful attention is needed during a field inspection to locate existing riprap that is in hydraulically similar conditions. As an example, a bridge abutment is proposed in very shallow water at the edge of a bay. During severe winter storms the abutment is on the leeward shore. The predominant hydraulic forces are due to wave action.

The ability of a wave to displace riprap is proportional to the wave height. Waves are highest in deeper water on the windward side of bays. Riprap adequate to withstand waves in deep water on the windward side may be overly large for the proposed abutment. The performance of existing riprap on the leeward side of the bay or nearby bays in shallow water would be a better indicator of site conditions.

The size of bed material transported by the stream at the specific location of the proposed riprap is determined by both the field inspection and geological exploration. This information is critical when sizing riprap. Many experienced designers do not select riprap with a $D_{50}$ less than two or three times the maximum diameter of the moving bed material at the location where the riprap will be placed. This practice is recommended.

### 15.5.1.7.2 Tractive Force Design Relationship

The tractive force design relationship is based on tractive force theory yet has velocity as its primary design parameter. The design assumptions are primarily based on the uniform or gradually varying flow typical of channels with fairly uniform cross-section. The relationships have been verified to be adequate for a variety of rivers, both in natural channels within floodplains and in channels confined within flood levees. This procedure is discussed in detail in the latest versions of references 6 and 7. This method is recommended for these applications where tractive forces due to streamflow control the riprap size:

- riverine embankments,
- channel linings, and
- bridge abutments.

*Note: This procedure provides realistic results in most subcritical flow situations. It may overestimate the needed riprap size in some near critical or critical flow applications. In these instances, the ODOT velocity based method or modified Isbash relationship may provide more realistic results. The pier riprap method should also be used to determine rock size on structures where the pier projects through the toe of the abutment protection riprap, and the larger calculated rock size should be used.*

The primary equation for the tractive force method follows:

$$D_{50} = \frac{0.001 C V_a^3}{d_{avg}^{0.5} K_1^{1.5}} \qquad \text{(Equation 15 - 6)}$$

Where:

$D_{50}$ = median riprap equivalent spherical diameter in feet,
$C$ = correction factor (described in following text),
$V_a$ = the average velocity in the main channel in feet per second,
$d_{avg}$ = the average depth in the main channel in feet, calculated as the main channel cross-sectional area in square feet divided by the main channel flow width,
$K_1$ = the ratio of the incipient motion tractive force on a sloping surface to the incipient motion tractive force on a level surface and is defined as:

$$\mathbf{K_1 = \left[1 - \frac{\sin^2 \Theta}{\sin^2 \Phi}\right]^{0.5}} \qquad \textbf{(Equation 15-7)}$$

Where:

$\Theta$ = the bank angle with the horizontal in degrees, and

$\Phi$ = 41 degrees (the angle of repose of ODOT standard riprap classes).

Table 15-8 lists the $K_1$ values for various bank angles.

**Table 15-8 $K_1$ Values**

| Bank Slope | Bank Angle ($\Theta$) | $K_1$ |
|---|---|---|
| | | |
| 1V: 1½H | 33.69º | 0.534 |
| 1V: 1¾H | 29.74º | 0.654 |
| 1V: 2H | 26.57º | 0.732 |
| 1V: 2½H | 21.80º | 0.824 |
| 1V: 3H | 18.43º | 0.876 |

The average flow depth and velocity used in Equation 15-6 are the main channel values. The main channel is defined as the area between the channel banks, as shown in Figure 15-8. The tractive force method can also be used for riprap in overbank areas if the average velocity and depth in the overbank area are used.

*Note: The riprap sizes in an overbank area are calculated using main channel values unless it is certain that the main channel will not shift in alignment and scour out the overbank area.*

Equation 15-6 is based on a rock riprap specific gravity of 2.65, and a stability factor of 1.2. Equations 15-8 and 15-9 are correction factors for other specific gravities and stability factors. The correction factors computed using Equations 15-8 and 15-9 are multiplied together to form the single correction factor **C** used in Equation 15-6.

$$\mathbf{C_{sg}} = \frac{\mathbf{2.12}}{\left(\mathbf{S_s - 1}\right)^{\mathbf{1.5}}} \qquad \textbf{(Equation 15-8)}$$

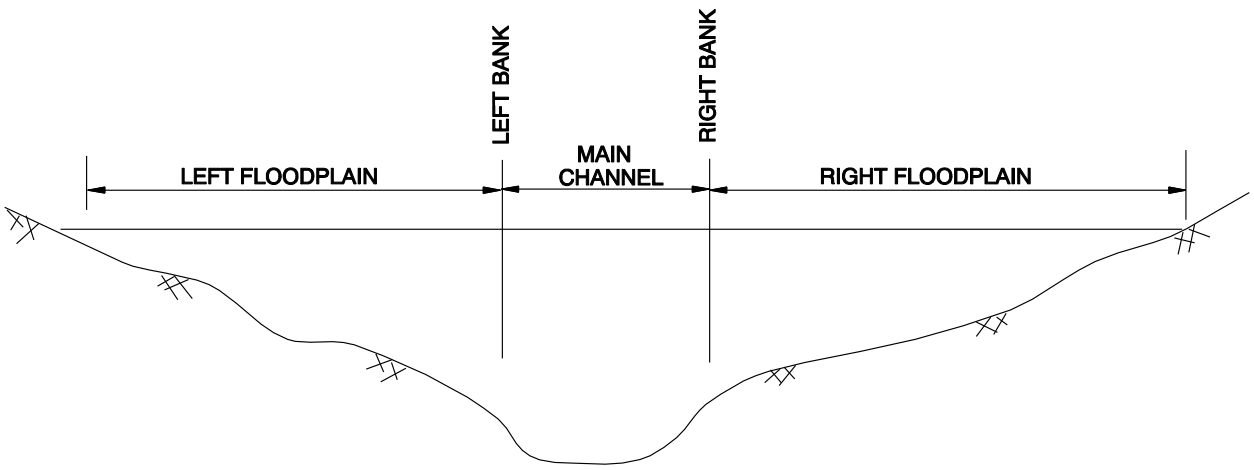



**Figure 15-8 Definition Sketch of Channel Flow Distribution**

Where:

$C_{sg}$ = correction factor for rock specific gravity, and
$S_s$ = specific gravity of the rock (see Subsection 15.5.1.2 for typical specific gravity values)

$$C_{sf} = \left(\frac{SF}{1.2}\right)^{1.5} \qquad \textbf{(Equation 15-9)}$$

Where:

$C_{sf}$ = correction factor for stability, and
SF = the stability factor to be applied.

The stability factor, SF, used in Equations 15-6 and 15-9 is defined as the ratio of the riprap material's critical shear stress and the average tractive force exerted by the flow field. As long as the stability factor is greater than 1, the critical shear stress of the riprap material is greater than the flow induced tractive stress and the riprap is considered to be stable. As mentioned in the preceding discussion, a stability factor of 1.2 was used in the development of Equation 15-6.

The stability factor is used to reflect the level of uncertainty in the hydraulic conditions at a particular site. Equation 15-6 is based on the assumption of uniform or gradually varying flow. However, design needs dictate that the relationship also be applicable in nonuniform, rapidly varying flow conditions often exhibited in natural channels. Flow conditions include debris and/or ice impacts, higher shear stresses at channel bends, or the cumulative effect of high shear stresses and forces from wind and/or boat generated waves. The stability factor is used to increase the design rock size when these conditions must be considered. When a stability factor is selected to accommodate wave action, it is recommended that the riprap stability also be checked with the Hudson relationship. Table 15-9 presents guidelines for the selection of an appropriate value for the stability factor.

**Table 15-9 Guidelines for the Selection of Stability Factors**

| Flow Condition | Stability Factor Range |
|---|---|
| Uniform flow; Straight or mildly curving reach (curve radius/channel width is more than 30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters. | 1.0 – 1.2 |
| Gradually varying flow; Moderate bend curvature (30 is more than curve radius/channel width is more than 10); Impact from waves, ice or floating debris moderate. | 1.3 – 1.6 |
| Approaching rapidly varying flow; Sharp bend curvature (curve radius/channel width is less than 10); Significant impact potential from floating debris and/or ice; Significant wind and/or boat generated waves (1 - 2 feet); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters. | 1.7 –2.0 |

Damage or impact from ice is also a factor to consider when selecting the stability factor. In general, ice damage is not a problem in Oregon although ice has the potential to be a problem in the high Cascades and in eastern Oregon. The concern with ice is that moving surface ice can cause crushing or bending forces as well as large impact forces. The tangential flow of ice along a riprap lined channel bank can also cause excessive shearing forces. Quantitative criteria for evaluating the impact ice has on riprap are unavailable although observations of ice flows in New England rivers indicate that riprap sized to resist design flow events will also resist ice

forces. Maintenance personnel should be contacted to determine if a site or watershed is subject to ice and if so, get their advice as to what size riprap has historically provided adequate performance when subjected to ice. Observations and experience of maintenance personnel can also be used to select an appropriate stability factor. Where ice flows have historically caused problems, a stability factor of 1.2 to 1.5 is usually adequate to increase the design rock size.

### 15.5.1.7.3 ODOT Velocity Based Design Procedure

This design method is based on the average channel velocity in the channel bank side slope. It was adapted from a U.S. Bureau of Public Roads (USBPR) procedure, and it has been used by ODOT for over thirty years. The original USBPR procedure was developed for channel linings and bank protection revetment on straight waterways. ODOT has adapted the procedure to be used on curved waterways and bridge abutments by use of a correction factor. This method is suitable for the following situations where tractive forces due to streamflow predominate:

- riverine embankments or channel linings in straight to moderately curving reaches (curve radius/channel width more than 10), or
- bridge abutments with no to moderate flow contraction through the bridge opening.

*Note: The tractive force design method is recommended for channels with sharp bend curvature. The modified Isbash relationship is recommended for abutments in locations with significant flow contraction. The pier riprap method should also be used to determine rock size on structures where the pier projects through the toe of the abutment protection riprap, and the larger calculated rock size should be used.*

The velocity/side slope/riprap size relationship is shown in Figure 15-9. To use the figure, enter the average channel velocity and side slope of the riprap and then select the appropriate ODOT standard riprap class. The $D_{50}$ values of ODOT standard riprap classes are plotted on Figure 15-9 and they represent the upper stability limits for the particular riprap classes. The next larger riprap class should be used when the required size of stone exceeds the $D_{50}$ of a particular riprap class. For example, for a velocity of 9 feet per second and a 1V: 1½H side slope, the embankment should be protected with Class 200 riprap. These average velocities are used:

- the average channel velocity for a straight channel or a slightly curved channel (curve radius/channel width more than 30), or riprap on the inside of a bend on a moderately curved channel,
- 1.33 times the average channel velocity for riprap on the outside of a bend on a moderately curved channel (curve radius /channel width more than 10), or
- 1.33 times the average bridge opening velocity for bridge abutments.

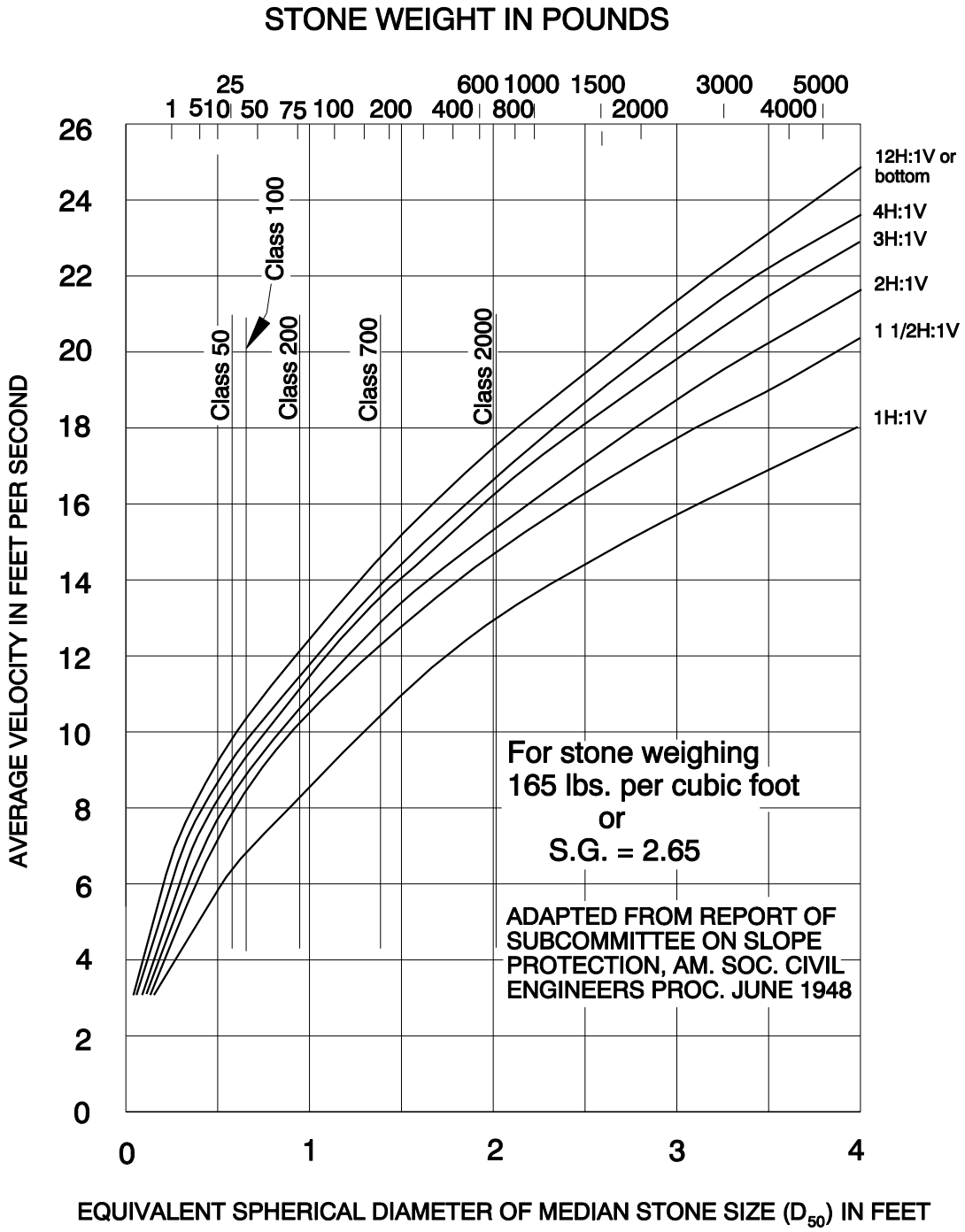



**Figure 15-9 Velocity Based Riprap Design Chart**

The stone size determined from Figure 15-9 should be corrected with Creager's equation when the unit weight of stone is other than 165 pounds per cubic foot, as follows.

$$\mathbf{D_w = \frac{102.5 D_s}{\gamma_w - 62.5}} \qquad \textbf{(Equation 15-10)}$$

Where:

$D_w$ = stone size in feet for stone of unit weight $\gamma_w$ in pounds per cubic foot, and
$D_s$ = stone size in feet from Figure 15-9.

#### 15.5.1.7.4 Hudson Relationship for Wave Erosion Protection

Waves generated by wind or boat traffic can cause bank erosion on inland waterways. The most widely used measure of riprap wave resistance is that developed by R. Y. Hudson. The so-called Hudson relationship is given by the following equation:

$$\mathbf{W_{50} = \frac{\gamma_s H^3}{2.20(S_s - 1)^3 \cot\Theta}} \qquad \textbf{(Equation 15-11)}$$

Where:

$W_{50}$ = median rock weight in pounds,
$\gamma_s$ = riprap rock unit weight (solid material) in pounds per cubic foot,
$H$ = the wave height in feet,
$S_s$ = specific gravity of riprap material, and
$\Theta$ = bank angle with the horizontal in degrees.

Assuming $S_s = 2.65$ and $\gamma_s = 165$ pounds per cubic foot, Equation 15-11 can be reduced to:

$$\mathbf{W_{50} = \frac{16.7 H^3}{\cot\Theta}} \qquad \textbf{(Equation 15-12)}$$

In terms of an equivalent diameter Equation 15-12 can be reduced to:

$$\mathbf{D_{50} = \frac{0.75 H}{\sqrt[3]{\cot\Theta}}} \qquad \textbf{(Equation 15-13)}$$

Where:

$D_{50}$ = median riprap size in feet.

The maximum wave heights, H, that ODOT standard riprap classes can tolerate for various slopes are listed in Table 15-10.

**Table 15-10 Bank Slope vs. Maximum Wave Height**

| | Maximum Wave Height (feet) | | | | |
|---|---|---|---|---|---|
| Bank Slope | Class 50 | Class 100 | Class 200 | Class 700 | Class 2000 |
| 1V: 1½H | 1.1 | 1.3 | 1.9 | 2.6 | 4.0 |
| 1V: 2H | 1.2 | 1.4 | 2.1 | 2.9 | 4.4 |
| 1V: 2½H | 1.3 | 1.6 | 2.3 | 3.1 | 4.7 |
| 1V: 3H | 1.4 | 1.7 | 2.4 | 3.3 | 5.0 |
| 1V: 4H | 1.5 | 1.8 | 2.7 | 3.6 | 5.5 |
| 1V: 5H | 1.6 | 2.0 | 2.9 | 3.9 | 5.9 |

The height of a wind generated wave on an inland waterway is influenced by the fetch length, the wind speed and duration, and the water depth. Procedures to determine these heights are in the latest revision of Reference 7.

.

*Note: The procedure in this chapter is intended for inland waterways such as lakes, reservoirs, large rivers, protected bays, estuaries, etc. Embankments subject to oceanic wave action should be designed using coastal protection procedures. References10, 11, and 12 provide useful information.*

### 15.5.1.7.5 Modified Isbash Relationship to Size Riprap at Bridge Piers and Abutments

This subsection contains design guidelines for sizing scour protection riprap at bridge piers and abutments. It is based on a modified Isbash relationship, and it is explained in detail in Reference 6.

#### 15.5.1.7.5.1 Bridge Piers

The modified Isbash relationship can be used to size pier protection riprap. Pier riprap is used most often in bridge maintenance and seldom in new designs. The ODOT standard detail for bridge pier riprap is shown in **Chapter 10**.

*Note: Spillthrough opening bridge piers are sometimes surrounded by the revetment protecting the abutment toe. This procedure should be used in addition to the abutment revetment sizing methods to calculate the needed rock size. The larger calculated rock size should be used.*

Determine the $D_{50}$ size of riprap using the rearranged Isbash equation:

$$\mathbf{D_{50} = \frac{0.692(KV)^2}{(S_s - 1)2g}} \qquad \textbf{(Equation 15-14)}$$

Where:

| | | |
|---|---|---|
| $D_{50}$ | = | median stone diameter in feet, |
| K | = | pier shape coefficient (K = 1.5 for round-nose piers, K = 1.7 for rectangular piers), |
| V | = | average velocity approaching the pier in feet per second, |
| $S_s$ | = | specific gravity of riprap (normally 2.65), |
| g | = | acceleration of gravity (32.2 feet per second per second). |

The velocity, V, should be the same velocity that was used to calculate local pier scour. Figure 15-10 can be used to determine the $D_{50}$ size of the riprap necessary to provide scour protection. The upper stability limits for ODOT riprap classes are plotted on Figure 15-10. The next larger riprap class should be used when the required size of stone exceeds the $D_{50}$ of a particular riprap class. For example, for a round-nose pier subjected to a velocity of 9.0 feet per second, the pier should be protected with Class 700 riprap. ODOT standard riprap classes can be used for a velocities up to 11.7 and 10.3 feet per second for round-nose and rectangular piers, respectively.

### 15.5.1.7.5.2 Bridge Abutments

The following equations for sizing riprap for abutments are based on research conducted by the FHWA in a hydraulic flume. This method determines rock size. Abutment protection details are in **Chapter 10**, Bridges. This method is applicable to applications where tractive forces due to streamflow predominate, including:

- all bridge abutment types, including those with significant flow contraction into the bridge opening.

*Note: The pier riprap method should also be used to determine rock size on structures where the pier projects through the toe of the abutment protection riprap, and the larger calculated rock size should be used.*

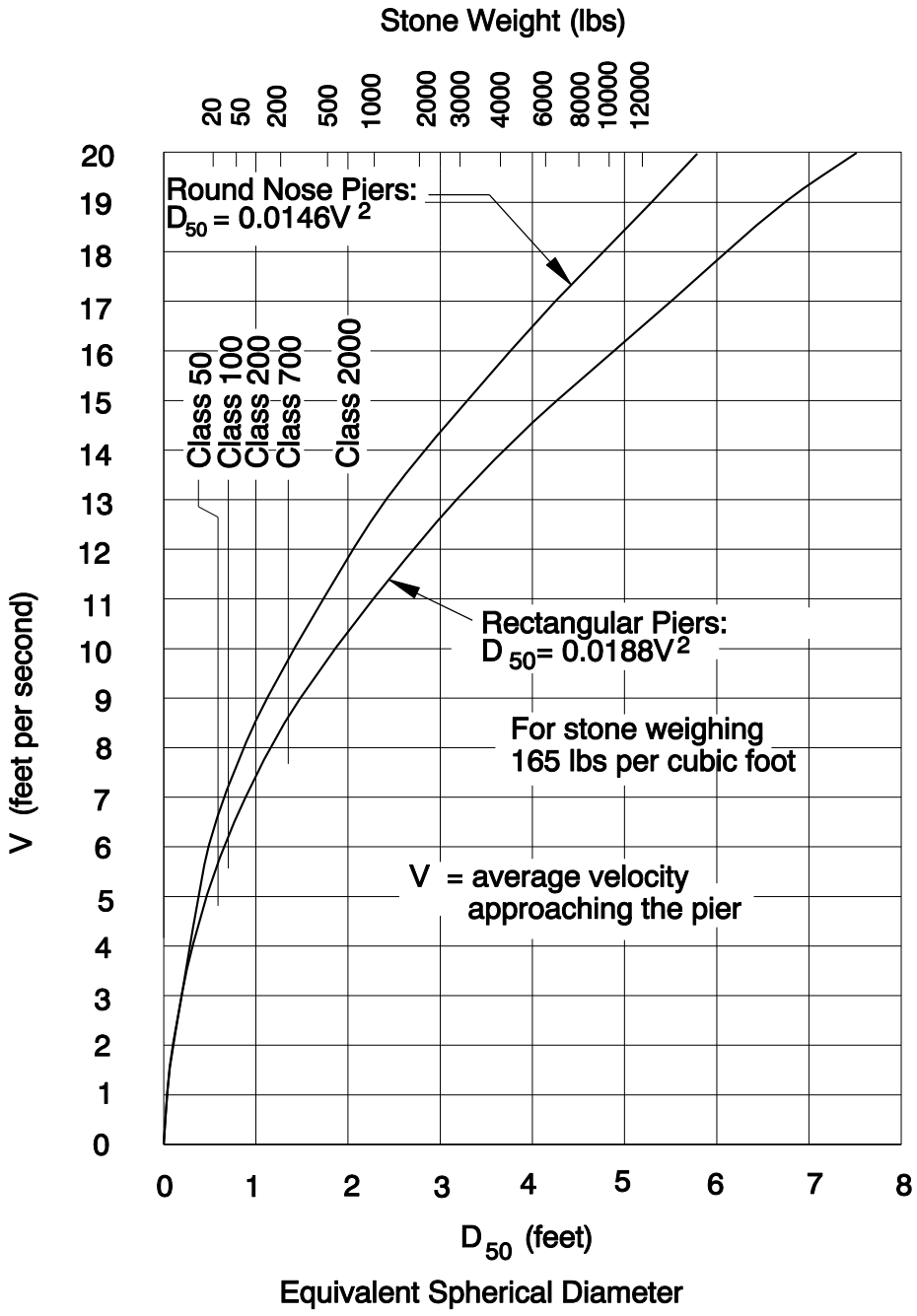



**Figure 15-10 Pier Scour Riprap Chart**

*The research was conducted using 1V: 2H slopes for spillthrough abutments. The riprap size should be compared against the riprap size obtained from the other procedures when applying the equations to spillthrough abutments with 1V: 1½H slopes. Engineering judgment should then be used to select the final riprap size. The same procedure is recommended for all spillthrough abutments and vertical abutments – compare the riprap size obtained with following equations to the riprap size obtained from all other applicable methods, and then apply engineering judgment to select the final riprap size.*

This form of the Isbash relationship is recommended for **Froude numbers, $\frac{V}{\sqrt{gy}}$ less than or equal to 0.80**, with either spill through or vertical abutments:

$$\frac{D_{50}}{y} = \left(\frac{K}{S_s - 1}\right)\left(\frac{V^2}{gy}\right) \qquad \textbf{(Equation 15-15)}$$

Where:

$D_{50}$ = median stone diameter in feet,
V = characteristic average velocity in the contracted section (explained in the remainder of this subsection), in feet per second,
$S_s$ = specific gravity of rock riprap,
g = gravitational acceleration, 32.2 feet/second$^2$
y = depth of flow in the contracted bridge opening in feet,
K = 0.89 for spillthrough abutments, and
K = 1.02 for vertical abutments.

**E**quation 15-16 is recommended for **Froude numbers more than 0.80,**

$$\frac{D_{50}}{y} = \left(\frac{K}{S_s - 1}\right)\left(\frac{V^2}{gy}\right)^{0.14} \qquad \textbf{(Equation 15-16)}$$

Where:

K = 0.61 for spillthrough abutments, and
K = 0.69 for vertical abutments.

In both equations, the coefficient K is a velocity multiplier to account for the apparent local acceleration of flow at the point of rock riprap failure. Both of the equations are envelope relationships that were forced to over predict 90 percent of the laboratory data.

The first step to determine the characteristic average velocity is to determine the set-back ratio, SBR, of each abutment. SBR is the ratio of the set-back length to average channel flow depth. The set-back length is the distance from the near edge of the main channel to the toe of the abutment. The average channel flow depth is the area of the main channel divided by the top width of the main channel.

$$\text{SBR} = \frac{\text{Set - back length}}{\text{Average channel flow depth}}$$

The characteristic average velocity depends on the set-back ratio, using the following guidelines.

1. Compute a characteristic average velocity, Q/A, based on the entire contracted area through the bridge opening, if **SBR is less than 5** for both abutments (Figure 15-11). This includes the entire upstream flow, exclusive of that which overtops the roadway.

2. Compute a characteristic average velocity, Q/A, for the respective overbank flow only, if **SBR is more than 5** for an abutment (Figure 15-12). Assume that the entire respective overbank flow stays in the overbank section through the bridge opening.

3. A characteristic average velocity determined in Step 1a for the abutment with SBR less than 5 may be unrealistically low if **SBR for one abutment is less than 5 and SBR for the other abutment is more than 5** (Figure 15-13). This would depend upon the opposite overbank discharge as well as how far the other abutment is set back. For this case, the characteristic average velocity for the abutment with SBR less than 5 should be based on the flow area limited by the boundary of that abutment and an imaginary wall located on the opposite channel bank. The appropriate discharge is bounded by this imaginary wall and the outer edge of the floodplain associated with that abutment.

### 15.5.2 Stream Barbs

This section contains design guidelines for designing stream barbs. Stream barbs, also referred to as bendway weirs or bank barbs, are low sills projecting out from a stream bank and across the stream's thalweg to redirect the stream flow away from an eroding bank towards the channel centerline. Stream barbs are normally not visible, especially at stages above low water and are intended to redirect flow by utilizing weir hydraulics over the stream barb. Flow passing over the barb is redirected so that flow leaving the barb is perpendicular to the barb centerline axis. In

order to direct the flow towards the channel centerline, stream barbs are always oriented upstream. The barbs are keyed into the channel bank to prevent flow from flanking the end of the barb. Stream barbs are commonly constructed from rock, but tree trunks have also been used. When stream barbs are used, the bank between barbs is often reshaped and then planted with vegetation.

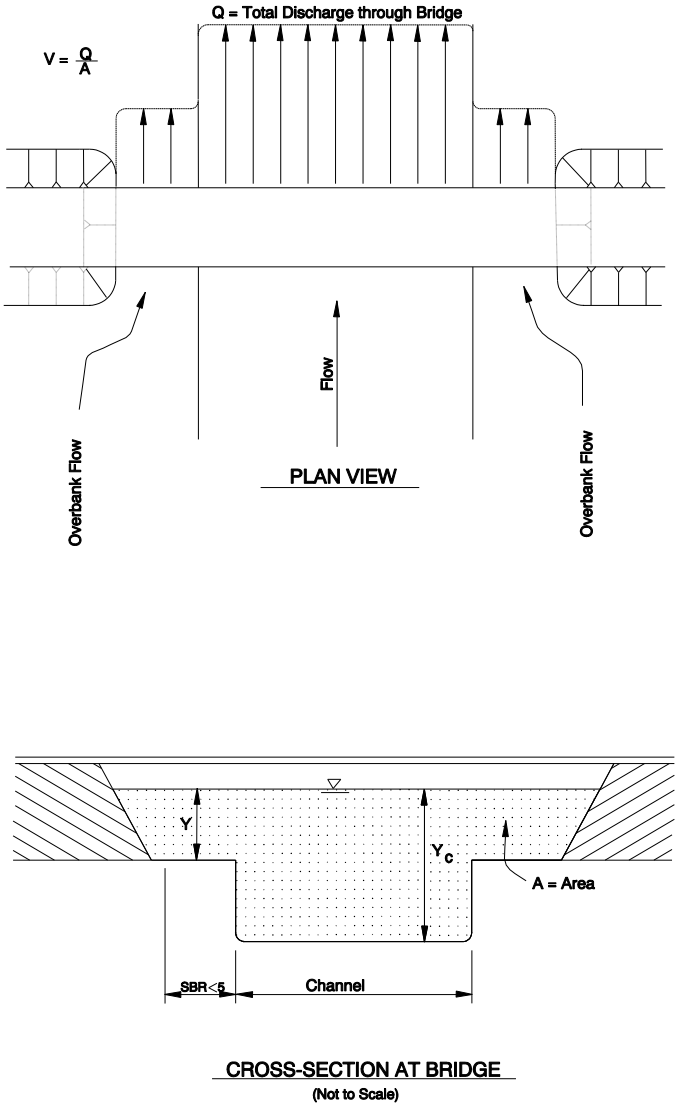



**Figure 15-11 Characteristic Average Velocity for SBR is less than 5**

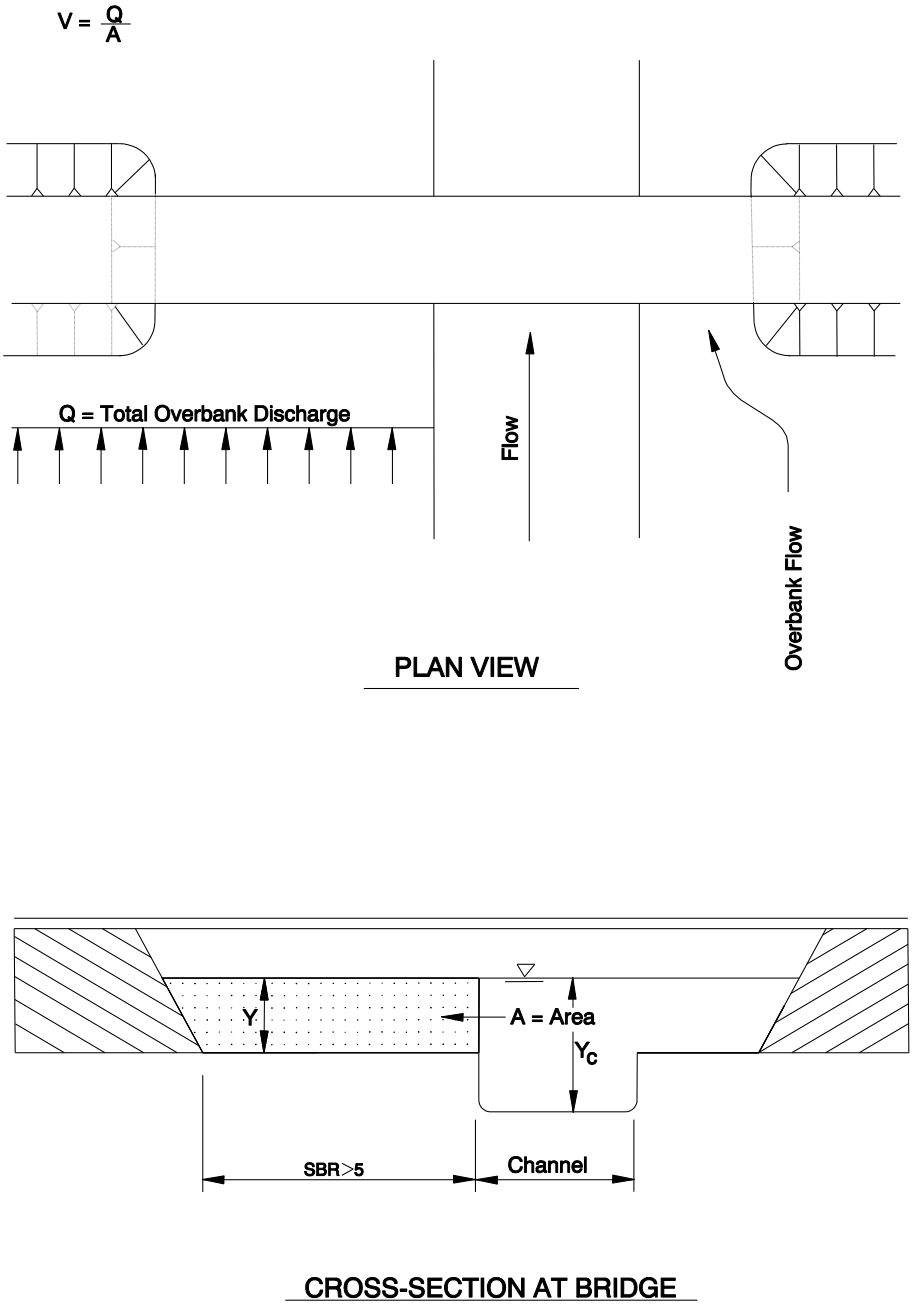



**Figure 15-12 Characteristic Average Velocity for SBR is more than 5**

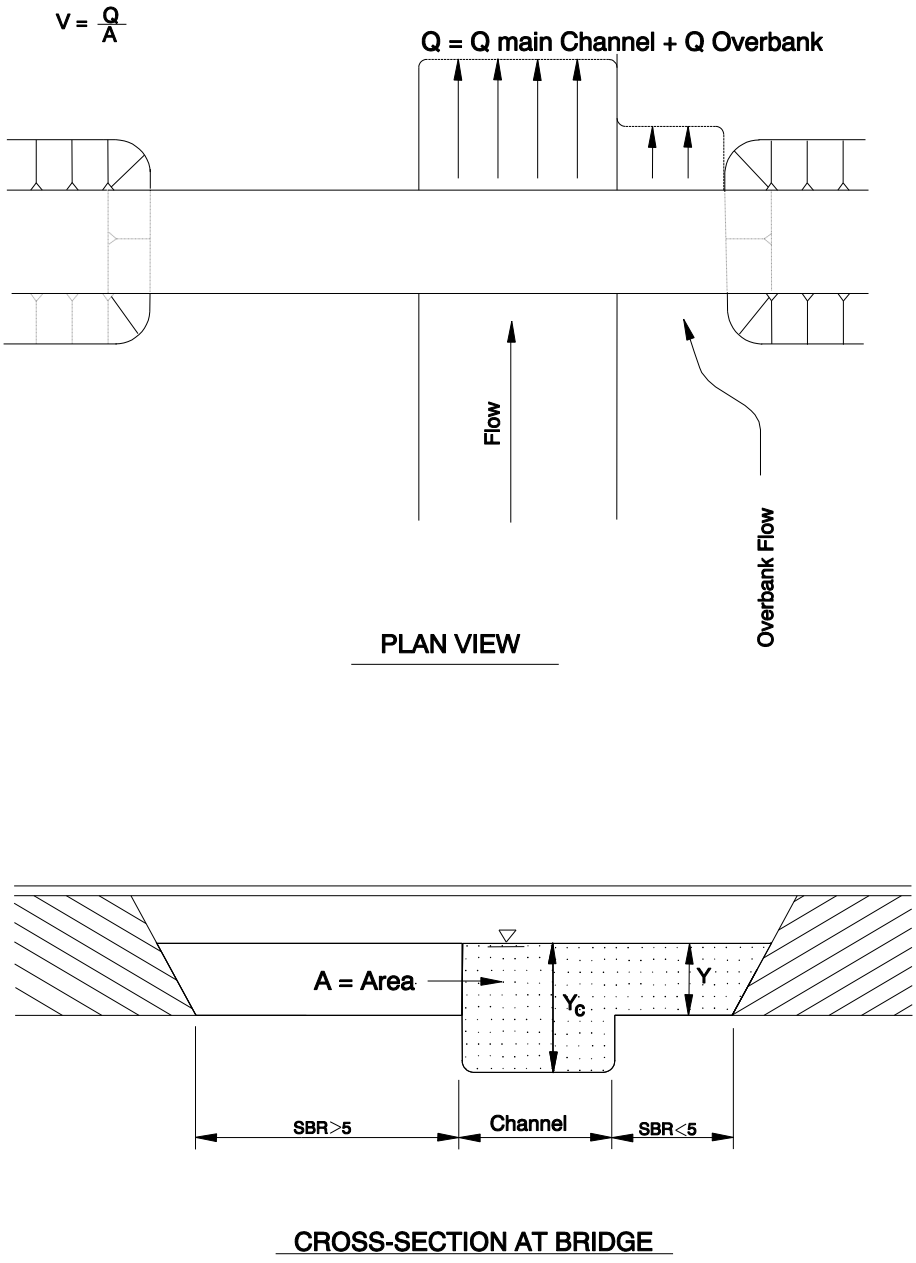



**Figure 15-13 Characteristic Average Velocity for SBR is more than 5 and SBR is less than 5**

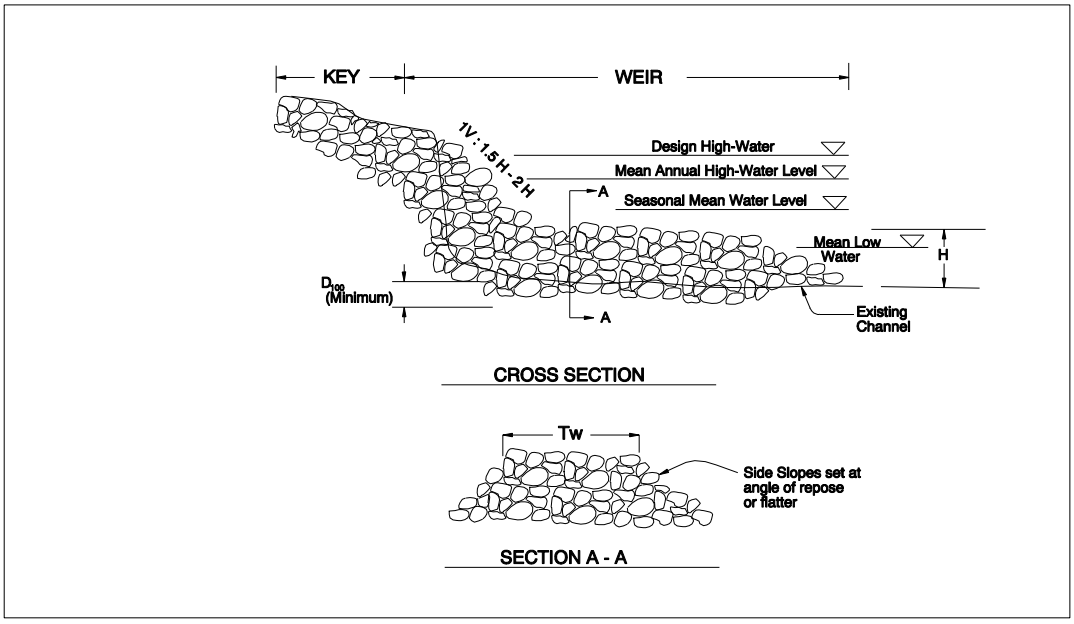
The channel stability needs to be evaluated before selecting barbs to protect the bank. Barbs should not be used in degrading channels. In degrading channels, the barbs are likely to be undermined and will fail. The approach channel also needs to be laterally stable in order to ensure long-term performance of the barbs. The best sites for stream barbs are long, uniform channel bends where the approach flow pattern will remain relatively constant over time.

#### 15.5.2.1 Height

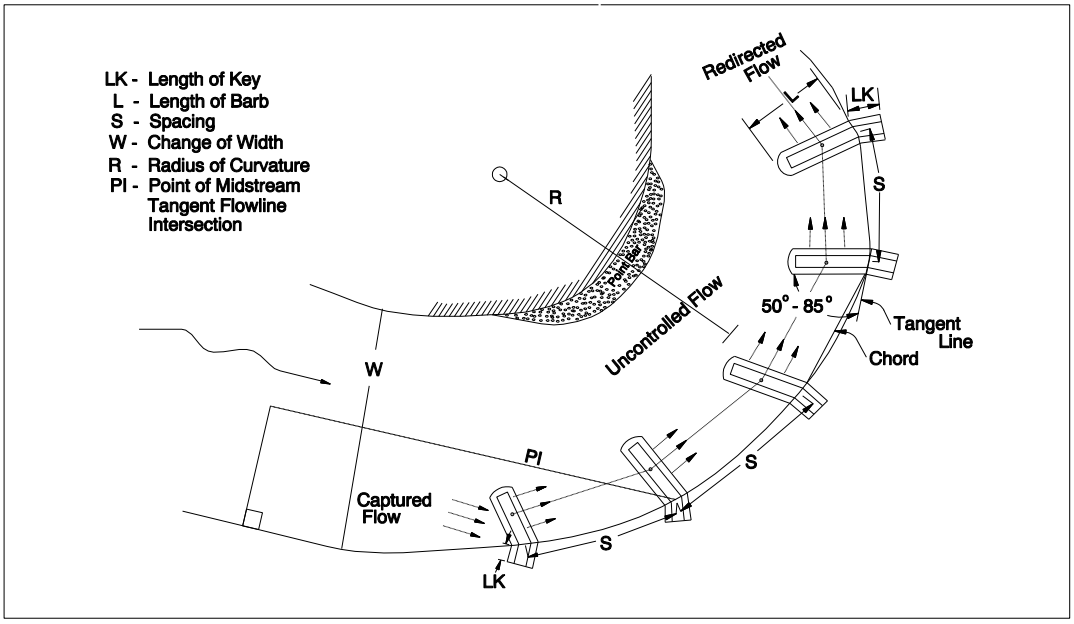
The height of the barb, H, is determined by analyzing various flow depths at the project site as shown in Figure 15-14. The stream barb height should be between 30 to 50 percent of the depth at the mean annual high water elevation. The height of the barb should also be below the normal or seasonal mean water elevation (ordinary high water) and should be equal to or above the mean low water elevation. The barb must be high enough to intercept a large enough percentage of the flow to produce the desired results. If barbs are placed in channels used for boat traffic, the barbs must be low enough to allow boats to safely pass over the barbs. The barb should be keyed a minimum depth of $D_{100}$ into the channel bottom.

#### 15.5.2.2 Angle of Projection

The angle of projection, Θ, between the stream barb axis and the upstream bankline tangent typically ranges from 50 to 85 degrees as shown in Figure 15-14. Rather than measure the angle from the bankline tangent, the angle of projection is often easier to measure from the centerline of the barb to the chord between two adjacent barbs. The chord is drawn from the points of intersection of the barbs and the bankline. The flow pattern entering the area of the proposed barb field should be analyzed and drawn in plan view for low and high flow conditions. The angle of projection,Θ, of the barb into the stream is based on the location of the barb in the bend and the direction of the flow lines of the approach flow. Ideally, the angle of attack of the flow lines at high-water flow should be no greater than 30 degrees and, at low flow, the angle of attack of the flow lines should be no less than 15 degrees to the normal of the centerline of the first few barbs. If the low-flow angle of approach is head-on (parallel to the barb) or close to head-on, the barb will be ineffective and serve to divide the flow. Bank scalloping and erosion will result. Also, the barbs downstream will be less effective due to turbulence and incorrect flow patterns entering their section of the barb field. If the angle of the approach flow line of high-water flow is too large, the weir will not be able to effectively redirect the flow to the desired direction. The upstream edge of the weir should be a well-defined straight line.




**a) Stream Barb Typical Cross Section**




**b) Stream Barb Typical Plan View**

**Figure 15-14 Stream Barbs**

### 15.5.2.3 Barb Length

The length of the barb does not have a definitive design, but the following criteria are offered as guidance. The maximum length should not exceed one third of the channel width as barbs this long can cause the flow pattern to impact the opposite bank. Barbs designed for bank protection need not exceed one fourth of the channel width. Additional guidance for barb length is that it should be long enough to extend across the thalweg. A length equal to 1.5 to 2.0 times the distance from the bank to the thalweg has proven satisfactory. The length of the barb will define the spacing between barbs.

### 15.5.2.4 Barb Location and Spacing

The first barb, which is shorter than the rest, should be located a distance, S, upstream from the point where a projection, PI, of the mid-channel flow line of the approach flow would impact the bank (see Figure 15-14). Additional barbs are then located at equal distances, S, based on site conditions and sound engineering judgment. The spacing, S, of the barbs is dependent on the streamflow leaving the barb and its impact point on the bank. The following formulas are given for guidance in establishing the spacing:

$$S = 1.5L\left(\frac{R}{W}\right)^{0.8}\left(\frac{L}{W}\right)^{0.3} \qquad \textbf{(Equation 15-17)}$$

$$S = (4 \text{ to } 5)L \qquad \textbf{(Equation 15-18)}$$

$$S_{max} = R\left[1-\left(1-\frac{L}{R}\right)^{2}\right]^{0.5} \qquad \textbf{(Equation 15-19)}$$

Where:

R = Channel radius
W = Channel width
L = Barb length

Maximum spacing is not recommended but, in cases where some erosion is permissible, the spacing may be set between S and $S_{max}$. The spacing and barb length, given by the formulas, should be investigated to determine if they would direct the flow as desired. Streamlines approaching and leaving the barbs should be analyzed and drawn in plan form. A suggested method for evaluating the barb spacing during construction is to construct the upstream barb

first. Then adjust the location of the next downstream barb based on where the resulting flow hits the stream bank. Continue this process with each barb in turn through the last barb.

### 15.5.2.5 Length of Bank Key

To prevent flanking flow, stream barbs should be keyed into the bank. The key length, LK, is based on a 20-degree flow expansion angle (Figure 15-15). Two guidance formulas for setting key length are given, based on the radius of curvature of the channel: For a large radius of curvature, R is more than 5W $S \text{ is more than } \frac{L}{\tan 20^{\circ}}$ and

$$LK = S \tan(20^{o}) - L \qquad \textbf{(Equation 15-20)}$$

For small radius of curvature, R is less than 5W $S \text{ is less than } \frac{L}{\tan 20^{\circ}}$ and

$$\mathbf{LK} = \left(\frac{\mathbf{L}}{\mathbf{2}}\right)\left(\frac{\mathbf{W}}{\mathbf{L}}\right)^{\mathbf{0.3}}\left(\frac{\mathbf{S}}{\mathbf{R}}\right)^{\mathbf{0.5}} \qquad \textbf{(Equation 15-21)}$$

LK should not be less than 1.5 times the total bank height. NRCS guidelines for barbs and short weirs specify that the key should be 8 feet or 4($D_{50}$) which ever is greater. The key may be inline with the weir as shown in Figure 15-15 or perpendicular to the tangent at the bank as shown in Figure 15-14. The depth of the key should extend down to the depth of the thalweg for the full length into the bank.

### 15.5.2.6 Top Width

The top width, $T_w$, of the weir may vary between 3 feet to 12 feet but should be no less than 2 to 3 times $D_{100}$. Where the length of the barb is so long that it cannot be constructed from the bank, the barb must either be made wide enough for the construction equipment to drive on it and work on top of the barb itself or the work area must be isolated so construction equipment can work in the channel.

### 15.5.2.7 Riprap Backing

Riprap backing is usually only needed between the key and the bank. Table 15-6 can be used to select the required riprap backing.

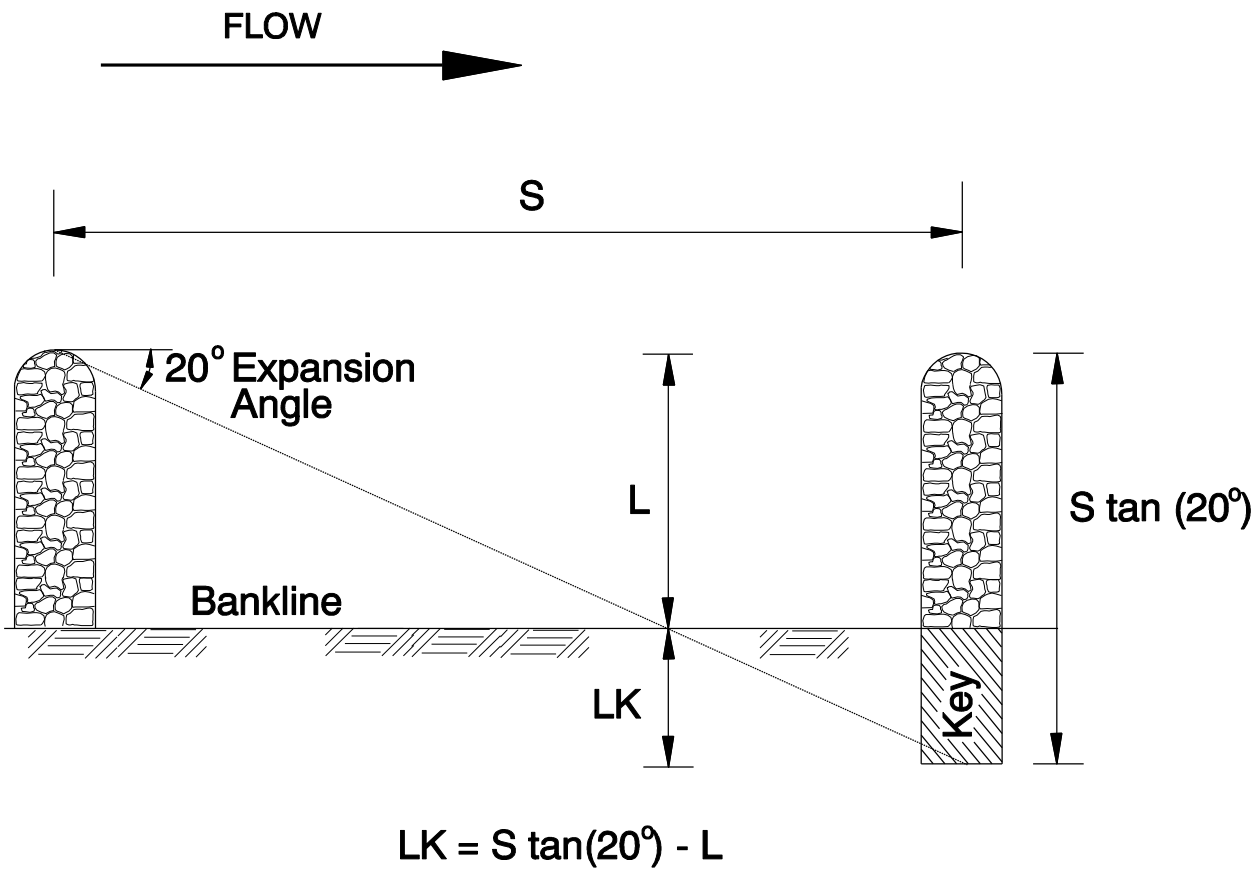



**Figure 15-15 Length of Bank Key for Mild Bends (R is more than 5W)**

### 15.5.2.8 Number of Barbs

The number of weirs should be sufficient to accomplish the task of stabilizing the bank. The weir length and spacing can be adjusted to minimize the number of barbs. Usually, a minimum of three barbs should be used.

### 15.5.2.9 Riprap Sizing

To determine the riprap size, it is recommended that the $D_{50}$ rock size obtained from the following three techniques be compared, and then the most reasonable rock size, based on

judgment and experience, should be selected. When applying the techniques listed below, use the bankfull velocity and bankfull flow depth.

1. Equation 15-6 with a stability factor of 2.0.

2. Double the rock size obtained with Figure 15-9.

3. Use the appropriate rearranged Isbash equation for sizing riprap for bridge abutment protection discussed in Subsection 15.5.1.7.5.2. It is recommended that the K factor for vertical abutments be used when applying the equations in Section 15.5.1.7.5.2.

Natural Resources Conservation Service (NCRS) recommends that the $D_{100}$ rock size should be 1.5 to 2 times the $D_{50}$ rock size, while the guidance Reference 6, recommends that the $D_{100}$ should be 3 times the $D_{50}$ rock size. Using a $D_{100}$ rock size equal to 2 times the $D_{50}$ rock size seems like a reasonable criterion. Rock should be well graded but with only a limited amount of material less than half the median stone size. The minimum rock size should not be less than the $D_{100}$ of the streambed material.

### 15.5.3 Grouted Rock

Grouted rock revetment consists of rock slope protection having voids filled with concrete grout to form a monolithic armor. Relative to loose riprap, grouted rock allows the use of smaller rock to provide the same level of protection. Since grouted rock is a rigid revetment, it will not conform to changes in the bank geometry due to settlement. As with other monolithic revetments, grouted rock is particularly susceptible to failure from undermining and the subsequent loss of the supporting bank material. Although it is rigid, grouted rock is not extremely strong and the loss of even a small area of bank support can cause failure of large portions of the revetment. Consequently, the stability of the underlying soil is critical to the long-term performance of grouted rock. If long-term stability problems are likely with the underlying soil, then grouted rock should not be used.

Experience has shown that grouted riprap will crack, cracking will be irregular, and cracks will likely extend within the grout matrix and around the periphery of larger stones. Cracking may cause enhanced weathering, including aggressive chemical reactions, but should not significantly diminish the effectiveness of the revetment if the underlying granular filter is properly designed and constructed to provide adequate drainage without loss of filter materials through cracks. Grouted rock should not be used at sites where frost heave or ice in the underlying native soil can be expected to cause a significant uplift force.

The most common application of grouted rock is to protect spillthrough abutments for bridges at sites where the velocity is too high to use loose riprap. Other applications include linings for steep gradient drainage ditches and scour protection for culvert and storm sewer outfalls. Grouted rock is also an option when a source of the required loose riprap size is unavailable.

#### 15.5.3.1 Bank Slope

Bank slopes for grouted riprap revetments should typically not exceed 1V: 1½H. Banks slopes as steep as 1V: 1H may be possible if no stability problems are expected with the supporting soil mass.

#### 15.5.3.2 Rock Size and Grout Penetration

Table 15-11 lists the maximum velocity that ODOT's standard riprap classes can withstand after being grouted to the penetration depth shown. Reference 7 was used to determine the maximum design velocity listed in the table.

#### 15.5.3.3 Hydrostatic Pressure Relief

Proper drainage is critical to the long-term performance of grouted rock. Therefore, weep holes should be provided in the revetment to relieve hydrostatic pressure buildup behind the grout surface. Weeps should extend through the grout surface to the interface with the granular filter blanket. Weeps should consist of 3-inch diameter pipes having a maximum horizontal spacing of 6-feet and a maximum vertical spacing of 10-feet. The buried end of the weep should be covered with wire screening or a fabric filter of a gage that will prevent the granular filter blanket from migrating into the weep. The minimum grout penetration equals one-half the blanket thickness.

#### 15.5.3.4 Riprap Backing

Riprap backing should be a granular filter blanket that conforms to the requirements listed in Table 15-6. For Class 50 and Class 100 riprap use the granular filter shown for Class 200 riprap.

A riprap geotextile should not be used in lieu of the granular filter blanket, but can be used as backing for the granular filter.

#### 15.5.3.5 Typical Grouted Rock Section

The typical section for grouted rock is the same as for loose riprap and is shown in Figure 15-4. The grouted rock layer should extend to the bottom of the toe trench. For example, if a 3-foot thick layer of Class 700 loose riprap is being grouted, the outer 18-inches of the 3-foot layer

should be grouted to the bottom of the toe trench. The remainder of the toe trench should be filled with loose riprap in order to provide a reservoir of rock that can provide protection against undermining as discussed in Section 15.5.1.5.

**Table 15-11 Grouted Rock**

| Standard Riprap Class | Maximum Design Velocity (feet per second) | Minimum Grout Penetration (inches) |
|---|---|---|
| Class 50 | 9 | 6 |
| Class 100 | 13 | 9 |
| Class 200 | 16 | 12 |
| Class 700 | 20 | 18 |
| Class 2000 | 24 | 24 |

### 15.5.4 Keyed Riprap

Keyed riprap, also referred to as plated riprap, is loose riprap that has been keyed in place by slapping the surface with a large piece of armor plating. A 5000 pound steel plate (approximately 4' x 5' x 6") is used to compact the rock into a tight mass and to smooth the revetment surface. The plate need only be dropped 3 to 4 feet to be effective. During the plating operation the larger stones are fractured, producing smaller rock sizes to fill voids in the blanket. Greater stability is afforded by keyed riprap over loose riprap because of the reduced drag on the individual stones and an increase in the angle of repose produced by the compacted mass of rock.

Riprap backing should be a granular filter blanket a minimum of 1-foot thick and should be 4" - 0 to 6" – 0 well graded stone embankment material. Stone embankment that is used as a granular filter blanket shall be an unweathered, hard, durable, free draining material and shall meet the test requirements of ODOT *Standard Specifications for Highway Construction*. If the underlying soil meets the gradation requirements of the granular filter blanket, then the filter blanket is not required. Using a geotextile as a riprap backing is not recommended, as it is likely to be damaged during the plating operation.

There is no design technique to determine the required keyed riprap size. As a general rule, keyed riprap can be one standard size smaller than loose riprap. For example, if a loose riprap

design calls for Class 2000 riprap, Class 800 keyed riprap will probably be adequate. Keyed riprap allows a thinner blanket thickness (usually 6 – 12 inches thinner) than loose riprap.

Since smaller rock can be used with keyed riprap, it is an option at sites where the required loose riprap size is unavailable. Keyed riprap is usually not cost effective unless there are large quantities of riprap to be placed due to the mobilization costs associated with keyed riprap. One disadvantage of keyed riprap is that it cannot be placed underwater.

Relative to loose riprap, keyed riprap gradation calls for a larger percentage of heavier stone sizes. Table 15-12 lists the gradation and blanket thickness for keyed riprap classes.

**Table 15-12 Standard Keyed Riprap Gradations**

| Percent by Weight | Class 250 | Class 800 | Class 1600 | Class 2700 |
|---|---|---|---|---|
| 27 | 250 - 150 | 800 - 400 | 1600 - 800 | 2700 – 1600 |
| 40 | 150 - 50 | 400 – 200 | 800 - 400 | 1600 – 800 |
| 23 | 50 - 25 | 200 – 25 | 400 – 50 | 800 – 100 |
| 10 | 25 - 0 | 25 – 0 | 50 – 0 | 100 –0 |
| **Blanket Thickness** | 18" | 24" | 30" | 36" |

*Note: Gradations specified in Table 15-12 are non-standard riprap classes.*

### 15.5.5 Impermeable Deflector Spurs

Spurs (sometimes referred to as jetties or groins) are defined as permeable or impermeable linear structures projecting into the channel from the bank. Spurs are used to improve channel alignment or channel width by altering the flow direction, providing channel bank protection, inducing deposition, and/or reducing flow velocity along the bank.

There are three classifications of spurs based upon their permeability: retarder spurs, retarder/deflector spurs, and deflector spurs. Deflector spurs are impermeable structures that function by diverting the flow away from the bank. Retarder/deflector spurs are permeable and function by retarding flow velocities along the bank and deflecting the flow away from the bank.

Retarder spurs are the most permeable spur type and function by retarding flow velocities near the bank. Retarder spurs rely on flow retardance and sediment deposition as their primary bank protection mechanism.

General guidance on site conditions best suited for spurs are as follows.

- Spur-type structures are typically not suited for channels less than 150 feet wide. Potential impacts when used on narrow channels are flow deflection towards the opposite bank that can initiate an erosion problem and excessive channel bed scour caused by constricting the flow. Spurs can be used effectively on small channels where their function is to shift the location of the channel.

- The use of spurs on channel bends with a radius less than 350 feet may not be cost effective when compared to other countermeasures. This is due to the short spur spacing that would be required.

- Spurs are best suited for protecting low (less than 10 feet) to medium-height (from 10 to 20 feet) banks. Higher banks may be protected, but may require special designs using construction materials other than riprap in order to limit the size (footprint) of the spur.

This section will present design guidelines for deflector spurs as they are generally best suited for the stream types and hydraulic conditions encountered in Oregon. The reader is referred to References 5 and 6 for design guidelines for permeable, as well as impermeable, spurs.

Spurs are generally used to halt meander migration at a channel bend. They are also used to channelize wide, poorly defined streams into well-defined channels. Where spurs project from highway embankments to decrease flow along the embankment, they are called embankment spurs. These may project into the floodplain rather than the channel, and thus function as spurs only during overbank flow. A common application is to better align the flow through bridge openings at sites where the channel has migrated and is misaligned with the bridge. The usual consequence of a misaligned channel relative to the bridge opening is erosion of the approach embankment and increased abutment scour.

Diverter spurs are classified as either hardpoints or transverse-dike spurs. The primary difference between the two types of spurs is the length of the spur. Hardpoints are short structures used to protect an existing bank line and are not long enough to be used for flow control or improved channel alignment. Transverse-dike spurs are longer than hardpoints and are used to move the thalweg in order to improve the channel alignment.

The most common construction material for diverter spurs is loose riprap. Other designs have been used such steel sheetpile, gabions, and rectangular or triangular timber cribs filled with rock. Abrasion of the gabion baskets is a concern and are not recommended at sites with an abrasive bed load

Spurs and stream barbs are sometimes mistaken for one another because they look similar and have similar functions. Spurs are typically bank height or near bank height structures and are significantly larger structures than stream barbs. Spurs also provide better flow control than stream barbs and are used at sites where an improved or previous flow alignment as well as bank protection is desired. Since spurs constrict/obstruct the channel more than stream barbs, deeper scour holes will develop at the tip of spurs and is a more critical design issue.

#### 15.5.5.1 Spur Length

The spur should be long enough to produce the desired effect; however, the projected length of the spur should be held to less than 20 percent of the channel width at bank full stage. The projected spur length and channel width are measured from the desired channel bank alignment. The projected spur length is the length normal to the channel. When spurs are being used to shift the channel thalweg or to improve the channel alignment at sites with eroded, irregular channel banks, the actual spur length will be greater than the projected spur length. As the projected spur length increases, the scour depth and velocity at the spur tip will also increase.

#### 15.5.5.2 Spur Spacing

Spur spacing is established by first drawing an arc representing the desired flow alignment as shown in Figure 15-16. This arc will represent the desired extreme location of the thalweg nearest the outside bank in the bend. This is shown as the low flow thalweg in the figure. The desired flow alignment may or may not differ from existing conditions, depending on whether there is a need to arrest erosion of the bank at its current location or improve the flow alignment. If the goal is to arrest the bank erosion at its current location, then hardpoints may be the proper treatment. If improving the flow alignment is the goal, then transverse-dike spurs should be used. The arc representing the desired flow alignment should provide a smooth transition between the upstream and downstream flow alignment.

The next step is to draw an arc that represents the desired bank line. This may approximately describe the existing bank alignment or a new desired bank line that protects the existing bank line from further erosion. Also, draw an arc connecting the crest of the nose or tip of the spur field. This arc should encroach no more than 20 percent of the desired channel width as discussed in Subsection 15.5.5.1. The distance from this arc to the arc describing the desired bank line, along with the flow expansion angle, fixes the spacing between spurs. The arc

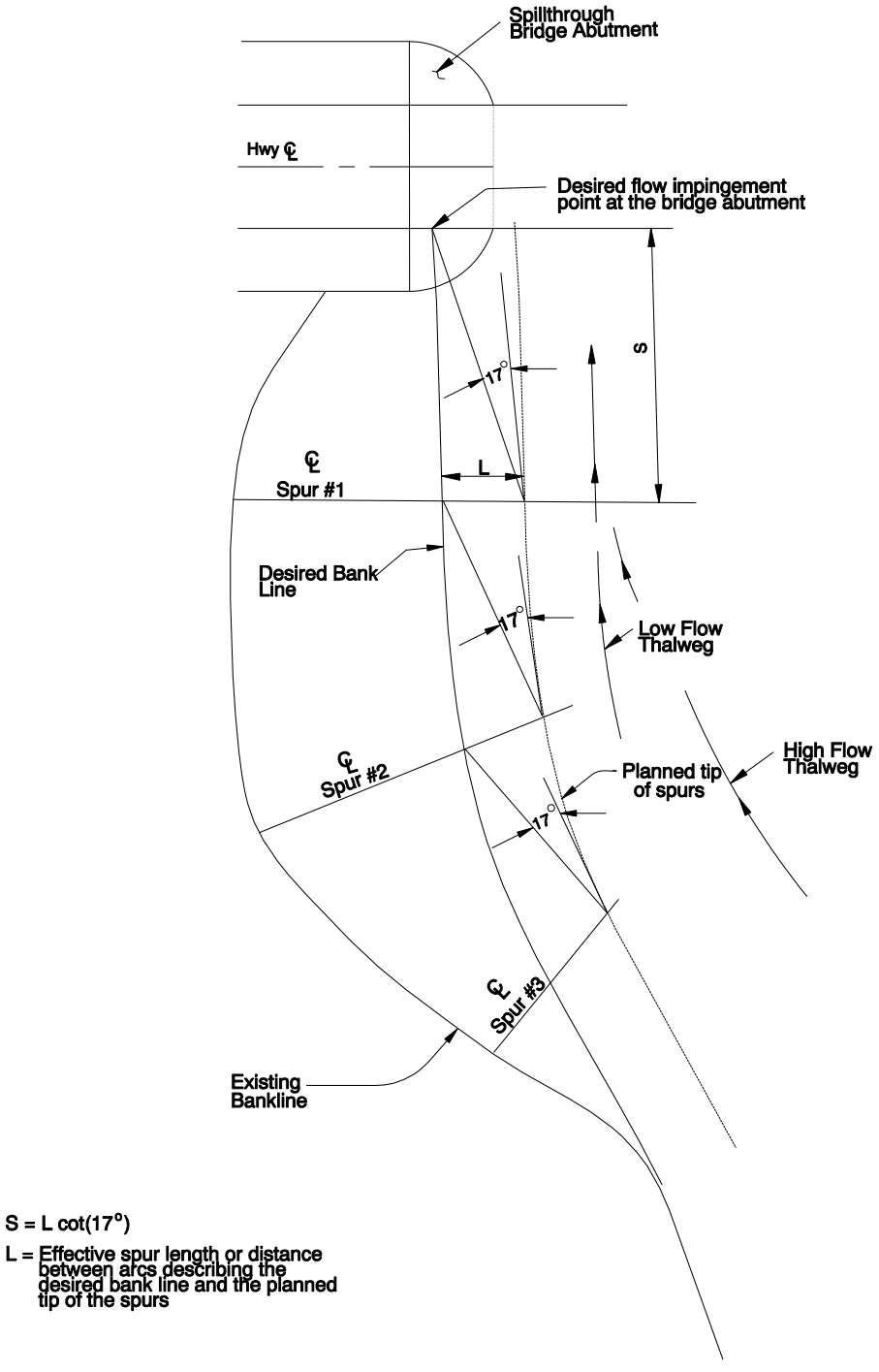



**Figure 15-16 Spur Spacing in a Channel Bend**

describing the tip of the spurs will be essentially concentric with the arc describing the desired flow alignment.

The next step is to establish the location of the spur at the downstream end of the spur field. The first step is to determine the desired flow impingement point downstream of the first spur. As previously mentioned a common application for spurs is to better align the flow at bridge sites were the channel has migrated and is misaligned with the bridge. For bridges with vertical abutments the desired impingement point will usually be the upstream corner of the abutment, but it may be any point within the width of the bridge or possibly a point downstream of bridge. For bridges with spillthrough abutments the desired impingement point will usually be a point along the abutment slope equal in elevation to the top of bank elevation at the upstream face of the bridge. As with vertical abutment bridges, the desired impingement point may be located within the width of the bridge or possibly along the bank downstream of the bridge. There may be cases that do not involve a bridge where a channel bend has migrated and is flowing adjacent to or impinging on the highway embankment. For this case the desired impingement point will be a point along the downstream channel bank judged to be stable.

*Note: The potential scour elevations within the waterway opening should be analyzed where spurs are used adjacent to structures. It should be verified that the spurs do not cause scour or revetment retention problems.*

For impermeable spurs the flow will expand at an angle of approximately 17 degrees from the tangent of the flow path at the tip of the spur. Therefore, it is important to determine the flow path at the tip of the spur. The flow path or thalweg needs to be estimated for low flow as well as high flow conditions because the thalweg has a tendency to shift during high flows as shown in Figure 15-16. The spacing between spurs and the distance to the first spur from the desired impingement point is determined with the following equation:

$$\mathbf{S = L\left(\cot 17^{\circ}\right) = 3.27L} \qquad \textbf{(Equation 15-22)}$$

Where:

S = Spacing between spurs at the crest of the spur tip, feet

L = Effective length of the spur or the distance between arcs describing the planned tip of the spur and the desired bank line, feet

ODOT experience with this spacing technique is that it is conservative and can sometimes result in very close spacing with the consequence that more spurs will be specified than are actually needed. Therefore, when determining the spacing, the designer may want to consider an impingement point between spurs that lies somewhere between the existing bank line and the desired bank line. The final spacing will be determined by the judgment of the designer. To assist

the designer in determining the final spacing of the spurs, it is recommended that the footprint of the proposed spurs be drawn on a plan view of the site.

### 15.5.5.3 Spur Height and Crest Profile

Impermeable spurs are generally designed not to exceed the bank height in order to protect against flanking of the spur by flow concentration and erosion behind the spur at high river stages. Where river stages are more than or equal to the bank height, the spurs should be equal to the bank height. If flood stages are lower than bank height, the spur should be high enough to prevent overtopping of the spur. Overtopping of spurs by as much as 3 feet does not affect the spurs' effectiveness.

The crest of impermeable spurs should slope slightly downward away from the bank line mainly because it is difficult to construct and maintain a level spur profile constructed with rock.

### 15.5.5.4 Bed and Bank Contact

The spur should be keyed a minimum of 6 feet into the channel bank to protect the spur from being flanked when flood stages overtop the spur. The nose or tip of the spur is vulnerable to being undermined by the scour hole that will develop due to the flow contraction that occurs at the spur tip. Therefore, the spur tip needs to be protected against undermining from scour. Two methods are used to provide scour protection. The preferred method is to key the spur tip into the channel bottom with a toe trench. A 4 – 6 foot deep toe trench will usually provide adequate protection. The alternate method is to place excess stone at the spur tip. As scour occurs the rock will self-launch or roll into the scour hole and protect the spur tip from being undermined. Adequate bed and bank contact is particularly important for the leading spur because it is subjected to the most intense hydraulic conditions.

### 15.5.5.5 Spur Orientation

Spur orientation refers to spur alignment with respect to the direction of the main flow current in the channel. Figure 15-17 defines the spur angle such that an acute angle means that the spur is angled in a downstream direction and an angle greater than 90 degrees means that the spur is oriented in an upstream direction.

There is no consensus regarding the orientation of impermeable deflector spurs. Spurs oriented at approximately 90 degrees have been found to force the thalweg farther from the bank than spurs oriented in an upstream or downstream direction. Therefore, more positive flow control is achieved with spurs oriented normal to the channel bank. Scour at the tip of a spur oriented in an upstream direction is greater than if oriented normal to the bank while spurs oriented in a

downstream direction experience less scour at the spur tip. Another effect that has been observed to be a function of spur orientation is the increased potential for bank erosion immediately downstream of the spur when the spur crest is overtopped. When overtopped, spurs oriented in a downstream direction tend to deflect the flow towards the bank, which can cause the bank to erode.

It is recommended that the spur furthest upstream be angled downstream to provide a smooth flow transition as the flow enters the spur field and to minimize scour at the tip of the spur. A spur angle of 30 degrees is suggested for the leading spur. If the leading spur will be overtopped, the designer should consider the need to protect the channel bank immediately downstream from the spur. The remaining spurs should all be set normal to the bank line or angled slightly upstream (±10 degrees) to minimize construction costs.

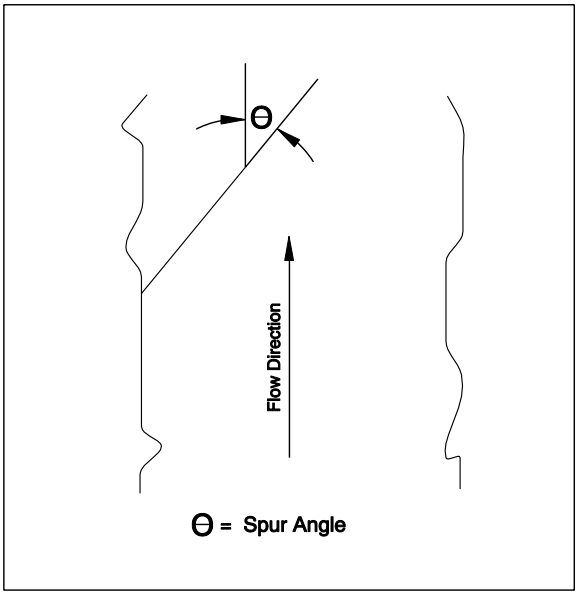



**Figure 15-17 Definition Sketch for Spur Orientation**

### 15.5.5.6 Riprap Sizing

To determine the riprap size it is recommended that the $D_{50}$ rock size obtained from the following three techniques be compared and then the most reasonable rock size, based on judgment and experience, should be selected. When applying the techniques listed below, use the

bankfull velocity and bankfull flow depth at the constricted channel cross-section at the tip of the spur.

Method 1: Equation 15-6 with a stability factor of 2.0.
Method 2: Double the rock size obtained with Figure 15-9.
Method 3: Use the appropriate equation for sizing riprap for bridge abutment protection discussed in Subsection 15.5.1.7.5.2. It is recommended that the K factor for spillthrough abutments be used when applying the equations in Subsection 15.5.1.7.5.2.

Use a standard riprap class if the $D_{50}$ is within the range of $D_{50}$ values shown in Table 15-3 otherwise design a special riprap gradation that meets the gradation requirements shown in Table 15-4.

### 15.5.5.7 Typical Rock Spur Details

Figures 15-18 and 15-19 show typical spur details. The side slopes of the spur should be no steeper than 1V: 1½H. The top of the spur should be a minimum of 4 feet wide. For cases where the top of the spur is used for construction access, the spur will need to be wide enough to accommodate the construction equipment. For construction access the spur may need the top to be 10 – 12 feet wide. Figure 15-20 shows rock spurs that were constructed to better align the flow with a bridge opening.

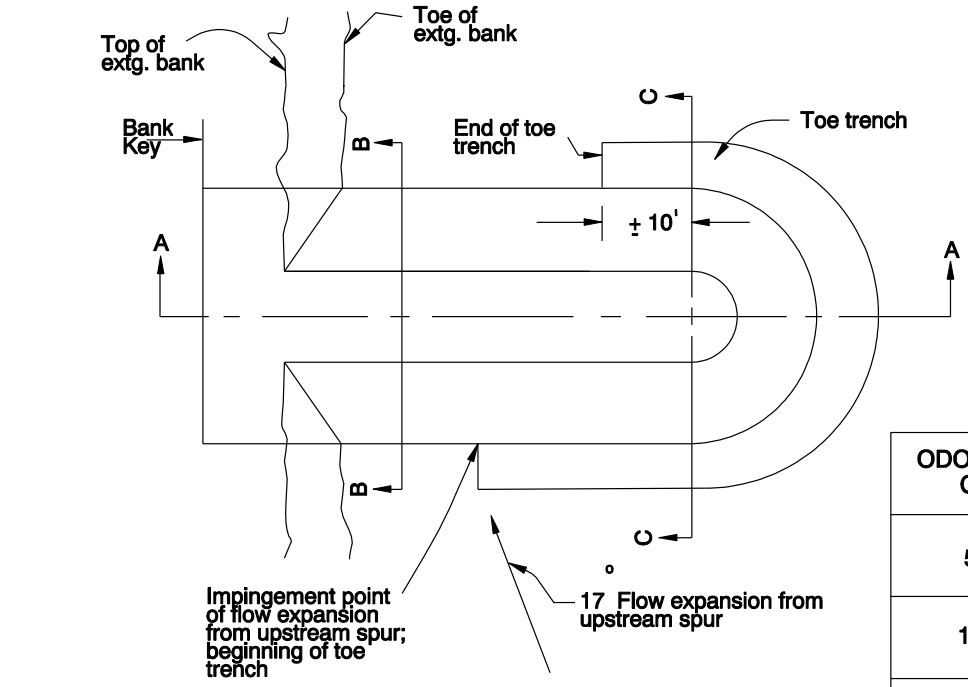
### 15.5.6 Proprietary Products

Several proprietary products are available for bank protection. This section will briefly discuss several of the products available. The reader is referred to the various manufacturers for more detailed information on the products discussed.

### 15.5.6.1 Gabions

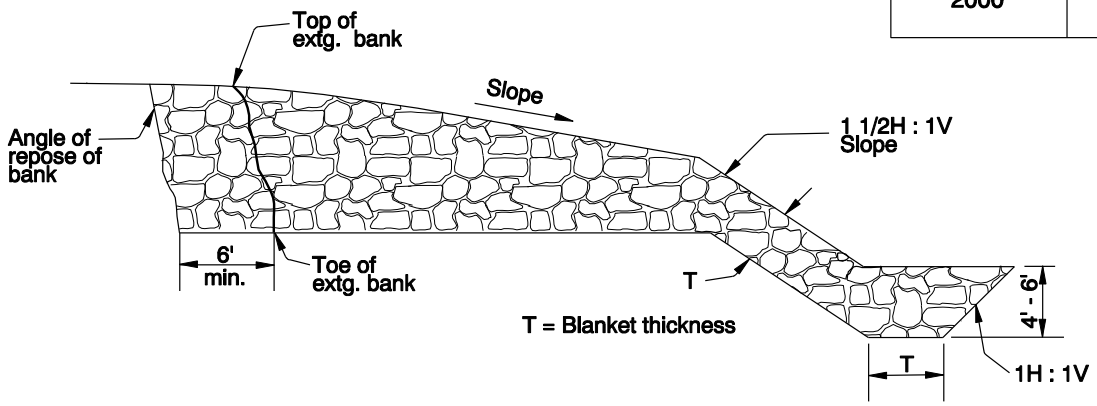
Gabions are rectangular wire mesh baskets filled with rock. The gabion baskets are commercially available from several manufacturers. Wire enclosed riprap is similar to gabions, but differs in that it is a continuous framework rather than individual interconnected baskets.

When used for bank protection, gabions are either placed as a mattress on a prepared slope or they are constructed as a retaining wall with a stepped block configuration as shown in Figure 15-21. To construction a gabion revetment empty gabions are placed in position, wired to adjoining gabions, filled with rock, and then folded shut and wired at the ends and sides.




SPUR PLAN
No Scale

| ODOT Riprap Class | T (feet) |
|---|---|
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |




SECTION A - A
No Scale

**Figure 15-18 Typical Rock Spur Details**

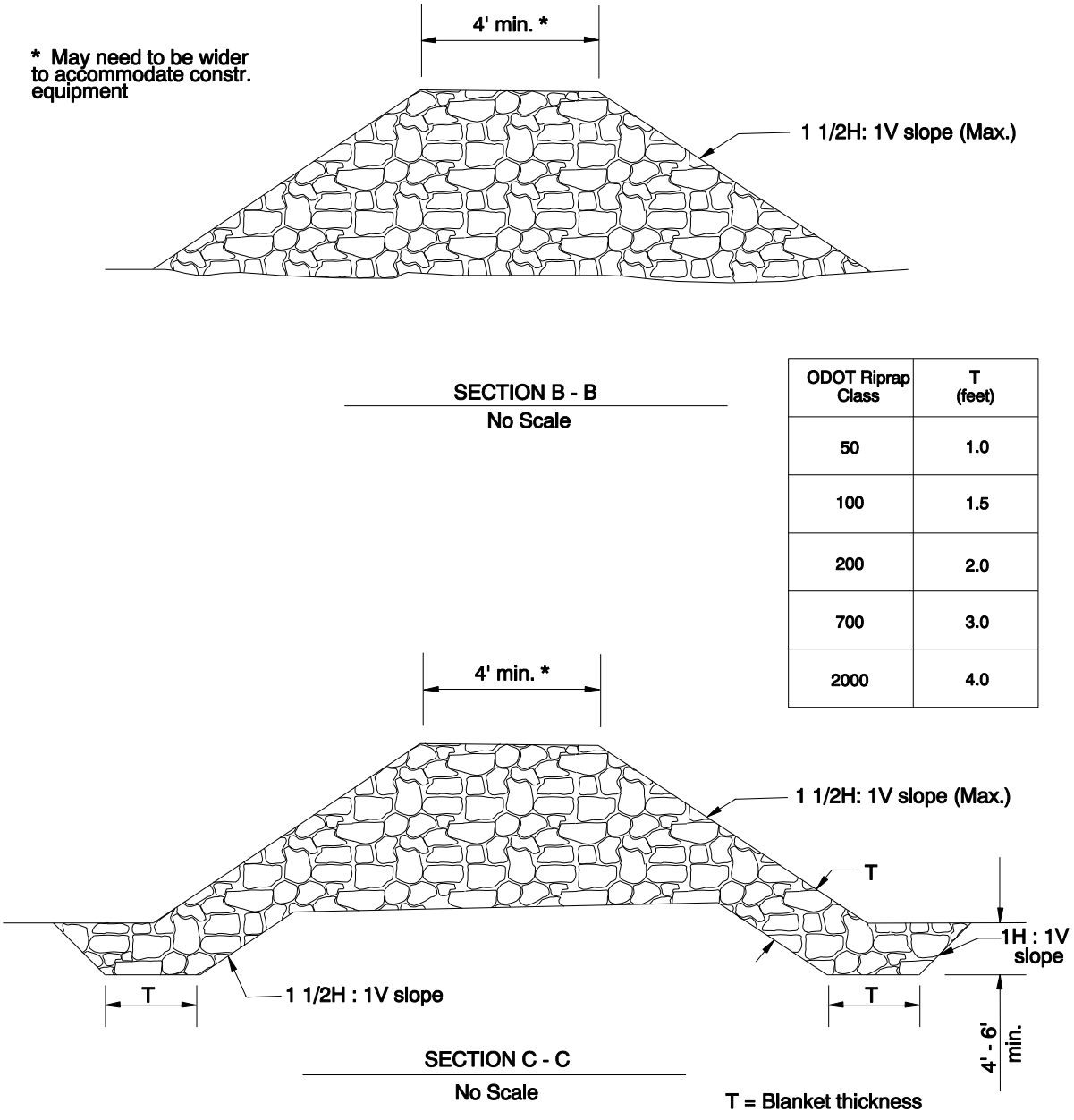



**Figure 15-19 Typical Rock Spur Details**

**Figure 15-20 Rock Spurs**

Material and construction specifications for gabions can be found in ODOT *Standard Specifications for Highway* Construction.

The following is a list of considerations and guidelines for the use of gabions.

- Regulatory agency standards and criteria may apply to gabions installed below the 2-year water surface elevation, and the design should satisfy these requirements.

- Experience with gabions is that they have a short service life in streams that have a high bed load. The bed load abrades and causes eventual failure of the wire basket. To avoid the abrasion problem gabions should not be used for toe protection on streams

- Gabions are available with stainless steel wire. This material may be cost-effective for submerged or partially submerged gabions.

- Gabions should be protected against impact from large woody debris and sharp objects, as impact from these materials tends to distort and break the gabions. As with abrasion this could lead to failure of the gabions.

- Gabions are an option at sites where a loose riprap design requires a rock size greater than what is locally available.

- When a stepped block retaining wall configuration is used, the stability of the gabions against overturning and sliding must be analyzed in a manner similar to gravity retaining walls. The allowable bearing pressure of the foundation material also needs to be considered in the design.

- When placed as a mattress, the underlying slope stability should be evaluated by a geotechnical engineer for slopes steeper than 1V: 2H.

- The stepped block configuration is useful where space is limited and a more vertical structure is required.

- For the stepped block configuration the vertical joints between adjacent rows should be staggered similar to a brick wall. The adjacent rows are typically staggered one-half of the basket length.

- Vegetation can be incorporated in gabions by placing live branch cuttings between the rows in a stepped block configuration or through the baskets when placed as a mattress. The live branches should range from 0.5 to 2.5 inches in diameter and must be long enough to reach beyond the back of the baskets into the backfill or underlying soil. If large woody vegetation is allowed to grow in gabions, there is a risk that the baskets will break when the large woody debris is uprooted or as the root and trunk systems grow. Thus, it is normally not acceptable to allow large woody vegetation to grow in the baskets. The maximum size of the vegetation should be limited to the wire-mesh opening size in the gabion basket. Wire-mesh opening sizes are shown in Table 15-14.

- For gabion mattresses the thickness of the mattress can be determined by considering three factors: the erodibility of the underlying bank soil, the maximum flow velocity and the bank slope. The minimum thickness required for various conditions is tabulated in Table 15-13. These values are based on observations of a large number of mattress installations that assume a filling material in the size range of 3 to 6 inches. The allowable velocity is also related to the $D_{50}$ of the rock used to fill the gabions. The allowable velocity increases as the $D_{50}$ increases. Gabion manufacturers provide guidance on stone sizing and allowable velocities and it is suggested that they be consulted when selecting the mattress thickness. NRCS recommends that a gabion mattress should be a minimum of 9 inches thick for stream velocities up to 9 feet per second and the thickness should be increased to a minimum of 18 inches for stream velocities from 10 to 14 feet per second.

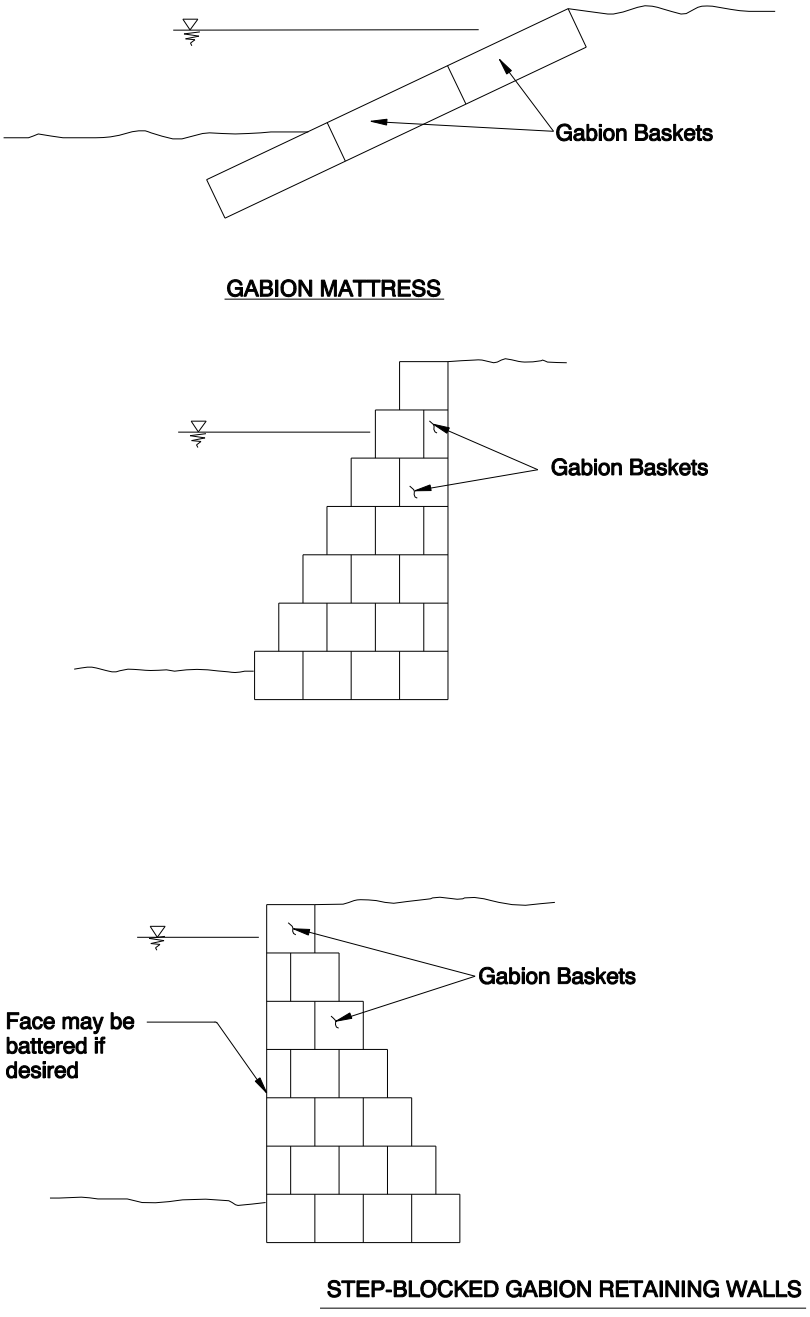



**Figure 15-21 Typical Gabion Configurations**

**Table 15-13 Criteria for Gabion Mattress Thickness**

| Bank Soil Type | Maximum Velocity (feet per second) | Bank Slope (V:H) | Minimum Required Mattress Thickness (inches) |
|---|---|---|---|
| Clays, heavy cohesive soils | 10<br>13 - 16<br>any | less than 3:1<br>less than 2:1<br>more than 2:1 | 9<br>12<br>More than or equal to 18 |
| Silts, fine sands | 10 | less than 2:1 | 12 |
| Shingle with gravel | 16<br>20<br>any | less than 3:1<br>less than 2:1<br>more than 2:1 | 9<br>12<br>More than or equal to 18 |

Table 15-14 summarizes standard gabion basket sizes that are commercially available. Alternative sizes can be manufactured when required. The 3-foot wide baskets are usually used for the stepped block retaining wall, while the 6-foot and 9-foot widths are used to construct mattresses. Additional information on gabions can be found at the various manufacturers' websites and in Reference 7.

### 15.5.6.2 Pre-Cast Concrete Blocks

Pre-cast concrete block revetments consist of pre-cast blocks that interlock with each other, are attached to each other, or butt together to form a continuous mat. The individual blocks are sometimes connected with steel cable, steel rods or synthetic fiber rope (e.g., nylon or polyester). When the individual blocks interlock or are connected with cable or rope, they are often referred to as articulating block mats. The blocks are typically 5 – 9 inches thick.

The various concrete block systems have a limited amount of flexibility that allows the mat to conform to minor changes in the bank geometry; however, their limited flexibility makes them vulnerable to undermining and/or flanking. Failures have occurred where a corner or edge of the mat is undercut, resulting in complete failure of the revetment. Consequently, protection against undermining or flanking is a critical part of the design. The permeable nature of these revetments permits free draining of the underlying soil. The blocks that comprise the mat are available with

open areas within and between the blocks that can be filled with soil and planted with vegetation. Pre-cast concrete blocks are an option to consider where the required loose riprap size is not readily available. Figure 15-22 shows an articulating block mat used to protect a bridge abutment. Additional information on pre-cast concrete blocks can be found at the various manufactures websites. Design guidelines can be found in References 6 and 7.

**Table 15-14 Standard Gabion Basket Sizes**

| Length (feet) | Width (feet) | Thickness (feet) | Wire-mesh Opening Size (inches x inches) |
|---|---|---|---|
| 6 | 3 | 3 | 3.25 x 4.5 |
| 9 | 3 | 3 | 3.25 x 4.5 |
| 12 | 3 | 3 | 3.25 x 4.5 |
| 6 | 3 | 1.5 | 3.25 x 4.5 |
| 9 | 3 | 1.5 | 3.25 x 4.5 |
| 12 | 3 | 1.5 | 3.25 x 4.5 |
| 6 | 3 | 1 | 3.25 x 4.5 |
| 9 | 3 | 1 | 3.25 x 4.5 |
| 12 | 3 | 1 | 3.25 x 4.5 |
| 4.5 | 3 | 3 | 3.25 x 4.5 |
| 9 | 6 | 0.5 | 2.5 x 3.25 |
| 12 | 6 | 0.5 | 2.5 x 3.25 |
| 9 | 6 | 0.75 | 2.5 x 3.25 |
| 12 | 6 | 0.75 | 2.5 x 3.25 |
| 9 | 6 | 1 | 2.5 x 3.25 |
| 12 | 6 | 1 | 2.5 x 3.25 |
| 99 | 6 | 1 | 3.25 x 4.5 |
| 99 | 9 | 1 | 3.25 x 4.5 |
| 99 | 6 | 1.5 | 3.25 x 4.5 |
| 99 | 9 | 1.5 | 3.25 x 4.5 |

**Figure 15-22 Articulating Block Mat**

### 15.5.6.3 Grout Filled Mattresses

Grout filled mattresses utilize a double-layer woven geotextile that serves as a flexible form for casting concrete. The mattress is positioned on the area to be protected and then inflated by pumping a structural grout into the mattress. Grout may be pumped into the mattress either underwater or in the dry. The mattresses are typically 3 – 8 inches thick and when pumped full of grout, the surface of the mattress has a rectangular block pattern. The mattresses are available with and without reinforcing cables. The reinforcing cables can be either steel or nylon and form a grid like pattern in the mattress. The mattresses usually have a built-in system to relieve hydrostatic pressure, but weep holes may be placed through the mattress if necessary. The mattresses are rigid structures although some mattresses offer a limited amount of flexibility. Their limited flexibility makes the mattresses vulnerable to undermining and/or flanking.

Consequently, protection against undermining or flanking is a critical part of the design. Additional information on grout filled mattresses can be found at the various manufactures websites. Figure 15-23 shows a grout filled mattress used to protect a bridge abutment. The reader is referred to Reference 6 for detailed design guidelines.

### 15.5.7 Biotechnical Stabilization

Biotechnical stabilization refers to the combined use of structural measures, such as riprap revetments or gabions, and vegetative plantings to provide bank protection. The structural measures are used to provide toe protection while the vegetative plantings are used to protect the remainder of the bank. Biotechnical stabilization is often called soil bioengineering although soil bioengineering is actually a component of biotechnical stabilization. The soil bioengineering component consists entirely of biodegradable material such as large woody debris, erosion control fabrics made from natural materials, and the vegetative plantings.

**Figure 15-23 Grout Filled Mattress**

This section addresses the design of biotechnical bank stabilization and discusses the various components of biotechnical stabilization including the stability of large woody debris, selecting an adequate erosion control mat, toe protection and the various vegetative techniques that are used. Typical designs are also presented.

*Note: Biotechnical methods will require routine maintenance and repairs. Access and maintenance should be evaluated and addressed during project development. The design should assure that proper easements, right-of-way acquisition, agreements, or other means are obtained to allow for access and maintenance activities and adequate area is allocated for these activities. The facilities required for access should also be incorporated into the project design. This could include turnouts, access roads, removable fences, etc.*

### 15.5.7.1 Use of Vegetation

Vegetation is an important component of biotechnical stabilization. Vegetation can effectively control erosion while providing environmental benefits. Environmental benefits include diverse and productive riparian habitats, shade to help maintain suitable water temperature for fish, cover for fish, source of nutrients for aquatic life, and improved water quality. Vegetation, if dense enough, can control erosion and provide bank protection by performing the following functions.

- The root system binds the soil together and increases the overall stability and shear strength of the bank.

- The exposed vegetation increases surface roughness and reduces local flow velocities, causing the flow to dissipate energy against the deforming plant rather than the soil. This also reduces the transport capacity and shear stress near the bank, thereby inducing sediment deposition.

- Vegetation deflects the high-velocity flow away from the bank and acts as a buffer against the abrasive effect of transported material.

Vegetative methods normally use un-rooted plant parts in the form of cut branches, although containerized plants and bare-root stock are also commonly used. Vegetative techniques often referred to as soil bioengineering include live stakes, live fascines (also called wattles), brush layering (also called branch packing), and brush mattresses. These techniques are most successful when constructed during the dormant season, which is typically mid-October to mid-March. Many of the vegetative techniques incorporate a natural or synthetic erosion control mat (see Subsection 15.5.7.3) to reinforce the stream bank and to provide protection until the vegetation is established. The reader is referred to References 8 and 9 for more detailed design guidance on vegetative techniques:

Live stakes are live, rootable vegetative cuttings that are inserted into the ground. The live stakes are typically 0.5 – 1.5 inches in diameter, 2 – 3 feet long, and are installed 2 –3 feet apart. When live stakes are driven through a layer of riprap, the planting technique is known as "joint planting". Live fascines are long, cylindrical bundles of branch cuttings that are bound together and placed in a shallow trench parallel to the bank. The bundles are typically 6 – 8 inches in diameter and secured to the bank with live and dead stakes.

Brush layering consists of dense layers of live cuttings, branches, and/or rooted stock between layers of compacted soil. Cuttings extend back into the bank and protrude slightly from the bank. They are particularly applicable in bank protection projects that require fill.

A brush mattress is a thick layer of overlapping live cuttings or branches placed on the surface of a graded stream bank and secured with a combination of twine, wire and live and dead stakes. The bottom edge of the mattress is often anchored with a live fascine.

ODOT's landscape architect should be consulted for guidance on plant selection. The plant materials should be native species that are suited to the soil, moisture, available sunlight and climatic conditions of the site. As an example, biotechnical methods may not be suitable under bridges because of the lack of sunlight that is needed for sustained growth. Flood tolerant plant species need to be used when planted in areas that may be partially or totally submerged for several weeks of the year. Species that root easily such as willow are required for measures such as live stakes and live fascines. As a general rule, the most successful planting plan is one that mimics the type and distribution of vegetation that is growing on adjacent channel banks that are stable.

*Note: The designer should visit the site and evaluate the pattern of plant growth on the adjacent channel banks. This pattern will reveal horizontal dividing line along the channel bank where and where not vegetation growth occurs. For successful application of biotechnical methods; biotechnical methods should be applied above this line and non-biotechnical methods should be applied below this line.*

Following the recommendation of planting during the dormant season will likely conflict with the in-water work period. Therefore, the required toe protection will probably need to be constructed during the in-water work period with the vegetative component constructed outside of the in-water work window unless irrigation or other means is provided to sustain the plants until the rainy season. Measures to isolate the work area from the stream channel will need to be in place prior to construction.

The performance of bank protection methods that use vegetation depends upon the ability of the plant's root system to reinforce the underlying soil. The current state-of-the-art, however, lacks a practical design method to quantify the performance or evaluate the safety factor of bank protection methods that use vegetation. Therefore, vegetation should be used only at sites that can tolerate a failure without endangering the public or causing extensive damage to the highway, structures, or other developed property. Vegetation will usually provide inadequate protection at sites subject to high velocities such as bridge abutments and the outside bank of channel bends. Where establishment of the vegetation is critical to the long-term stability of the bank, a commitment needs to be made to maintain the site until the vegetation gets established.

### 15.5.7.2 Toe Protection

As with other bank protection techniques, a stable toe is the most critical component of a biotechnical design as failure of the toe can lead to total failure of the bank protection. Riprap is the most common material used for toe protection. The riprap size can be determined as discussed in Subsection 15.5.1.7. The top of the riprap will typically extend slightly higher than the water surface elevation present during construction or up to the natural vegetation line of the adjacent bank. As a minimum the top of riprap should extend up to the vegetation line of the adjacent bank.

Limited ODOT experience with biotechnical protection indicated that a dense stand of plants on a channel bank will concentrate flow velocities at the toe. This occurs where biotechnical slope protection is used on the outside of bends. Larger riprap than the sizes calculated by methods in Subsection 15.5.1.7 may be needed.

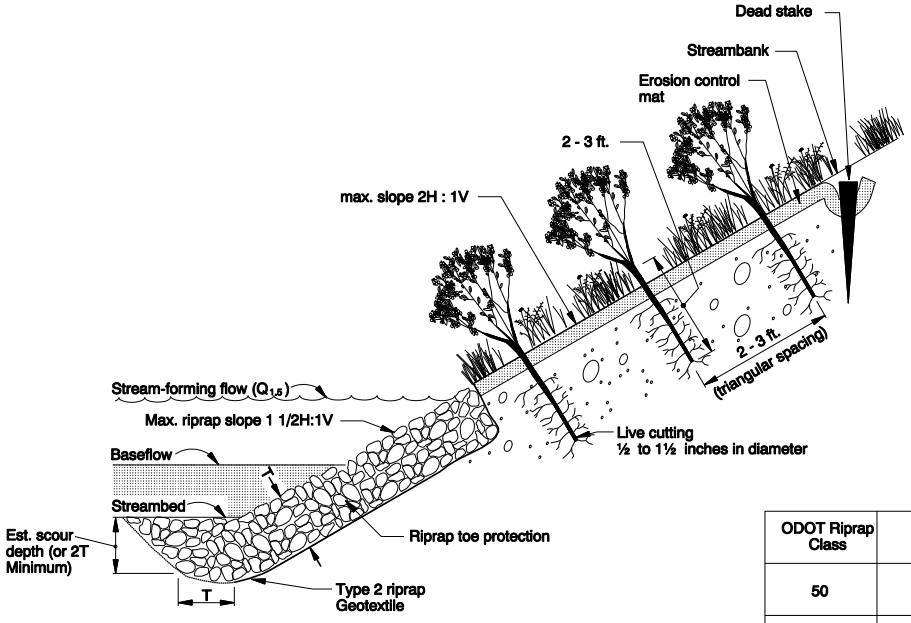
### 15.5.7.3 Selection of Erosion Control Mat

Erosion control mats, in conjunction with vegetation, are often used in biotechnical bank protection schemes. The role of the erosion control mat is to provide bank protection while the vegetation is becoming established. The mats are proprietary products and can be broadly classified as either degradable or non-degradable. Degradable mats are made from natural materials such as coir and photodegradable synthetic products and typically provide protection from 1 – 5 years. Non-degradable mats are made from synthetic materials and will typically provide protection for at least ten years after installation. Since the erosion control mats have a limited life, it is important that the vegetation becomes well established, particularly for the degradable mats. At some sites the erodibility of the underlying soil may be a concern in which case a mat should be selected that can essentially act as a filter and prevent the underlying soil from migrating through the erosion control mat.

To select erosion control mat first compute the shear stress the mat will be subjected to and then select a mat that can tolerate the computed shear stress. The shear stress is calculated as explained in **Chapter 8**, Channels. Permissible shear stresses for various materials can also be found in **Chapter 8**. Additional permissible shear stresses for various erosion control mats can be found in the Texas Department of Transportation's approved product list for flexible channel liners. The approved product list is available at the following website: www.dot.state.tx.us. The products tested had permissible shear stresses from 2 – 8 pounds per square foot. It should be noted that the products were tested after vegetation had been growing in the test trays for a period of 90 days. Therefore, the designer may want to apply a safety factor to the permissible shear stress to account for the lack of vegetation on the mats at the time of installation.

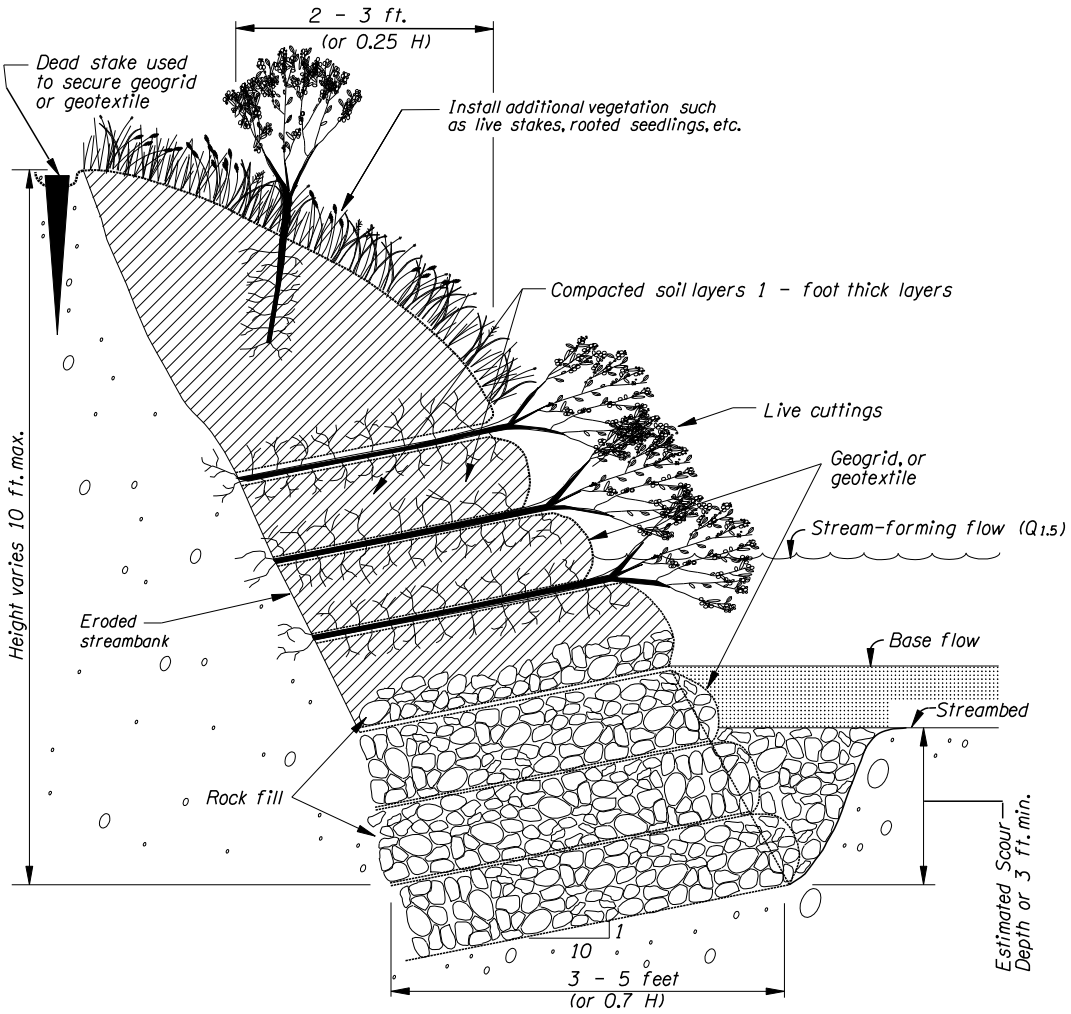
### 15.5.7.4 Typical Designs

The following figures show typical details for several biotechnical designs.




| ODOT Riprap Class | T (ft) |
|---|---|
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |

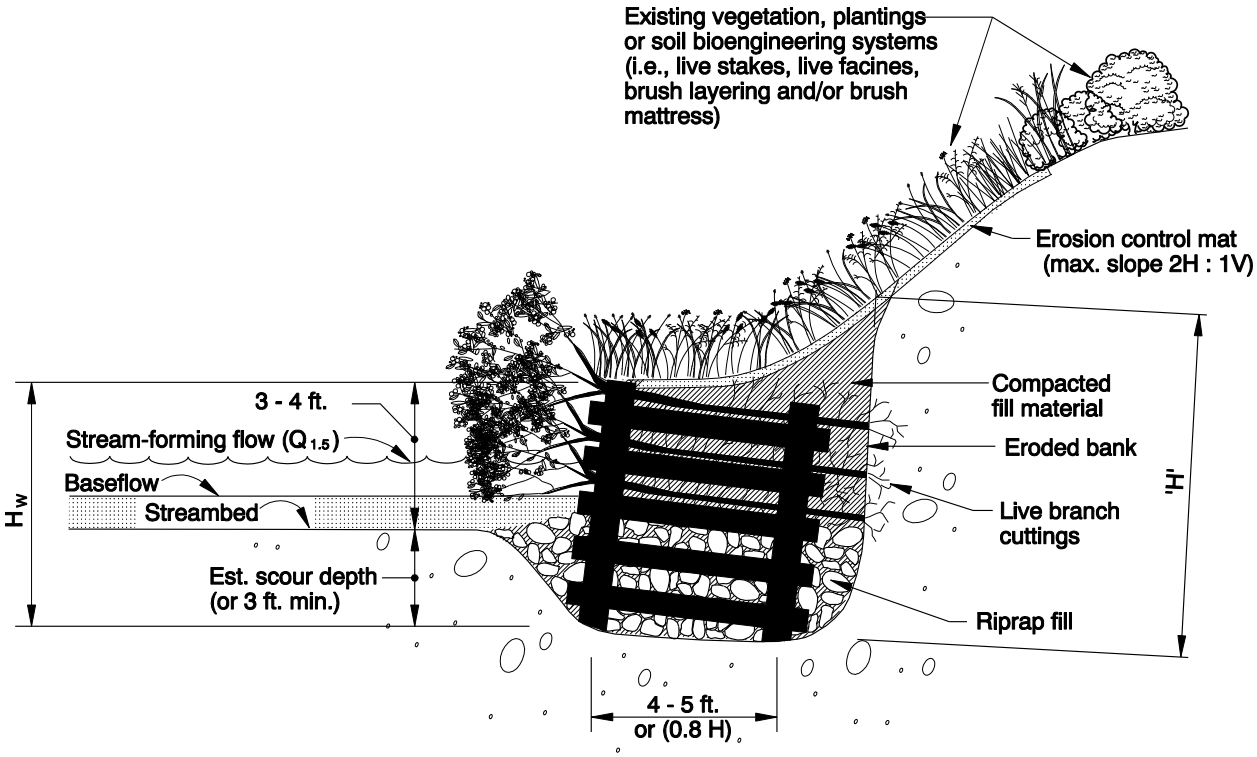
**Figure 15-24 Live Stakes with Riprap Toe Details**




Notes:

1. This detail is for over-steepened, eroded banks 10 feet or less height (H). Vegetated geogrids that are higher than 10 feet will require a stability analysis.
2. Rooted/leafed condition of the living plant material is not representative of the time of installation.
3. Adapted from USDA, NRCS, Engineering Field Handbook, Chapter 16

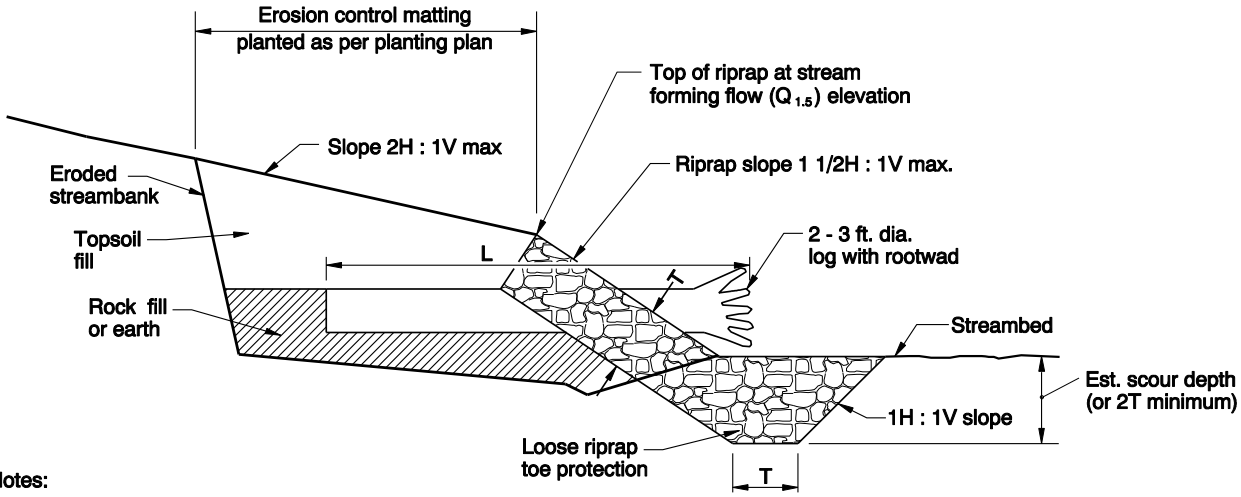
**Figure 15-25 Vegetated Geogrid Details**




Notes:

1. Design shown is for an over-steepened bank 6 feet or less in height ('H'). Cribwalls that are higher than 6 feet will require a stability analysis.
2. Cribwall is constructed with untreated log or timber members.
3. Batter cribwall 1H:10V.
4. Rooted/leafed condition of the living plant material is not representative of the time of installation.
5. Useful where space is limited and a more vertical structure is required.
6. Adapted from USDA, NRCS, Engineering Field Handbook, Chapter 16.

**Figure 15-26 Live Cribwall Detail**

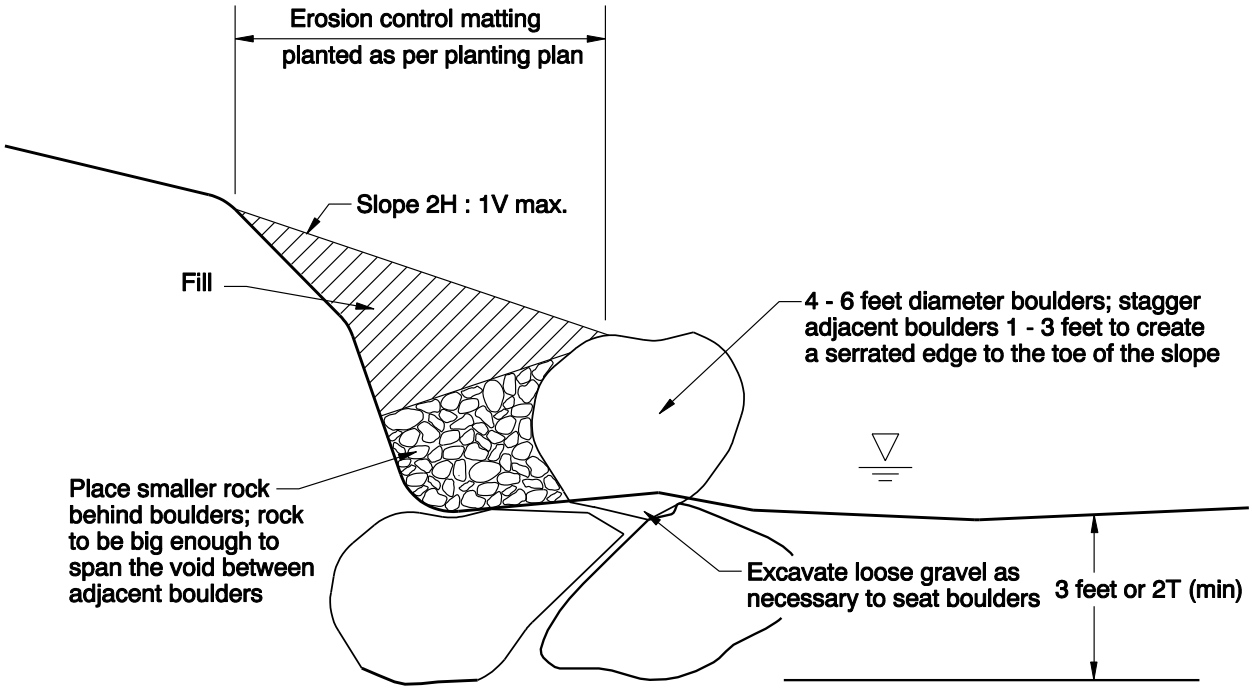



Notes:

1. Log length, L, must be embedded in embankment far enough to resist the buoyant forces. Logs are typically 15 - 20 feet long.

| ODOT Riprap Class | T (feet) |
|---|---|
| 50 | 1.0 |
| 100 | 1.5 |
| 200 | 2.0 |
| 700 | 3.0 |
| 2000 | 4.0 |

**Figure 15-27 Riprap with Rootwad Details**




Notes:

1. Stream forming flow ($Q_{1.5}$) should be no higher than the top of the boulders.

2. Do not use where scour at the toe will undermine and destabilize the boulders.

**Figure 15-28 Boulder Toe Details**

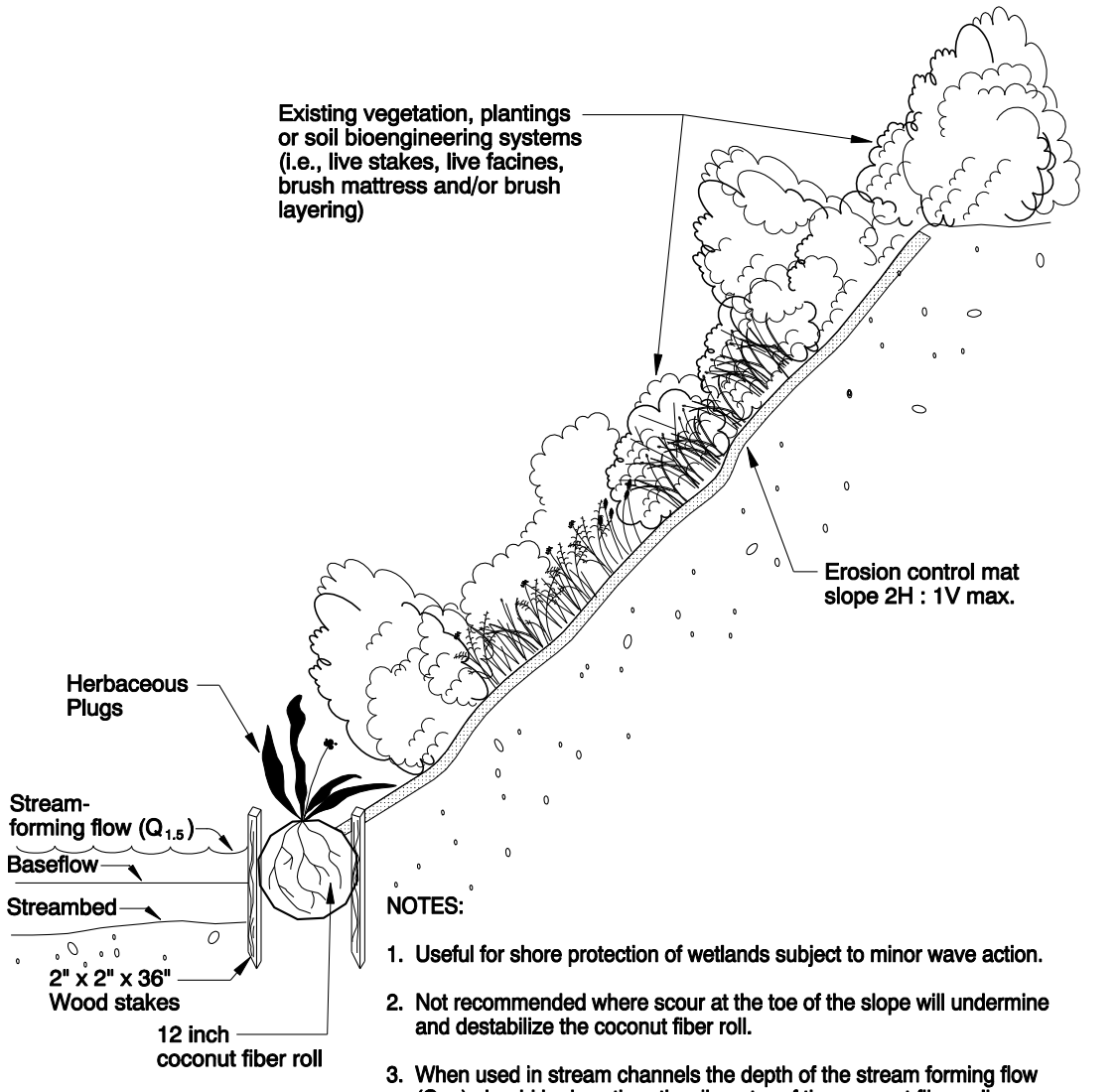



**Figure 15-29 Coconut (Coir) Fiber Roll Details**

### 15.5.7.5 Stability of Large Woody Debris

Large woody debris (logs with rootwads) is sometimes buried in riprap in order to provide better fish habitat than would be provided by the riprap alone. The woody debris is buried near the toe of the riprap with the rootwad projecting from the riprapped slope. The forces acting on the large woody debris need to be calculated in order to ensure that the woody debris will be stable. This section presents a method to calculate the stability of large woody debris. The following assumptions are made in the stability analysis presented.

- The analysis presented in the following examples is a buoyancy analysis. The horizontal drag force and other forces acting on the exposed woody debris are ignored in the analysis.

- The weight of the earth above the woody debris that resists the buoyant force is defined by a rectangular section with the width equal to the diameter of the woody debris (see Figure 15-30 in Example 1)

As a rule of thumb the woody debris will usually be stable if two thirds of the log is buried in the bank.

Table 15-15 tabulates the decay resistance, specific gravity and the unit weight for various species of trees that grow in Oregon.

In Table 15-15, $G_b$ is the basic specific gravity and is based on ovendry weight and green volume. Green refers to freshly sawed or undried wood. $G_{15}$ is the specific gravity for moisture content of 15 percent. The unit weight listed also corresponds to moisture content of 15 percent. A moisture content of 15 percent is a conservative estimate of the moisture content of large woody debris at the time of placement. Unless known otherwise, assume the moisture content is 15 percent. $G_{15}$ was calculated with the following equation, which adjusts $G_b$ for moisture content:

$$G_m = \frac{G_b}{1 - 0.265 a G_b} \qquad \textbf{(Equation 15-23)}$$

Where:

$G_m$ = specific gravity based on volume at moisture content M
$G_b$ = basic specific gravity

$$a = \frac{30 - M}{30}$$ M = Percent moisture content

When $M \geq 30$ percent, $G_m = G_b$.

The unit weight, $\gamma$, shown in Table 15-15 is calculated with the following equation:

$$\gamma = 62.4 G_m \left(1 + \frac{M}{100}\right) \qquad \textbf{(Equation 15-24)}$$

Equations 15-23 and 15-24 can be used to calculate the unit weight of the various species of wood for any moisture content. The source of the information in Table 15-15 and the equations used to calculate $G_m$ and $\gamma$ is Reference 10. This Reference can be viewed at the following website: www.fpl.fs.fed.us.

The maximum decay rate of wood occurs with alternate wetting and drying, or consistently damp conditions, which is the environment most large woody debris will be subjected to. If, however, the wood is kept continuously submerged, it does not decay significantly by the common decay fungi regardless of the wood species. Therefore, for maximum longevity, particularly at sites subject to wetting and drying, it is best to select a species with a decay resistance of "resistant or very resistant" or "moderately resistant". At sites where the wood will be continuously submerged, any of the species listed in Table 15-15 can be used.

When incorporating large woody debris in bank protection designs, it is recommended that the woody debris be placed normal to the bank or angled slightly upstream (15 - 20 degrees) in order to prevent overtopping flows from being deflected towards and possibly eroding the channel bank.

The following soil parameters are suggested for the analysis:

- Unit weight of soil, $\gamma_{soil} = 120$ pounds per cubic foot
- Submerged unit weight of soil, $\gamma'_{soil} = \gamma_{soil} - \gamma_{water} = 120 - 62.4 = 57.6$ pounds per cubic foot
- Unit weight of riprap, $\gamma_{riprap} = 165(0.7) = 115.5$ pounds per cubic foot (assumes riprap weighing 165 pounds per cubic foot has 30 percent voids)
- Submerged unit weight of riprap, $\gamma'_{riprap} = (165 - 62.4)\ 0.7 = 71.8$ pounds per cubic foot (assumes riprap weighing 165 pounds per cubic foot has 30 percent voids) For site-specific soil parameters, contact either the project geologist or the project geotechnical engineer.

## Table 15-15 Wood Properties

| Species | Decay Resistance | $G_b$ | $G_{15}$ | Unit Weight, γ (pounds per cubic foot) |
|---|---|---|---|---|
| Douglas Fir | Moderately Resistant | 0.45 | 0.48 | 34.3 |
| Western Red Cedar | Resistant or Very Resistant | 0.31 | 0.32 | 23.2 |
| Incense Cedar | Resistant or Very Resistant | 0.35 | 0.37 | 26.3 |
| Port Orford Cedar | Resistant or Very Resistant | 0.39 | 0.41 | 29.5 |
| Western Hemlock | Slightly or Nonresistant | 0.42 | 0.44 | 31.9 |
| Western Larch | Moderately Resistant | 0.48 | 0.51 | 36.8 |
| Ponderosa Pine Lodgepole Pine | Slightly or Nonresistant | 0.38 | 0.4 | 28.7 |
| Western White Pine | Slightly or Nonresistant | 0.35 | 0.37 | 26.3 |
| Sugar Pine | Slightly or Nonresistant | 0.34 | 0.36 | 25.6 |
| Engelmann Spruce | Slightly or Nonresistant | 0.33 | 0.35 | 24.8 |
| Sitka Spruce | Slightly or Nonresistant | 0.37 | 0.39 | 27.9 |
| Grand Fir | Slightly or Nonresistant | 0.35 | 0.37 | 26.3 |
| Pacific Silver Fir | Slightly or Nonresistant | 0.40 | 0.42 | 30.3 |
| Noble Fir White Fir | Slightly or Nonresistant | 0.37 | 0.37 | 27.9 |
| California Red Fir | Slightly or Nonresistant | 0.36 | 0.38 | 27.1 |
| Black Cottonwood | Slightly or Nonresistant | 0.31 | 0.32 | 23.2 |
| White Oak | Resistant or Very Resistant | 0.60 | 0.65 | 46.8 |

### 15.5.7.5.1 Example 1

Given:

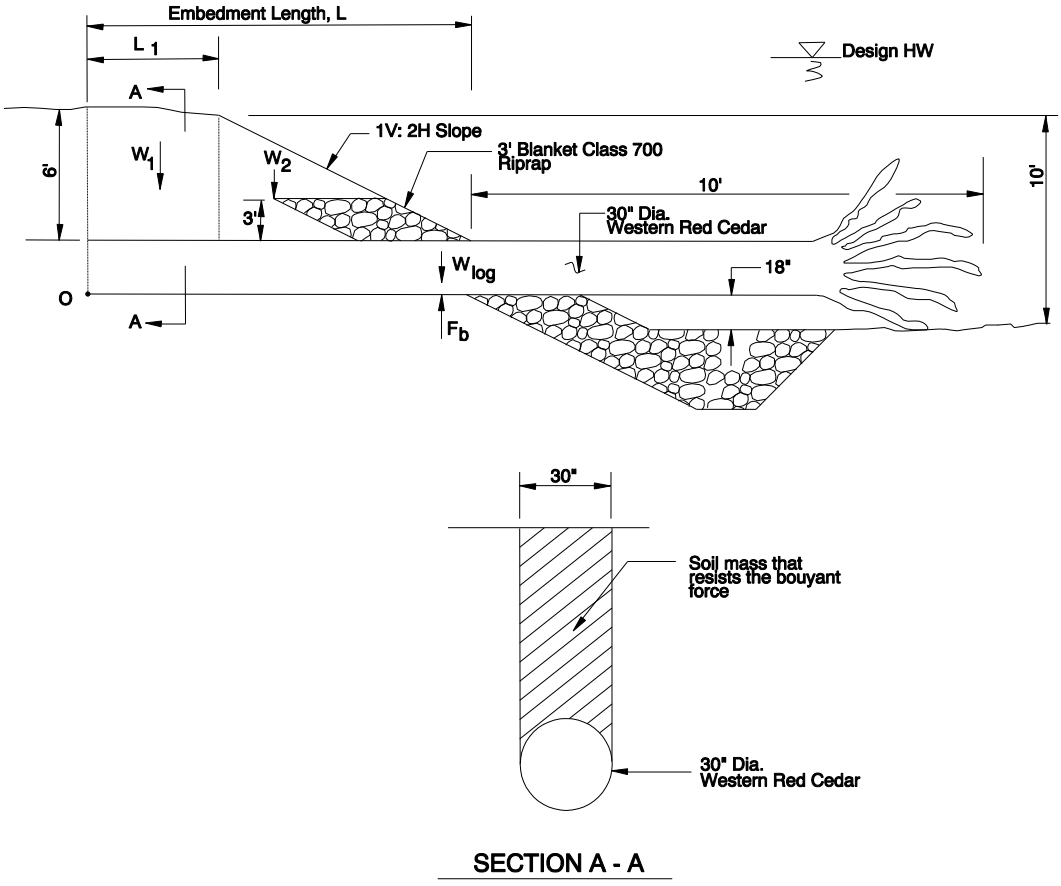



**Figure 15-30 Definition Sketch for Example 1**

Find: Embedment length for log to be stable

$\gamma_{soil}$ = 120 pounds per cubic foot

Solution:

To simplify the calculations assume the riprap above the log is soil. The analysis also conservatively assumes that the rootwad is solid wood with the same diameter as the log.

$\gamma_{log}$ = 23.2 pounds per cubic foot (from Table 15-15)

Submerged unit weight of soil = 120 pounds per cubic foot – 62.4 pounds per cubic foot = 57.6 pounds per cubic foot

$W_1 = 2.5(6)(L_1)(57.6) = 864L_1$

$W_2 = ½\ (12)(6)(2.5)(57.6) = 5{,}184$

$W_{log} = \pi(1.25)^2(10 + 12 + L_1)(23.2) = 113.9L_1 + 2{,}505.4$

Buoyant force, $F_b = \pi(1.25)^2(10 + 12 + L_1)(62.4) = 306.3L_1 + 6{,}738.7$

For the log to be stable, $\sum M_o = 0$

$$0 = W_1\left(\frac{L_1}{2}\right) + W_2\left(L_1 + \frac{12}{3}\right) + W_{log}\left(\frac{10+12+L_1}{2}\right) - F_b\left(\frac{10+12+L_1}{2}\right)$$

$$0 = 864L_1\left(\frac{L_1}{2}\right) + 5{,}184\left(L_1 + 4\right) + \left(113.9L_1 + 2{,}505.4\right)\left(\frac{22+L_1}{2}\right) - \left(306.3L_1 + 6{,}738.7\right)\left(\frac{22+L_1}{2}\right)$$

$$0 = 335.8L_1^2 + 950.95L_1 - 25{,}830.3$$

Solving for $L_1$ with the quadratic equation, $x = \dfrac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

$$L_1 = \frac{-950.95 \pm \sqrt{950.95^2 - 4(335.8)(-25{,}830.3)}}{2(335.8)}$$

$L_1 = 7.47$ feet

Minimum embedment length, $L = L_1 + 12 = 7.47 + 12 = 19.47 \approx 20$ feet

Use embedment length, L = 24 feet to provide factor of safety against overturning; recheck stability with L = 24 feet.

$$\sum M_o = 864(12)\left(\frac{12}{2}\right) + 5{,}184\left(12 + \frac{12}{3}\right) + 3{,}872\left(\frac{34}{2}\right) - 10{,}414.4\left(\frac{34}{2}\right)$$

$$\sum M_0 = 62{,}208 + 82{,}944 + 65{,}824 - 177{,}045 = 33{,}931 \text{ ft} - \text{lbs} > 0 \ldots \text{ log will not overturn}$$

$$\text{Safety Factor} = \frac{62{,}208 + 82{,}944 + 65{,}824}{177{,}045} = \frac{210{,}976}{177{,}045} = 1.19$$

Example 1 Summary:

Example 1 demonstrates cumbersome design process to determine embedment length as shown in Figure 15-30. It results in a 2:1 ratio of embedment to exposure. This should be a rule of thumb with 6-feet of cover on a 30 inch diameter log with rootwad.

### 15.5.7.5.2 Example 2

At some sites it may not practical or environmentally acceptable to disturb the stream bank the amount necessary to provide the embedment length as calculated in Example 1. Cabling concrete blocks or large rocks to the log can reduce the embedment length as shown in this example. This example is the same as Example 1 except that two concrete blocks have been added to provide additional ballast.

Given:

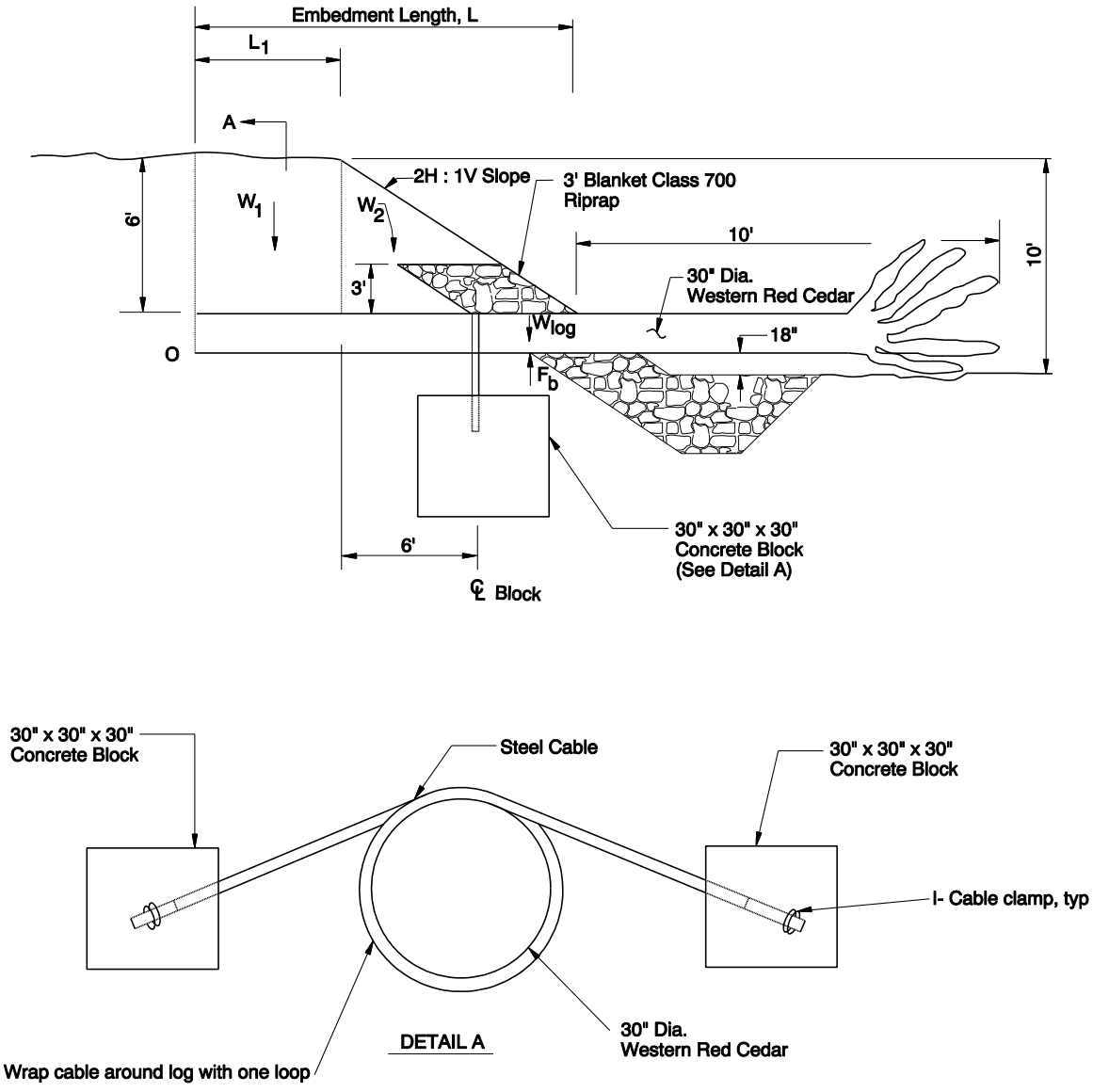



**Figure 15-31 Definition Sketch for Example 2**

Find: Embedment length for log to be stable.

Solution:

To simplify the calculations assume the riprap above the log is soil. The analysis also assumes that the rootwad is solid wood with the same diameter as the log.

$\gamma_{soil}$ = 120 pounds per cubic foot

$\gamma_{log}$ = 23.2 pounds per cubic foot (from Table 15-15)

$\gamma_{concrete}$ = 145 pounds per cubic foot

Submerged unit weight of soil = 120 pounds per cubic foot – 62.4 pounds per cubic foot = 57.6 pounds per cubic foot

Submerged unit weight of concrete = 145 – 62.4 = 82.6 pounds per cubic foot

$W_1 = 2.5(6)(L_1)(57.6) = 864L_1$

$W_2 = ½ (12)(6)(2.5)(57.6) = 5{,}184$

$W_b = 2(2.5)^3(82.6) = 2{,}581$

$W_{log} = \pi(1.25)^2(10 + 12 + L_1)(23.2) = 113.9L_1 + 2{,}505.4$

Buoyant force, $F_b = \pi(1.25)^2(10 + 12 + L_1)(62.4) = 306.3L_1 + 6{,}738.7$

For the log to be stable, $\sum M_o = 0$

$$0 = W_1\left(\frac{L_1}{2}\right) + W_2\left(L_1 + \frac{12}{3}\right) + W_b\left(L_1 + 6\right) + W_{log}\left(\frac{10+12+L_1}{2}\right) - F_b\left(\frac{10+12+L_1}{2}\right)$$

$$0 = 864L_1\left(\frac{L_1}{2}\right) + 5{,}184(L_1 + 4) + 2{,}581(L_1 + 6) + (113.9L_1 + 2{,}505.4)\left(\frac{22 + L_1}{2}\right)$$

$$- (306.3L_1 + 6{,}738.7)\left(\frac{22 + L_1}{2}\right)$$

$$0 = 335.8L_1^2 + 3{,}531.95L_1 - 10{,}344.3$$

Solving for $L_1$ with the quadratic equation

$$L_1 = \frac{-3{,}531.95 \pm \sqrt{3{,}531.95^2 - 4(335.8)(-10{,}344.3)}}{2(335.8)}$$

$$L_1 = 2.39 \text{ feet}$$

Minimum embedment length, $L = L_1 + \quad 12 = 2.39 + 12 = 14.39 \approx 15$ feet

Adding the concrete blocks reduced the embedment length calculated in Example 1 by 5 feet. In general cabling concrete blocks or large rocks to woody debris is most effective and often necessary at sites with a shallow layer of soil over the woody debris.

Example 2 summary:

Example 2 demonstrates a cumbersome design process to determine a reduced embedment length as shown in Figure 15-31 where two concrete blocks provide ballast to offset buoyant forces on the log. It results in a 1.5:1 ratio of embedment to exposure. This should be a rule of thumb with 2 concrete blocks and 6 feet of cover on a 30 inch diameter log with rootwad.

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# CHAPTER 16

# TRENCHLESS TECHNOLOGY

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## 16.1 Introduction

Trenchless technology consists of a variety of methods, materials, and equipment for inspection, stabilization, rehabilitation, and replacement of existing culverts and installation of new culverts with a minimum of excavation from the ground surface. Some of the methods were available in the early 1900's and with the addition of new techniques are coming into much more widespread use because of their inherent advantages. Trenchless methods minimize damage to the highway, cause little or no disruption to traffic, have less impact on the environment, and occasionally avoid or minimize the handling and disposal of contaminated soils.

Depending on the specific situation trenchless methods can be cost-effective alternatives to the more conventional open excavation. The cost is insensitive to depth of cover meaning that work under high fills will be more economical using trenchless technology. Many times the cost is not the main concern; factors such as safety, inconvenience of the motoring public, and environmental impacts outweigh the initial costs.

ODOT'S main application of this technology is in the rehabilitation, replacement, and installation of culverts and storm sewer lines in areas where open excavation is either more costly, not as environmentally friendly, or creates inconvenience for the motoring public.

## 16.2 Getting Started

The trenchless technology process starts with the identification of a problem. There is a drainage structure that requires some sort of fix. It may need stabilization, rehabilitation, or replacement. The first question to ask is what has caused this problem. Is the structure at the end of its design life, has it been damaged by mechanical abrasion or chemical erosion, have conditions changed and is it now under capacity, and are undesirable soil conditions causing problems?

It is always necessary to make a site investigation to determine what caused the problems identified. If you are not experienced in dealing with culverts ask for help. If the structure is less than 6.0 feet in diameter call the highway maintenance staff for that area. They are the local experts on what is going on with that structure. Contact the Region Bridge Inspector if the culvert is 6.0 feet in diameter or larger.

## 16.3 Site Investigation

Before trenchless methods can be considered for installation, replacement, or rehabilitation of culverts or storm sewer lines a thorough site investigation must be made. The existing culvert's condition must be determined. A visual inspection of the culvert's interior can be done if the culvert is at least 4 feet in diameter. If the culvert is less the 4 feet in diameter or other

circumstances prevent human entry, then a CCTV inspection, done by a trained technician must be performed. The resulting video inspection of the culvert can then be viewed by knowledgeable personnel for review and comment. In either case the embankment and surrounding area must also be examined thoroughly for subsidence and/or other anomalies at the time of inspection.

The underlying cause of the culvert's problems must be discovered. The following will provide guidelines on how to answer this most important question. The goal is to determine the underlying problems before a course of action is chosen.

Some of the more common problems found with culverts are:

- Loss of invert, walls, or ceiling due to corrosion and/or abrasion
- Leaking joints
- High debris load causing sedimentation and blockage.
- Settlement.
- Shape deformation.
- Scour and erosion of the streambed and embankments.
- Inadequate flow capacity.
- Joint separation
- Voids in the embankment around and above the culvert.

### 16.3.1 Roadway

- Look for defects in the roadway and embankment in the vicinity of the culvert. Check for sags, cracks in the pavement, pavement patches, dips in the guardrail, and erosion of the sideslopes. Probe the embankment looking for voids.

- Look for signs of recent high water; roadway overtopping, debris piled above the crown of the culvert, recent riprap placement around the inlet or outlet of culvert, and any other evidence that the culvert may be undersized or partially obstructed.

- Search through maintenance records. Region Bridge Inspectors inspect all culverts 6 feet & larger. District maintenance personnel inspect all of the culverts 6 feet and smaller.

### 16.3.2 Stream Channel

- In the stream channel look for scour holes at the inlet and outlet of the culvert.

- Examine the stream to determine size of the bed load passing through the culvert.

- Look at horizontal and vertical alignment of the channel for misalignment with the culvert inlet and outlet.

- Look for channel obstructions, such as, debris, slides, and fences. etc.
- Check for excessive embankment erosion, channel aggradation / degradation, and headcutting.
- Look for high water marks and if found above the culvert crown, look for changes in land use or drainage area.
- Examine headwalls, wingwalls, slope protection, and energy dissipators for misalignment and probe for possible voids. Check for scour, undermining of footings, and piping through the embankment.

### 16.3.3 Within Culvert

**Man Entry Possible**

- Look at the culvert invert for signs of abrasion and/or corrosion.
- Look for open or misaligned joints. Probe for voids behind open joints *
- Look for settlement
- Look for infiltration
- Look for sedimentation
- Look for blockage by debris.
- Look for cracking and spalling.
- Sound the culvert with a hammer to detect the formation of voids in the embankment surrounding the pipe.
- Look for evidence of fill material inside the culvert that may have migrated through an open joint.
- Look for high water marks within culvert.

**Note: If probing, boring, or any other invasive technique is needed to determine the culvert condition, the Region Bridge Inspector must be contacted first. He will initiate the procedure to obtain the proper permit of entry. Under no circumstances can these procedures be used without obtaining a permit first.*

**Man Entry Not Possible**

A closed circuit video (CCTV) inspection will need to be made by a qualified video inspection contractor. A video inspection is made with an audio commentary. The complete inspection should be viewed by knowledgeable personnel for comments and recommendations. The culvert inspection form (see Chapter 6, Appendix D) summarizes the above discussion and is recommended for use during site investigations.

## 16.4 Design

The critical component in determining what trenchless methods can be used is a thorough geotechnical investigation. The types of soil involved, the groundwater conditions, and the presence of large boulders or debris within the fill are all-important factors in determining method. Success with trenchless methods is very dependent on a more intensive site investigation with appropriate planning, design, and installation methods then would be required with open excavation methods. Field problems occurring during installation often result in much larger impacts to society, cost, and the environment than with open excavation methods. The results of all of the above observations will be factors in determining the best method of replacement or rehabilitation of the culvert and if trenchless technology can be used.

The next question is what other concerns exist? The answer may well change the entire design concept. Contact the Environmental Unit as early on in the project as possible so that environmental concerns can be incorporated into the design. The final design options will have to wait for the geotechnical data as well as the decisions of the environmental agencies before the plans can be approved.

Many ODOT projects that involve trenchless methods are dealing with existing drainage culverts that are nearing the end of their design life and are badly deteriorated. The first question is can the existing culvert be rehabilitated, or must it be replaced? Another alternative to consider is the rehabilitation of the existing culvert and the installation of additional culverts to increase flow capacity or enhance the passage of fish or wildlife.

### 16.4.1 Culvert Rehabilitation

The following list of conventional maintenance and repair methods is offered to remind that not every project will need trenchless methods to solve the problem:

#### 16.4.1.1 Conventional

- Cleaning
- Root removal
- Internal grouting
- External grouting
- Mechanical seals
- Reinforced shotcrete or concrete placement

### 16.4.2 Trenchless techniques (pipe not replaced)

- Sliplining with fusion welded continuous pipe.
- Short pipe sections.
- Cured-in-place-pipe.
- Deformed pipe.
- Spiral-wound pipe.
- Coatings.
- Spot repairs.

### 16.4.2.1 Sliplining with Fusion-Welded Continuous Pipe

**Figure 16-1 Sliplining with Fusion-Welded Continuous Pipe**

This process refers to a pipe sliplining technique where plastic pipe (HDPE) or PVC sections are joined by fusion welding outside of the pipe to be rehabilitated, and then pulled into place as one solid liner. The installation is quick, but this technique is limited to fusion of circular cross sections only. Any shape of host pipe can be sliplined with fused circular pipe, but since there is a high loss of cross-section area, particularly in smaller sizes, the design must be checked for flow capacity. The liner is generally smoother than the host pipe causing higher outlet velocities, which may require additional design to deal with the increased scour potential.

The outside diameter of the pipe used for sliplining is generally specified to be at least 4 inches smaller than the inside diameter of the host pipe so that a low-density cementatious grout can be injected into the annular space between the two pipes.

The entire annular space is grouted to prevent voids from forming in the pipe bed and embankment. The low-density nature of the grout and the application technique used to apply it

allows it to migrate out into the fill surrounding the host pipe and seal voids that exist. The grout also provides for structural rehabilitation, prevents liner collapse, prevents fatigue failure, stabilizes the pipe, extends the design life from uncertainty to at least 50 years, and resists temperature changes.

#### 16.4.2.2 Sliplining with Short Pipe Sections

Short pipe sections can be made of any material and are primarily used where access is restricted, such as, manhole to manhole installations. Installation is quick with a variety of cross sections to choose from. Some materials are easily damaged and may require special care at joints or when grouting in place.

#### 16.4.2.3 Cured-in-Place Pipe

**Figure 16-2 Cured-in-Place Pipe**

A felt tube saturated with a thermosetting resin is either pulled into the existing pipe or inverted through as water pressure pushes the tube tightly against the pipe wall. The water in the tube is then heated to the curing temperature of from 160-180 degrees Fahrenheit. The plastic resin on the tube cures to solid pipe inside the existing pipe creating a new lining. Installation goes quickly leaving no annular space to be sealed. Odd cross sections, bends, and minor deformations can be accommodated. This method is particularly useful when flow capacity must be maintained or slightly increased.

#### 16.4.2.4 Deformed pipe

**Figure 16-3 Deformed pipe**

The fold and form method utilizes PE or PVC pipe whose properties allow for the deformation of the pipe into a "U" shape prior to insertion into the host pipe. Once in place the pipe is heated with steam to expand it back to its original shape. A tight fit is formed against the wall of the original pipe. This process will line pipe up to a 24-inch diameter pipe.

Swagelining also takes advantage of the properties of PE pipe. A liner pipe with a slightly larger diameter than the existing pipe is pulled through a reduction die to temporarily reduce its diameter as it is inserted into the host pipe. After the liner has been pulled through the pipe the pulling force is removed allowing the PE pipe to expand back toward its original diameter. The expansion is stopped by the host pipe creating a tight fitting liner. This process will line pipe up to 48" in diameter.

Rolldown is a process like swagelining except that the pipe diameter is reduced by mechanical rolling instead of use of a reduction die. The liner is then pressurized with water at ambient temperature to revert it to its original size after it has been pulled through the existing pipe. This process will line pipe up to 20-inches in diameter.

#### 16.4.2.5 Spiral-wound pipe

**Figure 16-4 Spiral-wound pipe**

This method uses a continuous PVC strip, which can be introduced into the existing pipe by hand or by the use of a helical winding machine. It can accommodate variations in cross section and large radius bends. It is mandatory to grout the annular space between the liner and existing pipe in small lifts using structural strength low-density grout. Depending on what must be done structurally to stabilize the pipe, significant loss of capacity can occur if the cross-sectional area is significantly reduced.

**Coatings**

A wide variety of materials including cement mortar, epoxy resins, and polyurethane, are available to coat the interiors of pipes. These materials are usually sprayed onto the pipe for corrosion protection. Structural spray-on lining is also available made from quick setting epoxy resin or polyurethane material.

Pipe lines may be restored by treating manhole lengths with a chemical solution. The length is first sealed and filled with solution "A" which is then pumped out. The length is refilled with solution "B" which is subsequently pumped out. The chemical reaction between the two components seals joints and cracks in the pipe and stabilizes the surrounding soil.

**Spot Repairs**

Internal grouting of pipe joints and radial cracks can be accomplished with a packer. The packer is moved into place over the pipe joint or radial crack, then bladders are inflated at each end of the packer and grout is then injected into and around the damaged area. When the repair is completed the bladders are deflated and the packer removed from the system. Cementatious grout, resins, and urethane, are the common grout materials used.

Internal mechanical seals are also available. They are generally made from a special EPDM rubber gasket reinforced with internal stainless steel compression hoops. They form a tight yet flexible pressure seal over the damaged area.

A section of felt tube saturated with a thermosetting resin up to approximately 20 feet can be pulled into the existing pipe at the point of needed repair. This short tube section differs from a conventional cured-in-place liner by the curing process. No heat is required to cure the resin. Ambient temperature cures the pipe is just a few hours.

.

### 16.4.3 Trenchless Techniques (Culvert Replacement)

This section will discuss trenchless techniques used in the installation of new culverts. The following list is of popular techniques being used to install new pipe.

- Pipe Jacking
- Auger Boring

- Pipe Ramming
- Microtunneling
- Horizontal Directional Drilling
- Pipe Bursting
- Utility Tunneling

#### 16.4.3.1 Pipe Jacking

**Figure 16-5 Pipe Jacking**

Pipe Jacking is a method for installing a prefabricated pipe through the ground from a drive shaft to a reception shaft. The pipe is moved by jacks located in the drive shaft. The thrust power of the hydraulic jacks forces the pipe forward through the ground as the face is being excavated. After each pipe segment has been installed, the rams of the jacks are retracted so that another pipe segment can be placed in position for the jacking cycle to begin again.

As the excavation proceeds soil is transported out of the jacked pipe and drive shaft either manually or mechanically. The soil conveyance systems include wheeled carts or skips, belt or chain conveyors, slurry systems, auger systems, and vacuum extraction systems. Both the

excavation and spoil removal processes require workers to be inside the pipe during the jacking operation. For personnel health and safety, a minimum pipe diameter of 42-inches is recommended.

Excavation can be accomplished by hand mining or mechanical excavation within a shield or by a tunnel boring machine (TBM). The excavation method selection is based on soil conditions. If there is any possibility of the excavation face collapsing, soil stabilization techniques must be considered. Dewatering or grouting are both common methods of soil stabilization.

The design of the drive shaft is critical to the success of the project. The shaft floor and thrust reaction structure must be designed to withstand the large jacking forces required to push the pipe through the ground and withstand the weight of heavy pipe segments being placed on them repeatedly. Pipe jacking equipment that has a pipe lubrication system can decrease the jacking forces necessary by 20-30 percent.

The required working space must provide adequate space for storage and handling of the pipe and spoil and space for the shaft. Typically, the working space should be from 4 feet to 10 feet wider than the diameter of the pipe and from 10 feet to 25 feet longer than the length of pipe sections being installed. Shaft size will vary depending on the type of jacking and excavation equipment used.

The primary concern is the prediction of subsurface soil behavior. Unanticipated ground conditions require corrective measures that cause cost overruns and delays. Sandy clay is the most favorable soil condition if the water table is not above the pipe invert. With the use of the proper excavation methods many types of ground conditions can be overcome. Major factors to be considered are the presence of groundwater, unanticipated obstructions such as boulders, and changed soil conditions that would require different equipment to excavate.

Other concerns include proper design of the shaft to withstand the large jacking thrust and that jacking thrust is uniformly transferred through a properly designed joint material. The over-excavation above or ahead of the pipe is to be avoided, if over-excavation occurs or voids develop, external grouting is usually required.

### 16.4.3.2 Auger Boring

**Figure 16-6 Auger Boring**

Auger Boring is a method for driving a horizontal bore hole through an embankment from a drive shaft to a reception shaft. A rotating cutting head is attached to the leading end of an auger string. The spoil is transported back to the drive shaft area by the rotation of the helical-wound auger flights within the steel pipe casing being placed. Common practice is to jack the steel casing through the hole as it is being bored. Because of the possible hazards of the unsupported hole collapsing or personnel being hurt from unshielded auger flights, uncased auger boring is discouraged. The boring machine turns the auger string, which rotates the cutter head. A casing is advanced through the bore hole by hydraulic jacks located at the rear of the machine. The jacks push the whole assembly forward as the boring progresses. The soil is excavated, removed, and the casing installed in one continuous operation.

The method is unguided as to line and grade unless a steering head is used. With the steering head and a water-level grade monitoring system an accuracy of 1 percent of the length can be maintained in vertical grade. Horizontal grade is generally not controlled and obstructions such as boulders can cause large deflections. Technology is providing more accurately controlled articulating cutter heads and laser sensing systems which are coming into common use. With the new improvements, auger boring can rival microtunneling accuracy for relatively short drives in stable ground conditions.

A properly designed and constructed drive shaft is critical to the success of the project. The drive shaft provides a stable foundation to maintain the preset grade and to support the jacking process with a properly designed trust block. The boring machine sets on tracks placed on the same line and grade as the desired bore hole. If the foundation settles, the track will also deflect causing inaccuracy in borehole alignment.

The thrust block transmits the horizontal jacking forces from the tracks to the ground at the rear of the drive shaft. The trust block must be designed to distribute the force over a large enough area so that the allowable compressive strength of the soil is not exceeded. If the trust block moves, bore hole accuracy will be lost and binding forces could result.

The casing pipe is typically made of steel to resist damage that can be caused by rotation of the augers. The casing can be lined with any suitable material once it has been installed. Installed sizes typically range from 1 foot to 3 feet with typical bore lengths of 100 feet to 300 feet. Diameters of up to 5 feet and lengths of drive over 800 feet have been made.

Working space is required at both ends of the casing. The drive shaft is the working shaft with its size determined by bore hole diameter and length of casing segments used. Typical casing lengths range from 10 feet to 20 feet. For a casing that is 20 feet in length the shaft size should be from 30 feet to 35 feet in length and from 10 feet to 14 feet wide.

Operator skill is critical as the excavation and spoil removal process can't be seen. Much of the work is done by feel. Changing conditions that are not detected can cause subsidence of the ground by overexcavation. If soil conditions change from cohesive to noncohesive an experienced operator will adjust the location of the cutting head relative to the end of the casing to avoid removing too much soil. Excessive force applied to the excavation face can cause heaving of the ground above the excavation. The operator's skill and experience are the key element of success with this method.

A wide variety of soil conditions can be accommodated from wet sand to firm dry clay. Boulders as large as 1/3 of the casing diameter can be excavated, but deflection from line and grade must be carefully watched. Depending on soil conditions, casing diameter and length a typical advance rate would be from 3 feet to 40 feet per hour. The minimum depth of cover over the excavation should not be less than 2 feet.

### 16.4.3.3 Pipe Ramming

**Figure 16-7 Pipe Ramming**

With pipe ramming, the dynamic force and energy transmitted by a percussion hammer attached to the end of a thick-walled steel casing drives it from the drive shaft on one side of the embankment to the reception shaft on the other side of the embankment. The casing can be used without a pipe installed through it or a project pipe can be installed through the casing and grouted in place with low density grout. The casing can also be sprayed with a coating to retard corrosion, or it can be used untreated depending on the local conditions.

The process is simple and similar to pile driving operations. Continuous support is provided during the drive with no over-excavation needed. The spoils stay in the pipe and are mechanically removed when the drive is completed.

The initial setup is the critical factor in the success of any pipe ramming project. The drive shaft must be located on very stable ground or a concrete slab must be placed under the casing. The pipe is unguided so the floor of the drive shaft must be engineered to be on the same line and grade as the pipe to provide the accuracy needed.

Ground conditions encountered also effect the line and grade of the casing making it very important to know what the fill is composed of. There is a possibility of ground heaving and subsidence. Heaving usually is not a problem because there is minimum disturbance outside the pipe. Subsidence can occur in some soils due to the consolidation resulting from the vibratory action of the hammer.

The casing will have the tendency to drift downward slightly as the ramming proceeds due to gravitational forces. When possible it is advantageous to start the drive from the upstream side of the crossing.

Pipe ramming is particularly effective when the fill contains obstructions. Where a boulder or tree stump partially obstructs the pipe, the casing will usually break it up, swallow it into the casing, or push it aside as it continues through the embankment. The line and grade of the pipe can be affected by these obstructions if encountered before enough casing has been driven to give the pipe support. Larger pipe diameters are less likely to be deflected by obstructions. Pipe ramming can be used for a wide range of soil conditions including high groundwater table.

One percent of the length of the drive is the accuracy both vertically and horizontally that can be expected from an unguided ram. The lack of good line and grade control limits the typical length from 100 feet to 200 feet not to exceed 300 feet when pipe ramming is used. Oversize casings are sometimes installed so that a carrier pipe can be adjusted within the casing to correct for line and grade problems.

Recently a steering mechanism has been developed that is attached to the front of the driven pipe that is capable of maintaining accuracies to 0.1 percent of the grade. The largest ramming to date is a 12 feet steel casing, 225 feet long. Larger more powerful machines are now available that have the capability of ramming a 16 feet steel casing over 500 feet.

### 16.4.3.4 Microtunneling

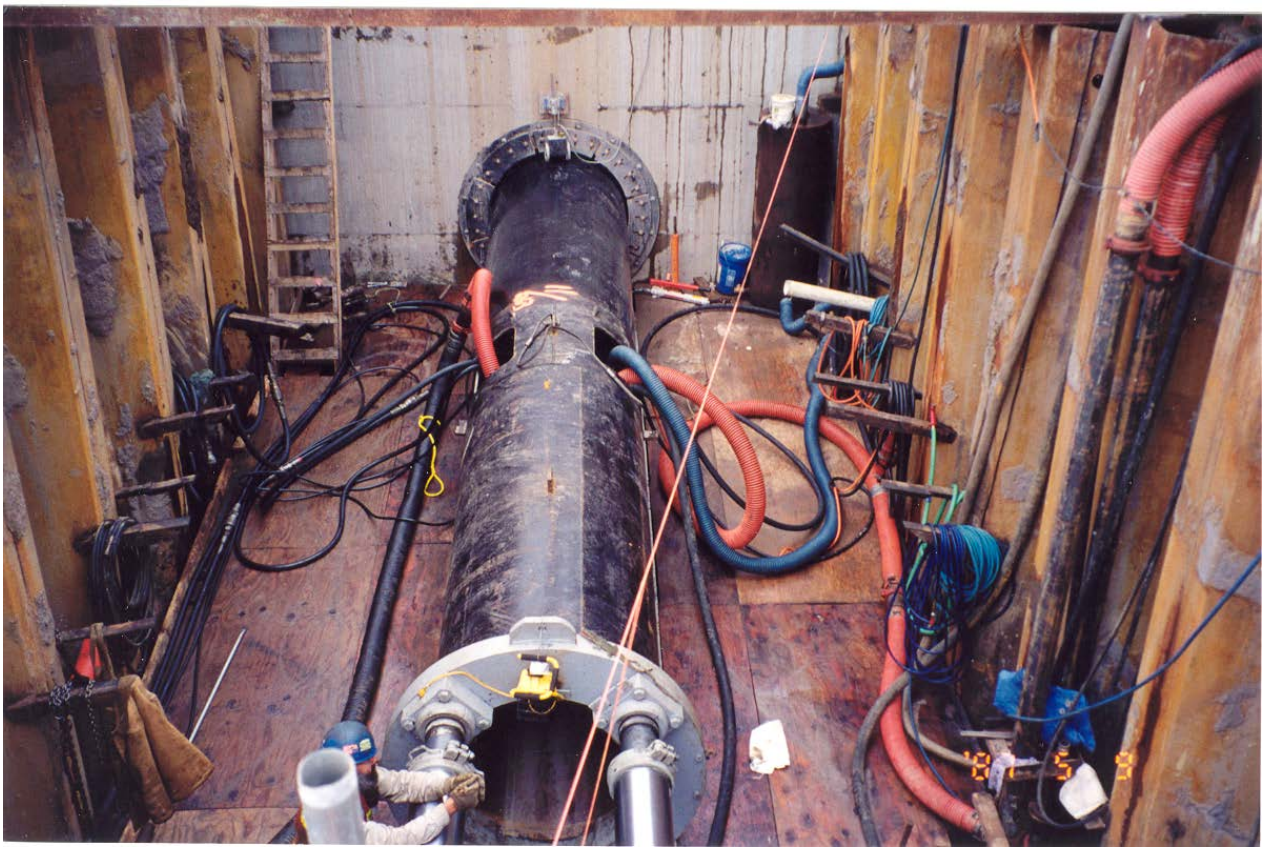


**Figure 16-8 Microtunneling**

Microtunneling (MT) is a method using a remotely controlled, guided pipe jacking process that provides continuous support to the excavation face. It can be used in a wide range of soil conditions while keeping close tolerances to line and grade. Spoils are removed by either a pumped slurry or by mechanical auger. Five independent systems are incorporated into Microtunneling systems. They are as follows:

- Microtunnel boring machine (MTBM).
- Jacking or propulsion system.
- Spoil removal system.
- Laser guidance and remote control system.
- Pipe lubrication system.

The MT procedure on a typical project is much the same as for Pipe Jacking.

The required working space must provided adequate space for storage and handling of the pipe and spoil and space for the shaft. Typically, the drive shaft would range from 16 feet to 50 feet wide and from 35 feet to 100 feet long depending on pipe diameter, length and equipment dimensions. Working space typically would range from 20 feet to 40 feet wide and 75 feet to 150 feet long.

The primary concern is the prediction of subsurface soil behavior. Unanticipated ground conditions require corrective measures that cause cost overruns and delays. Wet sand for slurry MT and stable sandy clay for auger MT are the most favorable soil conditions for each. A wide variety of MTBM cutter heads are available that provide the capability to handle a range of soil conditions, including boulders and solid rock. Major factors to be considered are the presence of groundwater, unanticipated obstructions such as boulders, and changed soil conditions that would require different equipment to excavate.

Microtunneling is very accurate. Line and grade can be maintained to 0.01 percent of the drive length depending on many factors, the most important being the skill of the machine operator.

#### 16.4.3.5 Horizontal Directional Drilling

**Figure 16-9 Horizontal Directional Drilling**

Horizontal Directional Drilling is a method that uses steerable soil drilling in a two-stage process. Stage 1 involves drilling a pilot hole approximately 1-inch to 5-inches in diameter along the proposed pipe alignment centerline. Concurrent to drilling the pilot hole, the contractor may elect to run a larger diameter "wash pipe" that will encase the pilot drill string. The wash pipe acts as a conductor casing providing rigidity to the smaller diameter pilot drill string and will also save the drilled hole should it be necessary to retract the drill string for bit changes.

In Stage 2 a backreamer is attached to the end of the drill string and pulled back through the pipe to enlarge the hole to the size necessary to pull back the new pipeline. The pipe can be pulled in behind the backreamer or the hole can be created first then the pipe pulled through. It may require several passes on different diameter backreamers to enlarge the bore to the diameter desired. The bore is drilled oversize approximately 50 percent larger than the diameter of the new pipe so that the pipe can be pulled into place without causing large frictional forces which can lead to binding of the pipe and/or collapse of the bored hole.

The drill rig itself sets on the surface and has an inclined carriage typically adjusted to an angle between 5 to 20 degrees with the ground. The drill string enters the ground at the preset angle and is then guided through the fill in a shallow arc. The drill path is monitored by an electronic package housed in the pilot drill string near the cutting head. Data is transmitted back to the surface where calculations are made as to the location of the cutting head. Installation accuracy depends on which electronic monitoring package is used, but an accuracy of within 1 percent of the bore length is considered acceptable. In actual field operations, such accuracy is not exceeded because of the high drilling speeds preferred by contractors and limitations due to operators' skill levels and the steering system. Greater accuracy is possible using more powerful sondes in combination with electrically charged surface grids with computer interface.

Large volumes of drilling slurry are used to provide the following:

- Transport of the drilling cuttings to the surface.
- Stabilize the hole against collapse
- Reduction of friction.
- Lubrication of the cutting head.
- Cooling and cleaning of the cutters.
- Providing power to the drill motor.

The slurries most commonly used are bentonite based. Bentonite is naturally occurring clay known for its hydrophilic characteristics. Often polymers and surfactants are also added to enhance certain slurry characteristics. The slurry is pumped downhole and circulates back to the surface and is may be collected in return pits. The slurry is pumped from the return pits to a settling and containment pit. The slurry is then passed through machinery that separates the cuttings from the slurry. This process involves a series of shaking sieves and various size hydroclones and centrifuges.

At the end of the project the slurry can be disposed of by:

- Use at another location.
- Spread onto the surrounding land for water retention improvement.
- Evacuate to a dump site.

A work space of approximately 120 feet x 175 feet should be provided at both the rig side and the outlet side of the project for storage, and required auxiliary equipment.

Directional drilling is best used in clays. Cohesionless fine sand and silt generally stay suspended in the drill fluid for a sufficient amount of time to reach the surface and are also suitable. It is more difficult to use in granular materials such as gravel and rock because the drill fluid may not carry the spoils to the surface causing the cutting head and "wash pipe" to plug. Higher fluid pressure, more powerful jets, and special drill heads can be used to offset the clogging problems, but other problems may occur in spoil removal, hole stabilization, and backreaming operations. Caution must also be used to prevent ground movement and loss of slurry in applications with shallow ground cover.

Directional drilling has the fastest boring rate among all the trenchless methods. In suitable conditions, 500 feet of pipeline can be installed in one day. It can be used under water and a number of different pipe materials can be pulled into place.

### 16.4.3.6 Pipe Bursting

**Figure 16-10 Pipe Bursting**

Pipe Bursting technology uses a rod, or cable connected to either a static bullet-shaped bursting head or a dynamic head made from a converted pipe ramming machine. The bursting head is pulled through the existing pipe by a rod or cable, simultaneously shattering the old pipe and pulling a replacement pipe behind it. It is a method developed for use where underground space is at a premium. The subsurface can be so congested with existing services and chambers that the existing hole becomes a valuable route. The existing defective pipe is split and the fragments are displaced into the embankment. A new pipeline of the same diameter, usually of HDPE, is drawn in behind the splitter. If the soil conditions are favorable a larger diameter pipe may be installed. Open trench methods will have to be used to expose any lateral lines coming into the existing pipe before the pipe bursting process begins. The lateral lines are reinstated into the new pipeline after it is installed.

Advantages include rapid installation, replacement with a variety of pipe materials, and the method is not dependent on conditions of existing soil for same size replacement. To reduce the possibility of soil heave above the pipe line, a minimum of 4 feet of cover is needed in most cases.

The host pipes must be brittle to use pipe bursting. Most ductile iron, concrete, reinforced concrete, clay, PVC, Asbestos Cement, and liners can be burst. Corrugated Metal Pipe generally cannot be burst. In some circumstances cuttings blades can be inserted into the pulling head and the CMP can be sliced into longitudinal pieces and pushed into the fill as the replacement pipe in pulled into place.

The length of run is usually from manhole to manhole with longer runs possible. A 500 foot run is common place with the longest run at present being approximately 5000 feet long.

Soil conditions greatly affect the length of run and any potential upsizing of the pipe for replacement. The best to worst soil conditions are original backfill material, expandable clay, loose cobble, beach and running sand, densely compacted clay, with sandstone being the worse.

Upsizing to a larger diameter pipe is dependent on soil conditions, original trench width, and depth. To ensure that the ground surface will not heave there should be at least 4 feet of cover. A rule of thumb is that the burst depth should not be less than ten times the upsize diameter. If the pipe is to upsized by 1 foot then it would require at least 10 feet of cover. This is a rule of thumb only, other conditions such as soil type, groundwater level, and proximity of existing utilities must also be considered.

Areas containing clays and other compressible soils are best suited for upsizing. Sands and gravel are very difficult to burst because the soils tend to collapse in on themselves in the void created by the expander. The collapsed soils cause very large values of skin friction on the pipe being pulled in behind the bursting unit. The friction can stop the bursting head altogether. Rocks in the fill can settle back onto the new liner and collapse it after the expander has passed by. Solid rock outcrops may not expand at all. The need for a thorough geotechnical subsurface investigation before upsizing is attempted cannot be overemphasized. Pipe upsizing from 0 to 25 percent of the diameter is common, 25 percent to 50 percent is challenging, and above 50 percent experimental.

The following should be considered when evaluating a project for proposed pipe bursting:

- Upsizing is harder, whether at depth or shallow.
- The burst length that is possible decreases with depth.
- Pipe frictional drag increases with depth
- Existing utilities can be damaged. Suggest at least 2 feet of clearance between the bursting head and the nearest utility.
- Failures, deep or shallow are more expensive.

- Entry and exit pit construction is affected by depth and soil conditions.
- Lateral connections will have to be disconnected before bursting usually by open trenching.
- Point repairs may be necessary. Bursting does not correct existing sags in host pipe.

Pipe bursting is used extensively in sewer rehabilitation, but has limited use in the replacement of highway cross culverts. Up to 4 foot diameter pipes have been installed using pipe bursting.

### 16.4.3.7 Utility Tunneling

Utility tunneling is a method of soil excavation similar to pipe jacking. The difference is in the lining used. In Pipe Jacking, the pipe is the lining where in utility tunneling special steel or concrete liner plates, wood box tunnels, steel rib and wood lagging systems are used to provide temporary ground support.

The process involves removing soil from the front cutting face and installing a liner to form a continuous support structure. The tunnel is normally constructed between two access shafts. The procedure consists of four major steps:

- Soil excavation.
- Soil removal.
- Segmental liner installation.
- Line and grade control.

**Soil Excavation**

Depending on conditions, soil excavation can be accomplished by hand mining, open-face mechanical excavation, or closed-face tunnel boring machines (TBMs).

**Hand mining**

Hand mining is the simplest form of soil excavation using picks, shovels, or pneumatic hand held tools at the face of the excavation. A protective shield is usually required to provide face stability during soil excavation. In a fixed shield, minor line and grade changes are accomplished by differential excavation in the desired direction. With an articulated shield, line and grade corrections are accomplished by activating hydraulic cylinders internal to the shield.

Hand mining operations are slow, but provide simplicity and the ability to handle difficult and varying soil conditions. Minimal work space is required and linings as small as 30” diameter have been installed. The method is usually limited to short drives where high groundwater table is not a problem.

### Open-face mechanical excavation

The soil excavation rate for open-face mechanical excavation is much faster than with hand mining. Special shields can be equipped with power excavation devices such as rotary cutter booms mounted on the front of the shield or modified hydraulic backhoes. Most shields provide personnel with access to the front face for adjustments of cutter heads for varying ground conditions as well as manual handling of unexpected obstructions. In unstable soil conditions or high groundwater table conditions, compressed air is sometimes used, and in some shields, the tunneling operator is not required to work inside the pressurized zone.

### Closed-face tunneling shields

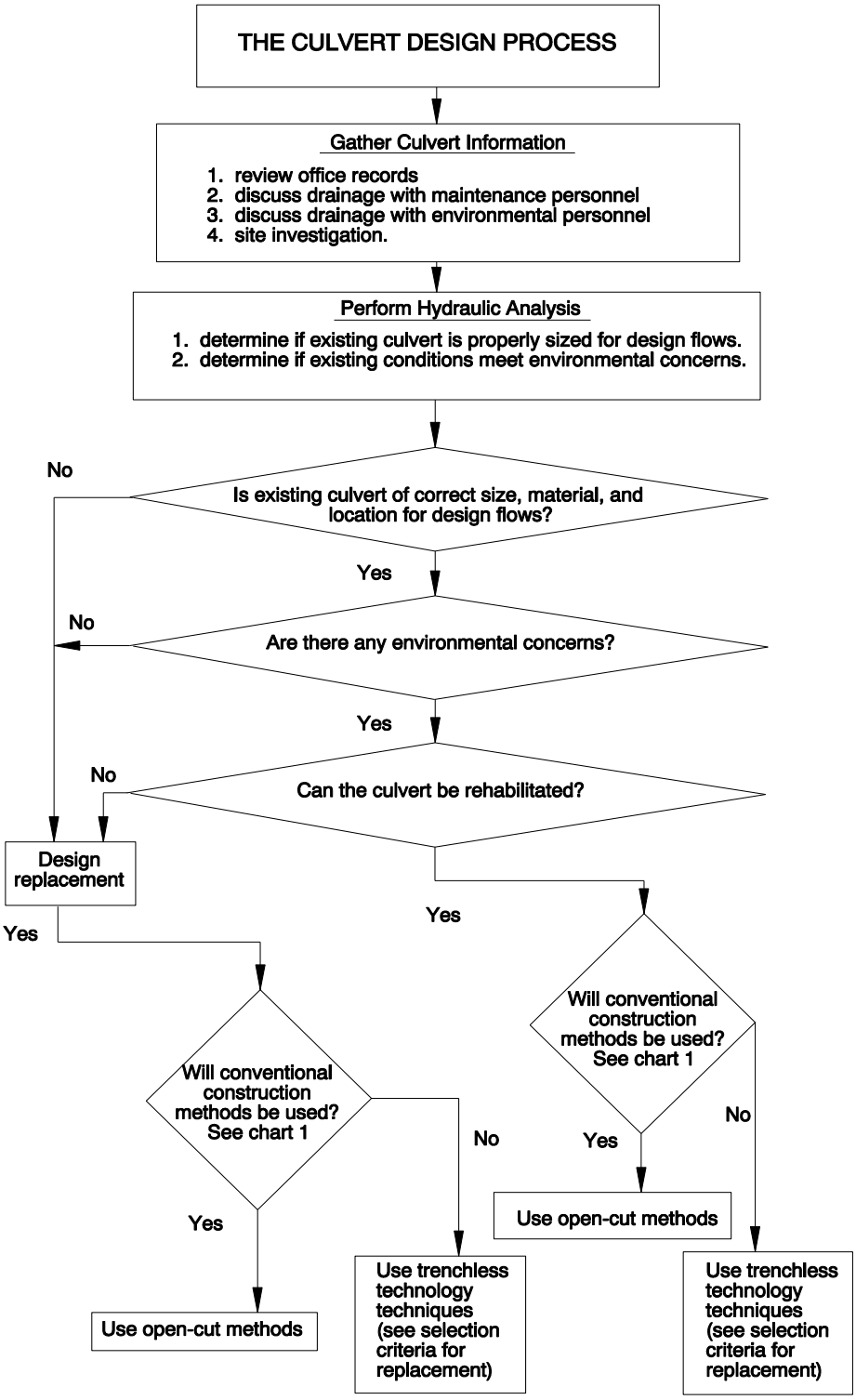
Closed face-tunneling shields are called Tunnel Boring Machines (TBMs) when equipped with hydraulically or electrically driven rotary cutter heads or disk cutters. The cut soil is forced inside the shield through slits in the cutter head as the shield advances. Generally, closed-face TBMs provide better face stability during soil excavation and are therefore more suitable for noncohesive soils below the water table. Some TBMs have crushing devices to excavate gravel and boulders. Major drawbacks include relatively high cost, limited face access, and are restricted to circular tunnels. The most sophisticated TBM systems incorporate a pressure chamber that provides a balance between the soil face pressure and external water pressure. At this level of sophistication this method becomes more of a microtunneling process then utility tunneling.

## Spoil Removal

Selection of the appropriate spoil removal system is determined by space requirements, method of excavation, the mechanism of face pressure balance, and total tunnel length.
The most common methods of removal are listed below:

- Wheeled carts or skips
- Belt and chain conveyors.
- Positive displacement pumping devices.
- Slurry systems.
- Auger systems.
- Vacuum extraction systems.

## Chart 16-1 The Culvert Design Process



**Segmental Liner Installation**

After the soil face has been excavated and removed or the shield is advanced and the jacking cylinders are retracted, new segmental liner sections are brought through the already erected lining to the face. The new liner sections are assembled and connected to the existing lining. If a tunneling shield is used, the retracted jacking cylinders are extended to make contact with the new lining and then mining continues.

The liner usually requires grouting of the annular space outside the liner/segment. The liner has prefabricated grout application holes installed to facilitate the grout application.

The liner plates are typically made of steel or precast reinforced concrete. The steel liner plates have flanged edges that allow the overlapping and bolting together of successive liner plates to form an integrated lining. Some concrete segments are bolted together through precast holes, and some are unbolted. Steel plates are more widely used than concrete plates because their strength to weight ratio is higher.

**Line and Grade Control**

The two most common directional measurement controls are the Theodolite and the Laser systems. The theodolite is a surveying instrument that monitors the current position of the tunnel face directly. The laser system also requires a skilled operator as well as a source of light. A laser system allows for any directional variation to be adjusted immediately, but it is sensitive to temperature variations along the line and can be dispersed over a long distance of dusty air. Neither system can be used continuously because they both cause temporary interruption of the mining work. More sophisticated inertia surveying systems, such as the gyroscope, have been adopted for curved tunneling.

During the soil excavation and shield advancing process, directional change is accomplished by applying different forces to the jacking cylinders controlling the advance of the tunneling shield.

Remote control of many elements of the tunneling process have been realized recently, however the installation and grouting of the segmental liners still requires manual operations. Generally, utility tunneling is a slow labor-intensive process, but is well adapted to rapid changes in soil conditions. The actual jacking pressures are low since only the tunneling shield is moving, which, makes possible a tunnel drive of any length.

## 16.5 Culvert Design Process Guidelines

**Chart 1**

Site specific considerations for the use of trenchless technology

- Are there environmental concerns to consider?
- Is disruption of traffic an important consideration?
- Does construction have to be completed quickly?
- Is a detour necessary?
- Is a high embankment fill involved?
- Will by-pass flows need to be maintained?
- Will dewatering of the site be necessary?
- Are there extra safety concerns on this project?
- Are utility conflicts a concern?
- Is the existing roadway in good condition?
- Are right of way problems present?
- Is the project not feasible using conventional methods?

If the answer to any of the above is yes, then trenchless technology may offer a solution to the problem.

### 16.5.1 Trenchless Method Selection Criteria For Replacement

**Table 16-1 Method By Pipe Diameter**

| TRENCHLESS METHOD | DIAMETER RANGE (Inches) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 12 | 48 | 60 | 72 | 120 | 144 | 168 |
| **Horizontal directional drill (HDD)** | X | X | | | | | |
| **Auger boring (AB)** | X | X | | | | | |
| **Pipe ramming (PR)*** | X | X | X | X | X | X | X |
| **Pipe Jacking (PJ)** | X | X | X | X | X | X | X |
| **Microtunneling (MT)** | X | X | X | X | X | X | X |

= N.A.

* more than 72 inch diameter use Horizontal pile driving equipment.

**Table 16-2 Method By Pipe Length**

| TRENCHLESS METHOD | LENGTH RANGE (Feet) | | | | |
|---|---|---|---|---|---|
| | 40 | 300 | 500 | 1000 | >1000 |
| **Horizontal directional drill (HDD)** | X | X | X | X | X |
| **Auger boring (AB)** | X | X | | | |
| **Pipe ramming (PR)** | X | X | | | |
| **Pipe Jacking (PJ)** | X | X | | | |
| **Microtunneling (MT)** | X | X | X | X | X |

= N.A.

**Table 16-3 Method By Soil Conditions**

| Soil Conditions | AB | HDD | MT | PJ | PR |
|---|---|---|---|---|---|
| **Soft clays, silts, and organics** | YES | YES | RISKY | RISKY | YES |
| **Medium to very stiff clays and silts** | YES | YES | YES | YES | YES |
| **Hard clays and weathered shales** | YES | YES | YES | YES | RISKY |
| **Loose sands above water table** | RISKY | YES | YES | RISKY | YES |
| **Medium to dense sands above water table** | YES | YES | YES | YES | YES |
| **Medium to dense sands below watertable** | NO | YES | YES | NO | YES |
| **Gravels and cobbles less than 4”** | YES | RISKY | YES | YES | YES |
| **Soils with significant cobbles greater than 4”** | RISKY | RISKY | RISKY | RISKY | YES |
| **Firmly cemented soils and weathered rocks** | YES | YES | YES | RISKY | RISKY |
| **Solid rock** | YES | YES | YES | NO | NO |

## METHOD BY DEPTH OF COVER

All trenchless replacement techniques require entrance and exit shafts, except HDD. Pipe can be installed at any depth using these methods.

**Table 16-4 Method By Pipe Type**

| Pipe Type* | AB | HDD | MT | PJ | PR |
|---|---|---|---|---|---|
| **HDPE (high density polyethelene)** | NO | YES | YES | NO | NO |
| **STEEL (A-36)** | YES | YES | YES | YES | YES |
| **PVC (poly vinyl chloride)** | NO | YES | YES | NO | NO |
| **VCP (vitrified clay pipe)** | NO | YES | YES | NO | NO |
| **DIP (ductile iron pipe)** | NO | YES | YES | NO | NO |
| **FRMP (fiber-reinforced mortar pipe)** | NO | YES | YES | YES | NO |

*pipe material selected must be designed to meet the project conditions.

### 16.5.1.1 Cost Guidelines (In 2004 Dollars)

Estimates include cost of installation, mobilization, and planning. Estimates do not include engineering or contingency costs, casing or carrier pipe cost, and cost for preparing entry or exit pits or dewatering costs.

**Table 16-5 Costs Per Foot By Method And Pipe Diameter**

| TRENCHLESS METHOD | DIAMETER RANGE IN INCHES | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 12 | 18 | 24 | 36 | 48 | 60 | 72 | 84 | 96 | 108 | 120 | 132 | 144 | 168 |
| **Horizontal directional drill (HDD)** | 180 | 215 | 250 | 375 | 500 | NA | NA | NA | NA | NA | NA | NA | NA | NA |
| **Auger boring (AB)** | 100 | 150 | 200 | 300 | 400 | NA | NA | NA | NA | NA | NA | NA | NA | NA |
| **Pipe ramming (PR)** | 100 | 150 | 200 | 300 | 400 | 600 | 850 | 1200 | 1700 | 2200 | 2700 | 3200 | 3700 | 4200 |
| **Pipe Jacking (PJ)** | 140 | 210 | 280 | 425 | 575 | 850 | 1220 | 1710 | 2500 | 3150 | 3850 | 4500 | 5300 | 6000 |
| **Microtunneling (MT)** | 240 | 360 | 480 | 720 | 1000 | 1200 | 1450 | 1700 | 2000 | 2400 | 3000 | 3500 | 5000 | 8000 |

***Note: Costs will vary based on location, contractor availability, contractor marketing strategies, time of year, etc. On bores larger that 72-inches in diameter these costs can exceed 50% of the estimate calculated from the following table because costs are totally dependent on soil and site conditions and how effective the contractor's machines are working with the existing conditions.***

### 16.5.2 Trenchless Method Selection Criteria For Rehabilitation

Estimates include cost of installation, mobilization, and planning. It does not include engineering or contingency costs, casing or carrier pipe cost, or cost for preparing entry or exit pits or dewatering costs.

*Note: Costs will vary based on location, contractor availability, contractor marketing strategies, time of year, etc*

**METHOD BY DEPTH OF COVER**

Minimum depth of cover for **pipe bursting** is 4 feet + ½ size increase.
**Fold and Form Liner** is not a structural repair and may fail if used where subjected to high water table. All other methods pipe can be designed to be installed at any depth.

**Table 16-6 Method By Pipe Diameter**

| TRENCHLESS TECHNOLOGY METHOD | DIAMETER RANGE IN INCHES | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 12 | 18 | 24 | 30 | 36 | 42 | 48 | 60 | 72 | 84 | 96 | 120 | more than 120 |
| **SLIPLINING *** | X | X | X | X | X | X | X | X | X | X | X | X | X |
| **CURED-IN-PLACE LINER** | X | X | X | X | X | X | X | X | X | X | X | | |
| **FOLD AND FORM PIPE LINER** | X | X | X | | | | | | | | | | |
| **SPIRAL WOUND LINER** | X | X | X | X | X | X | X | X | X | X | X | X | X |
| **COATING(INVERT LINING)** | | | | | X | X | X | X | X | X | X | X | X |
| **PIPE BURSTING** | X | X | X | X | X | X | | | | | | | |

= N.A.

*sliplining includes either HPDE or CMP pipe and grouting the annual space.

## Table 16-7 Method By Pipe Length

| TRENCHLESS TECHNOLOGY METHOD | LENGTH RANGE IN FEET | | | | |
|---|---|---|---|---|---|
| | 40 | 300 | 500 | 1000 | more than 1000 |
| **SLIPLINING *** | X | X | X | X | X |
| **CURED-IN-PLACE LINER** | X | X | X | X | X |
| **FOLD & FORM PIPE LINER** | X | X | X | X | X |
| **SPIRAL WOUND LINER** | X | X | X | X | X |
| **COATING(INVERT LINING)** | X | X | X | X | X |
| **PIPE BURSTING** | X | X | | | |

= N.A.

**Table 16-8 Method By Soil Conditions**

| Soil Conditions | SLIPLINING | CURED-IN-PLACE | FOLD & FORM PIPE | SPIRAL WOUND | INVERT PAVING | PIPE BURSTING |
|---|---|---|---|---|---|---|
| **Soft clays, silts, and organics** | YES | YES | YES | YES | YES | YES |
| **Medium to very stiff clays and silts** | YES | YES | YES | YES | YES | YES |
| **Hard clays and weathered shales** | YES | YES | YES | YES | YES | NO |
| **Loose sands above water table** | YES | YES | YES | YES | YES | YES |
| **Medium to dense sands above water table** | YES | YES | YES | YES | YES | YES |
| **Medium to dense sands below water table** | NO | YES | NO | NO | NO | YES |
| **Gravels and cobbles less than 4 inches** | YES | YES | YES | YES | YES | YES |
| **Soils with significant cob inches** | YES | YES | YES | YES | YES | YES |
| **Firmly cemented soils and weathered rocks** | YES | YES | YES | YES | YES | NO |
| **Solid rock** | YES | YES | YES | YES | YES | NO |

**Table 16-9 Method By Pipe Type ***

| PIPE TYPE* | SLIPLINING | CURED-IN-PLACE | FOLD & FORM PIPE | SPIRAL WOUND | INVERT PAVING | PIPE BURSTING |
|---|---|---|---|---|---|---|
| **HDPE (high density polyethylene)** | YES | NA | NA | NA | NA | YES |
| **STEEL (A-36)** | YES | NA | NA | NA | NA | YES |
| **PVC(poly vinyl chloride)** | YES | NA | NA | NA | NA | YES |
| **VCP (vitrified clay pipe)** | YES | NA | NA | NA | NA | NO |
| **DIP (ductile iron pipe)** | YES | NA | NA | NA | NA | NO |
| **FRMP (fiber-reinforced mortar pipe)** | YES | NA | NA | NA | NA | NO |

* Pipe material selected for liner must be designed to meet project conditions.

### 16.5.2.1 Cost Guidelines (In 2004 Dollars)

Estimates include cost of installation, mobilization, and planning. Estimates do not include engineering or contingency costs, casing or carrier pipe cost, cost for preparing entry or exit pits or dewatering costs.

*Note: Costs will vary based on location, contractor availability, contractor marketing strategies, time of year, etc.*

**Table 16-10 Costs Per Foot By Method And Pipe Diameter**

| TRENCHLESS TECHNOLOGY METHOD | PIPE DIAMETER IN INCHES | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 12 | 18 | 24 | 30 | 36 | 42 | 48 | 60 | 72 | 84 | 96 | 120 |
| **SLIPLINING *** | 100 | 150 | 180 | 200 | 225 | 250 | 300 | 350 | 400 | 460 | 560 | 700 |
| **CURED-IN-PLACE LINER** | 100 | 180 | 250 | 300 | 400 | 510 | 680 | 910 | 1200 | 1600 | 2200 | NA |
| **FOLD AND FORM PIPE LINER** | 165 | 205 | 250 | | | | | | | | | |
| **SPIRAL WOUND LINER** | 80 | 130 | 160 | 170 | 200 | 220 | 250 | 315 | 380 | 440 | 500 | 630 |
| **COATING (INVERT LINING)** | NA | NA | NA | NA | 80 | 85 | 95 | 110 | 130 | 140 | 145 | 150 |
| **PIPE BURSTING** | 200 | 300 | 400 | 480 | 600 | 800 | | | | | | |

*sliplining cost includes pipe and grouting the annual space.

# CHAPTER 17

# FIELD MARKERS

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## 17.1 Introduction

The purpose of this chapter is to outline a statewide process of field marking:

- Stormwater treatment facilities such as swales, ponds, filter strips, sedimentation basins, bioslopes, proprietary structures that utilize vaults and oversized manholes or tanks, and pretreatment manholes,
- Stormwater storage facilities such as ponds, tanks, and vaults,
- Low impact development (LID) best management practices (BMPs),
- Underground Injection Control (UIC) systems, and
- Culverts with a span less than 20 feet.

Oregon Department of Transportation (ODOT) owns, operates, and maintains hundreds of stormwater treatment and storage facilities and thousands of culverts across the state of Oregon. A process that includes assigning an identification number, placing field markers to locate and identify, and supports asset management data collected has been created for these ODOT drainage features. Other drainage features may be added to this process and updates would be made to this chapter.

**Figure 17-1 Stormwater Facility Field Marker**

**Figure 17-2 Culvert Field Marker**

## 17.2 Stormwater Treatment and Storage Facility Field Markers

There are three types of markers used to identify ODOT facilities or alert maintenance crews of the location of stormwater facility maintenance areas. A Type S1 marker is used to indicate the start and end of stormwater facility maintenance areas. The purpose of the Type S2 marker is to display the facility drainage identification number. Maintenance crews would be able to reference the appropriate Operation and Maintenance Manual using the ID number assigned to each facility. A Type S3 marker is used to stamp a drainage facility identification number onto the top of access covers of underground treatment and storage facilities that utilize vaults, oversized manholes, and tanks.

### 17.2.1 Responsibilities

The ODOT Regions or individuals within or outside of the agency have the following responsibilities:

- Project Hydraulics Engineer – Requests stormwater drainage facility IDs. Coordinates marking of new low impact development and conventional best management facilities. Includes drainage facility IDs on Stormwater Report and Operation and Maintenance Manual. Notifies applicable District Managers of all new stormwater facilities.

- District Manager – Coordinates marking existing stormwater facilities, and replacing/repairing damaged markers. Obtains drainage facility ID number(s) for existing stormwater facilities.

- Technical Services Geo-Environmental Section's Senior Hydraulics Engineer – Assigns and issues stormwater drainage facility IDs upon request. Maintains a database of assigned Drainage Facility IDs.

### 17.2.2 Definitions

Definitions of important terms are listed in this section.

Stormwater Facility Field Marker – Used to identify and locate ODOT stormwater facilities or alerts maintenance crews of stormwater facility maintenance areas. Three stormwater markers are recommended for identifying, locating, or alerting:

Type S1 – Marker using a non-reflective flexible plastic post.

Type S2 – Marker using a white background non-reflective aluminum paddle mounted onto one (1) steel post.

Type S3 – Marker created by stamping the drainage facility ID onto the top of manhole, vault, and/or tank access covers.

Drainage Facility ID (DFI) – A unique "DFI" is assigned to all stormwater treatment and storage facilities. It is used to associate or link the stormwater facility to an Operation and Maintenance Manual. The number is assigned by contacting the Geo-Environmental Section's Senior Hydraulics Engineer to obtain a unique "DFI". The Geo-Environmental Section will maintain a database of assigned Drainage Facility IDs.

Engineered Stormwater Treatment and/or Storage Facilities – A treatment or storage facility that requires engineering analysis to determine the hydrology, hydraulics, and design of the structure. Engineered treatment facilities include dry and wet detention basins, swales, treatment wetlands, and proprietary systems. Engineered storage facilities used for stormwater management include dry detention basins, vaults, and tanks.

Low Impact Development (LID) – LID is the concept of designing projects to minimize the effect on natural hydrology and water quality. This is primarily accomplished by minimizing impervious surface areas and applying LID best management practices which provide opportunities for infiltration of stormwater into vegetated soil.

Underground Injection Control (UIC) Systems – An underground injection control is any system, structure, or activity that is created to place fluid below the ground or sub-surface. Common stormwater underground injection systems or activities in Oregon include but are not limited to infiltration galleries, drywells, and subsurface drainfields. These types of facilities are considered stormwater disposal systems, not treatment systems, and have impacted groundwater quality in a number of communities across the nation.

Operation and Maintenance Manual – Summarizes a facility's inspection schedule and maintenance guidelines. It is prepared to assist personnel who maintain the facility. All engineered stormwater treatment and storage facilities must have an Operation and Maintenance Manual.

Stormwater Facility Footprint – The facility footprint includes the area needed to accommodate the pretreatment structure, storm drain piping, treatment zone (basin or pond, swale, filter strip, etc), primary and auxiliary outlets, outfalls, and a maintenance access road.

Stormwater Facility Maintenance Area – The stormwater facility maintenance area includes the stormwater facility footprint. These maintenance areas are applicable to all above ground stormwater treatment and storage facilities. Markers are placed at the start and end of each facility's maintenance area to alert maintenance crews of special maintenance requirements as outlined in the applicable Operation and Maintenance Manual. See Figure 17-4 for an example of a maintenance area.

### 17.2.3 General Field Marking Requirements

- All ODOT stormwater treatment and storage facilities must have a unique drainage facility ID. A Drainage facility ID number is not required for pollution control manholes that do not qualify as a water quality structure. See standard drawing RD340.

- Field marking is required when an existing ODOT stormwater facility has a prepared Operation and Maintenance Manual.
- Field marking is optional when an existing ODOT stormwater facility does not have a prepared Operation and Maintenance Manual.
- Field marking is required for all new ODOT conventional stormwater facilities, and low impactment development (LID) best management practices or other facilities that require special maintenance activities.

### 17.2.4 Type S1 Field Markers

A type S1 marker is a non-reflective flexible plastic post embedded into the ground. The purpose of this marker is to alert maintenance crews of the start and end of a stormwater facility maintenance area. A type S1 marker is shown in Figure 17-3. Standard placement of this marker is shown in Figure 17-4.

A type S1 marker color is red or green. A red marker indicates the beginning of a stormwater facility maintenance area. A green marker indicates the end of a stormwater facility maintenance area. Red and green marker coding is implemented with the direction of travel.

Placement of the red and green marker should be 4 to 6 feet from edge of pavement or face of curb. Note that these markers are not part of the mile point system or road delineator markers system.

Material and installation details are shown on Standard Drawing RD399.

### 17.2.5 Type S2 Field Markers

A type S2 marker uses an aluminum paddle mounted onto one (1) type 1U steel post. The purpose of a type S2 marker is to indicate the assigned facility drainage identification number. A type S2 marker is shown in Figure 17-3. Standard placement of this marker is shown in Figure 17-4.

The aluminum paddle or type S2 marker color will be white (non-reflective).

Placement of the type S2 marker should be at the right-of-way line and at the beginning of a stormwater facility maintenance area. Placement of a Type S2 marker at the end of a stormwater facility maintenance area is required if the facility's length is greater then 100 feet. Do not directly attach paddle to any historic or potentially historic structure when an alternate option is used to mount a paddle. Note that these markers are not part of the mile point system or road delineator markers system.

Installation details, paddle dimensions, and paddle mounting guidance are shown on Standard Drawing RD399.

### 17.2.6 Type S3 Field Markers

A Type S3 marker is used to mark the drainage facility identification number on an underground stormwater treatment system or facility. A type S3 marker is shown in Figure 17-3. Standard placement of this marker is shown in Figure 17-4.

The Type S3 marker places the drainage facility identification number on the access cover of a vault, manhole or tank for an underground stormwater treatment/storage facility. Ink stamping the drainage facility ID is not allowed on the top of access covers.

Installation details are shown on Standard Drawing RD399.

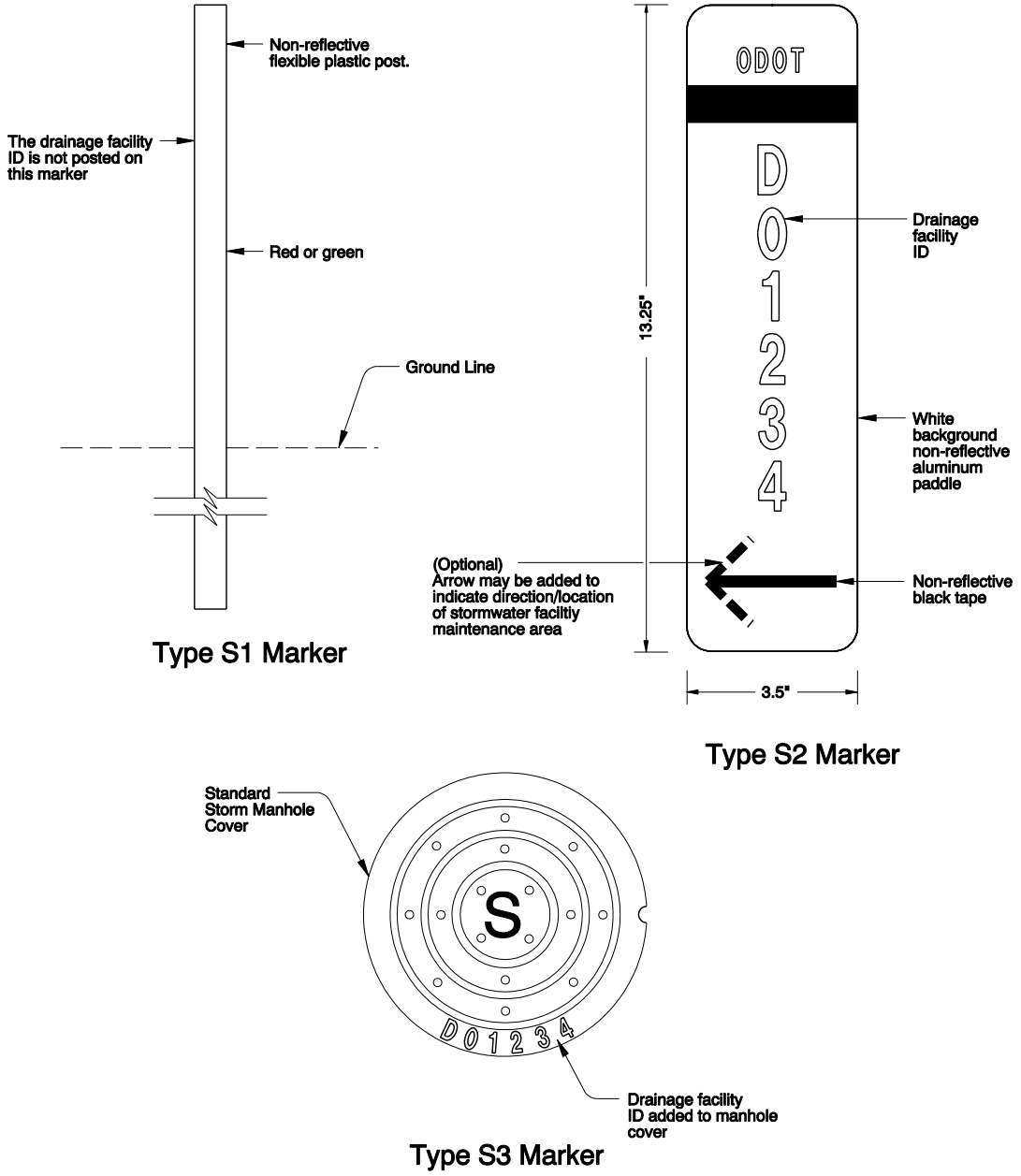



**Figure 17-3 Example of Stormwater Facility Field Marker**

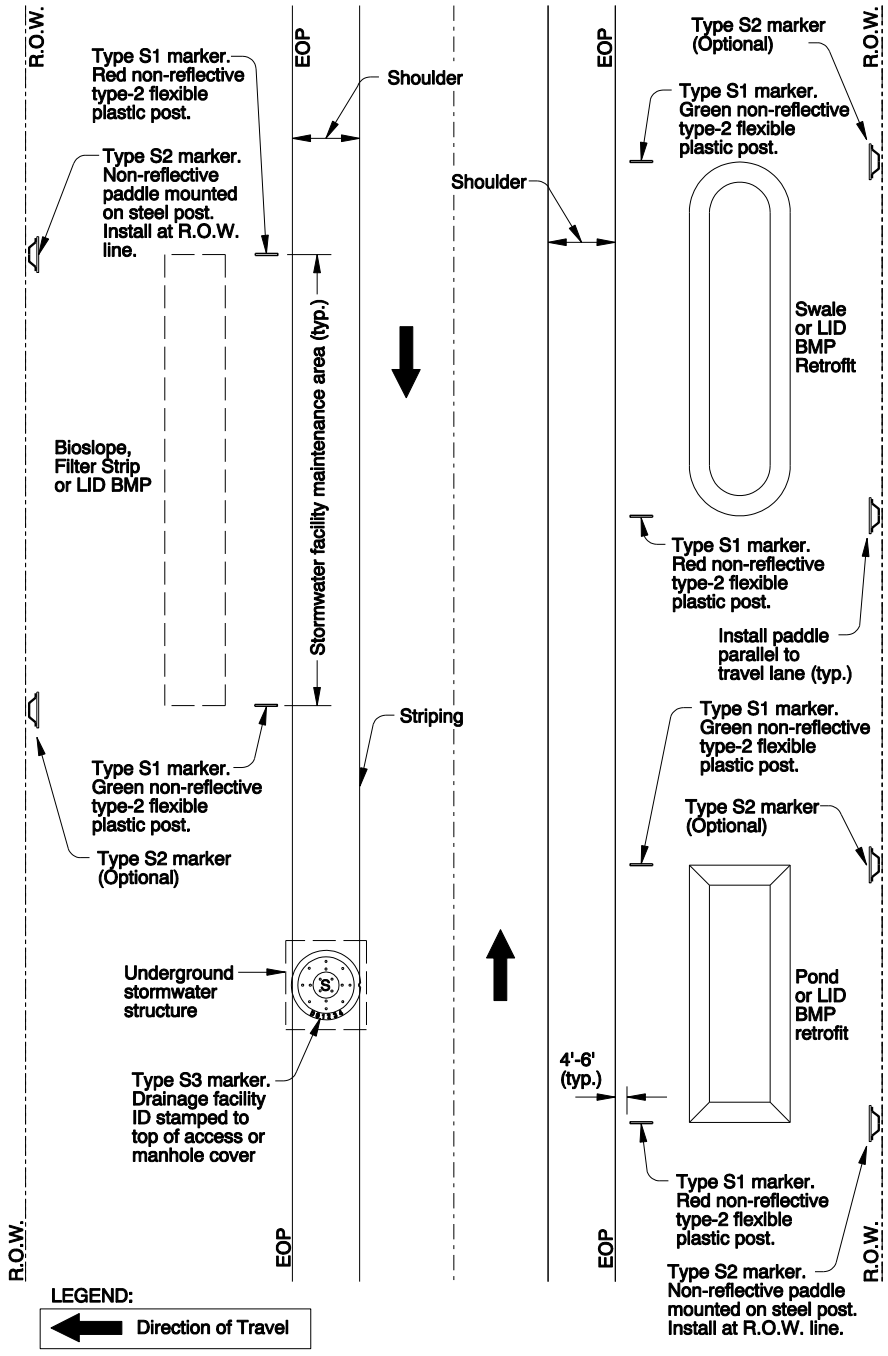



**Figure 17-4 Example of Stormwater Facility Field Marker Location**

**Figure 17-5 Stormwater Facility Field Markers (Swale along Hwy 22)**

### 17.2.7 Stormwater Treatment and Storage Facility Field Marking Procedure

The procedure for marking new and existing stormwater facilities and/or structures is outlined below:

**New Facilities**

**Step 1 -** The project hydraulics engineer requests drainage facility IDs when proposing engineered facility or low impact development systems.

**Step 2 -** Provide the following information for each facility needing a drainage facility ID:

- facility type(s)
- highway name
- highway number
- route number
- milepost when using a Type 3 marker or mileposts that equal the start and end of a stormwater facility maintenance area when applicable

- side of highway (side of highway is determined Right or Left in the direction of increasing mile points)
- is the facility a proprietary structure (yes or leave the cell blank)
- county
- region
- district, and
- requestor

Submit this information for each facility in an excel spreadsheet per Table 17-1.

| Facility Type | Hwy Name | Hwy # | Route # | Milepost (Begin) | Milepost (End) | Side of Highway | Proprietary | County | Region | District | Requested by |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Swale | Redwood Hwy | 25 | US 199 | 10.81 | 10.93 | Right | | Josephine | 3 | 8 | D. Sharp |

**Table 17-1 Example Excel Spreadsheet Drainage Facility ID Request Information**

Submit information to the Geo-Environmental's Senior Hydraulics Engineer and a drainage facility ID number(s) will be assigned to the requestor. Email address is available at the following website:

http://www.oregon.gov/ODOT/HWY/GEOENVIRONMENTAL/hydraulics1.shtml

**Step 3 -** Include drainage facility ID number(s) in the project Stormwater Report, Operation and Maintenance Manual, and appropriate Stormwater Control Special Provision(s).

**Step 4 -** Include drainage facility ID number(s) on Geo-Hydro stormwater plan sheets (Sheeting numbering: GJ). Reference Contract Plans Development Guide, Volume 1, Table 2-1).

**Step 5 -** Include Standard Drawing RD399 in the project design plans.

**Step 6 -** Provide project drafter the following information for each facility: facility stationing, milepoints, drainage facility IDs, paddle locations, red (begin) flexible plastic post locations, and green (end) plastic post locations. This information will be incorporated into a stormwater field marker table and added to the Geo-Hydro stormwater plan sheet section. The table can be found in the "hydro.cel" library within Microstation. See Table 17-2.

STORMWATER FIELD MARKER TABLE

| FACILITY LOCATION | | DFI # | TYPE S2 MARKER LOCATION | | TYPE S1 MARKER | |
|---|---|---|---|---|---|---|
| STATION | MP | | BEGIN | END | RED | GREEN |
| NB 13+800 | 12.45 | D01234 | ✓ | | ✓ | |
| NB 14+800 | 12.64 | D01235 | | ✓ | | ✓ |
| | | | | | | |
| | | | | | | |

✓ Check where appropriate
Red = Beginning of facility
Green = End of facility

**Table 17-2 Example Stormwater Field Marker Table**

**Step 7 -** Type 1, Type 2, and Type 3 markers are specified in Special Provision 00842.

**Existing Facilities**

**Step 1 -** District Manager requests drainage facility IDs for an existing facility from Geo-Environmental Section's Senior Hydraulics Engineer. Provide the information for each facility needing a drainage facility ID as described in Step 2 for NEW Facilities.

**Step 2 -** District Manager orders Type S2 markers from the ODOT Sign Shop according to the marker specifications outlined in Standard Drawing RD399. Provide a list of drainage facility IDs when submitting the sign request to the Sign Shop.

**Step 3 -** District Manager coordinates installation of markers as outlined in Section 17.2.

## 17.3 Culvert Field Markers

The purpose of the culvert field markers is to set a consistent statewide standard for marking culverts from twelve inches to twenty feet in culvert span. The underlying principle for the placing of the field markers is to provide a convenience to maintenance crews, scoping teams and asset management in locating culverts openings. The installation of culvert drainage markers is optional and will be installed as determined by persons listed in the responsibility section below. Each maintenance district and project team should consider installing drainage markers for all culverts, excluding private approaches.

### 17.3.1 Responsibilities

The ODOT Regions or individuals within or outside of the agency have the following responsibilities:

- Project Hydraulics Engineer – Determines location for Type 1 markers for culverts with a span greater then four feet. Coordinates with District Manager to determine the placement of the Type 2 markers. Requests culvert drainage facility ID's for the Type 2 markers.

- Project Roadway Engineer – Determines location for Type 1 markers for culverts with a span of four feet and less. Coordinates with District Manager to determine the need and placement of the Type 2 markers. Requests culvert drainage facility ID's for the Type 2 markers.

- District Maintenance Manager – Coordinates with project hydraulic engineer and roadway engineer during development of new projects to determine Type 2 marker needs and placement in relation to the culvert. Coordinates marking existing culverts that are not project related and replacing and repairing damaged Type 1 and Type 2 markers. Obtains drainage facility ID number(s) for Type 2 markers for exiting culverts.

- Technical Services - Geo-Environmental Section's Culvert-Hydraulic Engineer – Assigns and provides approved drainage facility ID numbers upon request.

### 17.3.2 Definitions

Definitions of important terms are listed in this section.

Culvert Field Marker - Used to locate and identify culverts along the ODOT's transportation system. Two types of culvert field markers can be used for identifying and locating culverts:

Type 1 – A green performed fused thermoplastic tape installed along the inlet edge of pavement.

Type 2 – Marker using a white background non-reflective aluminum paddle with back lettering and green strip that is mounted onto a steel post or permanent feature.

Drainage Facility Management System (DFMS) – A database developed by ODOT to track information on culverts within the transportation system. DFMS includes a graphical user interface that allows users to add, track and update information on culverts. DFMS is a web based system that can be accessed from any ODOT network.

Drainage Facility ID (DFI) – The Drainage Facility ID is used in DFMS to uniquely identify culverts within ODOT's highway system. The approved DFI will be either a Bridge Structure number or a DFI that begins with a 'D'. The DFI will default to the bridge structure number if one is assigned and entered into DFMS.

Cross Culvert – Culvert that crosses underneath the roadway.

Pipe Left and Pipe Right - Culverts that run perpendicular, parallel or at a skew to the highway but do not cross any travel lanes.

Public Approach Left & Right - Culverts that pass under public approach roads are termed 'Public Approach Left' & 'Public Approach Right'. These do not include private roadway or field approaches.

Bridge Structure Number – Identifying number of the ODOT Bridge Data System. Bridge structure numbers are given to culverts who bridge span is six feet and greater.

### 17.3.3 Field Marking Parameters

The Type 1 and Type 2 culvert markers are only to be used for cross culverts, pipe left and pipe right installations, and public approaches. These three categories can encompass many different kinds of culverts such as, streams, roadside drainage, and cattle passes. When marking culverts include those with grated or drop inlets with an open end at the outlet of the culvert.

Culverts that connect to other systems by means of manholes or blind connections are not to be located using the culvert markers. The following types of culverts are not be marked with the Type 1 and Type 2 culvert field markers:

- Slotted drains or perforated pipes
- Sanitary systems
- Stormwater systems attached via manholes.
- Private roadway or field approaches
- Culverts smaller than twelve inches in span.

### 17.3.4 Type 1 Field Markers

The Type 1 marker is a green preformed fused thermoplastic tape that measures from six to twelve inches in length by four inches wide. The marker is installed along the inlet edge of the pavement for quick identification of the culvert location from the road surface. Type 1 markers do not need a drainage facility ID before installation. A Type 1 drainage marker is shown in Figure 17-6.

Type 1 Placement Requirements:

The long portion of the pavement marking will be perpendicular to the center line of the highway and located at the inlet edge of the pavement. Place the pavement marking directly in line with the culvert inlet. Typical placement is shown in Figure 17-7.

- When there are three or more culverts within 50 feet of each other then mark the first and last drainage facility only.
- On non-divided highways place the pavement marker only on the culvert inlet side of the highway.

- For divided highways place the pavement marker on the culvert inlet side of the highway. Marking the culvert outlet side of the highway is optional.
- Do not place the pavement marker over the fog line or in the travel lane.

Material and installation details are shown on Standard Drawing RD398 and specified in Special Provision 00842.

### 17.3.5 Type 2 Field Markers

The Type 2 marker is a white background non-reflective aluminum paddle with black lettering that is mounted to a steel post or attached to a permanent feature. The paddle is marked with the facility's drainage ID, mile point, and a green strip near the top. All Type 2 markers must have an approved drainage facility ID and mile point from DFMS. A Type 2 marker is shown in Figure 17-6. The three different installation options for the Type 2 marker are listed below.

- Steel Posts: Aluminum paddle with the unique drainage facility ID and mile point mounted onto a Type 1U steel post.
- Guardrail Areas: Aluminum paddle with the unique drainage facility ID and mile point mounted on wood guardrail posts.
- Concrete Feature Areas: Aluminum paddle with the unique drainage facility ID and mile point mounted on a concrete culvert headwall or concrete barrier.

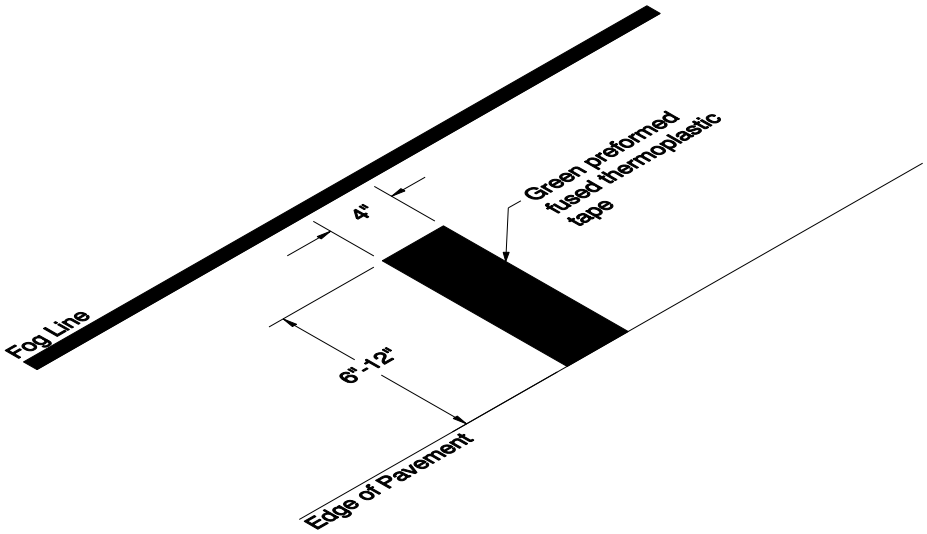
Type 2 Placement Requirements:

Drainage Markers are not part of the mile point system or road delineator marker system. The drainage markers will be located so that they are not noticed or conspicuous to the traveling public. Type 2 markers will not be attached directly to any historic structures which may include headwalls, bridges, and retaining walls. The markers only purpose will be to help identify the location of culverts for ODOT personnel. Typical placement is shown in Figure 17-7.

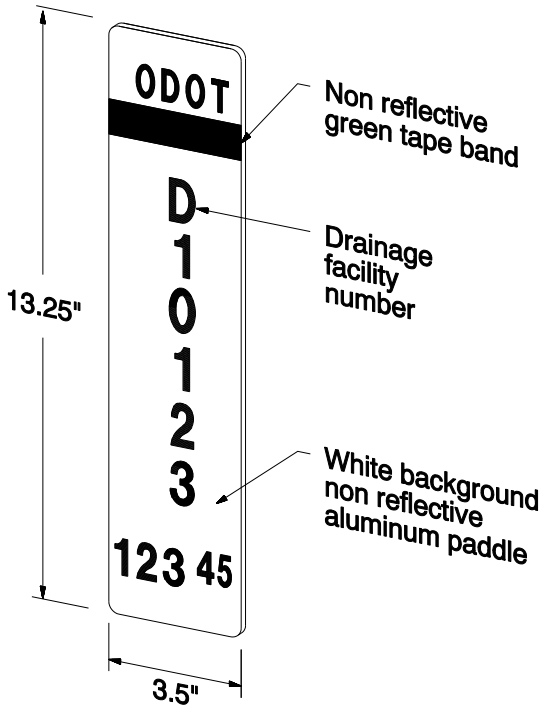
- When the Type 2 marker is mounted on a Type 1U steel post install the marker parallel to the travel lane.
- On non-divided highways place the Type 2 marker only on the culvert inlet side of the highway.
- For divided highways place the Type 2 marker on the culvert inlet side of the highway. Marking the culvert outlet side of the highway is optional.

Material and installation details are shown on Standard Drawing RD398 and specified in Special Provision 00842.

Material and installation details are shown on Standard Drawing RD398.

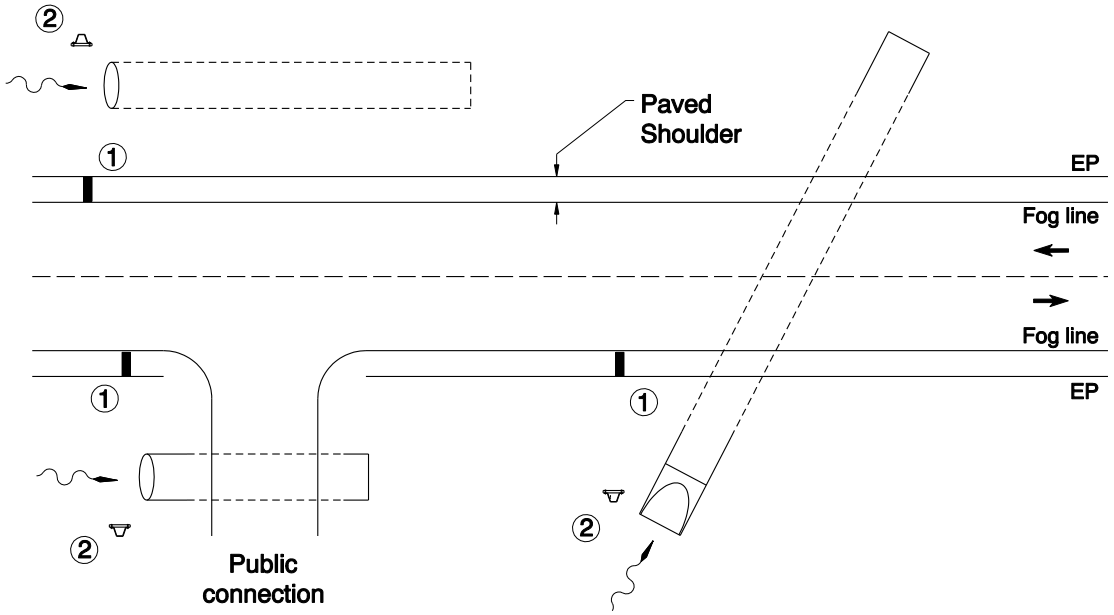



**Type 1 Marker**




**Type 2 Marker**

**Figure 17-6 Example of Culvert Field Marker**




① Type 1 Marker - Green preformed fused thermoplastic tape. Place in-line with culvert inlet between edge of pavement and fog line.

② Type 2 Marker - Non reflective paddle mounted on steel post. When used in conjunction with type 1 marker place type 2 marker near inlet.

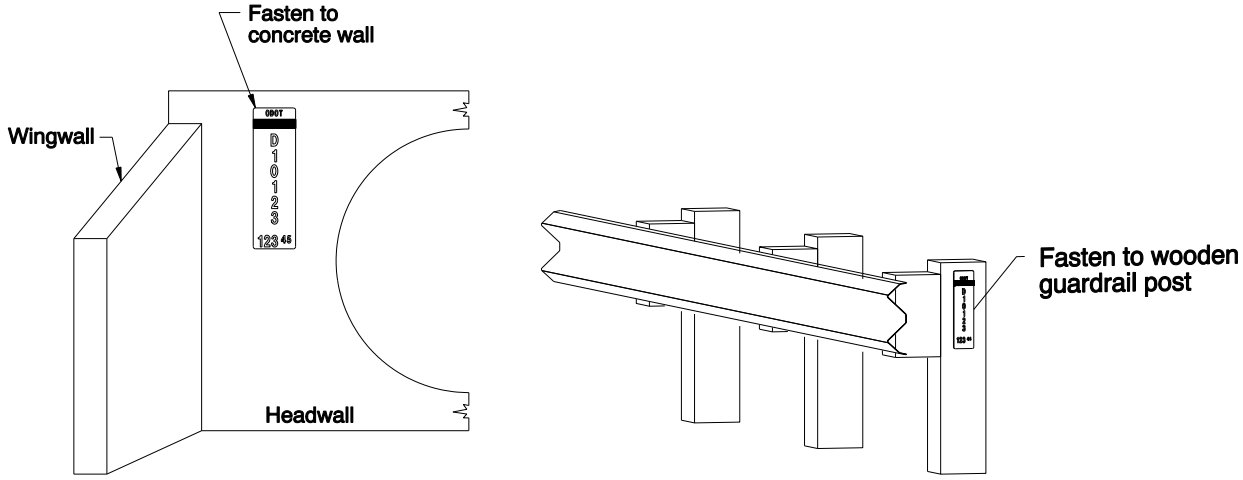



**Figure 17-7 Example of Culvert Field Marker Location**

### 17.3.6 Culvert Field Marking Procedure

The Type 1 and Type 2 culvert markers can be used independently or in combination with each other to provide additional culvert information and location. The procedure for marking new and existing culverts is outlined below:

**Project Based Culverts**

**Type 1 Culvert Marker:**

1. Provide project drafter with culvert stationing, milepoint(s), and inlet / outlet location(s) of the Type 1 marker. The project drafter will complete the DFI culvert marker table, located in the Microstation Workspace under the hydro cell library, and place in the roadway plan sheet section. Type 1 markers do not need a drainage facility ID before installation.

2. Include Standard Drawing RD398 in the project plans.

**Type 2 Culvert Marker:**

Culverts with a span of 6 feet and less than 20 feet:

1. The project hydraulic engineer requests a bridge structure number from ODOT Bridge Section. The drainage facility ID will default to the bridge structure number when assigned and placed on the Type 2 marker.

2. Include bridge structure number(s) on Geo-Hydro culvert plan sheets (Sheet numbering: GE). Reference Contract Plans Development Guide, Volume 1, Table 2-1. Use DFI title block cell located in the Microstation Workspace under the hydro cell library.

3. Provide project drafter the following information for each culvert: stationing, milepoint, bridge structure IDs, and Type 2 and Type 1 locations. The project drafter will complete the DFI culvert marker table, located in the Microstation Workspace under the hydro cell library, and place in Geo-Hydro culvert plan sheet section.

4. Include Standard Drawing RD398 in the project plans.

5. Provide the drainage facility ID and hydraulics plan sheet(s) to the Culvert-Hydraulic Engineer in Geo-Environmental Section.

Culverts with a span less than 6 feet:

1. The project hydraulic engineer / project roadway engineer requests a drainage facility ID from ODOT Geo- Environmental Section.

2. Provide the following information for each facility needing a drainage facility ID:

    - Highway number and milepoint
    - Frontage / Connection ID if needed; ex: 001AB
    - Number of Barrels
    - Barrel Rise, Span and Shape
    - Length of culvert
    - Material
    - Inlet side
    - Height of cover over inlet and outlet
    - Stream Name
    - Facility Usage
    - Stream Name
    - Requestor

3. Submit information to the Geo-Environmental's Culvert-Hydraulic Engineer and a drainage facility ID number(s) will be assigned to the requestor. Email address is available at the following website:

    http://www.oregon.gov/ODOT/HWY/GEOENVIRONMENTAL/culvert_dfms.shtml

4. Include drainage facility ID number(s) on appropriate sheet:

    a. Geo-Hydro culvert plan sheets (Sheet numbering: GE) - Reference Contract Plans Development Guide, Volume 1, Table 2-1. Use DFI title block cell located in the Microstation Workspace under the hydro cell library. Use for culverts with a span greater than 48 inches.

    b. Roadway drainage plan sheets – (Sheet numbering: 3) - Reference Contract Plans Development Guide, Volume 1, Table 2-1. Place drainage facility ID, marker type and location in culvert note. Use for culverts with a span 48 inches and less.

5. Provide project drafter the following information for each culvert: stationing, milepoint, drainage facility IDs, and Type 2 and Type 1 locations. The project drafter will complete the DFI culvert marker table, located in the Microstation Workspace under the hydro cell library, and place in the appropriate plan sheet section.

6. Include Standard Drawing RD398 in the project plans.

## Existing Culverts

**Type 1 Culvert Marker:**

1. District Manager coordinates installation of markers as outlined in Section 17.3. Type 1 markers do not need a drainage facility ID before installation.
2. Place Type 1 marker using installation details shown on Standard Drawing RD398.

**Type 2 Culvert Marker:**

1. District Manager requests drainage facility IDs for an existing facility based on culvert span.
    a. Culverts with a span of 6 feet and less than 20 feet: Request a bridge structure number from ODOT Bridge Section. The drainage facility ID will default to the bridge structure number when assigned and placed on the Type 2 marker.

        i. Provide the drainage facility ID and location(s) to the Culvert-Hydraulic Engineer in Geo-Environmental Section.

    b. Culverts with a span of less than 6 feet: Request a drainage facility ID from ODOT Geo-Environmental Section's Culvert-Hydraulic Engineer. Provide the information for each culvert needing a drainage facility ID as described in Project Based Type 2 Culvert Marker.

2. District Manager orders Type 2 markers from the ODOT Sign Shop according to the marker specifications outlined in Standard Drawing RD398.

3. District Manager coordinates installation of markers as outlined in Section 17.3.

### 17.3.7 Drainage Facility ID Guidelines for Type 2 Culvert Markers

The drainage facility ID is used to uniquely identify culverts within ODOT's highway system. Culverts that are signed with a Type 2 marker will include the approved drainage facility ID and milepoint from the DFMS. The approved drainage facility ID will be either a Bridge Structure number or a drainage facility ID that begins with a 'D'. The drainage facility ID will default to the bridge structure number if one is assigned and entered into DFMS.

| Drainage Facility ID | Culvert Span | Asset ID | Identifier Example | Type 2 |
|---|---|---|---|---|
| DFMS Culvert (ID assigned by Geo-Environmental - DFMS) | 12 in < X < 72 in | DFI 'D' | D026906 | Yes |
| Bridge Structure (ID assigned by ODOT's Bridge Section ) | 6 ft < X < 20 ft | Bridge Structure No. | 16199 or 03037A | Yes |
| Bridge Structure (ID assigned by ODOT's Bridge Section. | X > 20 ft | Bridge Structure No. | 16199 or 03037A | No |
| Temporary culverts will begin with a 'T' in the first position (These represent culverts that have been migrated over to DFMS). | 1 ft < X < 20 ft | ---------- | T002904 | No |

**Table 17-3 Example of Culvert Drainage Facility IDs for Type 2 Marker**

# GLOSSARY OF HIGHWAY-RELATED DRAINAGE TERMS

(Reprinted from the 1999 edition of the American Association of State Highway and Transportation Officials Model Drainage Manual)

## G.1 Introduction

This Glossary is divided into three parts:

- Introduction,
- Glossary, and
- References.

It is not intended that all the terms in this Glossary be rigorously accurate or complete. Realistically, this is impossible. Depending on the circumstance, a particular term may have several meanings; this can never change.

The primary purpose of this Glossary is to define the terms found in the *Highway Drainage Guidelines* and *Model Drainage Manual* in a manner that makes them easier to interpret and understand. A lesser purpose is to provide a compendium of terms that will be useful for both the novice as well as the more experienced hydraulics engineer. This Glossary may also help those who are unfamiliar with highway drainage design to become more understanding and appreciative of this complex science as well as facilitate communication between the highway hydraulics engineer and others.

Where readily available, the source of a definition has been referenced. For clarity or format purposes, cited definitions may have some additional verbiage contained in double brackets [ ]. Conversely, three "dots" (...) are used to indicate where some parts of a cited definition were eliminated. Also, as might be expected, different sources were found to use different hyphenation and terminology practices for the same words. Insignificant changes in this regard were made to some cited references and elsewhere to gain uniformity for the terms contained in this Glossary: as an example, "groundwater" vice "ground-water" or "ground water," and "cross section area" vice "cross-sectional area."

Cited definitions were taken primarily from two sources: W.B. Langbein and K.T. Iseri's "General Introduction and Hydrologic Definitions" *(37)* and a draft of a Glossary being developed by the Interagency Hydrology Committee. A few cited definitions were considered to be partially outdated; corrections are suggested where this occurred. Future plans of the AASHTO Task Force on Hydrology and Hydraulics are to integrate the current American Society of Civil Engineers Glossary of hydrologic terms with this Glossary.

Many terms are explained and defined in several ways—sometimes with considerable detail. This was done intentionally for several reasons:

- to facilitate understanding for those who respond better to a particular written format or detail of explanation,
- to highlight actual or apparent contradictions in current terminology,
- to avoid or minimize litigation problems from overly restrictive definitions, and
- to selectively augment certain subject matter in the AASHTO *Highway Drainage Guidelines* and *Model Drainage Manual*.

Some terms included in this Glossary will have limited application to highway drainage design. They have been included for several reasons:

- to facilitate communication with other, related sciences; and
- to preclude confusion with similar, but different hydraulics-related terms.

As often happens in any science, some practitioners have different names for the same thing. Every attempt has been made to sort out these colloquialisms and synonyms and assign all the definitions to one term. Cross-referencing of these terms was attempted, but is unlikely to ever be complete or to satisfy all practitioners. It is anticipated that errors and oversights will be resolved with revisions of this Glossary. The reader is encouraged to submit their experience with this Glossary to the AASHTO Task Force on Hydrology and Hydraulics. In particular, the following information is solicited:

- proposed terms and definitions,
- proposed revisions of present terms to include reasons where appropriate, and
- problems (particularly legal) with the present definitions.

In cross-referencing, two terms are used: "See" and "Compare with." The term "see" usually means one of two things: (1) The definition is provided under a different term; (2) The cross-referenced term provides additional information. The term "compare with" infers that the cross-referenced term may be at variance with or an antonym of the defined term; judgmental decisions may be needed in such cases.

This Glossary also attempts to "package" like terms having two or more words. This provides the added advantage of facilitating a comparison of terms without flipping back and forth between pages. As an example, see the section of this Glossary that addresses "Streams," "Gages," "Probability Distribution," or "Weir." In some instances this might prove inconvenient and add some length to the Glossary, but it was felt the advantages outweighed the disadvantages. An exception was made where a particular term might best be left with another, smaller grouping of like terms.

The scientific, and in particular the regulatory, world is inundated with acronyms and abbreviations. This Glossary attempts to define some of the more familiar acronyms and abbreviations likely to be encountered by the highway hydraulics engineer. In a few instances, they have different meanings (See TW, for instance). A few hydraulic variables and equations are also included.

And finally, since this is a Glossary of transportation-related hydraulic terms, an attempt has been made to provide this unique focus without distorting a term's meaning.

## G.2 Glossary

The following terms are provided to facilitate the application of the AASHTO *Highway Drainage Guidelines* and *Model Drainage Manual.*

**AASHTO** Acronym for American Association of State Highway Transportation Officials.

**ABLATION** The process by which ice and snow waste away from melting and evaporation or by which land wears away by the action of surface water.

**ABRASION** Removal of stream bank material due to entrained sediment, ice, or debris rubbing against the bank. Compare with Erosion, Scour, and Mass Wasting.

**ABSORPTION** The assimilation or taking up of water or other solutions by soil or other material; i.e., the entrance of water into the soil or rocks by all natural processes. It includes the infiltration of precipitation or snowmelt, gravity flow of streams into the valley alluvium (see Storage, Bank), sinks, or other large openings, and the movement of atmospheric moisture. The process by which substances in gaseous, liquid, or solid form dissolve or mix with other substances. Compare with Adsorption.

**ABSTRACTION** That portion of rainfall which does not become runoff. It includes interception, infiltration, and storage in depressions. It is affected by land use, land treatment and condition, and antecedent soil moisture.

**ABUTMENT** The superstructure support at either end of a bridge or similar type structure: usually classified as spillthrough or vertical. Considered part of the bridge substructure. See Spillthrough Abutment and Vertical Abutment.

**ACCRETION** Build-up of beach due to wave and wind action.

**ACRE-FOOT** The quantity of water required to cover 1 acre to a depth of 1 foot and equal to 43,560 cubic feet or about 326,000 gallons or 1233 $m^3$. Abbreviated as AF. See Hectare-meters.

**ACT** (ACTS) Written law, such as an Act of Congress.

**ACT OF GOD** In law, a direct, sudden or irresistible action of natural forces that could not reasonably have been foreseen and prevented.

**ACTION (HIGHWAY)** A highway action as it pertains to drainage design is any construction, reconstruction, rehabilitation, repair, or improvement in a watershed or on the transportation system that measurably changes such things as floodplain limits, established flood patterns, and runoff characteristics or that requires a design or review storm system to accommodate storm runoff.

**ADSORPTION** The adhesion in an extremely thin layer of molecules (such as gases, solutions, or liquids) to the surface of solid bodies or liquids with which they are in contact. Compare with Absorption.

**ADVERSE SLOPE** See Slope, Adverse.

**AERATION ZONE** See Zone of Aeration.

**AEROBIC** A condition in which molecular oxygen is a part of the environment.

**AF** Acronym for Acre-Foot. Common abbreviation is ac.-ft.

**AFFLUX** Backwater or height by which water levels are raised at a stated point, owing to presence of a constriction or obstruction, such as a bridge.

**AGGRADATION** General and progressive upbuilding of the longitudinal profile of a channel or within a drainage facility by the deposition of sediment. Compare with Sedimentation. Permanent or continuous aggradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place.

**AHW** Acronym for Allowable Headwater. Compare with HW.

**ALERT** Acronym for Automated Local Evaluation in Real Time. An automated, local flood warning system consisting of automatic self-reporting river and rainfall gages, a communications system based on line-of-sight radio transmission of data, and a base station. The base station consists of radio receiving electronic equipment and a microprocessor. Data analysis software is available to collect, quality control, and display data. A hydrologic model to provide simulation of streamflow is also available.

**ALFALFA** A deep-rooting plant, *Medicago Sativa*, native to Eurasia, having compound leaves with three leaflets and a cluster of small purple flowers. It is widely cultivated for forage and is used as a commercial source of chlorophyll. Its deep-rooting characteristics, commercial value, and environmental (forage and bee habitat) qualities often make it desirable as a ground cover to control erosion.

**ALGAE** Any of various primitive, chiefly aquatic, one-celled or multi-cellular plants that lack true stems, roots and leaves but usually contain chlorophyll.

**ALLOWABLE HEADWATER** See Allowable Headwater Depth.

**ALLOWABLE HEADWATER DEPTH** The depth or elevation of the flow impoundment for a drainage facility above which damage, some other unfavorable result, or a significant flood hazard could occur. Compare with Headwater Depth.

**ALLUVIAL** Referring to deposits of silts, sands, gravels, or similar detrital material which has been transported by running water.

**ALLUVIAL CHANNEL** Channel formed wholly in alluvium with no bedrock exposed in the channel at low flow or likely to be exposed by erosion. A channel whose processes are controlled by the flow and boundary interactions.

**ALLUVIAL FAN** A landform shaped like a fan in plan view and deposited where a stream issues from a narrow valley of high slope onto a plain or broad valley of low slope. Compare with Debris Cone.

**ALLUVIUM** Unconsolidated material such as clay, silt, sand, or gravel deposited by water in a channel, or on a floodplain, alluvial, fan, or delta.

**ALPHA** ($\alpha$) The kinetic-energy, velocity head coefficient. See Velocity Head Coefficient.

**ALTERNATE DEPTH** See Depth, Alternate.

**ALTERNATE BAR** See Bar, Alternate.

**AMC** Acronym for Antecedent Moisture Condition.

**AMPHIBIAN** Any of the various cold-blooded, smooth-skinned vertebrates (with backbone) organisms such as toads, frogs, and salamanders characteristically hatching as an aquatic larvae that breathe by means of gills and metamorphosing to an adult form having air-breathing lungs.

**ANABRANCH** Individual channel of an anabranched stream. A diverging branch of a river which reenters the main stream.

**ANABRANCHED STREAM** See Stream, Anabranched.

**ANAEROBIC** A condition in which molecular oxygen is absent from the environment.

**ANALYSIS, HYDRAULIC** An evaluation of a drainage-related circumstance or condition based on measured or computed findings coupled with prudent judgment. Compare with Assessment, Hydraulic.

**ANALYSIS, ECONOMIC** See Economic Analysis. Compare with Economic Assessment.

**ANASTOMOSING STREAM** See Stream, Anastomosing.

**ANCHOR ICE** Ice in the bed of a stream or upon a submerged body or structure *(57).*

**ANGLE OF FLARE** Angle between direction of wingwall and the centerline of a culvert barrel.

**ANGLE OF REPOSE** The maximum angle, as measured from the horizontal, at which granular particles can stand.

**ANGULARITY** The acute angle between the plane of the highway centerline along the bridge, and a line normal to the thread of the stream; i.e., the acute angle between the thread of the stream and a line normal to the centerline along bridge. Angle of skew if abutments are parallel to the flow line.

**ANNUAL FLOOD** See Flood, Annual.

**ANNUAL (FLOOD) SERIES** A list of annual events such as annual maximum floods and minimum flows. See Flood, Annual.

A general term for a set of any kind of data in which each item is the maximum, minimum, average, or some other consistent value in a year *(34).*

A frequency series in which only the largest value for a particular series of data in each year is used, such as the annual floods*(51).*

**ANNUAL RUNOFF** See Runoff, Annual.

**ANNUAL YIELD** See Runoff, Annual.

**ANTECEDENT MOISTURE** See Antecedent Moisture Condition.

**ANTECEDENT MOISTURE CONDITION** (AMC) The degree of wetness of a watershed's surface soils at the beginning of a storm.

**ANTECEDENT PRECIPITATION** Rainfall that occurred prior to the particular rainstorm under consideration.

**ANTECEDENT PRECIPITATION INDEX** An index of moisture stored within a drainage basin before a storm *(37) (43).*

**ANTIDUNE** A particular type of bed form caused by water flowing over a mobile material such as sand. A sand wave indicated on the water surface by a regular undulating wave. The ridges may move upstream and the surface waves become gradually steeper on the upstream sides until they break like surf and disappear. These surface waves are usually in series and often reform after disappearing. See Bed Form. Compare with Dune.

**APEX** The highest point, the vertex.

**APPROACH CHANNEL** The reach of channel upstream from a dam, bridge constriction, culvert, or other drainage structure. See Approach Section.

**APPROACH SECTION** A cross section of the stream channel, normal to thread of current and for the discharge of interest, located in the approach channel. See Approach Channel.

**APRON** Protective material laid on a stream bed to prevent scour commonly caused by some drainage facility. More specifically, a floor lining of such things as concrete, timber, and riprap, to protect a surface from erosion, such as the pavement below chutes, spillways, at the toes of dams, or at the outlets of culverts. Material placed on the banks is commonly termed a blanket. Compare with Channel Lining and Blanket.

**APRON, LAUNCHING** A flexible apron designed to settle and protect the side slopes of a scour hole or eroded channel bed both during and after settlement.

**AQUATIC LIFE** Wildlife living or growing on, in, or adjacent to water.

**AQUEDUCT** See Flume.

**AQUEOUS FORM** Dissolved in the water.

**AQUIFER** A porous, water-bearing geologic formation. Generally restricted to materials capable of yielding an appreciable supply of water.

**AREA** The size, in square feet or in square meters, of a flow section, channel, or waterway opening. Also, the size in square miles or kilometers, acres, or hectares, of drainage area or basin, catchment area, or drainage area. See Drainage Area.

**AREA CURVE** When plotted against some other variable such as discharge or velocity, a graph of: 1. the cross-sectional area of a stream at a gaging station or other section; 2. the surface area of a reservoir plotted against water-surface elevations; 3. the flow area of a drainage structure.

**AREA, HYDROLOGIC** See Hydrologic Area.

**AREA RAINFALL** The average rainfall over an area, usually as derived from or discussed in contrast with point rainfall *(51)*.

**AREA-CAPACITY CURVE** A graph showing the relation between the surface area of the water in a reservoir and the corresponding volume.

**ARID** Geographic areas that are dry, lacking moisture. Compare with Desert and Semi-arid.

**ARMOR** Artificial surfacing of channel beds, banks, or embankment slopes to resist stream bed scour and/or lateral bank erosion. Compare with Apron, Blanket, Channel Lining, and Revetment.

**ARMORING** A natural process whereby an erosion-resistant layer of relatively large particles is formed on a channel bank and/or channel bed due to the removal of finer particles by streamflow; i.e., the concentration of a layer of stones on the bed of the stream which are of a size larger than the transport capability of the recently experienced flow—the winnowing out of smaller material capable of being transported while leaving the larger sizes as armor that, for discharges up to that point in time, cannot be transported. Armoring may also refer to the placement of a covering on a channel bank and/or channel bed to prevent erosion.

**ARRAY** A list of data in order of magnitude; in flood-frequency analysis it is customary to list the largest value first, and in a low-flow frequency analysis the smallest first *(34).*

**ARROYO** Term of regional geographic usage. See Channel.

**ARTESIAN** Pertains to groundwater that is under pressure and will rise to a higher elevation if unconstrained.

**ARTICULATED CONCRETE MATTRESS** Rigid concrete slabs which can move without separating as scour occurs, usually hinged together with corrosion-resistant fasteners; primarily used for a lower bank protection armor blanket.

**ASPHALT, END DUMPED** Mass of uncompacted, asphalt chunks of various sizes that are usually dumped from a truck for upper bank protection or from a barge for lower bank protection. The purpose is to protect the bank against erosion. Note, there may be environmental prohibitions against using asphalt for such purposes.

**ASSESSMENT, HYDRAULIC** Primarily a prudent, judgmental evaluation of a drainage-related circumstance or condition using observations and experience but without a large amount of measured or computed information. Compare with Analysis, Hydraulic.

**ASSESSMENT, ECONOMIC** See Economic Analysis or Economic Assessment.

**AUGMENTATION, FLOW** The addition of water to a stream, especially to meet water quality or fish and wildlife needs.

**AUGMENTED FLOW** The increased volume of water entering a channel, or allowed to run overland as waste waters from the diversion of surface flow or as water from another stream or watershed; or from waters withdrawn or collected upstream and released after use.

**AVERAGE DISCHARGE** See Discharge, Average.

**AVERAGE VELOCITY** See Velocity, Average.

**AVULSION** A sudden change in channel course that occurs when a stream suddenly breaks through its banks; usually associated with a large flood or a catastrophic flood or event.

**BACKFILL** The material used to refill a ditch or other excavation, material placed adjacent to or around a drainage structure, or the process of doing so.

**BACKWATER** The increase in water surface elevation induced upstream from such things as a bridge, culvert, dike, dam, another stream at a higher stage, or other similar structures or conditions that obstruct or constrict a channel relative to the elevation occurring under natural channel and floodplain conditions. Stated another way, water backed up or retarded in its course as compared with its normal or natural conditions of flow. Also applies to the water surface profile in a channel or conduit. See Backwater Curve.

The difference between the observed stage and that indicated by the stage-discharge relation, is reported as backwater *(37)*.

Four other similar definitions and examples are offered: 1. in a general sense, a flow retarding influence due to a dam, other constrictions such as a bridge or culvert, or another stream; 2. the increase in water-surface elevation due to a bridge constriction above the normal unconstricted elevation at an approach section located one bridge length upstream from the bridge constriction; 3. water backed up or retarded in its course as compared with its normal or natural condition of flow; 4. in stream gaging, a rise in stage produced by a temporary obstruction such as ice or weeds, or by the flooding of the stream below.

**BACKWATER AREA** The low-lying lands adjacent to a stream that may become flooded due to backwater effects. Compare with Floodplain.

**BACKWATER CURVE** A particular form (or profile) of the surface curve (water surface) of a stream of water which is concave upward. It is caused by an obstruction in the channel such as those that cause backwater. The depth is greater at all points under the curve than the critical and the normal depth, and the velocities diminish downstream. The term is sometimes used in a generic sense to denote all water surface curves or profiles. Compare with Backwater and Water Surface Profile.

**BACKWATER FLOODING** The backup of water into a stream from a river, lake, or ocean having a higher water elevation *(33)*.

**BACKWATER RATIO** The ratio ($C_b$) of the amount of backwater ($h_l$*) created by a bridge constriction to the total drop ($\Delta h$) through the constriction.

**BACKWATER REACH** The reach of channel influenced by the backwater curve.

**BAFFLE** A structure constructed on the bed of a stream or drainage facility to deflect or disturb the flow. Vanes or guides, a grid, grating, or similar device placed in a conduit to check eddy currents below them, and effect a more uniform distribution of velocities. Also a device used in a culvert or similar structure to facilitate fish passage.

**BAFFLE-PIERS** Obstructions set in the path of high-velocity water, such as piers on the apron of a stilling basin, to dissipate energy and prevent bank erosion and stream bed scour.

**BANK** The side slopes or margins of a channel between which the stream or river is normally confined. More formally, the lateral boundaries of a channel or stream, as indicated by a scarp, or on the inside of bends, by the streamward edge of permanent vegetal growth.

The margins of a channel. Banks are called right or left as viewed facing in the direction of the flow [commonly downstream] *(37).*

See Bank, Upper; Bank, Lower; Bank of Channel Left; and Bank of Channel Right.

**BANK CAVING** See Mass Wasting.

**BANK, GUIDE** See Guide Bank.

**BANK, LEFT** See Left Bank of a Channel.

**BANK, LOWER** That portion of a channel bank having an elevation less than the mean water level of the stream or river. Compare with Bank; and Bank, Upper.

**BANK, MIDDLE** That portion of a channel bank having an elevation approximately the same as that of the mean water level of the stream or river.

**BANK, RIGHT** See Right Bank of a Channel.

**BANK, STREAM** See Bank, Right Bank of a Channel, and Left Bank of a Channel.

**BANK STORAGE** See Storage, Bank.

**BANK, UPPER** The portion of a channel bank having an elevation greater than the average (mean annual or dominant flow) water level of the stream. Compare with Bank; and Bank, Lower.

**BANKFULL DISCHARGE** Discharge that, on the average, fills a channel to the point of overflowing. Commonly considered as the mean annual discharge ($Q_{2.33}$) or two- to three-year discharge ($Q_2$, $Q_3$) in a channel that has been relatively stable for a number of years without the occurrence of a large, bank-destroying flood. Sometimes used as the dominant discharge. See Discharge, Dominant.

**BANKFULL STAGE** Stage at which a stream first overflows its natural banks. Compare with Flood Stage. Bankfull stage is a hydraulic term, whereas flood stage implies damage *(37).*

The water elevation that reaches the top of the banks along a stream *(33).* Compare with Bankfull Discharge.

**BAR** An elongated deposit of alluvium, not permanently vegetated, within or along the side of a channel. See Bar, Point; Bar, Middle; and Bar, Alternate.

**BAR, ALTERNATE** An alluvial deposit of sand and gravel lacking permanent vegetal cover occurring in an alternating pattern from bank to bank in a relatively straight channel reach.

**BAR, MIDDLE** A bar lacking permanent vegetal cover that divides the flow in a channel at normal stage.

**BAR, POINT** An alluvial deposit of sand or gravel lacking permanent vegetal cover occurring in a channel at the inside of a meander loop bendway and usually somewhat downstream from the apex of the loop. The longer, tapered portion, when present, commonly points upstream.

**BARRAGE** See Check Dam.

**BARREL WIDTH** Commonly the inside, horizontal extent of a drainage facility.

**BASE** The layers of specified material placed on the sub-base or subgrade to support the pavement, surface course, or a drainage facility.

**BASE DISCHARGE** See Base Flow.

**BASE FLOOD (or STORM)** See Flood, Base; and Flood (or Storm), Review.

**BASE FLOODPLAIN** Surface area flooded by the base flood.

**BASE FLOW** In the U.S. Geological Survey's annual reports on surface-water supply, the discharge above which peak discharge data are published. The base discharge at each station is selected so that an average of about three peaks a year will be presented *(37).* See Runoff, Base.

**BASE FLOW RECESSION** The declining rate of discharge of a stream fed only by base flow for an extended period. Typically, a base flow recession will be exponential *(20).* Compare with Depletion Curve and Recession Curve.

**BASE FLOW RECESSION HYDROGRAPH** A hydrograph that shows a base flow recession curve.

**BASE RUNOFF** See Runoff, Base.

**BASE STATION** The central location for receiving flood data from sensors in the ALERT [or IFLOWS] system. Gaging station providing basic hydrologic data *(33).*

**BASELINE MONITORING** The establishment and operation of a designed surveillance system for continuous or periodic measurements and recording of existing and changing conditions that will be compared with future observations.

**BASIC HYDROLOGIC DATA** Includes inventories of land and water features that vary only from place to place (topographic and geologic maps are examples) and records of processes that vary with both place and time (records of precipitation, streamflow, groundwater, and

quality-of-water analyses are examples). Basic hydrologic information is a broader term that includes surveys of the water resources of particular areas and a study of their physical and related economic processes, interrelations, and mechanisms.

**BASIC-STAGE FLOOD SERIES** See Partial-Duration Flood Series.

**BASIN, DEBRIS** A basin constructed in a channel to store debris, such as sand, gravel, silt, and driftwood. See Debris, Debris Barrier, and Debris Dam. Compare with Basin, Sedimentation; Basin, Recharge; Basin, Detention; and Basin, Retention.

**BASIN DRAINAGE** See Drainage Area.

**BASIN, DETENTION** A basin or reservoir incorporated into the watershed whereby runoff is temporarily stored, thus attenuating the peak of the runoff hydrograph. A stormwater management facility that impounds runoff and temporarily impounds runoff and discharges it through a hydraulic outlet structure to a downstream conveyance structure. Compare with Basin, Retention; Basin, Recharge; Basin, Debris; and Basin, Sedimentation.

**BASIN LAG** The amount of time from the centroid of the rainfall hyetograph to the hydrograph peak.

**BASIN, RECHARGE** A basin or pit excavated to provide a means of allowing water to soak into the ground at rates exceeding those that would occur naturally *(20).* A basin excavated in the earth to receive the discharge from streams or storm drains for the purpose of replenishing groundwater supply and/or attenuating flood discharge.

A basin or reservoir wherein water is stored for recharging the groundwater. It does not have an uncontrolled outlet. The stored water is disposed by such means as infiltration or injection (dry wells) designed for such purposes. Compare with Floodwater Retarding Structure; Retarding Reservoir; Basin, Detention; Basin, Sedimentation; Basin, Debris; and Basin Retention. See Recharge and Recharge Area.

**BASIN, RETENTION** A basin or reservoir wherein water is stored for regulating a flood. It does not have an uncontrolled outlet. The stored water is disposed by such means as infiltration, injection (or dry) wells, or by release to the downstream drainage system after the storm event. The release may be through a gate-controlled gravity system or by pumping. Compare with Floodwater Retarding Structure; Retarding Reservoir; Basin, Detention; Basin, Recharge; Basin, Sedimentation; and Basin, Debris.

**BASIN, SEDIMENTATION** A basin or tank in which floodwater or stormwater containing settleable solids is retained to remove by gravity or filtration a part of the suspended matter. Compare with Basin, Recharge; Basin, Detention; Basin, Retention; and Basin, Debris.

**BASIN, STILLING** See Stilling Basin.

**BEARD DISTRIBUTION** See Probability Distribution.

**BED** The bottom of a channel. The part of a channel not permanently vegetated which is bounded by banks and over which water normally flows. Compare with Substrate.

**BED FORM** A recognizable and commonly transient relief feature on the bed of a channel, such as a ripple, dune, or bar. A relief feature caused by water flowing over a mobile material such as sand and gravel.

**BED, INDURATED** A channel bed that has been made hard or has hardened. See Control.

**BED LAYER** A flow layer, several grain diameters thick (usually two) immediately above the bed.

**BED LOAD** Sediment that is transported in a stream by rolling, sliding, or skipping (saltating) along the bed or very close to it; considered to be within the bed layer. The quantity of silt, sand, gravel, or other detritus rolled along the bed of a stream, often expressed as weight or volume per time. Compare with Contact Load and Wash Load.

**BED LOAD DISCHARGE** The quantity of bed load passing a cross section of a stream in a unit of time. Compare with Sediment Discharge and Wash Load.

**BED MATERIAL** The sediment mixture of which a stream bed, lake, pond, reservoir, or estuary bottom is composed *(52).* Sediment consisting of particle sizes large enough to be found in appreciable quantities at the surface of a channel bed. Material found on the bed of a channel (may be transported as bed load discharge or sediment discharge). See Bed Load Discharge.

**BED SEDIMENT DISCHARGE** The part of the total sediment discharge that is composed of grain sizes found in the bed and is equal to the transport capability of the flow occurring at the time. Compare with Bed Load, Bed Load Discharge, Sediment Discharge, and Bed Load Discharge.

**BED SHEAR** See Tractive Force.

**BED SLOPE** The longitudinal inclination of a channel bottom. Compare with Superelevation.

**BEDROCK** The scour-resistant material underlying erodible soils and overlying the mantle rock, ranging from surface exposure to depths of several hundred miles or kilometers.

**BENCH-FLUME** A conduit on a topographical bench, cut into sloping ground. Compare with Flume.

**BENDWAY** A point of distinct curvature in a stream, river, or channel where the bankfull flow changes direction. Reach of sharpest curvature in a sinuous channel.

**BENDWAY SCOUR** See Scour, Bendway.

**BENEFICIAL USE** Relates to the beneficial use of a water and flood right. See Water Right and Flood Right. Beneficial use is such things as municipal use, agricultural use, recreation, and environmental purposes (instream flows).

**BERM** A narrow shelf or ledge; also a form of dike. A more detailed description might be: 1. the space left between the upper edge of a cut and the toe of an embankment; 2. a horizontal strip or shelf built into an embankment to break the continuity of an otherwise long slope.

Also may be the top surface or plane of a shoulder, ledge, or bank constructed in connection with the roadway embankment at bridge abutments, waste along fill slopes, and canal or ditch banks. Compare with Dike.

**BERNOULLI'S THEOREM** A proposition advanced by Daniel Bernoulli that the energy head at any section in a flowing stream is equal to the energy head at any other downstream section plus the intervening losses.

**BEST MANAGEMENT PRACTICES (BMPs)** Erosion and pollution control practices employed during construction to avoid or mitigate damage or potential damage from the contamination or pollution of surface waters or wetlands from a highway action.

**BETA (β)** The momentum velocity head coefficient. See Momentum Coefficient.

**BIOMASS** The amount of living matter (as in a unit area or volume of habitat) according to *Webster's New Collegiate Dictionary.*

**BIOSTIMULATION** To rouse any living organisms to activity.

**BIOTIC** Pertaining to life or specific life conditions.

**BIOTIC REGION** A geographic area having similar biotic conditions. See Mitigation Alternatives.

**BITUMINOUS MATTRESS** An impermeable rock-, mesh-, or metal-reinforced layer of asphalt or other bituminous material placed on a channel bank to prevent erosion. Note that there may be environmental prohibitions for such material.

**BLANKET** Material covering all or a portion of a channel bank to prevent erosion. Stream bank surface covering, usually impermeable, designed to serve as protection against erosion. Common pavements used on channel banks are concrete, compacted asphalt, and soil-cement. Compare with Apron, Apron Launching, Armor, and Channel Lining.

**BLANKET, CLAY** See Clay Blanket.

**BLANKET, FILTER** See Filter Blanket.

**BLM** Acronym for the Bureau of Land Management.

**BLOM DISTRIBUTION** See Probability Distribution.

**BMP (BMPs)** Acronym for Best Management Practices.

**BOG** A shrug peatland dominated by ericaceous shrubs, sedges and peat moss and usually having a saturated water regime or a forested peatland dominated by evergreen trees (usually spruces and firs) and/or larch. See Wetlands.

**BORE, HYDRAULIC** A wave of water having a nearly vertical front, such as a tidal wave, advancing upstream as a result of high tides in certain estuaries; a similar wave advancing downstream as the result of a "cloudburst," or the sudden release of a large volume of water from a reservoir, as in the Johnstown (PA) flood. The bore is analogous to the hydraulic jump in that it represents the limiting condition of the surface curve wherein it tends to become perpendicular to the bed of the stream.

**BOREAL REGION** The geographical area just below the arctic tundra and usually characterized by the evergreen forests.

**BOTTOM CONTRACTION** A channel contraction resulting from some protrusion across the bottom of a channel.

**BOTTOM MATERIAL** See Bed Material.

**BOULDER** A rounded or angular fragment of rock, the diameter of which is in the size range of 10 inches to 13 feet or 250 mm to 4000 mm according to the FHWA's *Highways in the River Environment Manual.*

**BRAID** A subordinate channel of a braided stream. See Stream, Braided. Compare with Anabranch.

**BRAIDED STREAM** See Stream, Braided.

**BRAIDING OF RIVER CHANNELS** See Stream, Anastomosing.

**BRANCH** Term of local geographic usage. See Channel.

**BREAKERS** The surface discontinuities of waves as they breakup. They may take different shapes (spilling, plunging, surging). Zone of break-up is called the surf zone.

**BRIDGE** A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a tract or passageway for carrying traffic or moving loads, and, for definition purposes (AASHTO), having an opening measured along the center of the roadway equal to or more than 6.1 m (20 feet) between undercopings of abutments or spring lines of arches, or extreme outside ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening. Also a structure that, while designed hydraulically using the principles of open channel flow to operate with a free water surface, may be inundated under flood conditions. The structure generally consists of a deck or superstructure supported on two or more abutments and often includes intermediate piers. Compare with Culvert.

**BRIDGE OPENING** The total cross section area beneath a bridge superstructure that is available for the conveyance of water. Compare with Bridge Waterway.

**BRIDGE, RELIEF** An opening through an embankment located on a floodplain for the purpose of permitting passage of overbank flow.

**BRIDGE WATERWAY** The area of a bridge opening available for flow as measured below a specified stage and normal to the principal direction of flow. Compare with Bridge Opening.

**BROAD-CRESTED WEIR** See Weir, Broad-Crested.

**BROKEN RECORD** A systematic record [of events such as annual floods] which is divided into separate, continuous segments because of a deliberate, discontinuation of recording for significant periods of time *(33).*

**BROKEN-BACK CULVERT** See Culvert, Broken-Back.

**BUBBLE GAGE** See Gage, Bubble.

**BUFFER ZONE** Areas of trees, grass or other vegetation located between the top of a channel bank and the adjacent highway; also called greenbelts.

**BULKHEAD** A steep or vertical wall that supports a natural or artificial embankment and may also serve as a protective measure against bank erosion.

**BYPASS FLOW** See Flow, Bypass.

**CAISSON** A chamber, usually sunk by excavating from within, for the purpose of gaining access to the subsurface geology selected to support a structure's substructure (the foundation). If the chamber is closed on top and the water excluded by air pressure, it is called a pneumatic caisson. Oftentimes in highway construction a caisson is filled with concrete or masonry to serve as the pier or abutment substructure. Compare with Cofferdam.

**CALIBRATION** The process of fitting a [computational] model to a set of observed data by changing unknown or uncertain model parameters systematically within their allowable ranges until a "best fit" of the model to the observed data is achieved *(1).*

**CALIFORNIA DISTRIBUTION** See Probability Distribution.

**CANAL** A constructed open conduit or channel for the conveyance of irrigation water which is distinguished from a ditch or lateral by its larger size. It is usually excavated in natural ground, although lined canals on berms are not uncommon. Compare with Channel and Swale.

**CAPACITY** A measure of the ability of a channel or conduit to convey water. Compare with Conveyance.

**CAPACITY CURVE** A graph of the volume of such things as a reservoir, tank, or detention pond as a function of elevation. The capacity of a reservoir is defined by reference to an elevation.

**CARRYOVER** See Flow, Bypass.

**CATASTROPHIC FLOOD (OR STORM)** See Flood (or Storm), Catastrophic.

**CATCH BASIN** A structure, sometimes with a sump, for inletting drainage from such places as a gutter or median and discharging the water through a conduit. In common usage it is a grated inlet, curb opening, or combination inlet with or without a sump. Note that sumps in catch basins may cause environmental hazards by further polluting "first flush" runoff and subsequent runoff passing through the catch basin.

**CATCHMENT** See Drainage Area.

**CATCHMENT AREA** See Drainage Area.

**CATCHMENT BASIN** See Drainage Area.

**CATION EXCHANGE CAPACITY (CEC)** The sum total of exchangeable cations that a soil can absorb; expressed in milliequivalents per gram, or per 100 g of soil.

**CAUCHY DISTRIBUTION** See Probability Distribution.

**CAUSEWAY** Rock or earth embankment carrying a roadway across water.

**CAVING BANK** See Mass Wasting.

**CAVITATION** A phenomenon associated with the vaporization of a flowing liquid in a zone of low pressure, wherein cavities filled with liquid vapor (vapor bubbles) alternately develop and collapse. Significant cavitation is unusual in highway drainage structures but, if present, may cause structural deterioration. Put another way, a condition wherein a vacuum, to any degree, exists as a result of flowing water. Complete cavitation occurs when the pressure within the affected part is reduced to that of the vapor pressure of the water.

**CEC** Acronym for Cation-Exchange Capacity.

**CELERITY, WAVE** The velocity of a wave measured relative to the liquid, known as celerity C, is equal to the wave length divided by the period [of the wave], or C = L/t *(13).*

**CELLULAR-BLOCK MATTRESS** Regularly cavitated, interconnected concrete blocks placed directly on a bank (or shore) to prevent erosion. The cavities can permit bank (or shore) drainage and the growth of either volunteer or planted vegetation when synthetic filter fabric is not used between the mattress and bank (or shore). See Apron and Blanket.

**CFR** Acronym for Code of Federal Regulations.

**CFS** (cfs) Acronym for cubic feet per second *(37).* See Cubic Feet per Second.

**CFS-DAY** (cfs-day) Often [erroneously] called a "second-foot-day." The volume of water represented by a flow of 1 cubic foot/s/day *(50).*

**CHAIN OR TAPE GAGE** See Gage, Chain or Tape.

**CHANNEL** The term "channel" has been defined numerous ways: 1. the bed and banks that confine the flow of surface water in a natural stream or artificial channel; also see River and Stream; 2. the course where a stream of water runs or the closed course or conduit through which water runs, such as a pipe; 3. An open conduit either naturally or artificially created which periodically or continuously contains moving water or which forms a connecting link between two bodies of water. River, creek, run, branch, anabranch, [arroyo, draw, wash] and tributary are some of the terms used to describe natural channels. Natural channels may be single or braided (see Braiding of River Channels). Canal [, ditch, lateral] and floodway are some of the terms used to describe artificial channels *(37).*

Channel has also been defined as an elongated, open depression in which water may, or does, flow. An elongated depression, either naturally or artificially created and of appreciable size, which periodically or continuously contains moving water or which forms a connecting link between two bodies of water. It must have a definite bed and bank, which serve to confine the water up to some bankfull discharge amount.

Local convention may use river, stream, arroyo, or branch. With constructed canals, the term ditch or lateral may be used. See Field Ditch.

Compare with Swale, Canal, and Waterway.

**CHANNEL, ALLUVIAL** See Alluvial Channel.

**CHANNEL, APPROACH** See Approach Channel.

**CHANNEL, BED** See Bed.

**CHANNEL COEFFICIENT** A roughness factor in the Kutter, Manning, Bazin, and other formulas expressing the character of a channel as it affects the friction slope of water flowing therein. More specifically, for highway drainage design in the United States, the roughness factor, n, in Manning's equation. See Manning's Equation.

**CHANNEL, COLLATERAL** Channels that are side-by-side; accompanying; from the same source; or similar but of subordinate nature.

**CHANNEL CONTRACTION** The degree of contraction imposed by a bridge type constriction on the river channel for a given discharge. It is measured by the channel-contraction ratio (M), which is defined as the ratio of that portion of flow directed at the contracted opening divided by the total flow that must pass through the contracted opening. Sometimes the inverse of this

ratio is used to define the channel contraction, so caution is warranted. Compare with Constriction, Contracted Section, Contraction, and Coefficient of Contraction.

**CHANNEL DIVERSION** The taking of water from a stream or other body of water into a canal, pipe, or other conduit *(37).* The removal of all or a portion of the flow from a natural or artificial (canal, ditch, field ditch, or lateral) channel.

**CHANNEL LINING** The material applied to the bottom and/or sides of a natural or constructed channel. Material may be such things as concrete, sod, grass, rock, or any of several other types of commercial linings. Covering of stones on a channel bed or bank (used in the AASHTO *Model Drainage Manual* with reference to natural covering). Compare with Apron, Blanket, and Armor.

**CHANNEL, LOW FLOW** Lower portion of natural or artificial watercourse often of perceptible extent with a definite bed and banks which confines and conducts continuously or periodically flowing water. A low flow channel is considered as that portion of a channel commonly lying below the plane of the ordinary highwater (OHW). A low flow channel may be adjoined by a floodplain. The bankfull capacity is often associated with the dominate discharge or mean annual discharge.

**CHANNEL, INCISED** See Incised Channel; Incised Reach; and Stream, Incised.

**CHANNEL, MEANDERING** See Meandering Channel.

**CHANNEL MIGRATION** Change in position of a channel by lateral erosion of one bank and the simultaneous accretion of the opposite bank. Systematic channel shifting in the direction of flow.

**CHANNEL, NATURAL** A surface or underground watercourse created by natural agents and conditions. The principal stream channel or channels and, if the stream is braided, its natural and customary overflow channels.

**CHANNEL, NON-UNIFORM** A channel where the flow streamlines are not straight and parallel. The velocity vector varies significantly with distance along a flow streamline at a given instance. With non-uniform channels, resistance and gravity forces are not in balance, the water surface is not parallel to the channel bottom or friction slope, $S_f$. Also, the rate of loss of total head will not equal the average channel bottom slope, $S_0$. Compare with Channel, Uniform.

**CHANNEL, OPEN** A channel having a water surface exposed at all points to atmospheric pressure. Any conveyance in which water flows with a free surface. Compare with Open Channel Flow.

**CHANNEL PATTERN** The aspect of a stream channel in plan view, with particular reference to such things as the degree of sinuosity, braiding, or anabranching.

**CHANNEL PROCESS** Behavior of a channel with respect to channel migration, erosion, and sedimentation.

**CHANNEL ROUTING** The process whereby a peak flow and/or its associated streamflow hydrograph is mathematically transposed to another site downstream taking into account the effect of channel storage.

**CHANNEL, REGIME** See Regime Channel.

**CHANNEL SLOPE** Fall per unit length along the channel centerline.

**CHANNEL, STABLE** A condition that exists when a channel has a bed slope and cross section which allows it to transport the water and sediment delivered from the upstream watershed without significant aggradation, deposition or bank erosion. See Regime, Regime Channel, and Regime of a Stream.

**CHANNEL STORAGE** See Storage, Channel.

**CHANNEL, TORTUOUS** A winding channel which is not free to shift its alignment. Compare with Flow, Turbulent.

**CHANNEL, UNIFORM** Channel with a uniform cross section and constant roughness. A constant slope is also a requirement for uniform flow and depth. See Depth, Normal.

[For highway drainage design,] a channel where the flow streamlines are [essentially] straight and parallel. Lack of [significant] variation in the velocity vector with distance along a streamline at a given instant. If a channel is uniform and resistance and gravity forces are in near balance, the water surface will be parallel to the bottom of the channel and to the friction slope, $S_f$, and the rate of loss of total head will equal the slope, $S_o$, of the channel bottom. This is the condition of uniform flow. The depth from the water surface to the bottom grade line of the channel, which because of the parallelism of the water surface and grade line is the same at all sections, is known as normal depth, $Y_o$ [$Y_n$]. The normal depth is a function of the shape and roughness of the channel, its slope, and the rate of discharge. For a given discharge, channel shape, and slope, the normal depth will vary according to the total effect of boundary resistance *(56).* Compare with Channel, Non-Uniform.

**CHANNEL, WANDERING** A channel exhibiting a more or less non-systematic process of channel shifting, erosion, and deposition, with no definite meanders or braided pattern.

**CHANNELIZATION** Straightening and/or deepening of a channel by such things as artificial cutoffs, grading, flow-control measures, river training, or diversion of flow into an artificial channel. See River Training Structures.

**CHECK DAM** A relatively low dam or weir across a channel for the diversion of irrigation flows from a small channel, canal, ditch, or lateral. A check dam can also be a low structure, dam or weir, across a channel for such things as the control of water stage or velocity or the control

of channel bank erosion and channel bed scour from such things as headcutting. Sometimes termed a barrage in Britain and former British Commonwealth countries.

**CHECK FLOOD** See Flood (or Storm), Review.

**CHECK FLOOD FOR BRIDGE SCOUR** See Flood (or Storm), Review.

**CHEMICAL GAGING** A process of measuring the flow of water by ascertaining the resulting degree of dilution of a chemical solution of known saturation introduced at a known rate into the stream.

**CHEMICAL STABILIZATION** Channel bank protection technique involving the application of chemical substances to increase particle cohesiveness and to shift the size distribution toward the coarser fraction. The net effect is to improve the erosion resistance of the material.

**CHEGODOYEV DISTRIBUTION** See Probability Distribution.

**CHEMICAL-HYDROMETRY** See Chemical Gaging.

**CHEMOCLINE** A salinity or other chemical gradient within surface waters. Chemoclines are usually thought of for lakes, but might include some ponds. See Lake and Pond.

**CHEZY FORMULA** An empirical formula for uniform flow expressing the relation between velocity of water, hydraulic radius, and friction slope; thus, $V = C(RS)^{1/4}$ where V = velocity; R = hydraulic radius; $S_n$ = energy or friction slope; and C = a coefficient.

**CHOKING (OF FLOW)** Severe backwater effect resulting from excessive constriction.

**CHUTE** An open or closed channel used to convey water, usually situated on the ground surface. Compare with Cutoff. Two other definitions are: 1. a high-velocity conduit for conveying water to a lower level; 2. an inclined drop or fall. See Drop.

**CHUTE CUTOFF** See Cutoff.

**CIPOLLETTI WEIR** See Weir, Cipolletti.

**CIVIL ACTION** Action presenting an issue to be resolved under civil law, as distinguished from criminal law, and/or brought to establish or recover private and civil rights or redress for damage; tort action.

**CIVIL LAW** The system of jurisprudence established by a nation, state, or commonwealth peculiarly for itself; the division of law regulating ordinary private matters, as distinct from laws regulating criminal, political or military matters. The civil laws regarding the management of naturally occurring waters established the rights or easements, both favorable and restrictive, of the riparian owners individually and with respect to others, and are directed toward equitable use and the preservation and continuation of natural drainage conditions. Compare with Common Law.

**CIVIL LAW DOCTRINE OR RULE** A rule of law pertaining to the disposal of drainage waters, under which the owner of higher land has the right or easement to dispose of the surplus or excess waters from his lands to lower lands, unobstructed by the owners thereof. Compare with Common Enemy Doctrine or Rule, Natural Drainage Doctrine or Rule, and Reasonable Use Doctrine and Rule.

**CLAY** Material passing the No. 200 (0.074 mm) U.S. Standard Sieve that exhibits plasticity (putty-like properties) within a range of water contents and has considerable strength when air-dry (Unified Soil Classification System) *(21,* FHWA *Highways in the River Environment Manual).*

**CLAY BLANKET** Layer of compacted clay placed over cohesionless bank soils to protect them against erosive streamflow.

**CLAY PLUGS** See Meander Plugs.

**CLEARANCE** An unobstructed horizontal or vertical space. Compare with Freeboard.

**CLIMATE** The sum total of the meteorological elements that characterize the average and extreme condition of the atmosphere over a long period of time at any one place or region of the earth's surface. The collective state of the atmosphere at a given place or over a given area within a specified period of time *(37).*

**CLIMATIC YEAR** A continuous 12-month period during which a complete annual cycle occurs, arbitrarily selected for the presentation of data relative to hydrologic or meteorologic phenomena. The climatic year is usually designated by the calendar year during which most of the 12 months occur *(37).* See also Water Year.

A year selected for the presentation of data on water supply, precipitation, etc.; the climatic year of the USGS extends from October 1 to September 30 following.

**CLOUDBURST** A torrential downpour of rain, which by its spottiness and relatively high intensity suggests the bursting and discharge of a whole cloud at once *(73).*

**COAST LINE** The line or interface forming the boundary between the land and water. See Shore.

**COASTAL ZONE** The strip of land that extends inland from a coast (or shore) line to the first major change in terrain features.

**COBBLE** A fragment of rock the diameter of which is in the size range of 2.5 inches to 10 inches or 64 to 250 mm *(21,* FHWA *Highways in the River Environment Manual).*

**CODE OF FEDERAL REGULATIONS** (CFR) Codifies and publishes at least annually Federal regulations currently in force. The CFR is kept up to date by individual issues of the *Federal Register.* The two publications must be used together to determine the latest version of any given rule. See *Federal Register.*

**COE** Acronym for the U.S. Army Corps of Engineers.

**COEFFICIENT OF CONTRACTION** The ratio of the smallest cross section area of the flow after passing the constriction to the nominal cross section area of the constriction. Compare with Channel Contraction.

**COEFFICIENT OF DISCHARGE** Ratio of observed to theoretical discharge. Also the coefficient used for orifice or other flow processes to estimate the discharge past a point or through a reach.

**COEFFICIENT OF ROUGHNESS** See Channel Coefficient.

**COEFFICIENT OF SKEW (SKEWNESS)** See Skew Coefficient.

**COFFERDAM** A barrier built in the water so as to form an enclosure from which the water is pumped to permit free access to the area within. Compare with Caisson.

**COLLATERAL CHANNEL** See Channel, Collateral.

**COMBINATION INLET** See Inlet, Combination.

**COMBINED SEWER** See Sewer, Combined.

**COMMON ENEMY DOCTRINE OR RULE** A common law rule recognized by some states, pertaining to the disposal of surplus or excess surface waters, which holds that such waters are a "common enemy," and, therefore, the land owner has the right to protect his lands from such waters coming from higher lands. Under this rule, surface waters are regarded as a common enemy which each landowner may fight as he deems best and without regard to the harm that may be caused to others. Compare with Civil Law Doctrine or Rule, Natural Drainage Doctrine or Rule, and Reasonable Use Doctrine and Rule.

**COMMON LAW** As distinguished from "Roman" or "Civil" law, the body of unwritten law, especially of England, based on long-standing usages and customs and the court decisions and decrees recognizing, affirming and enforcing such usages and customs. Compare with Civil Law.

**COMMON LAW, FEDERAL** A body of decisional law developed by the Federal courts, unencumbered by state court decisions.

**COMMON LAW RULES** Principles or maxims established under the common law doctrine or rule.

**COMMUNITY SPECIES LEVEL** The living part of the ecosystem having an ecological structure which exists in dynamic equilibrium with its environment.

**COMPOSITE HYDROGRAPH** See Hydrograph, Composite.

**CONCENTRATION TIME** See Time of Concentration.

**CONCORDANT FLOWS** See Flow, Concordant.

**CONCRETE PAVING** Plain or reinforced concrete slabs poured or placed on the surface to be protected.

**CONDUIT** An artificial or natural channel; usually a closed structure such as a pipe or culvert. A general term for any channel intended for the conveyance of water, whether open or closed; any container for flowing water. With highways, conduits are often considered as being a pipe, culvert, flume, channel, chute, or similar drainage facility.

**CONFIDENCE LIMITS** Computed [statistical] values on both sides of an estimate of a parameter that show for a specified probability the range in which the true value of the parameter lies.

**CONFLUENCE** The junction of two or more streams.

**CONJUGATE DEPTH** See Depth, Conjugate.

**CONSERVATION STORAGE** See Storage, Conservation.

**CONSTRICTION** A compressed or constricted section or reach of a channel may be a natural condition or one produced by raising the bottom (as a sill or dam), or contracting the width (as a highway embankment on a floodplain), or both. A control section, such as a bridge crossing, channel reach, sill or dam, with limited flow capacity in which the discharge is related to the upstream water surface elevation; a constriction may be either natural or artificial. See Contraction, Stream Contraction, and Stream Constriction. Compare with Channel Contraction, Contracted Section, and Contraction.

**CONSTRICTION GEOMETRY** A measure of the size, shape, and character of such things as the embankments and abutments of a bridge type constriction, expressed as dimensionless length ratios.

**CONSUMPTIVE USE** A term used mainly by irrigation engineers to mean the amount of water used in crop growth plus evaporation from the soil *(51).* See Evapotranspiration.

The quantity of water absorbed by the crop and transpired or used directly in the building of plant tissue together with that evaporated from the cropped area: U.S. Bureau of Reclamation. The quantity of water transpired and evaporated from a cropped area or the normal loss of water from the soil by evaporation and plant transpiration *(8).* The quantity of water discharged to the atmosphere or incorporated in the products of the process in connection with vegetative growth, food processing, or an industrial process *(45).*

**CONSUMPTIVE USE, NET** The consumptive use decreased by the estimated contribution of rainfall toward the production of irrigated crops *(62).* See Effective Precipitation. Net consumptive use is sometimes called crop irrigation requirement.

**CONSUMPTIVE WASTE** The water that returns to the atmosphere without benefiting man *(63).*

**CONTACT LOAD SEDIMENT** Particles that roll or slide along in almost continuous contact with the stream bed. Compare with Bed Load.

**CONTENTS** The volume of water in a reservoir [or lake]. Unless otherwise indicted, reservoir content is computed on the basis of a level pool and does not include bank storage *(37) (52).*

**CONTIGUOUS** Touching, adjacent, adjoining, bordering on. Adjacent things may or may not be in actual contact, but they are not separated by like things. That which is adjoining something and touches it at some point or along a line. Things are contiguous when they touch along the whole or most of one side.

**CONTINUITY EQUATION** Discharge equals velocity times cross section area ($Q = V \times A$). For steady flow there is a continuity of discharge through succeeding sections of channel, expressed as: $Q = (A_1)(V_1) = (A_2)(V_2) = (A_nV_n)$ = a constant.

**CONTINUOUS MODELS** Models which simulate the rainfall-runoff process over long periods of time (months or years) including dry periods between events *(1).*

**CONTINUOUS STREAM** See Stream, Continuous.

**CONTRACTED SECTION** A cross section within a constriction; for example, at the downstream side of a bridge opening, or at a culvert entrance. Compare with Contraction, Channel Contraction, and Constriction.

**CONTRACTING REACH** A reach of channel wherein flow is accelerating; where the velocity head at lower cross section exceeds the velocity head at the upper cross section.

**CONTRACTION** The effects of a channel constriction on flow. Also, see Stream Contraction and Stream Constriction. Synonymous with Constriction.

**CONTRACTION COEFFICIENT** See Coefficient of Contraction.

**CONTRACTION METHOD (OF FLOW MEASUREMENT)** A method of indirect measurement of peak discharge following a flood by field survey of highwater marks and channel and bridge geometry at a constriction, such as at a bridge. Discharge is computed on the basis of an evaluation of energy changes between the approach section and the downstream side of the constriction by methods given in U.S. Geological Survey Circular 284. See Indirect Methods (of Flow Measurement).

**CONTRACTION SCOUR** See Scour, Contraction.

**CONTROL** A natural constriction of the channel, a long reach of the channel, a stretch of rapids, or an artificial structure downstream from a gaging station that determines the stage-discharge relation at the gage. A control may be complete or partial. A complete control exists where

the stage-discharge relation at a gaging station is entirely independent of fluctuations in stage downstream from the control. A partial control exists where downstream fluctuations have some effect upon the stage-discharge relation at a gaging station. A control, either partial or complete, may also be shifting. Most natural controls are shifting to a degree, but a shifting control exists where the stage-discharge relation experiences frequent changes owing to impermanent bed or banks *(37).*

A feature downstream from the gage that determines the stage-discharge relation at the gage. This feature may be a natural constriction of the channel, artificial structure, or a uniform cross section over a long reach of the channel *(52).*

A control may relate to a drainage design exclusive of any gage considerations. A section or a reach of a conduit where conditions exist that make the water level about it a fairly stable index of discharge; i.e., a section or reach of an open conduit or stream channel which maintains a stable relationship between stage and discharge. A control may be partial or complete. A complete control is independent of downstream conditions and is effective at all stages. An overflow dam, a ledge of rock crossing a channel, a boulder-covered reach, and an indurated bed are examples. Controls may be either natural or artificial. See Control Section; Control Structure; and Bed, Indurated.

**CONTROL FLUME** See Flume, Control.

**CONTROL SECTION** A control section, such as a bridge opening, reach of channel, or dam, with a definable flow capacity, in which the discharge is related to some measurable depth(s) such as the upstream water surface elevation, tailwater elevation, and/or contracted flow depth. See Control.

**CONTROL STRUCTURE** A structure [in a riverine environment] on a stream or canal that is used to regulate the flow or stage of the stream, or to prevent the intrusion of salt water *(52).* See Control.

**CONTROLLED RIVER** See River, Controlled.

**CONTROLLED SPILLWAYS** See Spillway, Controlled.

**CONVENTION** A short conduit for uniting two others having different hydraulic elements; a transition. Not a common term.

**CONVEYANCE** A measure of the ability of a stream, channel, or conduit to convey water. A comparative measure of the water-carrying capacity of a channel; that portion of the Manning discharge formula which accounts for the physical elements of the channel. Conveyance is expressed as: $(1/n)AR^{2/3}$ where n is Manning's n, A is the cross section area of flow, and R is the hydraulic radius. See Manning's Equation.

A measure of the water transporting capacity of such things as a channel, floodplain, drainage facility, storm drain, and/or other natural or artificial watercourse feature traversed by flows such as runoff or irrigation water. With the review flood or storm, conveyance may include that associated with overtopping flows and inundation of a traveled way at cross-drainages. Compare with Capacity.

**CORE-WALL** A wall of such things as masonry, sheet-piling, or puddled clay built inside a dam or embankment to reduce percolation.

**CORPS** See COE.

**CORRELATION** The process of establishing a relation between a variable and one or more related variables. Correlation is "simple" if there is only one independent variable and "multiple" if there are more than one independent variable. For gaging-station records, the usual variables are the short-term gaging-station record and one or more long-term gaging-station records *(59).*

**CORRELATIVE ESTIMATE** A discharge determined by correlation. A correlative estimate represents a likely value of the discharge for any particular period according to a specified method of analysis *(36).*

**CORROSION** The deterioration of pipe or structure by chemical action.

**COST-EFFECTIVE** A measure of a drainage design strategy's acceptability is often based on a judgment where either expected first costs or, when appropriate, the economic analysis costs are weighed against the selected design criteria.

The relationship between the benefits derived from a system and the cost of purchasing, operating, and maintaining it *(33).*

**COUNTERMEASURE** A measure, either incorporated into the design of a drainage facility or installed separately at or near the facility, that serves to prevent, minimize, or control hydraulic problems.

**COVER** The vertical extent of soil above the crown of a pipe or culvert. [Depending on the context, may also be] the vegetation, or vegetational debris, such as mulch, that exists on the soil surface. In some classification schemes, fallow or bare soil is taken as the minimum cover class *(51).*

**CRADLE** A footing structure shaped to fit the conduit it supports.

**CREEK** Term of regional geographic usage. See Channel.

**CREST** The maximum elevation of a flood at a specific location *(33).* Other definitions are: 1. the top of a dam, dike, spillway, or weir; 2. the overflow portion of a road or embankment; 3. the summit of a wave; and 4. the peak of a flood.

**CREST-STAGE GAGE** See Gage, Crest-Stage.

**CRITERIA** See Design Criteria.

**CRIB** See Retard.

**CRIB DAM** See Dam, Crib.

**CRITERION** A standard, rule, or test on which a judgment can be based. Compare with Design Criteria.

**CRITERIA, DESIGN** See Design Criteria.

**CRITICAL DEPTH** When the energy head is a minimum and the velocity head equals one-half the mean depth, the corresponding depth is Belanger's critical depth *(3)*.

The depth at which the specific energy (depth + velocity head) for a particular discharge is a minimum. It is the depth at which, for a given energy content of the water in a channel, maximum discharge occurs or the depth at which, in a given channel, a given quantity of water flows with minimum content of energy.

**CRITICAL FLOW** That flow in open channels at which the energy content of the fluid is at a minimum. Also, that flow which has a Froude number of unity. Flow at critical depth. See Critical Depth.

**CRITICAL PUBLIC SERVICES** [Services] typically consisting of police and fire assistance, emergency medical services, communications, utilities, transportation, and public works *(33)*.

**CRITICAL SHEAR STRESS** The minimum amount of shear stress (tractive force) exerted by passing stream currents required to initiate soil particle motion. Compare with Tractive Force.

**CRITICAL SLOPE** That particular slope of a given uniform conduit operating as an open channel at which normal depth equals critical depth for a given discharge ($Q_c$). The slope required in a conduit to sustain flow at critical depth for a given discharge; critical slope ($S_c$) must equal the energy slope ($S_E$) for the given discharge flowing at critical depth.

**CRITICAL VELOCITY** Mean velocity ($V_c$) of flow at critical depth ($d_c$); in open channels the velocity head equals one-half the mean depth.

**CRITICAL-DEPTH METHOD (OF FLOW MEASUREMENT)** A method for indirect measurement of peak discharge, following a flood, by field survey of highwater marks and channel and control-section or control structure geometry. Discharge is computed on a basis of critical flow (flow at minimum energy) theory. See Indirect Method (of Flow Measurement).

**CROSS DRAINAGE** The runoff from contributing drainage areas both inside and outside the highway right of way and the transmission thereof from the upstream side of the highway facility to the downstream side.

**CROSS SECTION (STREAM OR VALLEY)** A diagram or drawing cut across a channel normal to the expected flow direction [for a particular flood magnitude] that illustrates the banks, bed, [vegetal cover, soils,] and water surface. The shape of a channel, stream, or valley, viewed across the axis. In watershed investigations and channel analyses it is determined by a line approximately perpendicular to the main path of water flow [for a particular flood magnitude], along which measurements of distance and elevation are taken to define the cross-sectional area [, conveyance properties] and shape. In hydraulic analyses, vegetal patterns, floodplain material, and bed material [as well as any other conveyance properties] are considered part of the cross section *(51).*

**CROSS-SLOPE** See Roadway Cross-Slope.

**CROSSING** See Crossover.

**CROSSOVER** The relatively short and shallow reach of a stream between bends; also called a crossing. The point where flows crossover from one low flow channel bank to the other.

**CUBIC FEET PER SECOND** A unit measurement of water flow. Sometimes erroneously called "second feet," primarily in oral discussions. See cfs.

A unit expressing rates of discharge. Cubic feet per second is equal to the discharge of a stream of rectangular cross section, a foot wide and one foot deep, flowing water at an average velocity of one foot per sec. Abbreviated cfs or cusec *(37).*

The rate of discharge representing a volume of one cubic foot passing a given point during one second and equivalent to 7.48 gallons per second or 448.8 gallons per minute or 0.028 32 $m^3/s$ *(52).*

**CUBIC FEET PER SECOND PER SQUARE MILE** The average number of cubic feet of water flowing per second from each square mile of area drained, assuming that the runoff is distributed uniformly in time and space *(52).*

**CUBIC FOOT PER SECOND-DAY** The volume of water represented by a flow of one cubic foot per second for 24 hours. It is equivalent to 86,400 cubic feet, approximately 1.9835 acre-feet, about 646,000 gallons, or 2445 $m^3$.

**CUBIC METER PER SECOND** A unit measurement of water flow, primarily in oral discussions.

A unit expressing rates of discharge. Cubic meter per second is equal to the discharge of a stream of rectangular cross section, a meter wide and one meter deep, flowing water at an average velocity of one meter per second. See cubic feet per second. Symbolized by $m^3/s$.

**CULVERT** A structure which is usually a closed conduit or waterway that is designed hydraulically to take advantage of submergence to increase hydraulic capacity. A structure used to convey surface runoff through such things as a highway or railroad embankment. Although there are borderline cases, a culvert is a structure, as distinguished from bridges, which is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the channel bed serving as the bottom of the culvert. A culvert commonly has a regular, uniform shape, where a bridge opening may not—in other words, a culvert is a relatively large pipe or conduit. A culvert usually has a large ratio of length to width. Usually in practice, and in certain localities by AASHTO's definition, a structure of less than 6.1 m or 20 feet span as measured along the road centerline is classified as a culvert. Compare with Bridge.

**CULVERT, BROKEN BACK** A culvert having two or more longitudinal structure profile slopes. Such culverts are sometimes effective in reducing outflow velocities by energy dissipation from a hydraulic jump near the outlet.

**CULVERT METHOD (OF FLOW MEASUREMENT)** A method of indirect measurement of peak discharge, following a flood, by field survey of highwater marks and channel and culvert geometry where temporary upstream flood storage effects can be quantified or are not significant. Discharge is computed on the basis of an evaluation of energy changes between the approach section (headwater) and the control section. See Indirect Methods (of Flow Measurement) and Flow Types.

**CULVERT, SAG** A culvert where the inlet and outlet flowline is above the barrel flowline. A culvert that "sags" in order to pass under a low highway grade line. More commonly used to convey irrigation flows; not suitable for drainage subject to freezing. In the common but incorrect highway vernacular, a "siphon" or "inverted siphon."

A pipeline crossing a depression or under a highway, railroad, canal, etc. that makes use of pressure flow. A closed conduit, a part of which rises above the hydraulic grade line. It utilizes atmospheric pressure to affect or control the flow of water through it. The term "inverted siphon" or "siphon" is commonly and incorrectly used in highway drainage as such structures have none of the properties of a true siphon; i.e., these two terms are misnomers.

**CUMULATIVE CONVEYANCE** A tabulation or graphical plot of the accumulated measures of conveyance at various points across a cross section proceeding from the landward edge of one floodplain (or stream bank) to the landward edge of the other floodplain (or stream bank) so as to encompass the water surface width of a particular flood. Compare with Flow Distribution.

**CURB OPENING INLET** See Inlet, Curb Opening.

**CURRENT** Water flowing through a channel. The generally downstream moving portion or vector of flowing water.

**CURRENT METER** An instrument for measuring the speed of flowing water. The U.S. Geological Survey uses a rotating cup meter *(33).*

A device that is lowered into a stream in order to record the rate at which the current is moving *(20).*

An instrument used to measure the flow velocity of a current. It is usually operated by a wheel equipped with vanes or cups which is rotated by the action of the impinging current. An indicating or recording device is provided to measure the speed of rotation which is correlated with the velocity of the current. A device or instrument for determining or measuring the velocity of flowing water by ascertaining the speed at which a stream of water rotates a vane or a wheel.

**CUSEC** The abbreviation for cubic feet per second is common in the British Commonwealth countries, but is generally not used in the United States *(37).* See Cubic Feet Per Second.

**CUT BANK** The concave wall of a meandering stream. The outside, eroding bank of a bendway.

**CUTOFF** A natural or artificial channel that shortens the length of a stream; natural cutoffs may occur either across the neck of a meander loop (neck cutoffs) or across a point bar (chute cutoffs). Compare with Cutoff Wall.

**CUTOFF, CHUTE** See Cutoff.

**CUTOFF, NECK** See Cutoff.

**CUTOFF WALL** A wall, collar, or other structure intended to reduce percolation of water along otherwise smooth surfaces, or through porous strata. May also be a wall, usually constructed of such things as sheet piling or concrete, that extends from the end of a drainage structure and/or flowline downward to below the expected scour depth, or to scour-resistant material. Compare with Cutoff.

**CYCLE** A regularly recurring succession of events such as the cycle of the seasons. Climatic cycle sometimes is used to describe a group of wet years followed or preceded by a group of dry years *(3).* Compare with Trend and Probability Distribution.

**$D_{16}$** The particle diameter at the 16 percentile point on a size versus weight distribution curve.

**$D_{50}$** Median size of riprap or granular material. The particle diameter at the 50 percentile point on a size versus weight distribution curve such that half of the particles (by weight) are larger and half are smaller.

**$D_{85}$** The particle diameter at the 85 percentile point on a size versus weight distribution curve.

**DAILY DISCHARGE** Discharge averaged over one day.

**DAM** A barrier to confine or raise water for storage or diversion, or to create a hydraulic head.

**DAM, CRIB** A barrier made of timber, forming bays or cells which are filled with stone or other suitable material.

**DAM, DEBRIS** The barrier constructed across a channel to form a debris basin.

**DAM, DIVERSION** A barrier constructed for the purpose of diverting part or all the water from a channel into a different course.

**DAM METHOD (OF FLOW MEASUREMENT)** A method of indirectly determining the peak discharge of a flood by using the field survey of highwater marks in the headwater and tailwater pools, and the dam or highway-embankment geometry. Discharge is computed by recognized head-on-weir formulas, adjusted for degree of submergence when necessary. See Indirect Methods (of Flow Measurement).

**DAM-BREAK ANALYSIS** The use of a [computer] model to calculate the effects of a flood caused by the actual or hypothetical failure of a dam *(33).*

**DAMAGE REACH** A length of floodplain or valley selected for damage evaluation *(51).*

**DATUM** Plane of reference for elevations.

**$d_c$** See Critical Depth.

**DEAD STORAGE** See Storage, Dead.

**DEBRIS** Any material transported by the stream, either floating or submerged, such as logs, brush, suspended sediment, bed load, or trash that may lodge against a structure. Compare with Detritus.

**DEBRIS BARRIER** See Drift Barrier.

**DEBRIS BASIN** See Basin, Debris.

**DEBRIS CONE** A fan-shaped deposit of soil, sand, gravel, and boulders at the point where a steep stream meets a valley, or where its velocity is reduced sufficient to cause such deposits. Compare with Alluvial Fan.

**DEBRIS DAM** See Dam, Debris.

**DEEP WATER (FOR WAVES)** Water of such a depth that surface waves are little affected by bottom conditions; customarily, water deeper than half the wave length.

**DEFLECTOR** Little used alternative term for “Spur Dike” or “Spur.” See Dike, Spur.

**DEGRADATION (STREAM BED)** General and progressive lowering of the longitudinal profile of the channel bed due to long-term erosion. A progressive lowering of the channel bed due to

scour. Permanent or continuing degradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place.

**DEMONSTRABLY** Where such things as an engineering analysis or assessment, sometimes coupled with an economic analysis or assessment, along with prudent judgment is used to support a drainage related finding.

**DENSITY OF WATER SEDIMENT MIXTURE** Bulk density (mass per unit volume) including both water and sediment.

**DENTAL** A tooth-like projection on an apron, stilling basin, or other surface, to deflect or break the force of flowing water; a form of baffle. See Apron and Baffle Piers.

**DENTATED SILL** A notched sill at the end of an apron or stilling basin to check the force of flowing water and thus reduce erosion downstream from the apron. Compare with Dental.

**DEPENDABLE YIELD, n-YEARS** The minimum supply of a given water development that is available on demand, with the understanding that lower yields will occur once in n-years, on the average *(53).*

**DEPENDENT VARIABLE** See Variable.

**DEPLETION** The progressive withdrawal of water from surface or groundwater reservoirs at a rate greater than that of replenishment. Compare with Recession Curve and Stream Flow Depletion.

**DEPLETION CURVE** See Recession Curve

**DEPOSITION** The settling of material from the streamflow onto the bed.

**DEPRESSION STORAGE** See Storage, Depression.

**DEPTH, ALTERNATE** For a given rate of flow and a given specific head, two depths of flow are possible. These two depths are alternate depths. These depths, one less than and one greater than critical depth, may be present in a channel or conduit for any given value of specific energy above the minimum. Compare with Depth, Conjugate.

**DEPTH, CONJUGATE** The alternate depth of flow involved with the hydraulic jump; i.e., The depth $d_1$ and $d_2$ before and after a hydraulic jump. Unlike the alternate depths for a given specific head, the conjugate depths for a hydraulic jump reflect the energy loss from the hydraulic jump. Compare with Depth, Alternate.

**DEPTH, CRITICAL** See Critical Depth.

**DEPTH, NORMAL** The depth of water in an open conduit that corresponds to uniform velocity for the given flow. It is a hypothetical depth under conditions of steady nonuniform flow; the depth for which the water surface and bed are parallel. Normal depth and velocity apply only

to uniform flow with a free water surface. These conditions will be approached with a steady discharge in a length of uniform channel that is sufficient to establish uniform flow. See Channel, Uniform.

**DEPTH OF SCOUR** The vertical distance a stream bed is lowered by scour below a reference elevation.

**DEPTH-AREA CURVE** A graph showing the change in average rainfall depth as the drainage area changes.

A graph showing the changes in average rainfall depth as the size of the area receiving precipitation changes *(51).* It may also be the relationship between water depth and the land surface area inundated.

**DESERT** A wild, uninhabited and uncultivated tract. An arid barren tract incapable of supporting any considerable population without an artificial water supply (*Webster's New Collegiate Dictionary*). Compare with Arid and Semi-arid.

**DESIGN/ANALYSES MODELS** Models used for simulation of the detailed performance of particular elements within a subcatchment *(1).*

**DESIGN CRITERIA** Criteria, coupled with prudent judgmental factors, that are used to design a drainage facility. Compare with Policy.

**DESIGN DISCHARGE** The maximum rate of flow (or discharge) for which a drainage facility is designed and thus expected to accommodate without exceeding the adopted design constraints. Maximum flow a bridge, culvert, or other drainage facility is expected to accommodate without contravention of the adopted design criteria. The peak discharge, volume, stage or wave crest elevation and its associated probability of exceedance (see Exceedance Probability) selected for the design of a road culvert or bridge over a channel, floodplain or along a shoreline. By definition, the design discharge, or wave, does not overtop the road. The design discharge headwater, or wave height, may be at an elevation lower than the road's profile grade in order to meet other design criteria such as the protection of property, accommodating land use needs, lowering of velocities, reducing scour, or complying with regulatory mandates.

With irrigation facilities, the design discharge is the water right plus any flood right. Where floodwaters can enter the irrigation ditch or canal, the design discharge would include the water right, flood right, and any floodwaters capable of being conveyed to the point of interest (without first escaping elsewhere).

Selected flood discharge and corresponding recurrence interval for designing a highway encroachment. The design flood for highway bridge scour analyses shall be the base (100-year) flood, and the review flood the 500-year event. Compare with Flood, Review; Flood, Catastrophic; and Superflood.

**DESIGN FISH** See Fish, Design.

**DESIGN FLOOD** See Design Discharge.

**DESIGN FLOOD FOR BRIDGE SCOUR** The flood flow equal to or less than the 100-year flood that causes the deepest scour at bridge foundations. The highway or bridge may be inundated at the stage of the design flood for bridge scour. The worse case scour condition may occur for the overtopping flood due to the potential for pressure flow.

**DESIGN FLOOD (OR STORM) SYSTEM** Drainage facility (or storm drain system) that uses natural terrain and constructed facilities to provide for the conveyance of runoff from a frequently occurring flood (or storm), up to and including the design flood (or storm runoff), in a practicable and cost-effective manner so as to avoid a significant increase in the existing flood hazard or causing a flood hazard greater than allowed by policy and design criteria. By definition this system also precludes overtopping of the highway as well as an unacceptable flood hazard due to an encroachment on a floodplain or spread on the traveled way of guttered highways and streets. See Encroachment and Spread.

The water conveyance elements (such as gutters, inlets, laterals, trunks, pipes, channels, and ditches) of a storm drain system. Storm drains generally are that portion of the total drainage system between the most remote point of interception of runoff, such as a gutter(s), to an outfall(s). Sometimes erroneously called storm sewer when sewage is not involved. Compare with Sewer, Storm; Drain, Storm; and Sewer, Combined.

**DESIGN FLOOD FREQUENCY (OR STORM FREQUENCY)** The frequency (recurrence interval) for the selected design discharges (storms) that is expected to be accommodated without contravention of the adopted design criteria. See Design Discharge.

**DESIGN FLOW** See Design Discharge.

**DESIGN HIGHWATER ELEVATION** The maximum water levels that can occur through a reach and at a culvert, bridge-type opening, or other drainage facility without contravention of the adopted design criteria. May also be the usual term used to describe the estimated water surface elevation or profile in the stream (or other surface waters) at the project site for the selected design discharge.

**DESIGN HIGHWATER LEVEL** See Design Highwater Elevation.

**DESIGN STORM** Selected storm of a given frequency (recurrence interval) used for designing a design storm system. See Design Flood (or Storm) System.

Hypothetical storm derived from intensity-duration-frequency curves by reading the rainfall intensity from these curves for various durations for the frequency of interest and rearranging these rainfall intensities to fit an assumed storm pattern and storm duration *(51).*

A given rainfall amount, areal distribution, and time distribution, used to estimate runoff. The rainfall amount is either a given frequency (25-, 50-year, etc.) or a special large [or specific frequency] value *(51).*

**DETENTION BASIN** See Basin, Detention.

**DETERMINISTIC** See Hydrology, Deterministic.

**DETOUR** A temporary change in the roadway alignment. It may be localized at a structure or may be along an alternate route.

**DETRITUS** Loose fragments, particles, grains, grasses twigs or similar material comprising the smaller fraction of debris carried by flowing water. Compare with Debris and Drift.

**DEVELOPMENT** Refers to any constructed change to improved or unimproved real estate, including but not limited to buildings or other structures, mining, dredging, filling, grading, paving, excavation, or drilling operations or storage of equipment or materials. Taken from the 1992 NFIP regulations.

Land use changes from those of an existing condition.

Compare with Direct and Indirect Support (of Floodplain Development), Land Treatment Measure, and Land Use.

**DIKE** An impermeable linear structure for the containment or control of overbank flow; such dikes trend parallel with a river bank and differ from a levee only in that such dikes extend for a much shorter distance along the bank. Relatively short dikes are also placed to contain and redirect flow such as into a culvert or down some other path. Compare with Levee.

**DIKE, EMBANKMENT SPUR** Relatively short dikes constructed normal to an embankment such as an approach fill to a bridge or otherwise along a highway. Their purpose is to impede flow and direct it away from such embankments. An embankment spur dike is placed at an angle to the roadway for the purpose of shifting the erosion characteristics of stream or floodplain flow away from a drainage structure or a roadway embankment. Compare with Dike, Riparian Spur.

**DIKE, FINGER** See Dike, Embankment Spur.

**DIKE, GUIDE** See Guide Bank.

**DIKE, RIPARIAN SPUR** River training structure used for bank protection. A permeable or impermeable, linear structure projecting into a channel from a bank. A dike of rock or other material constructed from the bank into the channel for protection or for channel improvement. A dike extending from a bank into a channel that is designed: 1. to reduce the stream velocity as the current passes through the dike, thus encouraging sediment deposition along the bank (permeable dike); or 2. to deflect erosive currents away from the stream bank

(impermeable dike). An embankment or wall constructed more or less perpendicular to a stream bank or shoreline (also termed "groin"). A structure in the form of a barrier placed oblique to the primary motion of water, designed to control movement of bed load. Riparian spur dikes (groins) are usually solid, although they may be constructed with openings to induce sedimentation and control the elevations of such sediments. An elongated, permeable or impermeable obstruction projecting into a stream to control shoaling and scour by deflection of currents and waves. A dike constructed of piles, rock, or other material extending into a stream, the sea, or at the mouths of rivers to induce scouring, bank building, or bank or protection. Compare with Dike, Embankment Spur; and Retard.

**DIKE, SPUR** An outdated term for a Guide Bank. See Dike; Dike, Riparian Spur; and Guide Bank.

**DIKE, TOE** See Dike, Training.

**DIKE, TRAINING** Embankments constructed to provide a transition from the natural stream channel or floodplain into, and/or away from a channel constriction such as a bridge crossing. Embankments constructed to prevent lateral flow from scouring the corner of the downstream side of an approach embankment to a bridge as well as the bridge abutment.

**DIMICTIC LAKE** See Lake, Dimictic.

**DIRECT AND INDIRECT SUPPORT** (of floodplain development) To encourage, allow, serve, or otherwise facilitate additional base (100-year) floodplain development. Direct support results from an encroachment. Indirect support results from a highway action outside the base floodplain that leads to development in the base floodplain. Compare with Development.

**DIRECT PRECIPITATION** Water that falls directly into a lake or stream without passing through any land phase of the hydrologic cycle *(20).*

**DIRECT RUNOFF** See Runoff, Direct. Compare with Rainfall, Effective.

**DIRT** See Soil.

**DISCHARGE** Volume of water passing a point during a given time. The rate a volume of flow passes a point per unit of time, usually expressed in cubic feet per second ($ft^3$/sec) or cubic meters per second ($m^3$/s). Four somewhat differently stated definitions are: 1. the quantity of water, silt, or other mobile substances passing along a conduit per unit of time; 2. rate of flow such as cubic feet per second ($ft^3$/sec) or cubic meters per second ($m^3$/s), gallons per second or liters per second, millions of gallons per day or millions of liters per day; 3. the act involved in water or other liquid passing through an opening or along a conduit or channel; 4. the water or other liquid which emerges from an opening or passes along a conduit or channel. See cubic feet per second or cubic meters per second.

In its simplest concept, discharge means outflow; therefore, the use of this term is not restricted as to course or location, and it can be applied to describe the flow of water from a

pipe or from a drainage basin. If the discharge occurs in some course or channel, it is correct to speak of the discharge of a canal [, channel,] or of a river. It is also correct to speak of the discharge of a canal [, channel,] or stream into a lake, a stream, or an ocean (see Streamflow and Runoff) *(37).* The volume of water (or more broadly, volume of fluid plus suspended sediment) that passes a given point within a given period of time *(52).*

The data in the reports of USGS on surface water represent the total discharge, and streamflow and runoff represent water with the solids dissolved in it and the sediment mixed with it. Of these terms, discharge is the most comprehensive. The discharge of drainage basins is distinguished as follows:

Yield -------- Total water runoff or crop; includes runoff plus underflow.

Runoff ------- That part of water yield that appears in streams.

Streamflow ---- The actual flow in streams, whether or not subject to regulation, or underflow. See Underflow.

Each of these terms can be reported in total volumes (such as cubic feet per second, cubic meters per second or acres-feet per year, hectare-meter per year). The differentiation between runoff as a volume and streamflow as a rate is not accepted. Compare with Runoff and Streamflow.

**DISCHARGE, AVERAGE** In the annual series of the USGS reports on surface-water supply (the arithmetic average of all complete water years of record whether or not they are consecutive). Average discharge is not published for less than 5 years of record. The term “average” is generally reserved for average of record, and “mean” is used for averages of shorter periods, namely, daily mean discharge.

**DISCHARGE CURVE** See Stage-Discharge Curve.

**DISCHARGE, DESIGN** See Design Discharge.

**DISCHARGE, DOMINANT** The channel forming (morphological sense) discharge in a specific channel for a specific channel feature. The dominant discharge for hydraulic geometry relationships is sometimes taken to be the bankfull discharge. With stable banks, the bankfull discharge has a return period of approximately 1.5 years, whereas the bankfull discharge is sometimes associated with the mean annual flood ($Q_{2.33}$) or two-year flood ($Q_2$).

**DISCHARGE RATING CURVE** See Stage-Discharge Curve.

**DISCHARGE, MEAN** See Discharge, Average.

**DISCHARGE, PEAK** See Peak Discharge.

**DISCHARGE, SEDIMENT** See Sediment Discharge.

**DISCHARGE, UNIT** Discharge per unit width (may be average over a cross section, or local at a point).

**DISK, SECCHI** See Secchi Disk.

**DISTRIBUTED SYSTEM** The opposite of a lumped system. See Lumped System.

**DISTRIBUTION, FREQUENCY, AND DISTRIBUTION, GRAPH** See Probability Distribution.

**DISTRIBUTION, HYDROGRAPH** See Hydrograph, Distribution.

**DISTRIBUTION, PROBABILITY** Various probability distributions that may be of interest to a hydrologist and highway hydraulics engineer are California, Cauchy, Exponential, Extremal, Extreme Value, Gumbel, Logarithmetically Transformed, Normal, Partly Bounded, Pearson, Student's t, Totally Bounded, and Weibull. See Probability Distribution.

**DISTRIBUTION SYSTEM** A [statistical] system in which at least some of the variations in space have been recognized and established *(1).* Also used to refer to such things as a drinking water system.

**DISTRIBUTION-FREE** Requiring no assumptions about the kind of probability distribution a set of data may have *(34).*

**DITCH** An artificial channel, usually distinguished from a canal by its smaller size. See Canal, Channel, and Field Ditch.

**DITCH, FIELD** See Field Ditch.

**DIVERSION** See Channel Diversion.

**DIVERSION DAM** See Dam, Diversion.

**DIVIDE** See Drainage Divide.

**DIVERSITY** An environmentally related term referring to the range and number of species types in the biomass of surface waters.

**DOCTRINE** A rule, principle, theory, or tenet of the law. See Rule and Rule of Law.

**DOMINANT DISCHARGE** See Discharge, Dominant.

**DOUBLE-MASS CURVE** A plot on arithmetic cross-section paper of the cumulated values of one variable against the cumulated values of another or against the computed values of the same variable for a concurrent period of time *(58).* A graph in which accumulated amounts of item X are plotted versus accumulated amounts of item Y, the amounts for given times being used *(51).*

**DRAIN** A conduit for carrying off surplus groundwater or surface waters. Closed drains are usually buried.

**DRAIN, STORM** A conduit for carrying off stormwaters. Compare with Sewer. See Drain; Design Flood (or Storm) System; Sewer; Sewer, Combined; and Major Storm Drain.

**DRAINAGE** Four definitions are provided: 1. the process of removing surplus groundwater or surface waters by artificial means; 2. the manner in which the waters of an area are removed; 3. the area from which waters are drained; 4. a drainage basin. See Drainage Area.

**DRAINAGE AREA** The catchment area for rainfall [and other forms of precipitation] which is delineated as the drainage area producing runoff; i.e., contributing drainage area. Usually it is assumed that base flow in a stream also comes from the same drainage area.

The drainage area of a stream at a specified location is that area, measured in a horizontal plane, which is enclosed by a drainage divide. Over the years, use of the term to signify drainage basin or catchment area has come to predominate, although drainage basin is preferred. Used alone, the term "watershed" is ambiguous and should not be used unless the intended meaning is made clear *(37).*

According to the *National Engineering Handbook (51)*: 1. the area draining into a stream at a given point. The area may be of different sizes for surface runoff, subsurface flow, and base flow, but generally the surface runoff area is used as the drainage area; 2. the area contributing direct runoff to a stream. Usually it is assumed that base flow in the stream also comes from the same area. However, the groundwater watershed may be larger or smaller.

The drainage area of a stream at a specified location is that area, measured in a horizontal plane, enclosed by a topographic divide from which direct surface runoff from precipitation normally drains by gravity into the stream above the specified point. Figures of drainage area given in [WRD Data Reports] include all closed basins or noncontributing areas within the area unless otherwise noted *(52).*

The area of land drained by a channel. An area confined by drainage divides, often having only one outlet for discharge; the total drainage area contributing runoff to a single point. The watershed area to include all other catchment physical characteristics. The term "catchment" is often used synonymously with other terms such as "drainage area" and implies all physical characteristics, including the contributing area.

An area surrounded by a continuous ridge [or drainage divide] within which all runoff is expected to join into a single flow stream and which extends to the point of junction of this flow stream (downstream) with the ridge. Natural boundaries, constructed boundaries, or minimum size of pipe are criteria which can be used to define the catchment *(1).*

Three other definitions of interest are: 1. the area drained by a stream or stream system; 2. synonymous with drainage area, drainage basins, or catchment area; 3. for the sake of

clearness the use of watershed to mean catchment basin or drainage basin is avoided by the USGS and others. Compare with Drainage Basin.

**DRAINAGE BASIN** A part of the surface of the earth that is occupied by a drainage system, which consists of a surface stream or a body of impounded surface water together with all tributary surface streams and bodies of impounded surface water *(37)*. The land area from which surface runoff drains into a stream system *(20)*. A part of the surface of the earth that is occupied by a drainage system, which consists of a surface stream or a body of impounded surface water together with all tributary surface streams and bodies of impounded surface water *(52)*. Compare with Drainage Area.

**DRAINAGE DENSITY** Length of all channels above those of a specified stream order per unit of drainage area *(37)*.

**DRAINAGE DIVIDE** The rim of a drainage basin. The divide separating one drainage basin from another and in the past has been generally used to convey this meaning. Drainage divide, or just divide, is used to denote the boundary between one drainage area and another *(37)*.

**DRAW** Term of regional geographical usage. See Channel.

**DRAWDOWN** The difference in elevation between the water surface elevation at a constriction in a stream or conduit and the elevation that would exist if the constriction were absent. Drawdown also occurs at changes from mild to steep channel slopes and at weirs or vertical spillways. Compare with Drawdown, Rapid.

**DRAWDOWN CURVE** A particular form of the surface curve of a stream of water which is convex upward, generally a conversion from tranquil flow upstream to rapid flow downstream in which the stream flow drops through critical depth, such as flow over a weir or spillway or a sharp increase in stream bed slope, proceeding downstream. Compare with Backwater Curve.

**DRAWDOWN, RAPID** Lowering the water against a bank more quickly than the bank can drain which can leave the bank in an unstable condition due to excessive pore pressure. Compare with Drawdown.

**DRIFT** Alternative term sometimes used (perhaps incorrectly) for “debris.” Debris that drifts on or near the water surface. Sometimes used as general term for all debris, that floated or otherwise, in evidence following a flood. Compare with Detritus and Debris.

**DRIFT BARRIER** An open structure constructed across a stream channel to catch drift or debris. It may be of any form from a simple wire fence to a barrier of massive piers with heavy cables strung between them. Where acceptable, the material caught is burned in place at low stages of the stream.

**DRIFT LINE** An accumulation of deposited water-carried debris and/or detritus along a contour, at the base of vegetation, or within the vegetation and other topographic objects that provides direct evidence of prior inundation and often indicates the directional flow of floodwaters. See Highwater Mark.

**DRIFT RATE** The mass of total macro-invertebrates drifting with the current per unit of time, usually grams/hour. Depending on context, may be the rate (velocity), such as feet per second or meters per second, that floodborne drift or detritus is transported downstream.

**DRIP GROOVE** Linear depression in the bottom of structure components to cause water flowing on the surface to drop.

**DROP** Two definitions are offered: 1. a vertical or inclined structure for dropping the water in a conduit or channel to a lower level and dissipating its surplus energy. An inclined drop in a channel is often termed a chute; 2. a fall in water-surface elevation between the upstream and downstream (as between headwater and tailwater) sides of a flow-contacting drainage facility such as at a bridge constriction or culvert or between two sections of a slope-area reach.

**DRY WEATHER FLOW** The flow in sanitary or combined sewers that contains no stormwater *(1)*.

**DUNE** A sand wave of approximately triangular cross section (in a vertical plane in the direction of flow) formed by moving water or wind, with gentle upstream slope and steep downstream slope. Dunes travel downstream by the displacement of sediments on the upstream slope and their subsequent deposition on the downstream slope. See Bed Form. Compare with Antidune.

**DURATION CURVE** See Flow-Duration Curve for one type *(37)*.

A graph showing the percentage of time that the given flows of a stream will be equaled or exceeded. It is based upon a statistical study of historic streamflow records *(20)*.

**DYNAMIC EQUILIBRIUM** The delicate balance of many factors which must occur in a stream reach so that the channel is neither aggrading or degrading. See Stream, Poised.

EAP Acronym for Emergency Action Plan *(33)*. See Emergency Action Plan.

**EARTH DAM** A barrier to streamflow composed of earth, clay, sand, or sand and gravel, or a combination of earth and rock.

**ECCENTRICITY** An expression descriptive of the position of a bridge constriction not centrally located in the floodplain. Eccentricity, if considerable, has an effect upon the discharge coefficient. High eccentricity tends to decrease the discharge coefficient and increase the backwater.

**ECCENTRICITY RATIO** The eccentricity ratio ($K_e$) is defined as the ratio of flow contracted from the smaller lateral region on one side of a stream to the flow contracted from the larger lateral region on the other side, and is expressed as $K_e = K_a/K_b \leq 1.0$, where $K_a$ and $K_b$ are the conveyances of the smaller and larger lateral regions (approach cross section sub-sections).

**ECONOMIC ANALYSIS** A probabilisitically based analysis that compares the estimated construction costs with those expected average flood-related operational costs and risks that can be quantified for the anticipated service life of a project to identify an optimum design flood frequency. Compare with Economic Assessment.

**ECONOMIC ASSESSMENT** An economic assessment is a less rigorous, more judgmental economic analysis. Compare with Economic Analysis.

**ECOREGION** Continuous geographical areas characterized by distinctive flora, fauna, land forms, climate, vegetation, and ecological climax.

**ECOSYSTEM** The living organisms and the nonliving environment interacting in a given area. The complexity of a community and its environment functioning as an ecological unit in nature.

**EDDY CURRENT** A vortex-type motion of a fluid flowing contrary to the main current, such as the circular water movement that occurs when the main flow becomes separated from the bank or emerges from a bridge or culvert opening.

**EDDY LOSS** The energy lost (converted into heat) by swirls, eddies, and impact, as distinguished from friction loss.

**EFFECT** According to *Webster's New Collegiate Dictionary:* 1. something produced by an agent or cause: Result; 2. an outward sign: Appearance; 3. a distinct impression.

What occurs to surface waters as a direct or indirect result of a highway action. See Significance. Compare with Impact.

**EFFECTIVE DURATION** The time in a storm during which the water supply for direct runoff is produced. Also used to mean the duration of excess rain *(51).*

**EFFECTIVE PARTICLE SIZE** The diameter of particles, equivalent to a spherical shape, equal in size and arranged in a given manner, of a hypothetical sample of granular material that would have the same transmission constant, shear resistance (see Tractive Force), erosion/scouring response, and fall velocity as the actual material under consideration.

**EFFECTIVE PRECIPITATION** See Precipitation, Effective.

**EFFECTIVE RAINFALL** See Rainfall, Effective.

**EFFLUENT** Sewage, water, or other liquid, partially or completely treated or in its natural state as the case may be, flowing out of a reservoir, basin, or treatment plant.

**EFFLUENT STREAM** See Stream, Gaining.

**ELECTROLYTES** Substances that dissociate into ions in solution or when fused, thereby becoming electrically conducting.

**EMBANKMENT END SLOPE** Conical slope at end of road approach embankment; bridge spillslope. See Spillthrough Abutment and compare with Fill Slope.

**EMBANKMENT SPUR DIKE** See Dike, Embankment Spur.

**EMERGENCY ACTION PLAN** A detailed description of the actions that must be taken to reduce flood losses [and hazards] and to disseminate information about an actual or expected flood hazard *(33).*

**EMERGENCY OPERATION CENTER (EOC)** The center [for] decision-making, information gathering and dissemination, initiating of emergency actions, and coordinating responses to emergencies. The center may be part of a public service or safety unit in the local government or may be activated only when emergencies arise *(33).*

**EMERGENCY SPILLWAY** See Spillway, Emergency.

**EMINENT DOMAIN** In law, the right of a government to take, or to authorize the taking of private property for public use, just compensation being given to the owner.

**ENCROACHMENT** A highway action within the limits of a base (100-year) floodplain. With a design or review storm system, encroachment is sometimes used when referring to the width gutter flow spreads onto a traveled way as measured perpendicular from either the edge of the traveled way or from the face of the curb. See Spread.

**ENCROACHMENT, LONGITUDINAL** An encroachment that roughly parallels a channel or floodplain.

**ENCROACHMENT, TRANSVERSE** An encroachment that crosses a channel or floodplain. Compare with Spread.

**END CONTRACTIONS** The contraction in the area of overflowing water caused by such things as the ends of a weir notch.

**END SECTION** A structure, commonly made of concrete or metal, that is attached to the end of a culvert for such purposes as retaining the embankment from spilling into the waterway, improving the appearance, providing anchorage, improving the discharge coefficient and limiting some scour at the outlet. Compare with Inlet, Flared.

**ENERGY** The capacity to perform work: kinetic energy is that due to motion, and potential energy is that due to position. In a stream the total energy at any section is represented by the sum of its potential and kinetic energies.

**ENERGY DISSIPATION** The phenomenon whereby energy is dissipated or used up. In highway drainage this is the energy in flowing water.

**ENERGY DISSIPATOR** See Stilling Basin.

**ENERGY EQUATION** The work-energy relationship, reduced to the simplified form from the Bernoulli equation (Bernoulli's Theorem which states that $P/\gamma + Z + v^2/2g = \text{constant}$). The equation is:

$$v_1^2/2g + \rho_1/\gamma + Z_1 = V_2^2/2g + P_2/\gamma + Z_2 + h_L$$

where subscripts 1 and 2 denote the upstream and downstream cross section.

**ENERGY GRADE LINE** A line joining the elevation of energy heads of a stream; a line drawn above the hydraulic grade line a distance equivalent to the velocity head of the flowing water at each cross section along a stream or channel reach or through a conduit. An inclined line representing the total energy of a stream flowing from a higher to a lower elevation. For open channel flow the energy grade slope is located (or plotted) a distance equal to the velocity head ($V^2/2g$) plus the flow depth above the water surface: V = velocity, and g = acceleration due to gravity. Slope of the foregoing line joining the elevations of total energy through the reach of a stream or channel or through a conduit of flowing water.

**ENERGY GRADE SLOPE** See Energy Gradient.

**ENERGY GRADIENT** The slope of the energy line with reference to any plane or, more simply, the slope of the energy grade line. The slope of this line represents the rate of loss of head, and it must always slope downward in the direction of flow. Equivalent to Energy Gradient. Compare with Hydraulic Gradient and Friction Slope.

**ENERGY HEAD** See Head, Energy.

**ENERGY, KINETIC** Energy due to motion. The kinetic energy of a given discharge is generally taken as proportional to the product of its weight per unit of time and the velocity head of its mean velocity. For a constant discharge, kinetic energy may be represented by a line at a distance above a flowing water surface proportional to the velocity head of its mean velocity. The elevation of such a line above any datum represents the total energy (potential plus kinetic) of the given discharge above that datum. Strictly, the kinetic energy of a given discharge is the integral of the kinetic energies of its particles.

**ENERGY LINE** See Energy Grade Line.

**ENERGY LINE, MINIMUM** An energy line corresponding to conditions of critical flow.

**ENERGY, POTENTIAL** Energy due to position. The potential energy of a given volume of immobile water with reference to any datum is proportional to the product of its weight and the elevation of the center of gravity above that datum. The potential energy per unit of time

of a given discharge at any instant with reference to any datum is proportional to the product of its weight per unit of time and the elevation of its hydraulic grade line above that datum at that instant.

**ENERGY SLOPE** See Energy Grade Line.

**ENERGY, SPECIFIC** The energy contained in a stream of water expressed in terms of head, referred to the bed of a stream. It is equal to the mean depth of water plus the velocity head of the mean velocity.

The energy of a stream referred to its bed; namely, depth plus the velocity head based on the mean velocity.

**ENERGY, TOTAL** See Energy.

**ENHANCEMENT** See Surface Water Enhancement.

**ENTRANCE HEAD** The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head; equivalent to headwater height; energy head at approach section to culvert or bridge.

**ENTRANCE LOSS** The head lost in eddies and friction at the inlet to a conduit or structure, expressed as a coefficient ($K_e$) times velocity head = $K_e$ ($V^2/2g$).

**ENTRENCHED STREAM** See Stream, Incised.

**ENVELOPE CURVE** See Flood Envelope Curve.

**ENVIRONMENTAL EFFECTS** Pertaining to the effects of highway engineering works on their surroundings and on nature.

**EOC** Acronym for Emergency Operation Center.

**EON** A geologic time interval containing two or more eras. Compare with Era.

**EPA** Acronym for the U.S. Environment Protection Agency.

**EPHEMERAL STREAM** See Stream, Ephemeral.

**EPILIMNION** The upper, circulating water layer of a stratified lake. See Thermal Stratification.

**EQUALIZER** An opening, such as a culvert or bridge, placed where it is desirable to equalize the water level on both sides of an embankment.

**EQUIVALENT CROSS-SLOPE** An imaginary, straight cross-slope that provides conveyance equal to that of an actual or proposed compound cross-slope. Common usage in highway drainage is to facilitate estimating the stormwater conveyance of a street gutter having a compound cross-slope.

**ERA** A division of geological history of highest rank. There are five eras: Cenozoic; Mesozoic; Paleozoic; Proterozoic; Archeozoic. Compare with Eon.

**EROSION** Displacement of soil particles on the land surface due to such things as water or wind action. The wearing away or eroding of material on the land surface or along channel banks by flowing water or wave action on shores. Compare with Abrasion, Scour, Mass Wasting, and Sloughing.

**EROSION, LATERAL** Erosion in which the removal of material has a predominately lateral component (see Erosion), as contrasted with scour in which the component is predominately vertical (see Scour).

**EROSION CONTROL MATTING** Fibrous matting (e.g., jute, paper, etc.) placed or sprayed on a channel bank or land surface for the purpose of preventing erosion or providing temporary stabilization until vegetation is established.

**ESTUARY** Tidal reach at the mouth of a river. That portion of a river channel occupied at times or in part by both sea and river flow in appreciable quantities. The water usually has brackish characteristics.

**ET** Acronym for evapotranspiration *(51).* See Evapotranspiration.

**EUTROPHIC LAKE** See Lake, Eutrophic.

**EVALUATION SERIES** A list of floods or storms that produced floods during a representative period [of time], and used in water project evaluation to obtain estimates of flood damages *(51).*

**EVAPORATION** The process by which water passes from the liquid to the vapor state *(20).*

The process by which water is changed from the liquid or the solid state into the vapor state. In hydrology, evaporation is vaporization that takes place at a temperature below the boiling point.

**EVAPORATION OPPORTUNITY** Relative Evaportranspiration: The ratio of the rate of evaporation from a land or water surface in contact with the atmosphere to the evaporativity under existing atmospheric conditions. It is the ratio of actual to potential rate of evaporation, generally stated as a percentage *(47).*

The opportunity for a given rate of evaporation to continue is determined by the available moisture supply *(49).*

**EVAPORATION, TOTAL** The sum of water lost from a given land area [drainage area] during any specific time by transpiration from vegetation and building of plant tissue; by evaporation from water surfaces, moist soil, and snow; and by interception [questionable — Ed.]. It has

been variously termed “evaporation,” “evaporation from land areas,” “evapotranspiration,” “total loss [questionable — Ed.],” “water losses,” and “fly off” *(39).*

**EVAPORATIVITY** Potential Rate of Evaporation: The rate of evaporation under the existing atmospheric conditions from a surface of water that is chemically pure and has the temperature of the atmosphere *(47).*

**EVAPOTRANSPIRATION** Plant transpiration plus evaporation from the soil. Difficult to determine separately, therefore used as a unit for study (see Consumptive Use) *(51).* The combined loss of water from a given area by evaporation from the land and transpiration from plants *(22).* The sum of evaporation plus transpiration *(20).*

Water withdrawn from a land area by evaporation from water surfaces and moist soil and plant transpiration. It is a coined word; probably the first recorded use is on page 296 of the *Transactions of the American Geophysical Union,* Part 2, 1934 *(37).*

**EVAPOTRANSPIRATION POTENTIAL** See Potential Evapotranspiration.

**EVENT** As used in data collection, [event] represents a point at which a gage reaches a preset value and records the occurrence or transmits it to a receiver: *(33).* Compare with Flood Event.

**EVENT MODELS** [Computer] Models which simulate the rainfall-runoff process for a single rainfall event, where pre-event conditions must be specified. Event models usually incorporate short time intervals in the simulation *(1).*

**EVERGREEN** Plants that retain their leaves at the end of the growing season and usually remain green through the winter.

**EXCEEDANCE FREQUENCY** The percentage of values that exceed a specified magnitude, 100 times exceedance probability *(37).*

**EXCEEDANCE INTERVAL** See Flood-Frequency.

**EXCEEDANCE PROBABILITY** Probability that a random event will exceed a specified magnitude in a given time period, usually one year unless otherwise indicated *(34).*

**EXCESS RAINFALL** See Rainfall Excess.

**EXCESSIVE PRECIPITATION** See Rainfall, Excessive.

**EXCESSIVE RAINFALL** See Rainfall, Excessive.

**EXFILTRATION** The process by which stormwater leaks or flows to the surrounding soil through such things as openings in a conduit, channel banks, or lake shores.

**EXPANDING REACH** A reach of channel wherein flow is decelerating; where velocity head at the lower cross section is less than the velocity head at upper cross section.

**EXPONENTIAL DISTRIBUTION** See Probability Distribution.

**EXTREMAL DISTRIBUTION** See Probability Distribution.

**EXTREME VALUES** The largest and the smallest value [from a sample of data] are commonly referred to as extreme or [external] values, and are often associated with floods, droughts, surplus or deficit, and similar economic and safety connotations *(74).* See Probability Distributions.

**EXTREME VALUE DISTRIBUTIONS** See Probability Distributions.

**FABRIC FILTER** See Synthetic Filter.

**FABRIFORM** Grout-filled fabric mattress used for stream bank protection.

**FALL VELOCITY** See Velocity, Fall.

**FALLING LIMB** The declining portion of a hydrograph following a crest *(33).*

**FALLOW** Cropland kept free of vegetation during the growing season. May be a normal part of the cropping system for weed control, water conservation, soil conditioning, etc. *(51).*

**FASCINE** See Retard.

**FAUNA** The animals of a particular region or time.

**FEDERAL COMMON LA**W See Common Law, Federal.

**FEDERAL REGISTER** A daily publication of the Federal Government making federal regulations, legal notices, Presidential Proclamations, Executive Orders, etc., known to the public as they are proposed and subsequently issued. See also Code of Federal Regulations.

**FEMA** Acronym for Federal Emergency Management Agency.

**FERN ALLIES** A group of nonflowering, vascular plants comprised of clubmosses, small clubmosses, and quillworts.

**FETCH** The effective distance the wind blows over water in generating waves. The area in which waves are generated by wind having a rather constant direction and speed; sometimes and incorrectly used synonymously with "fetch length." The horizontal distance (in the direction of the wind) over which wind generates waves and wind setup.

**FETCH LENGTH** Redundant terminology, as the term "fetch" implies a length. See Fetch.

**FHWA** Acronym for Federal Highway Administration.

**FIELD CAPACITY** See Field-Moisture Capacity.

**FIELD DATA** All stormwater data collected in the field regardless of whether or not it was analyzed in the field *(1).*

**FIELD DITCH** An irrigation channel for: 1. delivering water to the rows in a field; or 2. flooding a field. A field ditch commonly does not have a recorded water and flood right at its point of diversion. These ditches deliver the water to the field from an irrigation canal having a recorded water or flood right at its point of diversion. The water right for a field ditch can sometimes be estimated based on the area being irrigated by the ditch at a point in time. That portion of the water right delivered by a field ditch can change when landowners modify their field ditch system.

**FIELD**, JACK See Jack Field.

**FIELD-MOISTURE** CAPACITY The quantity of water which can be permanently retained in the soil in opposition to the downward pull of gravity *(25).*

**FIELD-MOISTURE DEFICIENCY** The quantity of water, which would be required to restore the soil moisture to field moisture capacity *(26).*

**FILL SLOPE** Side or end slope of an earth-fill embankment. Where a fill slope forms the streamward face of a spillthrough abutment, it is regarded as part of the abutment. Compare with Embankment End Slope and Spillthrough Abutments.

**FILTER** Layer of synthetic fabric, sand, gravel, and/or graded rock placed (or developed naturally where suitable in-place materials exist) between the bank revetment and soil for one or more of three purposes: 1. to prevent the soil from moving through the revetment by piping, extrusion, or erosion (exfiltrating); 2. to prevent the revetment from sinking into the soil; 3. to permit natural seepage from the stream bank, thus preventing buildup of excessive hydrostatic pressure. Also may be a device or structure for removing solid or colloidal material from stormwater and floodwater or preventing the migration of fine-grained soil particles as water passes through soil; i.e., the water is passed through a filtering medium—usually a granular material or finely woven or non-woven geotextile. Depending on context, may be used to remove material other than soils from a substance.

**FILTER BLANKET** One or more layers of graded, intermediate-size gravel or a geotextile material laid between fine-grained material and riprap to prevent the migration of the finer material (exfiltration). Compare with Synthetic Filter.

**FILTER CLOTH** See Synthetic Filter.

**FILTRATION** The process of passing water through a filtering medium consisting of either granular material or filter geotextiles for the removal of suspended or colloidal matter.

**FINE SEDIMENT LOAD** See Wash Load.

**FINGER DIKE** See Dike, Finger.

**FIRST FLUSH RUNOFF** See Runoff, First Flush.

**FISH** An aquatic animal (pl. fish or fishes) according to *Webster's New Collegiate Dictionary.*

**FISH, DESIGN** A hypothetical fish embodying predetermined size, swimming, and migration characteristics that are used in the design of drainage facilities to minimize adverse effects to the fishery. See Total Fish Length.

**FISH LADDER** A structure with pools and drops to facilitate the migration and movement of fish around culverts, chutes, dams, or other obstructions in channels. Compare with Fishway.

**FISH LENGTH, TOTAL** See Total Fish Length.

**FISH SCREEN** A device intended to prevent the entrance of fish into a conduit or area where they are not wanted.

**FISHWAY** A waterway, designed or natural, that facilitates the movement of fish. Compare with Fish Ladder.

**FIVE-HUNDRED YEAR FLOOD** See Flood, Five-hundred Year.

**FIXED BED AND BANKS (MODEL)** A channel whose bed and banks are considered to be unable to move under the forces of moving water. Fixed bed and bank analyses and computer models ignore any mobility (scour or erosion) of the channel bed or banks. Compare with Mobile Bed and Banks (Model).

**FLANKING** Erosion resulting from streamflow between the bank and the landward end of a river training or a grade-control (drop) structure.

**FLANKING INLET** See Inlet, Flanking.

**FLARE ANGLE** See Angle of Flare.

**FLARED INLET, OUTLET, AND** WINGWALLS See Inlet, Flared.

**FLASH FLOOD** See Flood, Flash.

**FLASH FLOOD WARNING** The public notification issued by the National Weather Service (NWS) on a county basis when flash flooding is imminent or has been reported *(33).* See Flood, Flash.

**FLASH FLOOD WATCH** The public notification issued by the National Weather Service (NWS) when climatic, ground, and topographic conditions indicate that flash floods are likely to occur in a given area *(33).* See Flood, Flash.

**FLASHY STREAM** See Stream, Flashy.

**FLOAT GAGE** See Gage, Float.

**FLOCCULATION** The process to cause soil or other substances to form a lump or mass to facilitate their removal.

**FLOOD** In common usage, an event that overflows the normal flow banks or runoff that has escaped from a channel or other surface waters. See Normal Flow and Bank. In frequency analysis it can also mean an annual flood that may not overflow the normal flow banks. In technical usage, it refers to a given discharge based, typically, on a statistical analysis of an annual series of events.

An overflow or inundation that comes from a river or other body of water and causes or threatens damage. Any relatively high streamflow overtopping the natural or artificial banks in any reach of a channel. A relatively high flow as measured by either gage height or discharge quantity.

An overflow or other body of water that causes or threatens damage *(6).* Any relatively high streamflow overtopping the natural or artificial banks in any reach of a stream *(40).* A relatively high flow as measured by either gage height or discharge quantity *(35).* See Floodwaters; and Flood, Annual.

**FLOOD, ANNUAL** The maximum momentary peak discharge in each year of record. May be maximum daily discharge or instantaneous discharge. The highest peak discharge in a water year. The maximum flow in one year *(34).*

**FLOOD, BASE** A flood (or storm) or reservoir pool elevation having a 1 percent chance of being exceeded in a one year period; commonly termed a 100-year event. See One-Hundred (100-) Year Flood.

**FLOOD (OR STORM), CATASTROPHIC** Any flood (or storm) well in excess of the base flood (or storm) and review flood (or storm) but not exceeding the probable maximum flood (or storm). A flood (or storm) capable of causing a large amount of unavoidable damage and destruction. Compare with Superflood; Flood (or Storm), Review; and Design Discharge.

**FLOOD (OR STORM), CHECK** See Flood (or Storm), Review.

**FLOOD CREST** See Peak Discharge.

**FLOOD ENVELOPE CURVE** A plot showing the upper and lower boundary limits of the maximum annual floods for the range of drainage areas in a hydrologic region.

**FLOOD, DESIGN** See Design Discharge.

**FLOOD EVENT** A flow of water in a stream constituting a distinct progressive rise, culminating in a crest, together with the recession that follows the crest. Compare with Event and Peak Discharge.

**FLOOD EXCEEDANCE PROBABILITY** Probability that a random flood event will exceed a specified magnitude in a given time period, usually one year unless otherwise indicated.

**FLOOD FREQUENCY** The average time interval between occurrences of a hydrological event of a given or greater magnitude, usually expressed in years. May also be called recurrence interval. *(52)*.

The average time interval, in years, in which a given storm or amount of water in a stream will be exceeded. Also, referred to as exceedance interval, recurrence interval, or return period. May be stated as: 1. the average time interval between actual occurrences of a hydrological event of a given or greater magnitude; 2. the percent chance of occurrence in any one year period, e.g., a 2 percent chance of flood. The chances that a specific flood magnitude (discharge) will be exceeded each year expressed as a percent; i.e., a 100-year flood has a flood probability of 1 percent of being exceeded each year. In the analysis of hydrologic data the flood frequency is simply called frequency and has years as a unit of measure. Note that flood frequency is not hyphenated when referring to a specific flood's frequency, but is when referring to such things as a "flood-frequency" curve.

An expression or measure of how often a hydrologic event of given size or magnitude should, on an average, be ...exceeded. For example a 50-year frequency flood should be ...exceeded in size, on the average, only once in 50 years. In drought or deficiency studies it usually defines how many years will, on the average, be ...less than a given size or magnitude *(37)*. Note, this reference incorrectly stated "equaled or exceeded," and "equal to or less than" where the three periods (...) appear (Ed.).

**FLOOD-FREQUENCY CURVE** "General Introduction and Hydrologic Definitions" *(37)* offers two definitions: 1. a graph showing the number of times per year on the average, plotted as abscissa, that floods of magnitude, indicated by the ordinate, are ... exceeded; 2. a similar graph but with recurrence intervals [frequency] of floods plotted as the abscissa. A graph indicating the probability that the annual flood discharge will exceed a given magnitude, or the recurrence interval corresponding to a given magnitude. Compare with Frequency Curve and Flood Frequency.

According to Dalrymple *(18)*: 1. a graph showing the number of times per year on the average, plotted as abscissa, that floods of magnitude, indicated by the ordinate, ...or exceeded; 2. a similar graph but with recurrence intervals of floods plotted as abscissa.

Note that Flood-Frequency is hyphenated when referring to a flood-frequency (flood versus frequency) curve or relationship, and not hyphenated when referring to a specific flood's frequency.

**FLOOD, FLASH** A flood that occurs in a short time (minutes to hours) after the causative event *(33)*.

**FLOOD, FIVE-HUNDRED YEAR** The flood due to a storm and/or tide having a 0.2 percent chance of being exceeded in any given year. Compare with Superflood.

**FLOOD, GREATEST OPENING** The greatest flood expected to flow through a drainage structure where roadway overtopping cannot occur from the probable maximum flood.

**FLOOD HAZARD** Potential consequences, hazards, and inconveniences encountered by the traveling public, imposed on adjacent property owners, and incurred by the environment from a flood or a highway action in areas subject to flooding; included are such things as potential property loss or damage, loss of life, temporary or long-term loss of a transportation facility, permanent or long-lasting environmental damage, circuitous or interrupted highway travel, hydroplaning and other roadway overtopping related hazards. Compare with Flow Hazard.

**FLOOD HYDROGRAPH** See Hydrograph.

**FLOOD INDEX** The amount of rainfall that will produce a flood stage in 6 hours; it is calculated by measuring soil moisture *(33).*

**FLOOD LOSS POTENTIAL** The potential loss of life and property from flooding *(32).*

**FLOOD, MAXIMUM POSSIBLE** Not an accepted term. See Greatest Opening Flood.

**FLOOD, MEAN ANNUAL** Maximum annual flood peak having a 2.33 year frequency interval (recurrence interval). Flood where, if the total population of floods were known, half would be larger and the remaining half would be smaller. Compare with Flow, Mean Annual; Mean Daily Discharge; and Mean Monthly Flow.

**FLOOD MEASUREMENT** See Indirect Method (of Flow Measurement).

**FLOOD OF RECORD** The maximum estimated or measured discharge that has occurred at a site.

**FLOOD, ONE-HUNDRED YEAR** Magnitude of the flood that has a 0.01 probability of being ...exceeded [see following note — Ed.] in any given year and has about a 63 percent chance of being exceeded during a 100 year [year period]. It is now in vogue to call the 100-year flood the one-percent probability (chance) flood *(16).* In highway drainage design the 100-year flood is sometimes termed the base flood. See Flood, Base. Note: it is more proper to say "...probability of being exceeded in any...," rather than "...exceeded." Compare with Flood, One-Thousand Year. See X-Percent Chance Flood and Flood Frequency.

**FLOOD, ONE-THOUSAND YEAR** Magnitude of the flood that has a 0.001 probability of being ...exceeded in any given year. Over an infinitely long period of time it would occur on an average once every thousand years. It should be noted that over a 1000 years [year period] there is about a 63 percent chance of at least one occurrence and there is a significant chance of two, three, or more floods of this magnitude occurring during the thousand years *(16).* Again, "...probability of being exceeded in any..." is more proper. See Flood, One-Hundred Year; and X-Percent Chance Flood.

**FLOOD, OVERTOPPING** Incipient discharge escaping via such things as over a highway, at a watershed divide, or through emergency relief facilities. Stated another way, the flood which, if exceeded, results in flow over a highway, bridge or culvert, over a watershed divide or dike, or through structures provided for emergency relief. The worse case scour condition may occur with the overtopping flood. The total flood magnitude cannot exceed the probable maximum flood and its frequency accuracy is limited by the state of the art capability to estimate a recurrence interval.

**FLOOD PEAK** See Peak Discharge.

**FLOOD PLANE** The position occupied by the water surface of a stream during a particular flood. Also, loosely, the elevation of the water surface at various points along the stream during a particular flood *(37).* Compare with Floodplain.

**FLOOD POOL** Floodwater storage in a reservoir. In a floodwater retarding reservoir, the temporary storage between the crests of the principal and emergency spillways *(51).*

**FLOOD PROBABILITY** See Flood-Frequency.

**FLOOD, PROBABLE MAXIMUM (PMF)** The currently accepted term for the most severe flood that is considered reasonably possible at a site as a result of hydrologic and meteorologic conditions. This flood would result from the greatest depth of precipitation that is physically possible ["physically possible" is not a good term — Ed.] at a particular site. In practice many assumptions/calculations have to be made about the most severe combinations of meteorologic conditions, such as barometric pressure, wind speed, temperature, etc. and such variables as antecedent moisture, average basin infiltration rates, etc. It should be noted that floods greater than the computed PMF have occurred *(16).*

A PMF is developed from the PMP in much the same way as a SPF is developed. However, assumptions concerning rainfall losses, snowmelt runoff, channel efficiency, etc., are adjusted to produce the largest flood reasonably possible. The PMF is used to design high hazard structures (top of dam and spillway capacity) where failure cannot be tolerated *(30).*

The largest flood for which there is any reasonable expectancy in this climatic era *(40).* The probable maximum flood is the greatest flood that may reasonably be expected, taking into collective account the most adverse flood related conditions based on geographic location, meteorology, and terrain. A very rare flood discharge value computed by hydrometeorological methods, usually in connection with major hydraulic structures.

A catastrophic flood which, in highway design, may be defined by the upper limits of the flood envelope curve for maximum floods that have occurred in a hydrologic area and physiographic region. With highway design the PMF is sometimes arbitrarily considered to be a 10 000-year event for computational purposes.

**FLOOD PROFILE** A graph of elevation of the water surface of a river in flood, plotted as ordinate, against distance, measured in the downstream direction, plotted as abscissa. A flood profile may be drawn to show elevation at a given time, crests during a particular flood, or to show stages of concordant flows *(37).* See Flow, Concordant.

**FLOOD, PROJECT** A flood discharge value adopted for the design of projects such as dams and flood control works. The term "Design Flood" is more common to highway drainage design. See Design Discharge. Compare with Flood, Project Design.

**FLOOD, PROJECT DESIGN** A term common to the design of major dams and flood control works, but not routine highway drainage design.

When used in connection with levees and floodwalls, the Project Design Flood (PDF) is the flood (discharge and elevation) that, when freeboard is added, establishes the top of levee or floodwall grade. (Other, larger floods are used to design for overtopping of these kinds of structures, and to establish overtopping impacts.) For rapid flow channels the PDF is generally defined in much the same way. For reservoirs, the PDF is the flood that, with controlled releases, would fill the designated flood control storage. Tranquil flow channels do not have a single PDF; instead, they have differing design objectives over a range of flood magnitudes. They are generally formulated and designed to continue to provide stage reductions in floods exceeding channel capacity *(30).*

Compare with Level of Protection as it is not necessarily synonymous with this term. Compare with Design Discharge. See Flood, Project.

**FLOOD, REGULATORY** See Regulatory Flood.

**FLOOD (OR STORM), REVIEW** A flood (or storm) used to review (check) a drainage facility designed to accommodate a lesser design flood (or storm) so as to judge whether a significant flood hazard due to a flood larger than the proposed design discharge has been overlooked. The review flood (or storm) for all but bridge scour analyses is the greater of either the base flood, or the greatest flood when overtopping does not occur from the probable maximum flood. Bridge scour analyses shall use the greatest flood or the 500-year flood, whichever is smaller as the review (check) flood.

A flood (or storm), larger or smaller than the selected design flood, which is used to assess the performance of a drainage facility under other than design conditions.

Review (check) flood for scour is the flood resulting from a storm and/or tide having a flow rate in excess of the design flood for scour. It is used in the design of a bridge foundation to determine whether the foundations can withstand the flow and its associated scour without failing. Generally a 500-year flood. Compare with Superflood; and Flood, Catastrophic.

**FLOOD RIGHT** Where a state has jurisdiction over its waters, the flood right is an adjudication by that state of any excess runoff waters to a specified user for beneficial use. Runoff waters in

excess of those required to satisfy all water rights on a stream or river may be adjudicated as a flood right. Compare with Water Right. See Beneficial Use.

**FLOOD ROUTING** The process of determining progressively the timing and shape of a flood wave at successive points along a river *(12).*

Determining the changes in a flood wave as it moves downstream through a valley or through a reservoir (then sometimes called reservoir routing). Graphic or numerical methods are used *(51).*

**FLOOD STAGE** The gage elevation of the lowest bank of the reach in which the gage is situated. The term "lowest bank" is, however, not to be taken to mean an unusually low place or break in the natural bank through which the water inundates unimportant and small areas *(43).* The [elevation or] stage at which overflow of the natural banks of a stream begins to cause damage in the reach in which the elevation is measured: U.S. Weather Bureau. Compare with Bankfull Stage.

**FLOOD WAVE** A distinct rise in stage culminating in a crest and followed by recession to lower stages *(37).* The rise and fall in streamflow during and after a storm *(51).*

**FLOOD ZONE** The land bordering a stream which is subject to floods of about equal frequency; for example, a strip of the floodplain subject to flooding more often than once but not as frequently as twice in a century *(37).* Compare with Backwater Area and Floodplain.

**FLOOD-CONTROL STORAGE** Storage of water in reservoirs to abate flood damage: "General Introduction and Hydrologic Definitions" *(37).* See also Retarding Reservoir and Retention Basin.

**FLOODPLAIN** Any plain which borders a stream and is covered by its waters in time of flood. Topographic area adjoining a channel that is covered by flood flows as well as those areas where the path of the next flood flow is unpredictable, such as a debris cone, alluvial fan or braided channel. A nearly flat, alluvial lowland bordering a stream and commonly formed by stream processes, that is subject to inundation by floods.

Kirk Bryan, in "Erosion and Sedimentation in the Papago Country, Arizona" *(11),* provides this definition of a floodplain: A strip of relatively smooth land bordering a stream, built of sediment carried by the stream and dropped in the slack water beyond the influence of the swiftest current. It is called a living floodplain if it is overflowed in times of highwater, but a fossil floodplain if it is beyond the reach of the highest flood.

The lowland that borders a river, usually dry, but subject to flooding *(28).* That land outside of a stream channel described by the perimeter of the Maximum Probable Flood *(71).* Compare with Flood Plane, Flood Zone, and Backwater Area.

**FLOODPLAIN DEVELOPMENT** A floodplain containing, zoned to contain or reasonably foreseen to contain development that may incur a significant flood hazard or cause significant conveyance changes in a stream or river reach. See Development and Floodplain. Compare with Floodplain, Developed; and Floodplain, Undeveloped.

**FLOODPLAIN, DEVELOPED** A floodplain that contains a significant amount of development. See Development and Floodplain Development.

**FLOODPLAIN, UNDEVELOPED** A floodplain expected to either remain in a native or rural condition or incur no development during the service life of the highway that will not result in a significant flood hazard from a highway action. See Development and Direct and Indirect Support (of Floodplain Development).

**FLOODPROOF** Limiting or preventing the intrusion of floodwaters into a proposed or existing facility to protect their contents and structural integrity against flooding.

**FLOODS ABOVE A BASE** See Partial-Duration Flood Series.

**FLOODWATER RETARDING STRUCTURE** A dam, usually with an earth fill, having a flood pool where incoming floodwater is temporarily stored and slowly released downstream through a principal spillway. The reservoir contains a sediment pool and sometimes storage for irrigation or other purposes *(51).* Compare with Retarding Reservoir, Retention Basin, and Detention Basin.

**FLOODWATERS** Waters which escape from a natural watercourse in great volume and flow over adjoining lands in no regular channel. The fact that such errant waters make for themselves a temporary channel or follow some natural channel, gully, or depression, does not affect their character as "floodwaters" or give to the course which they follow the character of a natural "watercourse." Compare with Flood and Riparian Waters.

**FLOODWAY** "General Introduction and Hydrologic Definitions" *(37)* offers two definitions: 1. a part of the floodplain, otherwise leveed, reserved for emergency diversion of water during floods; 2.a part of the floodplain which, to facilitate the passage of floodwater, is kept clear of encumbrances.

The channel of a river or stream and those parts of the floodplain adjoining the channel, which are reasonably required to carry and discharge the floodwater or floodflow of any river or stream *(19).* Compare with Regulatory Floodway.

**FLOODWAY, REGULATORY** See Regulatory Floodway.

**FLORA** The aggregate of plants growing in and usually peculiar to a particular biotic or physiographic region or period.

**FLOW** A stream of water; movement of such things as water, silt and/or sand; discharge; total quantity carried by a stream. Characterized by the haphazard movement of small elements of

a fluid undergoing translation. In terms of the Reynold's number, turbulent flow corresponds to high values of that number.

**FLOW, BYPASS** Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downstream. Sometimes termed carryover.

**FLOW, CHOKING** See Choking.

**FLOW CONCENTRATION** A preponderance of the streamflow. Compare with Flow Distribution.

**FLOW, CONCORDANT** Flows at different points in a river system that have the same recurrence interval or the same frequency of occurrence. It is most often applied to flood flows *(37).*

**FLOW, CRITICAL** See Critical Flow.

**FLOW DISTRIBUTION** The estimated or measured spatial distribution of the total streamflow from the landward edge of one floodplain or stream bank to the landward edge of the other floodplain or stream bank. Usually shown as a percent of accumulated flow from one edge (0%) to the other edge (100%). Same as the cumulative conveyance only in terms of discharge rather than conveyance. Compare with Cumulative Conveyance.

**FLOW, FRONTAL** The portion of flow which passes over the upstream side of a grate and is subsequently captured. Compare with Splash-Over.

**FLOW HAZARD** Flow characteristics (discharge, stage, velocity, or duration) that are associated with a hydraulic problem or that can reasonably be considered of sufficient magnitude to cause a hydraulic problem or to test the effectiveness of a countermeasure. Compare with Flood Hazard.

**FLOW, GRADUALLY VARIED** Flow in which changes in depth and velocity take place slowly over large distances, resistance to flow dominates, and acceleration forces are neglected.

**FLOW, LAMINAR** That type of flow in which each particle moves in a direction parallel to every other particle and in which the head loss is approximately proportional to the first power of the velocity. It is sometimes designated "streamline flow" or "viscous flow." Laminar flow is characterized by the steady, translatory motion of adjacent small elements of the fluid. Tendencies toward turbulence or instability in truly laminar flow are damped out by viscous shear forces. In terms of the Reynold's number, laminar flow corresponds to low values of that number.

Flow in which the head loss is proportional to the first power of the velocity *(22).*

That type of flow in which the fluid particles follow paths that are smooth, straight and parallel to the channel walls. In laminar flow the viscosity of the fluid damps out turbulent motion *(20).*

Laminar flow changes to turbulent flow in a pipe between the critical value of 2000 and about 50 000 (there is no definite upper limit). Compare with Flow, Turbulent.

**FLOW LINE** Four definitions are offered: 1. the hydraulic grade line; 2. a conduit, as a pipe, laid on the hydraulic gradient; 3. bottom invert of a conduit; 4. flowage line.

**FLOW, MEAN ANNUAL** The annual mean flow, $Q_a$, for the year based on the 12 monthly means. Compare with Flood, Mean Annual; Mean Daily Discharge; and Mean Monthly Flow.

**FLOW MEASUREMENT** See Indirect Method (of Flow Measurement).

**FLOW, NONUNIFORM** A flow, the velocity of which is undergoing a positive or negative acceleration. If the flow is constant, it is referred to as "steady non-uniform flow." A flow in which the velocities vary from point to point along the stream or conduit, due to variations in cross section, slope, etc. Compare with Flow, Uniform. See Flow, Steady.

**FLOW, NORMAL** See Normal Flow; and Depth, Normal.

**FLOW, ONE-DIMENSIONAL** See One-Dimensional Water Surface Profile and Two-Dimensional Water Surface Profile.

**FLOW, ORIFICE** Flow similar to that through an orifice. For highway drainage design, in culvert flow it corresponds to flow-type V; i.e., a culvert flowing part-full under high head.

**FLOW, OVERBANK** Water movement over the top of a bank either due to a rising stream stage or to inland surface water runoff. Compare with Flood; and Flow, Overland.

**FLOW, OVERLAND** The flow of rainwater [, melting hail,] or snowmelt over the land surface toward stream channels. After it enters a stream, it becomes runoff *(37)*.

The flow of water over a land surface due to direct precipitation. Overland flow generally occurs when the precipitation rate exceeds the infiltration capacity of the soil and depression storage is full. Also called Horton's Overland Flow *(20)*.

Runoff which makes its way over the land surface prior to concentrating in gullies and streams; sometimes termed Sheet Flow. Compare with Flow, Overbank.

**FLOW, PRESSURE** Where flows passing through a bridge type opening are contracted vertically by the superstructure to the extent that the flow has a pressure head and flow streamlines analogous to that occurring at a sluice gate.

**FLOW, RAPID** See Flow, Supercritical.

**FLOW, RAPIDLY VARIED** Flow in which changes in depth and velocity take place over short distances, acceleration forces dominate and energy loss due to friction is minor.

**FLOW REGIME** The system or order characteristic of streamflow with respect to velocity, depth, and specific energy. See Flow, Critical; Flow, Subcritical; and Flow, Supercritical.

**FLOW, RETURN** That part of irrigation water that is not consumed by evapotranspiration and that returns to its source or another body of water. The term is also applied to the water that is discharged from industrial plants. Also called return water *(37).*

**FLOW, SHEET** See Flow, Overland.

**FLOW, SINUOUS** See Flow, Turbulent.

**FLOW SLIDE** Saturation of a bank to the point where the soil material behaves more like a liquid than a solid; the soil/water mixture may then move downslope resulting in a bank failure. See Sloughing.

**FLOW, SLUICE** Flow through a culvert under high head whereby the entrance contracts the flow and causes a part-time flow condition through the barrel; also referred to as orifice-type flow and similar to flow from under a sluice gate.

**FLOW, STEADY** A flow in which the flow rate or quantity of fluid passing a given point per unit of time remains constant. A constant discharge with respect to time.

**FLOW, STEADY NONUNIFORM** See Flow, Nonuniform; and Flow, Steady.

**FLOW STREAM** Flow from a catchment in an open or closed conduit *(1).*

**FLOW, STREAMING** See Flow, Subcritical.

**FLOW, SUBCRITICAL** In this state, gravity forces are dominant so that the flow has a relatively low velocity and is often described as tranquil or streaming. Also, that flow which has a Froude number less than unity. Flow at velocities less than critical velocity; flow at depths greater than critical depth. Flow at velocities less than one of the recognized critical values; specifically, turbulent flow with a mean velocity less than Belanger's critical velocity; streaming flow. Compare with Flow, Supercritical.

**FLOW, SUPERCRITICAL** In this state, inertia forces are dominant so that flow has a high velocity and is usually described as rapid or shooting. Also, that flow which has a Froude number greater than unity. Flow at velocities greater than critical velocity; flow at depths less than critical depth. Flow at velocities greater than one of the recognized critical values; specifically turbulent flow with a mean velocity equal to or greater than Belanger's critical velocity; shooting flow; rapid flow. Compare with Flow, Subcritical.

**FLOW, TRANQUIL** See Flow, Subcritical.

**FLOW, TURBULENT** The flow condition in which inertial forces predominate over viscous forces and in which head loss is not linearly related to velocity *(22).* That type of flow in

which the fluid particles move along very irregular paths. Momentum can be exchanged between one portion of the fluid and another. That type of flow in which any particle may move in any direction with respect to any other particle and in which the head loss is approximately proportional to the second power of the velocity. See Turbulence. Compare with Flow, Laminar.

**FLOW, TWO-DIMENSIONAL** See Two-Dimensional Water Surface Profile.

**FLOW TYPE, BRIDGE** As applied to flow through bridges there are four principal flow types, enumerated as follows: 1. Type I—Subcritical approach flow with subcritical flow in the most contracted section of the bridge opening (backwater due primarily to opening geometry); 2. Type IIA—Subcritical approach flow with flow passing through the normal flood depth (but not critical depth) in the most contracted section of the bridge opening. This is a transition range where the opening geometry is still influential, but the backwater starts to become influenced primarily by the amount of contraction; 3. Type IIB—Subcritical approach flow with flow theoretically passing through the bridge opening at critical depth in the most contracted section of the bridge opening. Backwater is due primarily to the amount of contraction; 4. Type III—Supercritical approach flow with supercritical flow through the bridge opening at a greater depth than the approach or tailwater flow (no backwater possible).

**FLOW TYPE, CULVERT** As applied to flow through culverts, there are six principal flow types, enumerated as follows: 1. Type I flow—Part-full flow (low upstream head) with control at inlet; 2. Type II flow—Part-full flow (low upstream head) with control at outlet; 3. Type III flow—Part-full flow (low upstream head) under backwater conditions (tailwater control); 4. Type IV flow—Full-flow with both inlet and outlet of culvert submerged; 5. Type V flow—Part full, sluice-type flow under high upstream head; 6. Type VI flow—Full-flow under high upstream head.

**FLOW, UNIFORM** Flow of constant cross section and average velocity through a reach of channel during an interval of time. A constant flow of discharge, the mean velocity of which is also constant. Uniform flow is also referred to as “steady uniform flow.” It is an ideal condition that can, in reality, only be approximated. If the velocity of the constant discharge varies, the flow is defined as “steady non-uniform.” When the average velocities at successive points or sections in the direction of steady flow are the same, the flow is described as uniform. Truly uniform flow, although frequently assumed for computational convenience, seldom occurs in natural open channels. Constant depth flow through a straight reach of a uniform artificial canal is an example of reasonably uniform flow. Compare with Flow, Nonuniform. See Flow, Steady.

**FLOW, UNSTEADY** Flow of variable cross section and average velocity through a reach of channel during an interval of time. A changing discharge with respect to time; opposite of Steady Flow; frequently labeled “varied flow.” Compare with Flow, Steady.

**FLOW-CONTROL STRUCTURE** A structure within and/or outside a channel that controls the direction, depth, or velocity of flowing water. May act as a countermeasure.

**FLOW-DURATION CHART** A graph indicating the percentage of time during which a given discharge is exceeded.

**FLOW-DURATION CURVE** A cumulative frequency curve that shows the percentage of time that specified discharges are equaled or exceeded *(58).*

**FLUME** An open or closed channel used to convey water. An open conduit of such things as wood, concrete, or metal on a prepared grade, trestle, or bridge. A flume holds water as a complete structure. A concrete lined canal would still be a canal without the lining, but the lining supported independently would be a flume. A large flume is also termed an aqueduct. Compare with Bench-Flume.

**FLUME, CONTROL** An open conduit or artificial channel arranged for measuring the flow of water which generally includes a constricted section wherein critical depth exists. See: Flume, Parshall Measuring; and Flume, Venturi.

**FLUME, PARSHALL MEASURING** (Formerly termed the "Improved Venturi Flume") A calibrated device developed by engineers of the U.S. Department of Agriculture, of whom Ralph I. Parshall, Assoc. M. Am. Soc. C. E., has been the principal experimenter. Its purpose is to measure the flow of water in open conduits. It consists essentially of a contracting length, a throat (flume control), an expanding length. At the throat is a sill over which the water is intended to flow at critical depth. The upper head is measured a definite distance upstream and the lower head a definite distance downstream from the sill. The lower head need not be observed except where the sill is submerged more than about 67 percent. A special form of control flume. Compare with Flume, Venturi.

**FLUME, VENTURI** A type of open flume with a contracted throat that causes a drop in the hydraulic grade line; used for measuring flow. Compare with Flume, Parshall Measuring.

**FLUSH ENTRANCE** See Inlet, Flush.

**FLUVIAL GEOMORPHOLOGY** A study of the structure and formation of the earth's features which result from the forces of water. Sometimes river engineers abbreviate fluvial geomorphology in discussions to a simpler, but incorrect, term "morphology." Compare with Geomorphology. See Morphology and Morphology Problems.

**FLUVIAL SEDIMENT** See Sediment, Fluvial.

**FORD** A location where a highway crosses a channel by allowing high annual or larger flows to pass over the highway and lower flows to pass through a culvert(s). Often used with cutoff walls, roadway lane markers and paved roadway embankments and traveled way (and shoulders). Warning signs may be included, also.

**FOREST INFLUENCES** Effects resulting from the presence of forest or brush upon climate, soil, water, runoff, streamflow, floods, erosion and soil productivity *(37).*

**FORK LENGTH** Fork length of a fish is a standard measure of length advocated by some fishery experts; it is commonly measured from anterior most extremity (nose tip) to the notch in the tail fin of fork-tailed fishes (or to the center of the fin when the tail is not forked).

**FRAZIL ICE** A French-Canadian term for fine spicular ice, derived from the French for cinders which this variety of ice most resembles. When formed in salt water, it is known as lolly ice. It is composed of fine particles which, when first formed, are colloidal and not seen in the water in which they are floating *(5) (57).*

**FREE FLOW** A condition of flow through or over a structure not affected by submergence.

**FREE OUTLET** Those outlets whose tailwater is equal to, or lower than, critical depth at the outlet. For culvert type structures having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

**FREE WATER SURFACE** The water surface of flow in an open channel or in a closed conduit not flowing full.

**FREE WEIR** A weir that is not submerged; i.e., tailwater is below the crest, or the flow is not affected by the elevation of the tailwater.

**FREEBOARD** Vertical clearance between the lowest structural member of the bridge superstructure, the top culvert invert, or the point of escape in a canal or channel to the water surface elevation of a flood. Freeboard may also be the vertical distance above a design stage that is allowed for waves, surges, drift and other contingencies. The vertical distance between the level of the water surface, usually corresponding to the design discharge (or wave runup) selected for freeboard considerations and a point of interest such as a low chord of a bridge beam, specific location on the roadway grade, or top of a channel bank. The distance between such things as the normal operating level and the top of the sides of an open conduit or channel, or the crest of a dam that is left to allow for wave action, floating debris, or any other condition or emergency, without overtopping the structure. For irrigation flows intercepting runoff, freeboard is based on the expected water surface elevation determined for the sum of the water right, flood right and design discharge.

The marginal height provided above [the] design [discharge] lines, [stage] on levees and in certain channels, to insure, as fully as practicable, against overtopping due to uncertainties in [such things as] the state of project maintenance or flood flow characteristics. In appropriate circumstances, special increments of levee freeboard may be provided to achieve design objectives (e.g., to control, in such an extremity, the location where initial overtopping of a levee would take place; to reduce wave overtopping; to extend the interval between major maintenance efforts for removal of tree growth, sediment deposition, etc. from the channel the levee bounds). Added height to earth levees is sometimes provided to allow [freeboard] for

settlement. In project evaluation, one-half of the inundation reduction benefits creditable to the levee freeboard zone may be included *(30).*

**FREEBOARD ALLOWANCE** See Freeboard.

**FREQUENCY** See Flood-Frequency.

**FREQUENCY ANALYSIS** [The interpretation and analysis of] a past record of hydrologic events in terms of future probabilities of occurrence. [The estimate] of frequencies [of occurrence] of floods, droughts, storages, rainfalls, water qualities, waves, etc; the procedure involved is known as frequency analysis *(14).*

**FREQUENCY CURVE** A graphical representation of the frequency of occurrence of specific events. In flood studies, frequency is expressed as the recurrence interval (RI) which is the average number of years within which a given peak discharge or rainfall intensity will be exceeded. Compare with Flood-Frequency Curve. See Frequency Analysis.

**FREQUENCY CURVE, LOW FLOW** See Low-Flow Frequency Curve.

**FREQUENCY DISTRIBUTION** See Probability Distribution.

**FREQUENCY LINE** The line on probability paper that represents a series of events and their frequencies.

**FREQUENCY SERIES** A sequence or array of actual events (floods, etc.) suitable for use in frequency analysis, or a sequence or array of hypothetical events obtained from a frequency analysis *(37).* See Frequency Analysis and Probability Distribution.

**FRICTION LOSS (OR HEAD)** The head or energy loss as the result of disturbances set up by the contact between a moving stream of water and its containing conduit. For convenience, friction losses are best distinguished from losses due to such things as bends, expansions, obstruction and impacts, but there is no recognized line of demarcation between them, and all such losses are often included in the term "friction loss."

**FRICTION SLOPE** The friction loss (or head) per unit length of conduit. For most conditions of flow, the friction slope coincides with the energy grade line, but where a distinction is made between energy losses due to such things as bends, expansions and impacts, a distinction must also be made between the friction slope and the energy grade line. Friction slope is equal to the bed or surface slope only for relatively uniform flow in nearly uniform channels. Compare with Energy Grade Line and Hydraulic Grade Line.

**FRONTAL FLOW** See Flow, Frontal.

**FROUDE NUMBER** A dimensionless number (expressed as $F = V/(gy)^{1/2}$) that represents the ratio of inertial to gravitational forces; i.e., at a Froude number of unity the flow velocity and wave celerity are equal (see Celerity, Wave). High Froude numbers can be indicative of a high

velocity associated with supercritical flow and thus the potential for scour and high momentum forces. Stated another way, a number which varies in magnitude inversely with the relative influence of gravity on the flow pattern: F > 1.0 indicates rapid (supercritical) flow; F < 1.0 indicates tranquil (subcritical) flow.

**FULL HEIGHT (OR RETAINING) ABUTMENT** See Vertical Abutment.

**FUNCTIONS OF SURFACE WATERS** The functional environmental characteristics of surface waters include such things as riparian and floodplain habitat for terrestrial and aquatic wildlife, flood control, groundwater recharge, aesthetics, shore and bank line geometry, scenic and wild rivers, endangered species habitat and water pollution abatement ability. May also be termed natural and beneficial values. See Surface Waters. Compare with Values of Surface Waters.

**FUNCTIONAL HIGHWAY CLASSIFICATION** A grouping of highways into classes or systems according to the quality of service they are expected to provide.

**FUNGI (FUNGUS)** Any of numerous plants of the division of subkingdom Thallophyta, lacking chlorophyll, ranging in form from a single cell to a body mass of branched filamentous hyphae that often produce specialized fruiting bodies and including the yeasts, molds, smuts and mushrooms.

**FWS** Acronym for U.S. Fish and Wildlife Service.

**G** (or g) The acceleration of gravity, $ft/s^2$.

**GABION** A rectangular basket made of steel wire fabric or mesh which is filled with rock or similar material of suitable size and gradation. Used to construct such things as flow-control structures, bank protection, groins, jetties, permeable dikes and riparian spur dikes. When filled with cobbles, masonry remnants, or other rock or suitable size and gradation, the gabion becomes a flexible and permeable block with which the foregoing structures and devices can be built. Compare with Riprap, Wire-Enclosed.

**GALTON DISTRIBUTION** Sometimes called the "law of Galton." See Probability Distribution.

**GAGE (GAUGE)** Two definitions are provided: 1. a staff graduated to indicate the elevation of a water surface; 2. a device for registering water levels, flow, velocity and pressure. Compare with Gaging Station, Recorder, Register, and Indicator.

**GAGE, BUBBLE** An automated device that measures water pressure near the bottom of the stream; this pressure equates with water depth above the gage *(33).*

**GAGE, CHAIN OR TAPE** A device consisting of a tagged or indexed chain tape or other line attached to a weight which is lowered to touch the water surface, whereupon the gage height is read on a graduated staff or opposite an index. Especially suited to bridges with the graduated staff generally placed horizontally with the line running over a pulley. Bridges and

similar structures with significant contractions incur rapidly varied flow conditions which may preclude any meaningful discharge measurements when such devices are placed within the contraction.

**GAGE, CREST-STAGE** A simple and economical device used to record crest stages of floods or flows. It is usually constructed of a 2 inch (50-mm) pipe, with suitable intake holes, contains a wooden gage stick and has finely ground cork placed in the bottom of the pipe to float up and temporarily mark the stage on this gage stick. The cork adheres to the gage stick up to the crest elevation of a flood. Given this elevation, the discharge is estimated based on a rating curve developed for the reach using the Slope-Area Method. See Slope-Area Method. Compare with Gage, Staff; and Water-Level Recorder.

**GAGE DATUM** The elevation level that corresponds to stage 0.0 at a stream gage; it is often set at the stream bottom or the elevation of a very low flow *(33).*

**GAGE, FLOAT** A chain or tape gage in which a float is substituted for the weight. Measurement of the discharge of water by floats to determine velocities.

**GAGE HEIGHT** Height of the water surface above the zero reference mark on a gage. The water surface elevation [is] referred to some arbitrary gage datum. Gage height is often used interchangeably with the more general term stage although gage height is more appropriate when used with a reading on a gage *(37) (52).*

**GAGE, HOOK** A pointed hook attached to a graduated staff or vernier scale for accurately measuring the elevation of the surface of still water. The hook is submerged and then raised until the point makes a pimple on the water surface.

**GAGE, INCLINED** A staff gage placed on a slope (incline) and graduated to read (or indicate) vertical heights above the datum.

**GAGE, POINT** A sharp, pointed rod attached to a graduated staff or vernier scale for measuring the elevation of the surface of flowing water. The point is lowered until the tip barely touches the water, forming a streak.

**GAGE, PRESSURE** See Manometer.

**GAGE, RECORDING** See Water-Level (Stage) Recorder.

**GAGE, SELF-REPORTING** Instruments that automatically transmit rainfall or stream stage data from a remote gage to the base station *(33).*

**GAGE, SLOPE** See Gage, Inclined.

**GAGE, STAFF** A vertical board or structure with a graduated scale for measuring the depth of a river in feet *(33).* A graduated scale on such things as a staff, plank, metal-plate pier, or wall, by which the elevation of the water surface may be read. Compare with Gage, Crest-Stage.

**GAGE, STANDARD RAIN** Also "standard gage." The USWB [now the NWS] nonrecording rain gage, having an opening gage size 8 inches or 200 mm in diameter and holding capacity of 24 inches or 600 mm of rainfall. The gage is usually examined once daily at a regular time and the rainfall catch (if any) measured by depth in inches or millimeters.

**GAGE, STREAM** Instruments that measure the depth of the water in a stream *(33).*

**GAGE, TIPPING BUCKET** An electrical-mechanical device that accumulates a small, precise precipitation gage amount of rainfall before tipping to spill the water. The spill triggers an electrical signal that is counted and/or transmitted to a base station. Each count represents a preset rainfall accumulation amount (1 mm) and is tagged with the time of occurrence *(33).*

**GAGING** The act of measuring the flow of streams.

**GAGING/SAMPLING** Refers to the measurement of precipitation, flow and/or water quality parameters *(1).*

**GAGING STATION** A particular site on a stream, canal, lake, or reservoir where systematic observations of gage height or discharge are obtained *(52).* Used synonymously with Gage *(37).* See Stream-Gaging Station.

A location on a stream where measurements of stage or discharge are customarily made. The location includes a reach of channel through which the flow is nearly uniform, a control downstream from this reach and usually a small building to house the recording instruments. A plane on a stream, including suitable gaging appurtenances, where systematic records of stream-flow are collected.

**GAGING, CHEMICAL** See Chemical Gaging.

**GAINING STREAM** See Stream, Gaining *(37).*

**GAUSS-LAPLACE DISTRIBUTION** Normal Distribution *(34).* See Probability Distribution.

**GAUSSIAN DISTRIBUTION** Normal Distribution *(34).* See Probability Distribution.

**GENERAL SCOUR** See Scour, General.

**GENERALIZED SKEW COEFFICIENT** A skew coefficient derived by a procedure which integrates values obtained at many locations *(34).*

**GEOMORPHOLOGY** A study of the structure and formation of the earth's features. That branch of both physiography and geology that deals with the form of the earth, the general configuration of its surface and the changes that take place due to erosion of the primary elements and in the buildup of erosional debris. Compare with Fluvial Geomorphology. See Morphology, Geomorphology, and Morphology Problems.

**GEOTEXTILE FILTER** See Synthetic Filter.

**GLACIER** Bodies of land ice that consist of recrystallized snow accumulated on the surface of the ground and move slowly downslope *(46).*

**GRADE** Three definitions are suggested: 1. the longitudinal slope of a road, channel, or natural ground; 2. the finished surface of a canal bed, road bed, top of embankment, or bottom of excavation; 3. any surface prepared for the support of such things as conduit paving, ties, or rails.

**GRADE CONTROL STRUCTURE** Structure placed bank to bank across a stream channel (usually with its central axis perpendicular to flow) for the purpose of controlling bed slope and preventing scour or headcutting. Compare with Drop and Sill.

**GRADED FILTER** An aggregate filter which is proportioned by particle size to allow water to pass through at a specified rate while preventing the migration of fine-grained soil particles without clogging.

**GRADED STREAM** See Stream, Poised.

**GRADIENT** Change of elevation, velocity, pressure, or other characteristics per unit length; slope. Compare with Energy Grade Line.

**GRADUALLY VARIED FLOW** See Flow, Gradually Varied.

**GRATE INLET** See Inlet, Grate.

**GRAVEL** Particles, usually of rock, whose diameter is between .07 inch and 2.5 inches or 2 mm and 64 mm. The term gravel is also applied to a mixture of sizes (gravel with sand or gravel with cobbles) in which the dominant or modal fraction is the gravel size range: FHWA *Highways in the River Environment Manual.*

**GRAVITY DAM** A dam depending solely on its weight to resist water pressure and any momentum forces.

**GREATEST OPENING FLOOD** See Flood, Greatest Opening.

**GREENBELT** See Buffer Zone.

**GRINGORTEN DISTRIBUTION** See Probability Distribution.

**GROIN** See Dike, Riparian Spur.

**GROOVE, DRIP** See Drip Groove.

**GROUNDWATER** Subsurface water occupying the saturation zone, from which wells and springs are fed. A source of base flow in streams. In a strict sense the term applies only to water below the water table. Water at and below, the water table; basal or bottom water; phraetic water. Used also in a broad sense to mean all water below the ground surface.

Water in the ground that is in the zone of saturation, from which wells, springs and groundwater runoff are supplied *(47).* The Groundwater Subcommittee offers numerous definitions: 1. that part of the subsurface water that is in the saturated zone; 2. loosely, all subsurface water as distinct from surface water; 3. all water which occurs below the land surface [and] it includes both water within the unsaturated and saturated zones; 4. means water below the land surface in a zone of saturation; 5. groundwater is the water contained within an aquifer; 6. all water which occurs below the land surface; 7. all subsurface water as distinct from surface water; 8. subsurface water that fills available openings in rock or soil materials to the extent that they are considered water-saturated; 9. water below the land surface in a zone of saturation; 10. water in a saturated zone or stratum beneath the surface of land or water.

The water contained in interconnected pores located below the water in an unconfined aquifer or located in a confined aquifer *(20).*

The water in the saturated zone beneath the water table. A source of base flow in streams *(51).*

**GROUNDWATER DISCHARGE** That part of the discharge from a drainage basin that occurs through the groundwater. The term "underflow" is often used to describe the groundwater outflow that takes place in valley alluvium (instead of the surface channel) and thus is not measured at a gaging station *(37).* See Underflow.

**GROUNDWATER OUTFLOW** See Groundwater Discharge.

**GROUNDWATER, PERCHED** The Groundwater Subcommittee offers this definition *(22).* [Unconfined] Groundwater separated from an underlying body of groundwater by an unsaturated zone. Its water table is a perched water table. Perched groundwater is held up by a perching bed whose permeability is so low that water percolating downward through it is not able to bring water in the underlying unsaturated zone above atmospheric pressure.

**GROUNDWATER RUNOFF** That part of the runoff which has passed into the ground, has become groundwater and has been discharged into a stream channel as spring or seepage water. See also Base Runoff and Direct Runoff *(37).*

**GROUT** A fluid mixture of cement and water or of cement, sand and water used to fill joints and voids.

**GUIDE** A book that explains, outlines, or gives practical instruction in some subject according to the *Reader's Digest Great Encyclopedic Dictionary.* Something that provides a person with guiding information according to *Webster's New Collegiate Dictionary.* A manual containing guidelines on some subject. Compare with Guideline.

**GUIDE BANK** Formerly termed spur dike. Relatively short embankments generally in the shape of a quarter of an ellipse and constructed at the upstream side (and sometimes the downstream

side) of either or both bridge ends as an extension of the abutment spillslope. The purpose is to align the flow with the bridge opening so as to decrease scour at the bridge abutment by spreading the flow and any resultant scour throughout the bridge opening. May also be a training dike (usually when constructed downstream). See Dike, Training. Sometimes referred to using the outdated term "Spur Dike." See Dike, Spur.

**GUIDELINE** Any suggestion, rule, etc that guides, directs, or sets a standard: [such as] government guidelines designed to regulate wage or price increases according to *Reader's Digest Great Encyclopedic Dictionary.* Compare with Guide.

**GUMBEL DISTRIBUTION** See Probability Distribution.

**GUTTER** That portion of the roadway section adjacent to the curb which is utilized to convey stormwater runoff.

**HABITAT** The area or type of environment in which an organism or biological population normally lives or occurs.

**HARD POINT** A channel bank protection technique whereby "soft" or erodible materials are removed from a bank and replaced by stone or compacted clay. Some hard points protrude a short distance into the channel to direct erosive currents away from the bank. See Dike, Riparian Spur. Hard points also occur naturally along channel banks as passing currents remove erodible materials leaving nonerodible materials exposed. Natural hard points also result from clay plugs deposited near the cutoff points when a meander is cutoff, as well as from such things as rock outcrops.

**HAZEN DISTRIBUTION** See Probability Distribution.

$\mathbf{H_E}$ Entrance head loss, m.

**HEAD** The height of water above any point, plane, or datum of reference. Used also in various computations, such as energy head, entrance head, friction head, static head, pressure head, lost head, etc. The height of the free surface of a body of water above a given point.

**HEAD, ELEVATION** The elevation of a given point in a column of liquid above a datum *(22).*

**HEAD, ENERGY** The elevation of the hydraulic grade line at any section plus the velocity head of the mean velocity of the flow in that section. The energy head may be referred to any datum or to an inclined plane, such as the bed of a conduit. Also the total head above a datum at any cross section. Compare with Energy Grade Line.

**HEAD, FRICTION** See Friction Loss.

**HEAD, HIGH** A general term applied to culvert flow denoting that culvert entrance is submerged with "free getaway" (no adverse tailwater conditions) downstream from culvert. Compare with Head, Low.

**HEAD, LOST** The energy of a given flow that is lost (converted into heat and, therefore useless) as a result of friction, eddies and impact expressed as a head; that is, as the height through which that flow would have to fall to produce an equivalent amount of energy.

**HEAD, LOW** A general term applied to culvert flow, denoting that the culvert entrance is not submerged. Compare with Head, High.

**HEAD, PIEZOMETRIC** Elevation plus pressure head, total head at any cross section minus the velocity head at that cross section; equivalent to water surface elevation in open channel flow; equivalent to elevation of hydraulic grade line at any point.

**HEAD, PRESSURE** Hydrostatic pressure expressed as the height of a column of water that pressure can support at the point of measurement *(22)*. Compare with Head, Static; and Head, Total. The head at any point in a conduit represented by the height of the hydraulic grade line above that point.

**HEAD, STATIC** The height above a standard datum of the surface of a column of water (or other liquid) that can be supported by the static pressure at a given point. The static head is the sum of the elevation head and the pressure head *(22)*. Compare with Head, Total; and Head, Pressure.

The total head without deduction for velocity head or losses; for example, the difference in elevation of headwater and tailwater of a power plant. Compare with Head, Piezometric.

**HEAD, TOTAL** The total head of a liquid at a given point is the sum of three components: 1. the elevation head, which is equal to the elevation of the point above a datum; 2. the pressure head, which is the height of a column of static water that can be supported by the static pressure at the point; and 3. the velocity head, which is the height at which the kinetic energy of the liquid is capable of lifting the liquid *(22)*.

**HEAD, VELOCITY** The distance a body must fall freely under the force of gravity to acquire the velocity it possesses; the kinetic energy, in feet of head or meters of head, possessed by a given velocity. In flowing water, the velocity squared divided by twice gravity ($V^2/2g$).

**HEADCUT** The relatively fast drop (as compared to the average channel bed profile slope through a channel reach) in a channel bed profile that is, or has been, headcutting. See Headcutting.

**HEADCUTTING** Channel degradation associated with abrupt changes in the bed elevation (headcut) that migrates in an upstream direction. Channel bed erosion moving upstream through a basin indicating that a readjustment of the basin's profile slope, channel discharge and sediment load characteristics is taking place. Headcutting may be evidenced by the presence of waterfalls or rapidly moving water through an otherwise placid stream or river, provided there is flow present. In dry channels the presence of a relatively steep drop in the channel bed in an erodible channel is evidence of a headcut. Headcuts may range from 0.3 m or less to 3 m or more. Headcutting often leaves channel banks in an unstable condition as it

progresses through a reach as evidenced by large amounts of mass wasting. Compare with Nick Point. See Mass Wasting.

**HEADLOSS** A loss of energy in a hydraulics system.

**HEADWALL** The structural appurtenance usually applied to the end of a culvert inlet and outlet or storm drain outlet to retain an adjacent highway embankment and protect the culvert ends or storm drain outlet and highway embankment or storm drain outfall from bank erosion and channel bed scour.

**HEADWATER** See Headwater Depth.

**HEADWATER DEPTH** Depth of water above the inlet flow line at the entrance of a culvert or similar structure. Depth of water upstream of a contraction such as occurs at a bridge or similar structure. Natural flow depth plus backwater caused by a drainage structure.

That depth of water impounded upstream of a culvert, bridge, or similar contracting structure due to the influence of the structures constriction, friction and configuration. The water depth upstream from a structure. Compare with Allowable Headwater Depth.

**HEADWATER ELEVATION** Water surface elevation of the headwater.

**HEADWATER HEIGHT** See Headwater Depth.

**HEADWATERS** The uppermost reaches for the source of water flowing in a stream. The geographic regions near the divide of a watershed.

**HECTARE-METER** The quantity of water required to cover an area of 1 ha to a depth of 1 m and is equal to 9996 $m^3$ or about 10 x $10^6$ L. Common abbreviation is ha·m. See Acre-feet.

**HELICAL FLOW** Three-dimensional movement of water particles along a spiral path in the general direction of flow. These secondary currents are of most significance as flow passes through a bend; their net effect is to remove soil particles from the cut bank in a bendway and deposit this material on a point bar, alternate bar or middle bar.

**HERBACEOUS VEGETATION** Vegetation that has a fleshy stem as distinguished from the woody tissue of shrubs and trees and that generally dies back at the end of each growing season.

**HERBICIDE** A substance used to destroy plants, especially weeds.

**HETEROGENEITY** A characteristic of a medium in which material properties vary from point to point *(22).* Pertaining to a substance having different characteristics in different locations (A synonym is nonuniform) *(20).*

$\mathbf{H_f}$ The friction headloss, m.

**HIGH HEAD** See Head, High.

**HIGHWATER MARK** A mark left as evidence of the height to which a flood reached; usually in the form of such things as deposited sediment, debris and detritus. See Drift Line.

**HIGHWATER ELEVATION** The water surface elevation that results from the passage of flow. It may be an "observed highwater mark elevation" as a result of someone actually viewing and recording a runoff event or a "calculated highwater elevation" as part of a design process. See Indirect Method (of Flow Measurement).

**HIGHWAY** See Road and Street.

**HIGHWAY ACTION** See Action (Highway).

**HISTORIC WETLAND LOSSES** See Wetlands, Historic Losses.

**HISTORICAL FLOOD** A past flood event of known or estimated magnitude. A known flood event predating systematic flow measurements at a given site.

**HISTORICAL SERIES** A list of all actual storms (or floods) that caused flood damage in a watershed, in a given period of years, with the data of each storm of flood being known *(51).*

$\mathbf{H_L}$ Total energy head loss, m. See Head, Energy; and Energy Grade Line.

$\mathbf{h_o}$ The height of the hydraulic grade line above the outlet invert of a drainage structure, m.

**HOMOGENEOUS** See Homogeneity.

**HOMOGENEITY** Records from the same populations *(34).* A characteristic of a medium in which material properties are identical everywhere *(22).* Pertaining to a substance having identical characteristics everywhere (a synonym is Uniform) *(20).*

**HOOK GAGE** See Gage, Hook.

**HYDRAULIC** Moved, operated, or effected by means of water; relating to hydraulics as in engineer [note that *Webster's* uses the plural form when referring to an engineer]; relating to water or other liquid in motion as in erosion; operated by the resistance offered or the pressure transmitted when a quantity of water, oil, or other liquid is forced through a comparatively small orifice or through a tube. These definitions have been taken from *Webster's New Collegiate Dictionary.* Compare with Hydraulics.

**HYDRAULIC BORE** See Bore, Hydraulic.

**HYDRAULIC ELEMENTS** The depth, area, perimeter, mean depth, hydraulic radius, velocity, energy and other flow related quantities pertaining to a particular stage of flowing water.

**HYDRAULIC FRICTION** A force-resisting flow which is exerted on contact surface between a stream and its containing channel. It usually includes the normal eddies and cross-currents attendant upon turbulent flow occasioned by the roughness characteristic of the boundary surface, moderate curvature and normal channel variations. Wherever possible, the effects of excessive curvature, eddies and impact, obstructions and pronounced channel changes are segregated from the effects of hydraulic friction.

**HYDRAULIC GRADELINE** In a closed conduit, a line joining the elevation to which water could stand in risers. In an open conduit, the hydraulic gradeline is the water surface; piezometric head line. Compare with Energy Gradeline. See Head, Piezometric.

A profile of the piezometric level to which the water would rise in piezometer tubes along a pipe run. In open channel flow, it is the water surface.

**HYDRAULIC GRADIENT** The slope of the hydraulic grade line; the slope of the water surface in uniform, open channel flow.

The change in total head with a change in distance in a given direction. The direction is that which yields a maximum rate of decrease in head *(20).* The slope of the hydraulic gradeline through a channel reach or drainage structure. Compare with Energy Gradeline and Friction Slope.

**HYDRAULIC HEAD** See Head.

**HYDRAULIC JUMP** The sudden and usually turbulent passage of water from a stage below critical depth (supercritical flow) to a stage above critical depth (subcritical flow) during which the velocity passes from supercritical to subcritical. It represents the limiting conditions of the water surface curve (or profile) wherein it tends to become perpendicular to the stream bed.

A hydraulic phenomenon, in open channel flow, whereby supercritical flow is converted to subcritical flow. This can result in a relatively abrupt and turbulent rise in the water surface. See Depth, Conjugate; Depth, Alternate; and Critical Depth.

**HYDRAULIC MODEL** A small-scale physical representation of a flow situation.

**HYDRAULIC PERFORMANCE CURVE** Computed estimates of how a drainage facility will perform over a wide range of discharges. Commonly these may include discharge and recurrence interval versus headwater, velocity, scour and/or stage (depth of flow).

**HYDRAULIC PROBLEM** The effect from such things as channel flow, tidal flow, or wave action on a crossing such that traffic is immediately or potentially disrupted or some other detrimental effect is expected or caused.

**HYDRAULIC RADIUS** In simplest terms, the cross section area of a stream divided by its wetted perimeter. The cross section area of a stream of water (normal to flow) divided by the length

of that part of its periphery in contact with its containing conduit; the ratio of area to wetted perimeter. A measure of the boundary resistance to flow, computed as the quotient of cross section area of flow divided by the wetted perimeter. For wide shallow flow, the hydraulic radius can be approximated by the average flow depth. Compare with Wetted Perimeter.

**HYDRAULIC ROUGHNESS** A composite of the physical characteristics which influence the flow (or conveyance) of water across the earth's surface, whether natural, channelized, or in a conduit. It affects both the time response of a watershed and drainage channel, or conduit as well as the channel or conduit storage characteristics. Compare with Manning's n.

**HYDRAULIC STRUCTURE** A facility used for such things as to impound, accommodate, convey, or control the flow of water, such as a dam, weir, intake, culvert, channel, or bridge.

**HYDRAULIC-FILL DAM** A dam composed of such materials as earth, sand and gravel, sluiced into place; generally the fines are washed toward the center for greater imperviousness.

**HYDRAULICS** The applied science concerned with the behavior and flow of liquids, especially in pipes, channels, structures and the ground. In highway drainage, the science addressing the characteristics of fluid mechanics involved with the flow of water in or through drainage facilities.

A branch of science that deals with practical applications (as the transmission of energy or the effects of flow) of water or other liquid in motion according to *Webster's New Collegiate Dictionary*. Compare with Hydraulic.

**HYDRAULICS DESIGNER** A hydraulics engineer or in some cases a technician who designs hydraulics structures under the supervision of a more experienced hydraulics engineer.

**HYDRAULICS ENGINEER** An engineer whose practice is limited primarily to hydraulics and river mechanics. See Hydraulic, Hydraulics, and River Mechanics.

**HYDRIC SOIL** Soil that, in its undrained condition, is saturated, flooded, or ponded long enough during a growing season to develop an anaerobic condition that supports the growth and regeneration of hydrophytic vegetation.

Some wetland biologists and/or regulators may prefer a soil that is saturated, flooded or ponded long enough during the growing season to develop anaerobic conditions in the upper part, which influences plant growth.

**HYDROGRAPH** The graph of stage or discharge versus time. A graph showing, for a given point on a stream or for a given point in any drainage system, the discharge, stage, velocity or other property of water with respect to time.

A graph showing [relating] stage, flow, velocity, or other property [characteristics] of water with respect to time *(37)* [Groundwater Subcommittee *(22)*].

A graph that shows some property of groundwater or surface water as a function of time *(20).* A graph showing, for a given point on a stream or for a given point in any drainage system, the discharge, stage, velocity, or other property of water with respect to time *(51).* A chart that shows the relationship between streamflow or water elevation to time at a certain location *(33).* Compare with Storm Hydrograph and Hyetograph.

**HYDROGRAPH, COMPOSITE** A plot of mean daily discharges for a number of years of record on a single year time base for the purpose of showing the average occurrence of high and low flows.

**HYDROGRAPH, DISTRIBUTION** A unit hydrograph of direct runoff modified to show the portions of the volume of runoff that occurs during successive equal units of time *(29).*

**HYDROGRAPH, STORM** A graph of the discharge of a stream over the time period when, in addition to direct precipitation, overland flow, interflow and return flow are adding to the flow of the stream. The storm hydrograph will peak due to the addition of these flow elements *(20).* Compare with Hydrograph.

**HYDROGRAPH, SYNTHETIC** A hydrograph determined from empirical rules. Usually a hydrograph based on the physical characteristics of the basin. A graph developed for an ungaged drainage area, based on known physical characteristics of the watershed basin.

**HYDROGRAPH, UNIT** The hydrograph of direct runoff from a storm uniformly distributed over the drainage basin during a specified unit of time; the hydrograph is reduced in vertical scale to correspond to a volume of runoff of 1 inch from the drainage basin *(2).*

The hydrograph of surface runoff (not including groundwater runoff) on a given basin due to an active rain falling for a unit of time *(61).* A discharge hydrograph coming from 1 inch of direct runoff distributed uniformly over the watershed, with the direct runoff generated at a uniform rate during the given storm duration. A watershed may have 1-hour, 2-hour, etc. unit hydrographs *(51).*

A typical streamflow hydrograph of a river basin produced by 1 inch of surface runoff uniformly distributed over the watershed during a specified period of time *(33).*

A hydrograph of a direct runoff resulting from 1 inch of effective rainfall generated uniformly over the watershed area during a specified period of time or duration. The discharge hydrograph resulting from 1 mm of direct runoff generated uniformly over the tributary area at a uniform rate during a specified period of time. Compare with Hyetograph.

**HYDROLOGIC AREA** A geographic area having homogeneous topographical, soil, vegetation and meteorological properties as they relate to its flood-frequency relationship; i.e., subareas within such an area have very similar flood-frequency relationships.

**HYDROLOGIC BUDGET** An accounting of the inflow to, outflow from and storage in, a hydrologic unit, such as a drainage basin, aquifer, soil zone, lake, reservoir, or irrigation project *(37).*

**HYDROLOGIC CHARACTERISTICS OF WATERSHED** The parameters that control the runoff [and flood-frequency relationship] in a watershed. These include [such things as] the basin size, ground cover conditions, slope of the land, stream lengths [, stream slopes, soil infiltration characteristics], topographic and geologic features and physical features constructed by man that alter runoff *(33).*

**HYDROLOGIC CYCLE** A convenient term to denote the circulation of water from the sea, through the atmosphere, to the land, and thence, with many delays, back to the sea by overland and subterranean routes and in part by way of the atmosphere; also, the many short circuits of the water that are returned to the atmosphere without reaching the sea *(48).*

**HYDROLOGIC EQUATION** The equation balancing the hydrologic budget *(37).* An expression of the law of mass conservation for purposes of water budgets which may be stated as inflow equals outflow plus or minus changes in storage.

**HYDROLOGIC MODELS** Mathematical equations, algorithms and/or logic that represents the rainfall runoff process in a watershed *(33).*

**HYDROLOGIC SOIL GROUP** A group of soils having the same runoff potential under similar storm and cover conditions *(51).* Compare with Hydrologic Soil-Cover Complex.

**HYDROLOGIC SOIL-COVER COMPLEX** A combination of a hydrologic soil group and a type of cover *(51).* Compare with Hydrologic Soil Group.

**HYDROLOGIC STUDIES** Studies to determine the runoff and flood characteristics to be expected at a highway drainage site. A most important step prior to the hydraulic design of a highway drainage structure. Such studies are necessary for determining the rate of flow, runoff, or discharge that the drainage facility will be required to accommodate.

**HYDROLOGIC UNIT** A geographic area representing part or all of a surface drainage basin or distinct hydrologic feature as delineated by the Office of Water Data Coordination on the State Hydrologic Unit Maps; each hydrologic unit is identified by an eight digit number *(52).*

**HYDROLOGIST** A person who studies water on or through the earth's surface *(33).*

**HYDROLOGY** The science and study concerned with the occurrence, circulation, distribution and properties of the waters of the earth and its atmosphere, including precipitation, runoff, and groundwater. The science dealing with the waters of the earth in their various forms: precipitation, evaporation, runoff and groundwater. In highway drainage, the science dealing with the runoff and flood-producing process. In practice the study of the water of the oceans

and the atmosphere is, in some cases, considered part of the sciences of oceanography and meteorology. Hydrologic (adj.). Hydrologically (adv.).

The science encompassing the behavior of water as it occurs in the atmosphere, on the surface of the ground and underground *(2).* The science that relates to the water of the earth *(47).* The science treating [dealing with] the waters of the earth, their occurrence, distribution and movements *(35).*

The science that deals with the occurrence and behavior of water in the atmosphere, on the ground and underground. Rainfall intensities, rainfall interception by trees, effects of crop rotations on runoff, floods, droughts, the flow of springs and wells, are some of the topics studied by a hydrologist *(51).*

**HYDROLOGY, DETERMINISTIC** [The] deterministic process of hydrology is the result of physical, chemical and biological deterministic laws, but primarily the result of fluid mechanics and thermodynamics laws and regularities *(74).* Compare with Hydrology, Stochastic.

**HYDROLOGY LOSS** In hydrology, a loss for one purpose is usually a gain for another, so that the net effect may be more important than the loss. At various times, evapotranspiration, initial abstraction, infiltration, surface storage, direct runoff, seepage, etc., have been called losses according to the aims of a water user. See Water Loss *(51).*

**HYDROLOGY, STOCHASTIC** [The] laws of chance and the sequence of various variables that describe [the] ...random phenomena [of such things as] precipitation, evaporation, runoff, groundwater levels, sediment transport, lake levels, snow and ice accumulation and melt, water quality properties and properties of porous environments, river basin geomorphic forms, etc. ...are stochastic processes *(74).*

**HYDROMETEOROLOGY** The branch of hydrology concerned with the relationship of precipitation and runoff to climate and weather.

**HYDROMETRY, CHEMICAL** See Chemical Gaging.

**HYDROPHYTE** Plant life growing in water or on a substrate that is periodically flooded, causing deficiency of oxygen. These plants are typically found in wetlands and other aquatic habitats. Compare with Hydrophytic Vegetation.

**HYDROPHYTIC VEGETATION** A plant growing in water or a substrate that is at least periodically deficient in oxygen during a growing season as a result of excessive water content. Compare with Hydrophyte.

**HYDROSTATIC PRESSURE** See Pressure, Hydrostatic.

**HYETOGRAPH** Graphical representation of rainfall intensity against time *(37).* A graph plotting rainfall amounts or intensities during various time increments versus time *(1).* A graphical

representation of average rainfall, rainfall-excess rates or volumes over specified areas during successive units of time during a storm. Compare with Hydrograph.

**HYPOLIMNION** The lower layer, noncirculating water, of a stratified lake. Compare with Thermal Stratification.

**HW** Acronym for Headwater Depth. Compare with AHW.

**ICE, FRAZIL OR LOLLY** See Frazil Ice.

**ICING** Masses or sheets of ice formed on the frozen surface of a river or floodplain. When shoals in the river are frozen to the bottom or otherwise dammed, water under hydrostatic pressure is forced to the surface where it freezes.

**IFLOWS** [Acronym] Abbreviation for Integrated Flood Observing and Warning System. A network of [100 county] automated local flood warning systems. The county systems consist of automatic radio reporting rain gages, radio relays or repeaters (if required), a radio receiver and system software. Each county is usually capable of collecting and displaying real-time precipitation data. All of the counties are linked to a designated State Emergency Operation Center and offices so data can be retrieved by any county, State, or NWS office *(33)*.

**IMPACT** The striking together of two masses. When particles or streams of water suffer impact, energy losses result. According to *Webster's New Collegiate Dictionary,* the force of impression or operation of one thing on another; Effect.

Impact can also refer to the short- and long-range changes and their significance to surface waters and related social and environmental relationships resulting from an effect(s) brought about by a highway drainage facility. Compare with Effect.

**IMPACT LOSS** The head lost as a result of the impact of particles of water; included in and scarcely distinguishable from eddy loss.

**IMPERMEABLE STRATA** A strata in which texture is such that water cannot move perceptibly through it under pressures ordinarily found in subsurface water.

**IMPERVIOUS** Impermeable to the movement of water.

**IMPERVIOUSNESS** That quality or condition of a material that minimizes percolation.

**IMPROVED INLET** See Inlet, Improved.

**INCISED CHANNEL** Those channels which have been cut relatively deep into underlying formations by natural processes. Characteristics include relatively straight alignment and high, steep banks such that overflow rarely occurs, if ever. See Stream, Incised.

**INCISED REACH** The stretch of river with an incised channel that only rarely overflows its banks. See Incised Channel.

**INCISED STREAM** See Stream, Incised.

**INCLINED GAGE** See Gage, Inclined.

**INCOMPLETE RECORD** A streamflow record in which some peak flows are missing because they were too low or high to record or the gage was out of operation for a short period because of flooding *(34).*

**INDEPENDENT VARIABLE** See Variable.

**INDEX OF LOSS** See Wetland Loss Index.

**INDICATOR** A device that shows by such things as an index, pointer and/or dial, the instantaneous value of such quantities as depth, pressure, velocity, or the movements or positions of water-controlling devices; a gage. Compare with Recorder, Register, and Gage.

**INDIRECT METHOD (OF FLOW MEASUREMENT)** A method of determining peak discharge, other than by current meter such as with the slope-area, contraction, culvert, dam, or critical-depth methods; methods are usually based on survey of highwater marks following the flood.

**INERT** Unable to move or react.

**INFILTRATION** The flow of a fluid into a substance through pores or small openings. It connotes flow into a substance in contradistinction to the word percolation, which connotes flow through a porous substance *(26).*

The downward entry of water into the soil or rock *(22).*

Rainfall minus interception, evaporation and surface runoff. The part of rainfall that enters the soil *(51).*

That part of rainfall that enters the soil. The passage of water through the soil surface into the ground. Compare with Percolation.

**INFILTRATION CAPACITY** The maximum rate at which a soil or rock is capable of absorbing water or limiting infiltration *(22).* The maximum rate at which the soil, when in a given condition, can absorb falling rain or melting snow *(25).*

The maximum rate at which infiltration can occur under specific conditions of soil moisture. For a given soil, the infiltration capacity is a function of the water content *(20).*

**INFILTRATION INDEX** An average rate of infiltration, in inches per hour, equal to the average rate of rainfall such that the volume of rainfall at greater rates equals the total direct runoff *(38).*

**INFILTRATION RATE** The rate at which water enters the soil under a given condition. The rate is usually expressed in inches per hour or inches per hour; feet per day or meters per day; cubic feet per second or cubic meters per second.

**INFLOW** The rate of discharge arriving at a point (in a stream, structure, or reservoir).

**INFLOW DESIGN FLOOD** An inflow design flood is the flood hydrograph (in combination with a starting lake level, spillway and outlet works operation and freeboard [where applicable]) that establishes the top of dam elevation [or highway drainage facility geometry and size] required to meet hydrologic requirements. In some older documents [of the Corps of Engineers] this may be referred to as a spillway design flood *(30).*

**INFLUENT STREAM** See Stream, Losing *(20).*

**INITIAL ABSTRACTION** When considering Surface Runoff [, the initial abstraction] $I_a$ is all the rainfall before runoff begins. When considering direct runoff, $I_a$ consists of interception, evaporation and the soil-water storage that must be exhausted before direct runoff may begin. Sometimes called "initial loss" *(51).* See Loss, Hydrology.

**INITIAL LOSS** See Initial Abstraction.

**INLET** Consider four definitions: 1. a surface connection to a closed drain; 2. a structure at the diversion end of a conduit; 3. the upstream end of any structure through which water may flow; 4. an inlet structure for capturing concentrated surface flow. Inlets may be located in such places as along the roadway, a gutter, the highway median, or a field.

**INLET, COMBINATION** Drainage inlet usually composed of two or more inlet types, e.g., such combinations as curb opening and grate inlet, grate, and slotted drain inlet.

**INLET CONTROL** A condition where the relation between headwater elevation and discharge is controlled by the upstream end of any structure through which water may flow. For example, a culvert on steep slope and flowing part full as in inlet control. Compare with Outlet Control.

**INLET, CURB OPENING** Drainage inlet consisting of an opening in a curb.

**INLET, DROP** Drainage inlet with a horizontal or nearly horizontal opening that is generally flush with the street or land surface.

**INLET EFFICIENCY** The ratio of flow intercepted by an inlet to the total flow.

**INLET, FLANKING** Inlets placed upstream and on either side of a storm drain inlet that is located at the low point in a sag-vertical curve. The purpose of these inlets is to intercept debris as the longitudinal gutter slope decreases and to act as an emergency relief for the sump inlet at the low point of the vertical curve.

**INLET, FLARED** A specially fabricated culvert end appurtenance at the inlet and outlet, or a special end feature of box culverts where the walls flare outward from the culvert sides at the culvert inlet and outlet. This type of inlet is effective in reducing the calculated headwater caused by less efficient inlet types where inlet control prevails. It also serves to retain the roadway embankment. The walls form an angle to the centerline of the culvert. A type of culvert design having an inlet or outlet larger than the main barrel. Compare with Improved Inlet and End Section.

**INLET, FLUSH** Culvert barrel whose entrance (or outlet) does not project beyond the plane of the slope or headwall.

**INLET GRATE** Drainage inlet composed of a grate in the roadway section or at the roadside, in a low point, swale, or ditch.

**INLET, IMPROVED** Flared, depressed, or tapered culvert inlets which decrease the amount of energy needed to pass the flow through the inlet and thus increase the capacity of culverts with inlet control or supercritical flow.

**INLET, MITERED** A flush-entrance culvert where the barrel is mitered to the slope of the embankment.

**INLET, PARALLEL WING WALL** A culvert with wing walls parallel to the culvert centerline.

**INLET, PROJECTING** Culvert barrel projects beyond the plane of the slope or headwall; sometimes referred to as a "re-entrant" entrance.

**INLET, SLOTTED DRAIN** Drainage inlet composed of a continuous slot built into the top of a pipe which serves to intercept, collect and transport the flow. Often used in conjunction with a single grate inlet for clean out access.

**INLET, SQUARE-EDGED** An approximately 90□ corner formed by the inside of the barrel and the upstream end of the culvert. Small chamfers ordinarily used in concrete construction are considered as producing a square-edged entrance; i.e., no hydraulic improvement.

**INLET, SUBMERGED** See Submerged Inlet.

**INLET, SUMP** Inlet located at the low point in a sag-vertical curve.

**INLET, TAPERED** A type of culvert design having an entrance face area larger than the main barrel.

**INLET TIME** The time required for stormwater to flow from the most distant point in a drainage area to the point at which it enters a storm drain.

**INSTANTANEOUS DISCHARGE** A discharge at a given moment. The discharge at a particular instant of time *(52).*

**INSULATED STREAM** See Stream, Insulated *(37).*

**INTEGRATION METHOD** A means of determining the mean velocity at a vertical depth of a stream by noting the total number of revolutions of a current meter vane and the time consumed, while the meter is slowly lowered from the surface to the bed and returned one or more times.

**INTENSITY** The rate of rainfall upon a watershed, usually expressed in inches per hour.

**INTERCEPTING CHANNEL** A channel excavated at the top of earth cuts, or at the foot of slopes, or at other critical places to intercept surface flow; sometimes termed a catch-drain.

**INTERCEPTION** The process and the amount of rain or snow stored on leaves and branches and eventually evaporated back to the air. Interception equals the precipitation on the vegetation minus stemflow and throughfall *(24).* See Stemflow and Throughfall.

Precipitation retained on plant or plant residue surfaces and finally absorbed, evaporated, or sublimated. That which flows down the plant to the ground is called stemflow and not counted as true interception *(51).*

**INTERFLOW** The lateral movement of water in the unsaturated zone during and immediately after a precipitation event. The water moving as interflow discharges directly into a stream or lake *(20).*

**INTERMITTENT STREAM** See Stream, Intermittent.

**INTERRUPTED STREAM** See Stream, Interrupted.

**INTERSTICE** A narrow or small space between things or parts, such as riprap; crevice.

**INUNDATE** To cover or fill as with a flood.

**INVERT** The flow line in a channel cross section, pipe, or culvert. The lowest point in the channel cross section or at flow control devices such as weirs or dams. The floor, bottom, or lowest part of the internal cross section of a conduit. Compare with Soffit.

**INVERTEBRATES** Animals that have no backbone or spinal column.

**INVERTED SIPHON** See Sag Culvert.

**IRRIGATED AREA** The gross farm area upon which water is artificially applied for the production of crops, with no reduction for access roads, canals, or farm buildings *(62).*

**IRRIGATION** The controlled application of water to arable lands to supply water requirements not satisfied by rainfall *(27).*

**IRRIGATION EFFICIENCY** The percentage of water applied that can be accounted for in soil-moisture increase *(54).*

**IRRIGATION POOL** Reservoir storage used to store water for release as needed for irrigation.

**IRRIGATION REQUIREMENT** The quantity of water, exclusive of precipitation, that is required for crop production. It includes surface evaporation and other economically unavoidable wastes *(8).*

**IRRIGATION, SUPPLEMENTAL** Commonly, irrigation as carried on in ...humid areas. The term means that the irrigation water is supplementary to the natural rainfall source of moisture as in the arid and semi-arid West. Supplementary irrigation is used generally to prevent retardation of growth during periods of drought *(30).*

**IRRIGATION, SUPPLEMENTAL SOURCES** When irrigation water supplies are obtained from more than one source, the source furnishing the principal supply is commonly designated the primary source and the sources furnishing the additional supplies, the supplemental sources *(27).*

**ISLAND** A permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Some islands originate by establishment of vegetation on a bar and other originate by channel avulsion or at the junction of minor tributaries with a stream.

**ISOHYET** See Isohyetal Line.

**ISOHYETAL LINE** A line drawn on a map or chart joining points that receive the same amount of precipitation *(37).* A line on a map, connecting points of equal rainfall amounts *(51).*

**JACK** Devices cabled together in near parallel rows for flow control and protection of banks against lateral erosion. A Kellner jack has six mutually perpendicular arms formed by three steel struts rigidly fixed at the center. Steel jacks are strung with wire. Concrete jacks are made of three reinforced concrete beams bolted together at the midpoints, or sometimes cast as a monolithic unit. See Jack Field.

**JACK FIELD** Multiple rows of jacks tied together with cables. Generally one or more rows are parallel with the low flow bank or along a line where re-establishment of the low flow bank is desired. The remaining jack rows are placed perpendicular or at an angle to this row of low bank jacks. This combination of rows is termed a “jack field.” Jack fields may be placed outside a channel on the floodplain and/or within a channel where it is desired to re-establish a low flow channel bank and attendant floodplain. See Jack.

**JACKSON TURBIDITY UNIT** See Turbidity.

**JETTY** See Dike, Riparian Spur.

**JTU** Acronym for Jackson Turbidity Unit. See Turbidity.

**JUMP** See Hydraulic Jump.

**JURISDICTIONAL SURFACE WATERS** See Navigable Waters.

**KARST TOPOGRAPHY** Irregular topography characterized by sinkholes, streamless valleys and streams that disappear into the underground, all developed by the action of surface and underground water in soluble rock such as limestone.

**KINETIC ENERGY** See Energy, Kinetic.

**KINEMATIC VISCOSITY** Dynamic viscosity M (mu) divided by the mass density ρ (rho) of the liquid.

**KUTTERS FORMULA** An empirical formula expressing the value of the coefficient C, in the Chezy formula, in terms of the friction slope, hydraulic radius and a coefficient of roughness.

**LACUSTRINE** of or pertaining to a lake. See Lake. Compare with Limnology.

**LAG** See Lag Time.

**LAG TIME** [Lag time, $T_L$, is] Variously defined as time from beginning (or center of mass) of rainfall to peak (or center of mass) of runoff *(2).*

The difference in time between the centroid of the excess rainfall (that rainfall producing runoff) and the peak of the runoff hydrograph. Often estimated as 60 percent of the time of concentration ($T_L = 0.6T_c$) *(51).*

**LAKE** An area of open, relatively deep water sufficiently large to produce somewhere on its periphery a barren, wave-swept shore. See Lacustrine. Compare with Pond.

**LAKE AREA** See Lake Surface Area.

**LAKE, DIMICTIC** Lakes with a directly stratified epilimnion in summer and an inversely stratified epilimnion in winter. See Epilimnion.

**LAKE, EUTROPHIC** Lakes that have large supplies of nutrients and heavy layers of organic bottom sediment.

**LAKE, MEROMICTIC** Lakes in which some water remains partly or wholly unmixed with the main water mass at [during] circulation periods is said to be meromictic. The process leading to a meromictic state is termed meromixis. The perennially stagnant deep layer of a meromictic lake is called the monimolimnion. The part of a meromictic lake in which free circulation can occur is called the mixolimnion. The boundary between the monimolimnion and the mixolimnion is called the chemocline *(32).*

Lakes that have perennially stagnant water below a steep salinity gradient.

**LAKE, MESOTROPHIC** Lakes at an in-between stage nutritionally, with ecosystems functioning in a stable fashion, supporting a diverse community of aquatic plant and animal life.

**LAKE, MONOMICTIC** Lakes that have a temperature minimum of 4□C. Water circulates freely in the winter time and stratifies during the summer.

**LAKE, OLIGITROPHIC** Lakes where plant growth is limited by a low chemical concentration of nutrients.

**LAKE, OXBOW** Lakes formed by an oxbow. See Oxbow.

**LAKE, PLAYA** Lakes having a nearly level area at the bottom of a basin in an arid or semi-arid region; the beach or bank of a river. An intermittent pond or lake with no outlet. An undrained basin that, at times, becomes a temporary, shallow lake. The term "Playa" is more common. See Playa Lake.

**LAKE SHORE** See Coastal Zone and Shore.

**LAKE SURFACE AREA** That area of a lake outlined on the latest U.S. Geological Survey topographic map as the boundary of the lake and measured by a planimeter in hectares. In localities not covered by topographic maps, the areas are computed from the best maps available at the time planimetered. All areas shown are those for the stage when the planimetered map was made *(52).*

**LAKE TURNOVER** A seasonal change in heat distribution occurring in most North American Lakes. When air temperatures drop in the autumn, epilimnion, and hypolimnion temperatures equalize and achieve the same densities. As surface waters become cooler and heavier, they begin to mix with the water below. This mixing, combined with the movement due to winds and currents, results in a total turnover or reversing of the epilimnion and hypolimnion. Another turnover occurs in the spring in northern regions when ice melts and water temperatures become uniform throughout the lake.

**LAMINAR FLOW** See Flow, Laminar.

**LAMINAR VELOCITY** That velocity below which, in a particular conduit, laminar flow will always exist and above which the flow may be either laminar or turbulent depending on circumstances. See Flow, Laminar; and Flow, Turbulent.

**LAND LINE** Relates to communications: typically telephone service but includes any communication lines [land line].

**LAND MANAGEMENT AGENCY** A State or Federal government agency responsible for the process of planning, organizing, programming, coordinating, directing, and controlling land use actions for its government on lands for which it is responsible under the statutes, regulations, rules, and mandates that provide its authority.

**LAND TREATMENT MEASURE** A tillage practice, a pattern of tillage or land use, or any land improvement with a substantial effect of reducing runoff and sediment production or of improving use of drainage and irrigation facilities. Examples are contouring, improved crop rotations, controlled grazing, land leveling, [and] field drainage. In hydrologic computations, nonbeneficial measures (such as straight-row, poor-rotation corn) are included for convenience in evaluation. In general conservation work "land treatment measure" has a broader meaning that includes measures to improve the soil, control sheet erosion, [and] increase soil fertility *(51).* Compare with Development and Land Use.

**LAND USE** A term which relates to both the physical characteristics of the land surface and the human activities associated with the land surface *(1).* A highway facility to accommodate land uses is termed a land use structure or facility. See Land Use Facility.

A land classification. Cover, such as row crops or pasture, indicates a kind of land use. Roads may also be classified as a separate land use *(51).* Compare with Development and Land Treatment Measure.

**LAND USE FACILITIES** With highways, stockpasses and machinery passes under a road are often termed land use facilities. See Land Use.

**LAND-SURFACE DATUM** Is a datum plane that is approximately at the land surface at each groundwater observation well *(52).*

**LAPLACE-GAUSS DISTRIBUTION or LAPLACEAN DISTRIBUTION** See Normal Distribution.

**LATERAL** A conduit, ditch, canal, or channel conveying water diverted from a main conduit, canal or channel for delivery to distributaries; sometimes considered a secondary ditch. See Channel.

**LATERAL EROSION** See Erosion, Lateral.

**LATERAL-FLOW SPILLWAY** See Spillway, Side-Channel.

**LAUNCHING** Release of undercut material (stone riprap, rubble, slag, etc.) downslope; if sufficient material accumulates on the stream bank face, the slope can become effectively armored.

**LAW, WATER** See Water Law.

**LEFT BANK OF A CHANNEL** The left-hand bank of a channel when the observer is looking downstream. Compare with Right Bank of a Channel.

**LEGAL LIABILITY** Liability between litigants recognized and enforced by the courts. Compare with Liable.

**LEVEE** An embankment, generally landward of a top bank, that confines flow during highwater periods, thus preventing overflow into lowlands. A linear embankment outside a channel for containment of flow. Longer than a dike. Compare with Dike.

**LEVEL OF SIGNIFICANCE** The [statistically computed] probability of rejecting a hypothesis when it is in fact true. At a "ten-percent" level of significance the probability is 1/10 *(34).*

**LEVEL OF PROTECTION** This is the flood level at which flood damages and/or other adverse effects not eliminated by a project are considered relatively minor. Level of protection is a convenient term to express the flood control effectiveness of a project and may be based on flood discharge, stage, volume, duration, or any combination of factors that express project functional characteristics. Most commonly, it is expressed as the frequency (statistically estimated) of occurrence of the flood discharge and attendant levels thereof that the project will accommodate with acceptably minor residual flood damages in such an occurrence (e.g., ...the 1 percent chance or 100-year flood, etc). However, to fully define how a project is expected to function requires describing project impacts at several flood levels and locations. The terms "level of protection" and "project design flood" are not necessarily synonymous; separate reaches of a project may have different levels of protection (variance in the scope of project response to the flood threat) and level of protection may change over time (because of new upstream development or other changed conditions). There is no minimum level of protection required for Corps [COE] projects. However, in urban areas... ...it may well be desirable to consider alternatives providing a higher level of protection if... ...significant portions of the urban area [are] within the residual 100-year floodplain; or if, with overtopping or failure... ...there would be attendant risk to the lives of many, which could not be reasonably guarded against without a higher level of protection *(30).*

Not a term commonly used in highway drainage design. See Project Flood. Compare with Project Design Flood since it is not necessarily synonymous with this term. Also compare with Economic Analysis and Economic Assessment.

**LIABLE** Subject to civil action against, or for redress from infringement of private rights. Compare with Legal Liability.

**LIABILITY** See Liable.

**LIMNETIC ZONE** The deeper open water zones of lakes or lake size ponds. A relatively large expanse of open water above the profundal zone. Compare with Littoral Zone and Profundal Zone. See Pond.

**LIMNOLOGY** That branch of hydrology pertaining to the study of lakes. Compare with Lacustrine.

**LINE OF SIGHT** An unobstructed straight line from a radio transmitter to a receiver site *(33).*

**LINEAR SYSTEM** A system in which the principle of superposition applies and system response is only a function of the system itself (e.g., unit hydrograph model) *(1).*

**LINING** See Channel Lining. Compare with Blanket and Apron.

**LITTORAL DRIFT** The transport of material along a shoreline. Sometimes termed "long-shore sediment transport." The movement of sediments in the near shore zone by waves and currents. The movement can be parallel to the shore (long-shore transport) or perpendicular to the shore (onshore-offshore transport).

**LITTORAL TRANSPORT** See Littoral Drift.

**LITTORAL ZONE** A shore or coastal zone. Also the shallower depths of a lake where sunlight penetrates so that rooted plants can grow. Compare with Limnetic Zone and Profundal Zone.

**LIVESTOCK** Domestic animals, such as cattle, horses, hogs and chickens that are raised for home use or for profit. Compare with Wildlife.

**LOAD (or SEDIMENT LOAD)** Amount of sediment being moved by a stream.

**LOCAL SCOUR** See Scour, Local.

**LOG PAPER** Short for "full-logarithmic graph paper," which is a graph paper (available commercially) that has logarithmic scales on both horizontal and vertical axes. Sometimes called "log-log paper." The scales may be any number of cycles, but usually in [cycle] combinations like 1x1, 2x2, 3x3, 3x5, 4x7, etc. *(51).* Compare with Semilog Paper and Log-Normal Paper.

**LOG PEARSON DISTRIBUTION** See Probability Distribution.

**LOGARITHMICALLY TRANSFORMED DISTRIBUTION** See Probability Distribution.

**LOG-LOG PAPER** See Log Paper.

**LOG-NORMAL** Short for "logarithmic-normal probability distribution" *(51).*

**LOG-NORMAL PAPER** Graph paper used in estimating frequencies of floods, etc. Has a logarithmic scale for the flood (or other) amounts and a cumulative distribution scale (also called frequency or percent chance scale) for the probability plotting positions *(51).* Compare with Log Paper and Semilog Paper.

**LOLLY ICE** See Frazil Ice.

**LONG-PERIOD VARIATIONS** Secular when a cycle or a change in trend is completed within a century; climatic when the period of change runs through centuries or a few millennia; geologic when the period runs into geological time *(72).* See Trend.

**LONG-SHORE SEDIMENT TRANSPORT** See Littoral Drift.

**LONG-TERM DEGRADATION** See Short-Term Degradation.

**LONGITUDINAL PROFILE** The profile of a stream or channel drawn along the length of its centerline. In drawing the profile, elevations of the water surface or the thalweg are plotted against distance as measured from the mouth or from an arbitrary initial point.

**LOSING STREAM** See Stream, Losing.

**LOSS, HYDROLOGY** See Hydrology Loss.

**LOST HEAD** See Head, Lost.

**LOW FLOW CHANNEL** See Channel, Low Flow.

**LOW HEAD** See Head, Low.

**LOW-FLOW FREQUENCY CURVE** A graph showing the magnitude and frequency of minimum flows for a period of given length. Frequency is usually expressed as the average interval, in years, between recurrences of an annual minimum flow equal to or less than that shown by the magnitude scale. Compare with Flood-Frequency Curve.

**LOWER BANK** See Bank, Lower.

**LUMPED SYSTEM** A [statistical] system in which the variations in space either do not exist or have been ignored (opposite of distributed system) *(1)*.

**LYSIMETER** Structure containing a mass of soil and designed to permit the measurement of water draining through the soil *(23)*.

**MACROPHILE** Rooted aquatic plants.

**MAIN STEM** Main branch of the watershed (drainage area) stream system. Compare with Tributaries.

**MAJOR IRRIGATION FACILITIES** Major irrigation facility is an agency-specific term which might be defined by using arbitrary definitions such as where: 1. a water right is recorded for a conveyance facility, such as a canal and any appurtenant structures; 2. complex system and/or structure geometries are required; 3. significant sediment and/or erosion problems occur or are expected; or 4. complex hydraulic analysis practices are needed.

**MAJOR STORM DRAIN** A major storm drain system is an agency-specific term which might be defined by using arbitrary definitions such as where: 1. either three or more inlets enter a common trunkline and outfall; or 2. judgment indicates the need for a storm drain system to avoid a significant flood hazard. See Drain, Storm. Compare with Design Flood (or Storm) System.

**MANHOLE** Considered to be a gender-neutral term for a structure by which one may access a drainage system. Also referred to as access hole.

**MANNING'S EQUATION** An empirical formula devised by Manning, based upon original work by Ganguillet and Kutter, for computing flow in open channels and pipes. In its present form it has been modified to: $v = (1/n)R^{2/3}S^{1/2}$ where v = velocity, R = hydraulic radius or A/Wp where A = cross section area and Wp = wetted perimeter and S = Hydraulic Gradient. See Manning's n.

**MANNING'S** n A coefficient of roughness, used in a Manning's equation for estimating the capacity of a channel to convey water. Generally, "n" values are determined by inspection of the channel *(51).* The roughness coefficient, n, in the Manning equation for determination of a discharge. Compare with Hydraulic Roughness. See Manning's Equation.

**MANOMETER** A tube containing a liquid, the surface of which moves proportionally to changes of pressures; a tube type of differential pressure indicator; a pressure gage.

**MASS CURVE** A graph of the cumulative values of a hydrologic quantity (such as precipitation or runoff), generally as ordinate, plotted against time or date as abscissa *(34)*. See also Double-Mass Curve and Residual-Mass Curve.

**MASS INFLOW** Curve A graph showing the total cumulative volume of stormwater runoff plotted against time for a given drainage area.

**MASS WASTING** The collapse of a bank by undercutting due to wearing away of the toe or an erodible soil layer above the toe. The ongoing undercutting of stream banks by erosion and scour followed by the slumping and subsequent erosion of upper bank material. Sudden collapse of a bank due to an instability condition such as removal of a portion of the bank by scour. Compare with Erosion, Abrasion, Scour, and Sloughing.

**MASTER DRAINAGE PLAN** Planning model for drainage in a particular regional or local geographic area.

**MATHEMATICAL MODEL** A symbolic representation of a flow situation using mathematical equations.

**MATTRESS** A covering of concrete, wood, stone, or other material used to protect a stream bank against erosion. See Lining and Blanket.

**MAXIMUM POSSIBLE FLOOD** See Flood, Maximum Probable.

**MAXIMUM PROBABLE FLOOD** See Flood, Maximum Probable.

**MEAN ANNUAL FLOOD**, $Q_{2.33}$ See Flood, Mean Annual.

**MEAN ANNUAL FLOW**, $Q_a$ See Flow, Mean Annual.

**MEAN DAILY DISCHARGE** The average of mean discharge of a stream for one day. Usually given in $m^3/s$ *(51).* Compare with Flood, Mean Annual; Flow, Mean Annual; and Mean Monthly Flow.

**MEAN DEPTH** Cross section area of a channel divided by its surface width.

**MEAN DISCHARGE** The arithmetic mean of individual daily mean discharges during a specified period *(52).*

**MEAN MONTHLY FLOW** Average monthly flows, expressed in percent of annual flow, are determined for each of nearby gaged basins in the same hydrologic region. The overall percentage for each month is computed for the gaged sites, and these averages are multiplied by the estimate of mean annual flow, $Q_a$, to determine the mean monthly streamflows at the ungaged site. Compare with Flow, Mean Annual ($Q_a$); Mean Daily Discharge; and Flood, Mean Annual.

**MEAN SEA LEVEL** Mean sea level is the plane about which the tide oscillates. It is determined from tidal observations by averaging the recorded hourly heights of the tide over a period of several years. Mean sea level varies with locality of observation and method of computation, and hence it has no absolute constant value. in order to simplify the computation, mean tide level is sometimes used as a substitute for mean sea level *(14).* Compare with Mean Tide Level.

The mean of all hourly ocean elevations over a 19-year period *(33).*

**MEAN TIDE LEVEL** Mean Tide Level is simply equal to the average of the observed high and low waters, and it is considered a poor substitute [for the mean sea level] in many cases because of the appreciable influence of short-period tides *(14).* Compare with Mean Sea Level.

**MEAN VELOCITY** Two definitions are provided: 1. the velocity at a given section of a stream obtained by dividing the discharge of the stream by the cross section area at that section; 2. mean velocity may also apply to a reach of a stream by dividing the discharge by the average area of the reach.

**MEAN-SQUARE ERROR** Sum of the squared differences between the true and estimated values of a quantity divided by the number of observations. It can also be defined as the bias squared plus the variance of the quantity *(34).*

**MEANDER** The winding of a stream channel *(37).* The changes in direction and winding of flow, usually in an alluvial channel which is sinuous in character. Any reverse or letter-S channel pattern fashioned in alluvial materials by erosion of the concave bank, which is free to shift its location and adjust its shape as part of a stage in the migratory movement of the channel as a whole down an erodible, alluvial valley. A meander is characterized by curved flow patterns and alternating shoals and bank erosion.

**MEANDER AMPLITUDE** Distance between points of maximum curvature of successive meanders of opposite phase in a direction normal to the general course of the meander belt, measured between center lines of channels *(37).*

**MEANDER BELT** Area between lines drawn tangential to the extreme limits of fully developed meanders *(37).*

**MEANDER BREADTH** The distance between the lines used to define the meander belt *(37).*

**MEANDER LENGTH** Twice the distance between successive points of inflection of the meander wave *(42).* Distance, following the general, sinuous course of the meanders, between corresponding points of successive meanders of the same amplitude.

**MEANDER LOOP** An individual loop of a meandering or sinuous channel lying between inflection points with adjoining loops.

**MEANDER PHASE** See Meander Amplitude.

**MEANDER PLUGS** Deposits of cohesive materials in old channel bendways due to a cutoff. These plugs, sometimes termed "clay plugs," are sufficiently resistant to erosion to serve as essentially semipermanent geological controls to advancing channel migrations. See Clay Plug.

**MEANDER SCROLL** Topographical markings on old floodplains resembling a cross section of the edge pattern of a partly unrolled sheet of paper or having a spiral or coiled form, which have been left on a floodplain as a result of the historic migratory movement of the channel. Stated another way, low concentric ridges and swales on a floodplain, marking the successive positions of former meander loops.

**MEANDERING CHANNEL** A channel exhibiting a characteristic process of bank erosion, crossover and point bar deposition associated with systematically shifting meanders. See Meander and Meander Belt.

A channel having a sinuosity greater than some arbitrary value, herein placed at 1.25. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops.

**MEANDERING STREAM** See Meandering Channel.

**MEASURABLE** Computing or measuring a change that is more than the inherent error encountered in accepted hydraulics practices but exclusive of such errors found in flood predicting methods.

**MEASURED SCOUR** See Scour, Measured.

**MEASURING POINT** An arbitrary, permanent reference point from which the distance to the water surface in a well is measured to obtain the water level *(52)*.

**MEASURING WEIR** See Weir, Measuring.

**MEDIAN DIAMETER** See $D_{50}$.

**MEROMICTIC LAKE** See Lake, Meromictic.

**MESOTROPHIC LAKE** See Lake, Mesotrophic.

**METEOR BURST** Radio transmission uses the phenomenon of meteors entering the upper atmosphere to bounce signals back to earth *(33)*.

**METEOROLOGICAL SENSOR DATA** Collected weather data such as rainfall, humidity, temperature, and wind *(33)*.

**METER, CURRENT** See Current Meter.

**METERS PER SECOND**, m/s Velocity of flow.

**METHOD OF MOMENTS** A standard statistical computation for estimating the moment of a distribution from the data of a sample *(34)*.

mg/L Abbreviation for milligrams per liter (mg/L). See Parts Per Million.

**MICROPROCESSOR** A small computer that is usually more powerful in processing ability and more flexible in using other equipment than a typical home computer *(33)*.

**MICROWAVE RADIO** A radio whose transmission [frequency] requires line-of-sight. The frequency has high resistance to atmospheric interruption (telephone tower relay stations are an example) *(33)*.

**MID-CHANNEL BAR** See Bar, Middle.

**MIDDLE BANK** See Bank, Middle.

**MIDDLE BAR** See Bar, Middle.

**MIGRATION, CHANNEL** See Channel Migration.

**MILD SLOPE** See Slope, Mild.

**MINIMIZE** Reducing to the smallest practicable amount.

**MINIMUM ENERGY LINE** See Energy Line, Minimum.

**MITERED ENTRANCE** See Inlet, Mitered.

**MITIGATE** The act of lessening, offsetting, or compensating an impact on surface waters. To moderate (a qualifying or condition) in force or intensity. To decrease or rectify an adverse condition or action. See Mitigation Alternatives, Mitigation Measures, and Mitigation Methods.

**MITIGATION ALTERNATIVES** Environmental mitigation alternatives for surface waters in order of priority are currently (1992) defined as: 1. avoidance; 2. on-site mitigation; 3. off-site mitigation within the same drainage area; 4. off-site mitigation within the same drainage and biotic region; 5. no mitigation. See Biotic Region.

**MITIGATION MEASURES** Mitigation measures for surface waters are defined as the site-specific action or construction necessary to accomplish the mitigation to the extent practicable. See Practicable.

**MITIGATION METHODS** Mitigation methods for surface waters are defined as either the on-site or offsite: 1. construction of new surface waters; 2. enhancement of existing surface waters; 3. acquisition in perpetuity and enhancement of existing surface waters; 4. combinations thereof.

**MOBILE BED AND BANKS (MODEL)** A channel whose bed and banks are free to move under the forces of flowing water. Mobile bed and bank analyses and computer models estimate channel hydraulics taking into account this mobility. See Fix Bed and Banks (Model).

**MODEL** Reference 22 provides two definitions: 1. a conceptual, mathematical, or physical system obeying certain specified conditions, whose behavior is used to understand the physical system to which it is analogous in some way; 2. a conceptual description and the associated mathematical representation of a system, subsystem, components, or condition that is used to predict changes from a baseline state as a function of internal and/or external stimuli and as a function of time and space.

Relates output to input by attempting to describe the characteristics and processes of the system in the form of mathematical algorithms usually attempting to reflect real cause-effect relationships *(1).*

**MODEL, COMPUTER** The representation of a drainage system with computer software. Compare with Model, Physical.

**MODEL PARAMETER** A constant whose value varies with the circumstances of its application (e.g., Manning n-value) *(1).*

**MODEL, PHYSICAL** The representation of a drainage system with a hydraulically scaled laboratory model. Compare with Computer Model.

**MODEL VARIABLE** A term [which] has no fixed value (e.g., rainfall) *(1).*

**MOISTURE** Water diffused in the atmosphere or the ground.

**MOISTURE EQUIVALENT** The ratio of 1. the weight of water which the soil, after saturation, will retain against a centrifugal force 1000 times the force of gravity, to 2. the weight of the soil when dry. The ratio is stated as a percentage *(47).*

**MOMENTUM** The impetus of a moving body; the quantity of motion in a body as measured by the product of its mass by its velocity.

**MOMENTUM COEFFICIENT (β)** A correction factor (beta) applied to the flow momentum, to correct for non-uniformity of momentum in a cross section. For a fairly straight, nearly uniform channel β varies from 1.01 to 1.12 and as may be as high as 1.33 for overflooded river valleys *(13)*. Compare with Velocity Head Coefficient (α).

**MONOMICTIC LAKE** See Lake, Monomictic.

**MORPHOLOGY** The biological study of the form and structure of living organisms. May also be shortened term hydraulics engineers, for convenience, often used (or misused) when referring to fluvial geomorphology (technically this is incorrect but commonly used).

**MORPHOLOGY PROBLEMS** These are fluvial geomorphology problems related to such things as channel aggradation or degradation, bendway migration, bank erosion, bed scour and bendway cutoffs. See Fluvial Geomorphology, Geomorphology, and Morphology.

**MUD** A soft, saturated mixture mainly of silt and clay.

**MUDFLOW** A well-mixed mass of water and alluvium which, because of its high viscosity and low fluidity as compared with water, moves at a much slower rate, usually piling up and spreading over the [alluvial] fan like a sheet of wet mortar or concrete *(73).*

**MUSSEL** Any of several marine bivalve mollusks, especially the edible *Mytilus edulis,* having a blue-black shell.

**n (MANNING'S)**, or n Value See Manning's n.

**NAPPE** A sheet or curtain of water overflowing such things as a weir or drop structure. A stable nappe has an upper and a lower water surface exposed to the atmosphere. See Nappe, Flapping.

**NAPPE, FLAPPING** A nappe whose lower water surface periodically loses contact with the atmosphere which results in a negative pressure thereby causing the nappe to oscillate up and down in a flapping motion. Hydraulic structures with overflows should be designed to insure the nappe remains aerated throughout the design range of flows. See Nappe.

**NATIONAL GEODETIC VERTICAL** Datum of 1929. A geodetic datum derived from a general adjustment of the first order level nets of both the United States and Canada. It was formerly called "Sea level Datum of 1929" or "mean sea level" [in the annual WRD data reports]. Although the datum was derived from the average sea level over a period of many years at 26

tide stations along the Atlantic, Gulf of Mexico and Pacific Coasts, it does not necessarily represent local mean sea level at any particular place *(52)*.

**NATURAL AND BENEFICIAL FLOODPLAIN VALUES** Such things as wildlife, plants, open space, natural beauty, outdoor recreation, agriculture, aquaculture, forestry and artifacts as well as natural flood control and water quality control features such as wetlands and groundwater recharge areas. Floodplains also provide these values as well as supporting foodchains for aquatic life as well as provide other aquatic-related needs.

**NATURAL DRAINAGE DOCTRINE OR RULE** The precept in civil law dealing with the management and use of naturally occurring waters, based on preservation and continuance of the natural drainage system and runoff conditions. Compare with Common Enemy Doctrine or Rule, Civil Law Doctrine or Rule, and Reasonable Use Doctrine and Rule.

**NATURAL LEVEE** A low ridge along a stream channel, formed by deposition as floods abate, that slopes gently away from the channel.

**NATURAL SCOUR** See Scour, Natural.

**NATURAL WATERCOURSE** See Channel, Natural.

**NAVIGABLE WATERS** Primarily a regulatory term as it applies to highway drainage planning, design and construction in jurisdictional surface waters. The term "navigable waters" refers to jurisdictional surface waters as used in PL 92-500 and defined in Section 502(7) as "waters of the United States including the territorial seas." The territorial seas; coastal and inland waters, lakes, rivers and streams that are navigable waters of the United States, including adjacent wetlands; tributaries to navigable waters of the United States, including adjacent wetlands; interstate waters and their tributaries, including adjacent wetlands; and all other waters not identified above, the degradation or destruction of which could affect interstate commerce (33 CFR 323.3, 42 FR 37144, 1977); also see 80 Stat. 941, Volume 80 of the U.S. Statutes at Large, page 941; 49 USC. 1651 et seq.: Title 49, United States Code, Section 1651 and that which follows (or "and following"); PL 92-500: Public Law number 500 enacted by the 92nd Congress; 40 FR 55810: Volume 40 of the *Federal Register*, page 55810; 40 CFR 126: Title 40 of the Code of Federal Regulations, part 126. Those waters of the United States that are subject to the ebb and flow of the tide and/or those waters of the United States that are presently used, or have been used in the past, or may be susceptible to use in the future to transport interstate or foreign commerce. (33 CFR 322.2, 42 FR 37139, 1977).

Further, navigable waters are considered to be territorial seas of the United States and internal waters of the United States that are subject to tidal influence and internal waters of the United States not subject to tidal influence that: 1. are or have been used, or are or have been susceptible for use by themselves or in connection with other waters, as highways for substantial interstate or foreign commerce, not withstanding natural or man-made obstructions that require portage, or 2. a governmental or non-governmental body, having expertise in

waterway improvement, determines to be capable of improvement at a reasonable cost (a favorable balance between cost and need) to provide, by themselves or in connection with other waters, highways for substantial interstate or foreign commerce (40 FR 49327; 33 CFR 2.05-25).

**NECK CUTOFF** See Cutoff.

**NETWORK** An assembly of gaging/sampling points located and operated in such a manner as to provide a representative and transferable "sample" of the spatial and temporal variability of processes occurring within a geographic region *(1).*

**NEPHELOMETRIC TURBIDITY UNITS** See Turbidity.

**NFIP** Acronym for National Flood Insurance Program.

**NIBBLE EFFECT** A minor or insignificant (see Significant) amount of detrimental change in the functions and values of surface waters which may not be immediately detrimental, but the accumulation of many such small changes over a period of time will eventually cause detrimental problems.

**NICK POINT** A relatively small, localized point of scour in the thalweg of a vegetated swale or channel characterized by a small or shallow vertical area of channel degradation. If nick points enlarge up- and downstream so as to join together, a headcut forms and headcutting begins. Nick points occur when the vegetation is unable, for various reasons, to resist the eroding action of the flows occurring in the swale.

**NOMINAL SEDIMENT** Equivalent spherical diameter of a hypothetical sphere of the same volume as a given stone or sand particle.

**NON-LINEAR SYSTEM** A system in which the system response depends both upon the system itself and the input intensity (e.g., equations of gradually varied, open channel flow) *(1).*

**NON-STRUCTURAL MEASURES** These include: 1. flood warning and preparedness; 2. temporary or permanent evacuation and relocation; 3. emergency flood fighting and financial relief; 4. land use regulations including floodway delineation, floodplain zoning, subdivision regulations, and building codes; 5. flood proofing with or without land use regulations; 6. area renewal and conversion to open space; 7. flood insuring. The fundamental goal is to develop, define and recommend a robust solution that has public and institutional support (having appropriately determined how well an economical plan can be made to function, how capable are the responsible interests to operate and maintain it and how safe will be the people who will depend on it). Methods of reducing damage from floods include local flood warning and response systems, temporary permanent evacuation and relocation of people or property, emergency flood fighting and financial relief, land use regulations and building codes, flood proofing, area renewal and conversion to open space and flood insurance *(30).* Compare with Structural Measures.

**NON-UNIFORM CHANNEL** See Channel, Non-Uniform.

**NON-UNIFORM FLOW** See Flow, Non-Uniform.

**NONPARAMETRIC** Statistically, the same as distribution-free.

**NORMAL** A central value (such as arithmetic average or median) of annual quantities for a 30-year period ending with [on] an even 10-year, thus 1921-50; 1931-60 and so forth. This definition accords with that recommended by the Subcommittee on Hydrology of the Federal Interagency Advisory Committee on Water Data (the Subcommittee on Hydrology was formerly under the Federal Interagency Committee on Water Resources) *(37).*

A mean or average value established from a series of observations, for purposes of comparison of some meteorological or hydrological event *(51).*

**NORMAL DEPTH** See Depth, Normal.

**NORMAL DISTRIBUTION** See Probability Distribution.

**NORMAL FLOW** Flow at normal depth. Average flow prevailing during the greater part of the year. See Normal Stage; and Depth, Normal.

**NORMAL STAGE** The average water stage prevailing during the greater part of the year. The water surface elevation corresponding to the Normal Flow. See Normal Flow; and Depth, Normal.

**NORMAL VELOCITY** Mean velocity of flow at normal depth. See Depth, Normal.

**NORMAL WATER SURFACE** The free surface associated with flow in natural streams. The natural water surface. See Depth, Normal.

**NATURAL WATER SURFACE** See Normal Water Surface.

**NTU** Acronym for Nephelometric Turbidity Units.

**NEPHELOMETRIC TURBIDITY UNIT** Used in the direct reading measurement of the suspended particles. See Turbidity.

**NUTRIENT** Something that nourishes, especially, a nourishing ingredient in a food. Something found in surface waters that nourishes aquatic life.

**NWS** Acronym for the National Weather Service. More specifically, the United States Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service; formerly the U.S. Weather Bureau.

**OLIGITROPHIC LAKE** See Lake, Oligitrophic.

**ON-SHORE, OFF-SHORE TRANSPORT** See Littoral Drift.

**ONE-HUNDRED YEAR FLOOD** See Flood, One-Hundred Year.

**ONE-THOUSAND YEAR FLOOD** See Flood, One-Thousand Year.

**ONE-DIMENSIONAL WATER SURFACE PROFILE** An estimated water surface profile which recognizes flow only in the upstream-downstream direction: vertical and transverse velocity vector components are ignored. Compare with Two-Dimensional Water Surface Profile.

**OPEN CHANNEL** See Channel, Open.

**OPEN-CHANNEL CONSTRICTION** In application the term is used to refer to width constrictions, such as a highway bridge with long approach fills across a floodplain. In a general sense it applies to any type of constriction in an open channel, whether natural or constructed such as a dam, bridge, or culvert.

**OPEN CHANNEL FLOW** Flow in any open or closed conduit where the water surface is free; that is, where the water surface is at atmospheric pressure. Compare with Channel, Open.

**ORDINARY HIGHWATER (OHW)** A term for defining a regulatory-related water surface for a natural channel or the shore of standing waters. This intersection reflects the highest level water reaches in an average runoff year as indicated by such things as erosion, shelving, change in the character of soil, destruction of terrestrial vegetation or its inability to grow, the presence of litter and debris; or in the absence of such evidence, an arbitrarily estimated water surface might be used such as that associated with the mean annual flood. For the purposes of this Glossary, in no instance will the ordinary highwater (OHW) be considered as exceeding the estimated water surface level of the mean annual flood unless so mandated by the cognizant regulatory agency(ies). The sum of the water right, flood right, and mean annual flood may be used to arbitrarily determine the maximum OHW for irrigation channels intercepting runoff.

**ORGANIC MIXTURES AND MULCHES** Any of a number of agents (e.g., petrochemicals or vegetative matter) used to stabilize a stream bank against erosion by providing protection and nutrients while vegetation becomes established. These agents, which may be in the form of liquids, emulsions, or slurries, are normally applied by mechanical means.

**ORIFICE** Two definitions are pertinent: 1. a hole or opening, usually in a plate, wall, or partition, through which water flows, generally for the purpose of control or measurement; 2. the end of a small tube, such as the orifice of a pitot tube, or piezometer.

**ORIFICE PLATE** A plate containing an orifice. In pipes, the plate is usually inserted between a pair of flanges. The orifice is smaller than the pipe and the drop in the hydraulic grade line caused thereby is an index of the discharge.

**ORIFICE FLOW** See Flow, Orifice.

**OTHERS** A generic term sometimes used for such things as another governmental agency or quasi governmental agency (such as a flood control district, or irrigation district), as well as party(ies) or individual(s) from the private (non-governmental) sector of society.

**ORIFICE, SUBMERGED** See Submerged Orifice.

**OUTFALL** The point where: 1. water flows from a conduit; 2. the mouth (outlet) of a drain or sewer; 3. drainage discharges from a channel or storm drain.

**OUTLET CONTROL** A condition where the relation between headwater elevation and discharge is controlled by the conduit, outlet, or downstream conditions of any structure through which water may flow. In culvert flow, outlet control exists for flow-type II, III, IV and VI. Compare with Inlet Control.

**OUTLET, SUBMERGED** See Submerged Outlet.

**OUTLIER** Outliers (extreme events) are data points which depart from the trend of the rest of data *(34).* See Extreme Events and Probability Distribution.

**OVERBANK FLOW** See Flow, Overbank.

**OVERLAND FLOW** See Flow, Overland.

**OVERTOPPING FLOOD** See Flood, Overtopping.

**OXBOW** The abandoned bow-shaped or horseshoe-shaped reach of a former meander loop, that is left when the stream cuts a new shorter channel across the narrow neck (see Cutoff) between closely approaching bendways of the meander. See Oxbow Lake.

**PALIOLIMNOLOGISTS** Those whose study the history of lakes.

**PARALLEL WING WALL INLET** See Inlet, Parallel Wing Wall.

**PARAMETER** A characteristic descriptor, such as a mean or standard deviation *(34).* Sometimes considered as a variable comprised of the product of two or more variables.

The *Reader's Digest Great Encyclopedic Dictionary* offers three mathematical definitions: 1. a constant whose value determines the operation or characteristics of a system. In $y = ax^2 + bx + c$, a, b and c are the parameters of a family of parables; 2. a variable t, such that each of a related system of variables may be expressed as a function of t; 3. a fixed limit or guideline.

[Statistically], a number which describes the universe is called a parameter. [As an example] the value of 66 mm, which we found as the standard deviation of the heights of the Harvard students is a statistic. But if we were to say that the average birth weight of all newborn male babies is 3.4 kg, our number would be a parameter *(69).* Compare with Variable and Statistic.

**PARSHALL MEASURING FLUME** See Flume, Parshall Measuring.

**PARTIAL-DURATION FLOOD SERIES** A list of all flood peak discharges that exceed a chosen base stage or discharge, regardless of the number of peaks occurring in a year (also called basic-stage flood series, or floods above a base) *(37).* Compare with Partial-Duration Series.

A list of all events, such as floods, occurring above a selected base, without regard to the number, within a given period. In the case of floods, the selected base is usually equal to the smallest annual flood of a series, in order to include at least one flood in each year *(51).* Compare with Partial-Duration Flood Series.

**PARTIAL-DURATION SERIES** See Partial-Duration Flood Series.

**PARTS PER MILLION** The ratio used in expressing the concentration of suspended sediment ...given in either parts per million (PPM) or percent. Methods of determination: 1. weight of dried sediment divided by weight of sample; 2. weight of the dried sediment divided by weight of distilled water with a volume equal to that of the sampler; 3. weight of the dried sediment divided by the weight of the water in the sample, including the dissolved material. Either method 1. or 2. is used when the concentration is less than one percent. Current practice is to use milligrams per liter (mg/L) rather than parts per million.

**PARTIAL-RECORD STATION** A particular site where limited streamflow and/or water-quality data are collected systematically over a period of years for use in hydrologic analyses *(52).*

**PARTIALLY BOUNDED DISTRIBUTION** See Probability Distribution.

**PARTICLE SIZE** See $D_{16}$, $D_{50}$, and $D_{85}$.

**PATHOGENS** Any agent that causes diseases, especially a microorganism such as a bacterium or fungus.

**PAUCITY** Smallest of number, fewness, lack of.

**PAVEMENT** See Blanket and Mattress. Compare with Apron.

**PAVING** See Channel Lining.

**PDF** Acronym for Project Design Flood.

**PEAK DISCHARGE** The highest value of the stage or discharge attained by a flood; thus, peak stage or peak discharge. Flood crest has nearly the same meaning, but since it connotes the top of the flood wave, it is properly used only in referring to stage—thus, crest stage, but not crest discharge *(37).*

Maximum discharge rate on a runoff hydrograph for a given flood event. The instantaneous, maximum discharge of a particular flood at a given point along a stream.

In a frequency study of annual floods, it is the maximum instantaneous discharge rate reached during the year.

**PEAK FLOOD** See Peak Discharge.

**PEAK STAGE** See Flood Stage.

**PEAKED STONE DIKE** Riprap placed parallel to the toe of a stream bank (at the natural angle of repose of the stone) to prevent erosion of the toe and induce sediment deposition behind the dike. Compare with Reinforced Revetment.

**PEARSON DISTRIBUTION** See Probability Distribution.

**PERCENT CHANCE** A probability multiplied by 100 *(34).*

A name often given to the probability scale on log-normal paper. A 2-percent chance flood is a 50-year frequency flood (see Frequency) since 100 (percent chance = frequency in years) *(51).* See Log-Normal Paper.

**PERCHED GROUNDWATER** See Groundwater, Perched.

**PERCHED STREAM** See Stream, Perched.

**PERCHED WATER TABLE** See Groundwater, Perched; and Water Table.

**PERCOLATING WATERS** Those waters which pass through the ground beneath the surface of the earth without any definite channel and do not form a part of the body or flow of any surface or subterranean water course. They may be either infiltrating rain waters or snowmelt, or waters that have seeped through the banks or bed of a stream to a distance where they lose their character as streamflow.

**PERCOLATION** The flow of a fluid through a substance via pores or small openings. Two definitions are offered by the Groundwater Subcommittee: 1. the downward movement of water through the unsaturated zone; 2. the downward flow of water in saturated or nearly saturated porous medium at hydraulic gradients of the order of 1.0 or less.

Movement of water through the interstices of a substance, as through soils. The movement or flow of water through the interstices or the pores of a soil or other porous medium.

The movement, under hydrostatic pressure, of water through the interstices of a rock or soil, except the movement through large openings such as caves *(47).* Compare with Infiltration.

**PERENNIAL FLOW OR STREAM** See Stream, Perennial.

**PERMAFROST** Perennially frozen ground, occurring wherever the temperature remains at or below 0□C for two or more years in a row *(20).*

**PERMEABILITY** The property of a material that permits appreciable movement of water through it when it is saturated and movement is actuated by hydrostatic pressure of the magnitude normally encountered in natural subsurface water.

**PERMISSIBLE VELOCITY** See Velocity, Permissible.

**PERVIOUS SOIL** Soil containing voids through which water will move under hydrostatic pressure, percolate, or infiltrate.

pH The reciprocal of the logarithm of the hydrogen ion concentration. The concentration is the weight of hydrogen ions, in grams, per liter of solution. Neutral water (or soil) has a pH value of 7.

**PHOTOSYNTHESIS** The process by which chlorophyll-containing cells in green plants convert incident light to chemical energy and synthesize organic compounds from inorganic compounds, especially carbohydrates from carbon dioxide and water with the simultaneous release of oxygen.

**PHREATIC LINE** The upper boundary of the seepage water surface landward of a stream bank.

**PHYSICAL MODEL** See Hydraulic Model.

**PHYSIOGRAPHIC REGION** In highway hydrology considerations, a geographic area whose pattern of landforms and other runoff-producing features for its contiguous subregions are homogeneous but differ significantly from such feature(s) of an adjacent physiographic region(s).

**PIER SHAFT** The main part of a pier above the footing or foundation.

**PIEZOMETER** An instrument for measuring pressure head, usually consisting of a small pipe tapped into the side of a closed or open conduit and flush with the inside, connected with a pressure gage, mercury, water column, or other device for indicating pressure head.

**PIEZOMETRIC HEAD** See Head, Piezometric.

**PILE** An elongated member, usually made of such things as timber, concrete, or steel, that serves as a structural component of a river-training structure or bridge foundation. Compare with Pile Bin.

**PILE BIN** A pier composed of piles capped or decked with a timber grillage or with a reinforced-concrete slab forming the bridge foundation. See Pile.

**PILE PIER** See Pile Bin.

**PIPING** The action of water passing through or under an embankment and carrying some of the finer material with it to the surface at the downstream face. Removal of soil material through subsurface flow or seepage water that develops channels or “pipes” within the soil bank.

**PITOT TUBE** A device for measuring and observing the velocity head of flowing water, consisting essentially of an orifice held so as to point upstream in flowing water and connected with a tube by which the rise of water in the tube above the water surface may be observed. It may

be constructed with an upstream and a downstream orifice and two water columns, the difference of water levels being an index of the velocity head.

**PLANKTON** Plant and animal organisms, generally microscopic, that float or drift in great number in fresh or salt water.

**PLANNING MODELS** Models used in large-scale applications such as for metropolitan master plans *(1).*

**PLAYA** See Lake, Playa.

**PLAYA LAKE** Playa means playa lake thereby making this term redundant. See Lake, Playa.

**PLOTTING POSITION** The point computed by an equation and used to locate given data on probability [graph] paper *(51)*. See Probability Plotting Position.

**PMF** Acronym for Probable Maximum Flood.

**PMP** Acronym for Probable Maximum Precipitation.

**POINT BAR** See Bar, Point.

**POINT GAGE** See Gage, Point.

**POINT RAINFALL** See Rainfall, Point.

**POISED STREAM** See Stream, Poised.

**POLLUTANT** Anything that pollutes, such as grease, oil and toxic substances.

**POLICY** A definite course of action or method of action, selected to guide and determine present and future decisions whereas criteria are the standards by which a policy is carried out or placed in action; design criteria are needed for design, policy statements are not. Compare with Design Criteria.

**POND** Very small, very shallow bodies of standing water in which quiescent water and extensive occupancy by higher aquatic plants are common characteristics. Regional usage may refer to a lake as a pond. Compare with Lake.

**PONDAGE** Small-scale storage at a water power plant to equalize daily or weekly fluctuations in riverflow or to permit irregular hourly use of the water for power generation to accord with fluctuations in load *(37)*.

**POOL** A small, rather deep body of quiescent water, as a pool in a stream.

A deep reach of a stream. The reach of a stream between two riffles. Natural streams often consist of a succession of pools and riffles *(37).*

**POPULATION** The entire (usually infinite) number of data from which a sample is taken or collected. The total number of past, present and future floods at a location on a river is the population of floods for that location even if the floods are not measured or recorded *(34).* Compare with Sample.

**PORE WATER PRESSURE** See Pressure, Pore Water.

**POROSITY** Reference 22 offers two definitions: 1. the ratio, usually expressed as a percentage, of the total volume of voids of a given porous medium to the total volume of the porous medium; 2. the volume percentage of the total bulk not occupied by solid particles.

The ratio of the volume of void spaces in a rock or sediment to the total volume of the rock or sediment *(20).*

Stated two other ways: 1. an index of the void characteristics of a soil or stratum as pertaining to percolation; degree of perviousness; 2. ratio of void volume to total volume of a soil, or rock, generally expressed in percentages.

**POSSIBLE MAXIMUM FLOOD** See Flood, Maximum Possible.

**POTENTIAL ENERGY** See Energy, Potential.

**POTENTIAL EVAPOTRANSPIRATION** Water loss that will occur if at no time there is a deficiency of water in the soil for use of vegetation *(65).*

**POTENTIAL NATURAL WATER LOSS** The water loss during years when the annual precipitation greatly exceeds the average water loss. It represents the approximate upper limit to water loss under the type and density of vegetation native to a basin, actual conditions of moisture supply and other basin characteristics, whereas potential evapotranspiration represents the hypothetical condition of no deficiency of water in the soil at any time for use of the type and density of vegetation that would develop *(66).*

**POTENTIAL RATE OF EVAPORATION** See Evaporativity.

**POTOMOLOGY** The science of surface streams *(14).* Compare with River Mechanics.

**PPM** Acronym for Parts Per Million.

**PRACTICABLE** Capable of being accomplished within prudent natural, social, or economic constraints using readily available resources and reasonably reliable technology and practices: available and can be economically applied.

**PRECIPITATION** The process by which water in liquid or solid state falls from the atmosphere. The total measurable supply of water received directly from clouds, as rain, snow and hail; usually expressed as depth in a day, month, or year and designated as daily, monthly, or annual precipitation. Not synonymous with Rainfall. Compare with Rainfall.

As used in hydrology, precipitation is the discharge of water, in liquid or solid state, out of the atmosphere, generally upon a land or water surface. It is the common process by which atmospheric water becomes surface or subsurface water. The term "precipitation" is also commonly used to designate the quantity of water that is precipitated *(47).* Precipitation includes rainfall, snow, hail and sleet and is therefore a more general term than rainfall *(37).*

**PRECIPITATION, EFFECTIVE** Two definitions from "General Introduction and Hydrologic Definitions" *(37)* are: 1. that part of the precipitation that produces runoff; 2. a weighted average of current and antecedent precipitation that is "effective" in correlating with runoff. As described by U.S. Bureau of Reclamation, that part of the precipitation falling on an irrigated area that is effective in meeting the consumptive use requirements. Compare with Runoff, Direct; and Rainfall, Effective.

**PRECIPITATION, PROBABLE MAXIMUM** Probable Maximum Precipitation (PMP) is an estimate that approaches the theoretically largest storm physically possible. Development of the PMP considers all storms of record and the observed precipitation is increased by maximizing the moisture inflows to the storm system. Generalized depth, area, duration and season relationships for the continental U.S. are published by the National Weather Service in a series of hydrometeorological reports (HMR).

**PREDATOR** An animal that lives by preying upon others.

**PRESCRIPTION** Acquirement of the title or right to something through its open and continued use or actual possession from time immemorial or over a legally recognized or prescribed period. Also the diversion of water by a person at a point upstream from the land of a riparian owner, under the above conditions would give such person an appropriate right, perfected by prescription, to the use of the water, as against the owner.

**PRESCRIPTIVE DRAINAGE EASEMENT** A prescription (prescriptive right) which has been established through long, uninterrupted and undisputed use of a drainage facility or channel and thus precludes increasing the existing flood hazard, or changing the drainage pattern or amount at such a drainage facility or channel for any recurrence interval. The free or unencumbered use of some drainage facility for drainage or other purposes.

**PRESCRIPTIVE RIGHT** See Prescription.

**PRESERVE** To perpetuate an existing condition. In highway drainage design, maintaining the natural environmental functions of surface waters as close as practicable to their existing state. See Practicable.

**PRESSURE** Total load or force acting upon a surface; also appropriately used to indicate intensity of pressure or force per unit area.

**PRESSURE HEAD** See Head, Pressure. See Subsurface-Water Flow and Solute Transport in the Federal Glossary of Selected Terms *(22).*

**PRESSURE, HYDROSTATIC** The pressure exerted by the weight of water at any given point in a body of water at rest *(22).*

**PRESSURE AND MOMENTUM FORCE** The force due to the sum of the pressure and momentum forces caused by moving (flowing) water. See Pressure and Momentum.

**PRESSURE, PORE WATER** Unit stress (pressure) carried by the water in the pores of a soil mass. Excess negative pressure will displace soil particles. Can be measured with piezometers.

**PRESSURE RIDGES** Ridges on an ice-sheet over a body of water caused by expansion and consequent upheaval of the ice.

**PRINCIPAL SPILLWAY** See Spillway, Principle.

**PROBABILITY** [The science that] deals with the measure of chance or likelihood based on the sampled data *(14).* Compare with Statistic and Statistics.

The ratio of observed or expected events to all possible events; it is expressed as a decimal less than or equal to one *(33).* See X-Percent Chance Flood and Probability.

**PROBABILITY DISTRIBUTION** Function describing the relative frequency with which events of various magnitudes occur *(34).*

Probability distributions of interest to a hydraulics engineer are: Normal Distribution; Pearson Distributions; Extremal [Extreme] Distributions; Logarithmically Transformed Distributions.

Normal Distribution—A probability distribution that is symmetrical about the mean, median and mode (bell-shaped). It is the most studied distribution in statistics (even though most data are not exactly normally distributed) because of its value in theoretical work and because many other distributions can be transformed into normal. It is also known as Gaussian, the Laplacean, the Gauss-Laplace, the Laplace-Gauss distribution, or the Second Law of Laplace.

This is a symmetrical, bell-shaped, continuous distribution, theoretically representing the distribution of accidental errors about the mean, or so-called Gaussian law of errors [the area under the probability curve is between the variates of -□ and x].

Pearson Distributions—Karl Pearson ...derived a series of probability functions to fit virtually any distribution. Although these functions have only slight theoretical basis, they have been used widely in practical statistical works to define the shape of many distribution curves, [under certain conditions] the resulting Pearson distribution is identical with the normal distribution. Types I and III distributions are often used in hydrologic frequency analysis *(14).*

· Type I—This is a skew [probability] distribution with limited range in both directions, usually bell-shaped but may be J-shaped or V-shaped.

· Type III—This is a skew [probability] distribution with limited range in the left direction, usually bell-shaped but may be J-shaped.

Extremal Distributions—The probability distribution of extreme values. See Extreme Values and Probability Distributions. Extreme value distributions of interest are Type I, II and III *(14).*

· Type I—This [extreme value probability] distribution results from any initial distribution of exponential type which converges to an exponential function as x increases. [-□ < x < □]. Examples of such distributions are normal, the chi-square and the log normal distributions. The Type I distribution is sometimes known as Gumbel distribution since Gumbel first applied it to flood frequency analysis.

· Type II—This [extreme value probability] distribution results from an initial distribution of [the] Cauchy type which has no moments from a certain order and higher [0 □ x < □].

· Type III—This [extreme value probability] distribution results from a type of initial distribution in which x is limited by x □ ε [-□ < x □ ε]. Type III is known as Weibull distribution since Weibull first applied it to the description of the strength of brittle materials although Gumbel also applied it later to drought frequency analysis.

Logarithmically Transformed Distributions—Many probability distributions can be transformed by replacing the variate with its logarithmic value. Three transformed distributions commonly used in hydrologic studies are [Log normal; Log extremal; Truncated Log normal] *(14).*

· Log normal—This is a transformed normal [probability] distribution in which the variate is replaced by its logarithmic value. This distribution represents the so-called Law of Galton because it was first studied by Galton as early as 1875.

· Log extremal—[This is a probability distribution where the] variate x in [the] Type I [Pearson] distribution is replaced by a linear function of the logarithm of x and x - ε [so that] the resulting logarithmically transformed distributions become [Pearson] Type II and Type III distributions respectively.

· Truncated Log normal—[These are] two truncated and shifted logarithmically transformed normal distributions for hydrologic frequency analysis. One is called the partly bounded distribution which has the positive direction of the variate. The other is called the totally bounded distribution which has the maximum and minimum limits of fluctuations from the mean.

Student's t Distribution—A distribution used in [the] evaluation of variables which involve sample standard deviation rather than population standard deviation *(34).* Sometimes referred to as t distribution.

**PROBABILITY OF EXCEEDANCE** See Exceedance Probability.

**PROBABILITY PAPER** Any graph paper prepared especially for plotting magnitudes of events versus their frequencies or probabilities. See Log-Normal Paper *(51).*

The cumulative probability of a distribution may be represented graphically on a probability paper which is designed for the distribution. On such paper the ordinate usually represents the value of x [flood magnitude] in [a] certain scale and the abscissa represents the probability P(X □ x) or P(X □ x) [actually P(X > x) or P(X < x)—Ed.] or the recurrence interval T [or frequency T]. The ordinate and abscissa scales are so designed the distribution plots as a straight line and the data to be fitted appear close to a straight line. The objective of using the probability paper is to linearize the distribution so that the plotted data can be easily analyzed for extrapolation or comparison purposes. In the case of extrapolation, however, the effect of sampling errors is often magnified *(14).* See Probability Distribution and Probability Plotting Position.

**PROBABILITY PLOTTING POSITION** When a probability paper is chosen for use, the plotting of data on the paper requires the knowledge of [probability] plotting positions. Numerous methods have been proposed for the determination of plotting positions. Most of them are empherical *(14).* Nine probability plotting position formulae are of interest to hydrologists. With N = total number of items, m = order number of items arranged in descending magnitude (i.e., m = 1 for the largest item): 1. California, N/m; 2. Hazen, 2N/(2m - 1); 3. Weibull (and Gumbel), (N + 1)/m; 4. Beard, $1/(1 - 0.5^{1/n})$; 5. Chegodayev, (N + 0.4)/(m - 0.3); 6. Blom, (N + 1/4)/(m - 3/8); 7. Tukey, (3N + 1)/(3m - 1); 8. Gringorten, (N + 0.12)/(m - 0.44). Paraphrased from *Handbook of Applied Hydrology (14).*

**PROBABLE MAXIMUM FLOOD** See Flood, Probable Maximum.

**PROBABLE MAXIMUM PRECIPITATION** See Precipitation, Probable Maximum.

**PROFILE** A graphical representation of elevation plotted against distance. In open channel hydraulics a water surface profile is a plot of water surface elevation against channel distance. See Hydraulic Grade Line and Water Surface Profile.

**PROFILE GRADE** The trace of a vertical plane intersecting any given roadway surface as shown on the plans. Profile grade means either elevation or gradient of such trace according to the context.

**PROFUNDAL ZONE** The underlying deep, dark region of the waterbody where light penetration is insufficient to support the production of green plants. Compare with Littoral Zone and Limnetic Zone.

**PROJECT** The specific section of highway, together with all appurtenances and construction to be performed under a contract.

**PROJECT DESIGN FLOOD** See Flood, Project Design.

**PROJECT FLOOD** See Flood, Project.

**PROJECT HYDROLOGY** The designated design flood predicting practice(s) to be used for the design of drainage facilities on a particular highway project.

**PROJECTING ENTRANCE** See Inlet, Projecting.

**PROMONTORY** A high ridge of land or rock extending out into a sea or other expanse of water.

**PROTOZOA** Any of the single-celled, usually microscopic organisms of the phylum or subkingdom.

**PULS METHOD** See Storage-Indication Method.

**QUARRY-RUN-STONE** Natural material, often used for stream bank protection, as received from a quarry without regard to gradation requirements.

**RACK** A screen composed of parallel bars to catch floating debris.

**RADIO RELAYS** Repeating or relaying devices that transfer data from a self-reporting gage to a base stations *(33).*

**RADIO REPEATERS** See Radio Relays.

**RAILBANK PROTECTION** A type of countermeasure composed of rock-filled wire fabric and supported by steel rails or posts driven into the channel bed and used for such things retards, riparian spur dikes and training dikes.

**RAIN** Liquid precipitation *(37).* Also, precipitation in the form of water. Usage includes snow and hail in the term.

**RAINFALL** The quantity of water that falls as rain only. Not synonymous with precipitation *(37).* Compare with Precipitation.

**RAINFALL, EFFECTIVE** [Sometimes used as] another term for direct runoff. Usually not the same quantity on upland streams as on downstream rivers because of variability of seepage flows *(51).* Compare with Runoff, Direct; and Precipitation, Effective.

**RAINFALL EXCESS** The volume of rainfall available for direct runoff. It is equal to the total rainfall minus interception, depression storage and absorption *(2).* Not the same as excessive rainfall. Direct runoff at the place where it originates *(51).* Compare with Rainfall, Excessive; and Rainfall, Effective.

**RAINFALL, EXCESSIVE** Rainfall in which the rate of fall is greater than certain adopted limits, chosen with regard to the normal precipitation (excluding snow) of a given place or area. In

the National Weather Service, it is defined, for States along the southern Atlantic coast and the Gulf coast, as rainfall in which the depth of precipitation is 0.9 inches (22 mm) at the end of 30 minutes and 1.5 inches (38 mm) at the end of an hour and for the rest of the country as rainfall in which the depth of precipitation at the end or each of the same periods is 0.5 inch (13 mm) and 0.8 inch (20 mm), respectively *(37).*

Standard USWB (NWS) term for "rainfall in which the rate of fall is greater than certain adopted limits, chosen with regard to the normal precipitation (excluding snow) of a given place or area." Not the same as excess rainfall *(51).* Compare with Rainfall, Excess.

**RAINFALL INTENSITY** Amount of rainfall occurring in a unit of time, converted to its equivalent in inches per hour at the same rate.

**RAINFALL, POINT** Rainfall at a single rain gage *(51).*

**RAPID DRAWDOWN** See Drawdown, Rapid.

**RAPIDLY VARIED FLOW** See Flow, Rapidly Varied.

**RAPIDS** One of two definitions may apply: 1. a term used by some for "chute"; 2. swift and turbulent flow, without pronounced falls.

**RAPTOR** A bird of prey.

**RARE SURFACE WATERS** See Surface Waters, Rare.

**RATE OF RISE** Indicates how quickly a stream is rising, typically expressed in feet per hour or meters per hour; feet per day or meters per day *(33).*

**RATING** Three definitions are provided: 1. the relation, usually determined experimentally, between two mutually dependent quantities, such as stage and discharge of a stream; 2. a calibration of current-meter vane revolutions versus water velocity, etc.; calibration; 3. the taking of measurements or the making of observations to establish a rating or calibration.

**RATING CURVE** A graph of the discharge of a river at a particular point as a function of the elevation of the water surface *(33).* A graphic (or tabular) representation of rating; a calibration; a curve (table) relating stage to discharge. Compare with Stage-Discharge Curve.

**RATING FLUME** There are two types of rating flumes: 1. an open conduit built in a channel to maintain a consistent regime for the purpose of measuring the flow and developing stage-discharge relation; 2. a flume containing still water for rating current meters, Pitot tubes, etc.

**RATIONAL FORMULA** An empherical equation for estimating the flood discharge given as Q = CIA/360, where Q = peak discharge, C = a runoff coefficient, I = rainfall intensity in inches per hour for a duration equal to the concentration time of the basin and A = area of basin in

acres. The original formula is based on the approximation that 1 inch/hour/acre = 1 cubic foot a second

**RE-REGULATING RESERVOIR** See Reservoir Re-Regulating.

**REACH** A segment of stream or valley, selected with arbitrary bounds for purposes of study. A comparatively short length of a stream or channel.

"General Introduction and Hydrologic Definitions" *(37)* offers five definitions: 1. the length of channel uniform with respect to discharge, depth, area and slope; 2. the length of a channel for which a single gage affords a satisfactory measure of the stage and discharge; 3. the length of a river between two gaging stations; 4. more generally, any length of a river; 5. a length of stream or valley, selected for convenience in a study. See Damage Reach and Stream Reach.

**REASONABLE USE DOCTRINE AND RULE** A rule under which, in some jurisdictions, a riparian owner, acting in good faith and for a legitimate purpose, may use and/or alter the natural flow of water from his land without liability to other owners, so long as such use and action is reasonably necessary and beneficial and reasonable care is taken to avoid unnecessary injury or hindrance to the use of the land below. Compare with Common Enemy Doctrine or Rule, Civil Law Doctrine or Rule, and Natural Drainage Doctrine and Rule.

**RECEIVING WATER** "Natural" [Any—Ed.] body of water that one or more catchments enter into (i.e., stream, tributary, river, estuary, bay, lake, et cetera) *(1).*

**RECESSION CURVE** [That portion of] a hydrograph showing the decreasing rate of runoff following a period of rain or snowmelt. Since direct runoff and base runoff recede at different rates, separate curves, called direct runoff recession curves and base runoff recession curves, respectively are generally drawn. The term "depletion curve" in the sense of base runoff recession is not recommended *(37).*

The receding portion of a hydrograph, occurring after excess rainfall has stopped *(51).*

**RECHARGE** Addition of water to the zone of saturation from precipitation or infiltration *(22).* The process of adding water to the saturated zone; also, the water added. For constructed recharge facilities, see Basin, Recharge.

**RECHARGE AREA** An area in which water reaches the zone of saturation by surface infiltration *(22).* See Recharge.

An area in which there are downward components of hydraulic head in the aquifer. Infiltration moves downward into the deeper parts of an aquifer in a recharge area *(20).*

**RECHARGE BASIN** See Basin, Recharge.

**RECORDER** A device that makes a graph of the stage, pressure, depth, velocity, or the movement or position of water-controlling devices. See Indicator and Register.

**RECORDING GAGE** See Water-Level (Stage) Recorder.

**RECOVERY AREA** A clear zone which includes the total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope and/or a clear run-out area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry.

**RECTANGULAR WEIR** See Weir, Rectangular.

**RECURRENCE INTERVAL** See Flood Frequency.

**REFUSAL** Erosion-resistant material placed in a trench (excavated landward) at the upstream end of a revetment to prevent flanking.

**REGIME** The condition of a stream and its channel as regards to their stability. A river or canal is "in regime" if its channel has reached a stable form as a result of its flow characteristics.

General pattern of variation around a mean condition, as in such things as flow regime, tidal regime, channel regime and sediment regime; used also to mean a set of physical characteristics of a river. The system or order characteristic of a stream; its behavior with respect to such things as velocity and volume, form of and changes in channel, capacity to transport sediment and amount of material supplied for transportation. Compare with Regime Channel; Channel, Stable; and Stream, Poised.

**REGIME CHANGE** A change in channel characteristics resulting from such things as changes in imposed flows, sediment loads or slope.

**REGIME CHANNEL** Alluvial channel that has attained more or less a state of equilibrium with respect to erosion and deposition. Compare with Stream, Poised; and Channel, Stable.

**REGIME FORMULA** A formula relating stable alluvial channel dimensions or slope to discharge and sediment characteristics.

**REGIME OF A STREAM** The system or order characteristic of a stream; in other words, its habits with respect to velocity and volume, form of and changes in channel, capacity to transport sediment and amount of material supplied for transportation. The term is also applied to a stream which has reached an equilibrium between erosion and deposition or, in other words, to a graded stream *(11).* See Stream, Poised; Stream Graded; and Regime Channel.

**REGIME THEORY** "Regime theory" is a theory of the forming of channels in material carried by the streams. As used in this sense, the word "regime" applies only to streams that make at least part of their boundaries from their transported load and part of their transported load from their boundaries, carrying out the process at different places and times in any one stream in a balanced or alternating manner that prevents unlimited growth or removal of boundaries. A stream, river, or canal of this type is called a "regime stream, river, or canal." A regime channel is said to be "in regime" when it has achieved average equilibrium; that is, the

average values of the quantities that constitute regime do not show a definite trend over a considerable period (generally of the order of a decade). In unspecialized use, "regime" and "regimen" are synonyms *(10).* See Stream, Poised; Regime, Channel; and Channel, Stable.

**REGIMEN** In unspecialized use "regime" and "regimen" are synonyms *(37).* See Regime, Regime Theory, and Regime of a Stream.

**REGIONAL ANALYSIS** Flood-frequency [relationships] lines for gaged watersheds in a similar [homogeneous physiographic] area or region are used to develop a flood-frequency line for an ungaged watershed in that [same] region. Also used with other types of hydrologic data. Method is a simple (usually graphical and freehand) form of "regression analysis" used by statisticians *(51).*

A statistically based regional study of gaged stream data from a homogeneous physiographic region which produces regression equations relating various watershed and climatological parameters to such things as discharge frequency for application on ungaged streams. Used to formulate methods of predicting flood-frequency relationships for the hydraulic design of drainage facilities in hydrologically similar ungaged watersheds having characteristics similar to those used in the regression analysis.

**REGIONAL FLOOD OF RECORD** Maximum flood known or recorded in a drainage area. Peak flood flows from thousands of sites (Crippin and Bue, WSP-1987) in the conterminous U.S. were plotted against drainage area. An enveloping curve for each hydrologic area provides an estimate of maximum floods within drainage basins *(16).*

**REGISTER** One of two definitions may apply: 1. a device that notes quantities; it may make a graph or a printed or stamped record by figures or symbols on a dial or on an assembly of dials, indicate by pointer, index, or otherwise note such quantities as stage, pressure, velocity, depth, or quality; it may note the movement or position of water-controlling devices as gates or valves; a gage, indicator, or recorder; 2. to note such quantities. See Indicator, Recorder, and Gage.

**REGULATED RIVER OR STREAM** See River, Regulated.

**REGULATION** The artificial manipulation of the flow of a stream *(37).*

**REGULATIONS** Formal instructions or rules governing the application and administration of specified legislative acts, which have the force and effect of law. As used in the CFR, rule and regulation have the same meaning.

**REGULATORY FLOOD** The 100-year flood, which was adopted by the FEMA, as the base flood for most floodplain management purposes. See Flood, Base.

**REGULATORY FLOODWAY** The floodplain area that is reserved in an open manner by Federal, State, or local requirements, i.e., unconfined or unobstructed either horizontally or vertically,

to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount. May also be considered as: 1. a channel and floodplain regulated by an agency having jurisdiction over dredge and fill activities; or 2. a channel and floodplain requiring approval for a channel change or flood storage. Compare with Floodway.

**REGULATORY WATERS** See Navigable Waters.

**REINFORCED REVETMENT** A channel bank protection method consisting of a continuous stone toe-fill along the base of a bank slope with intermittent fillets of stone placed perpendicular to the toe and extending back into the natural bank. Compare with Peaked Stone Dike.

**REINFORCED-EARTH RETAINING WALL** A retaining structure consisting of vertical panels and attached to reinforcing elements embedded horizontally in compacted backfill for supporting a natural or artificial channel bank (a specific type of bulkhead).

**RELIEF BRIDGE** See Bridge, Relief.

**RESERVOIR** A pond, lake, or basin, either natural or artificial, for the storage, regulation and control of water *(37).* Reservoirs regulate floods downstream from the dam by temporarily storing some part of the flood volume and releasing it later. The impact downstream is to lower flood stages, increase the duration of flooding and shift the flood to a later time. It is normal for dam and reservoir projects to effect some control on and lower flood stages for, all magnitudes of floods *(30).*

**RESERVOIR, RE-REGULATING** A reservoir for reducing diurnal fluctuations resulting from the operation of an upstream reservoir for power production *(37).*

**RESERVOIR, RETARDING** Ungated reservoir for temporary storage of floodwaters. Sometimes called detention reservoir *(37).* Compare with Floodwater Retarding Structure; Basin, Retention; and Basin, Detention.

**RESERVOIR ROUTING** Flood routing through a reservoir *(51).* Flood routing of a hydrograph through a reservoir taking into account reservoir storage, spillway, and outlet works discharge relationships.

**RESIDENCE TIME** The time that water stays in lake with no outlet, or with a very limited outlet. May also refer to the time floodwaters for a given frequency flood are expected to be detained in a retention basin, retarding reservoir, recharge basin, or flood control reservoir.

**RESIDUAL-MASS CURVE** A graph of the cumulative departures from a given reference such as the arithmetic average, generally as ordinate, plotted against time or date, as abscissa *(37).* See Mass Curve.

**RESPONSE SYSTEM** The planned protective reaction to flooding or the threat of flooding *(33).*

**RESTORABLE SURFACE WATERS** Surface waters and wetlands having functions and values diminished by human impacts that can be restored through various management techniques.

**RESTORE** To re-establish to the extent practicable a geometry, setting, or environment in which the essential elements of the natural surface waters such as floodplains, shores, riparian areas, and channel as well as any previously existing constructed features can again function as they did prior to a highway action. See Practicable.

**RETAINING WALL** With drainage design, a structure used to maintain an elevation differential between the water surface and top bank while at the same time preventing bank erosion and instability.

**RETARD** A channel bank protection technique consisting of such things as wire mesh, chain-link, steel rails, or timber-framed fence attached to a series of posts, sometimes in double rows; the space between the rows may be filled with rock, brush, or other suitable permeable materials. Fences may be placed either parallel to the bank and/or extended into the channel; in either case these structures decrease the stream velocity and encourage sediment deposition as the flow passes through the fence. Explained another way, a frame structure, filled with earth or stone ballast, designed to absorb energy and to keep erosive channel flows away from a bank. A retard is designed to decrease velocity and induce sediment deposition or accretion. Retard type structures are permeable structures customarily constructed at and parallel to the toe of a highway fill-slope and/or channel banks. A permeable or impermeable linear structure in a channel, parallel with the bank and usually at the toe of the bank, intended to reduce flow velocity, induce deposition, or deflect flow from the bank.

**RETARDING RESERVOIR** See Reservoir, Retarding.

**RETENTION BASIN** See Basin, Retention.

**RETURN FLOW OR WATER** See Flow, Return.

**RETURN PERIOD** See Flood Frequency.

**REVETMENT** Rigid or flexible armor placed on a bank or embankment as protection against scour and lateral erosion. Compare with Armor, Blanket, and Mattress.

**REVIEW FLOOD (OR STORM) SYSTEM** Drainage facility (or storm drain system) that uses both natural terrain and constructed features to provide for conveyance of runoff in excess of the design flood (or storm system) so as to minimize an increase in the flood hazard from a catastrophic flood (or storm). Flood control facility (or storm drain system) designed through use of prudent judgment and the review flood (or storm runoff).

**REVIEW FLOOD (OR STORM)** See Flood (or Storm), Review.

**REVIEW STORM SYSTEM** See Review Flood (or Storm) System.

**REYNOLDS NUMBER** The effect of viscosity relative to inertia or R = (VL)/v where V is the velocity of flow, L is a characteristic length, and v (nu) is the kinematic viscosity of the liquid.

**RFR** Acronym for Regional Flood of Record or Review Flood of Record.

**RI** Acronym for Recurrence Interval.

**RIFFLE** A rapid in a stream *(37).* Shallow rapids in an open channel, where the water surface is broken into waves by obstructions wholly or partly submerged. A natural shallow flow area extending across a channel bed in which the surface of flowing water is broken by waves or ripples. Typically, riffles alternate with pools along the length of a channel.

**RIFFLE-POOL RATIO** The sum of the riffle lengths divided by the sum of the pool lengths expressed in percent for a given reach. These lengths are usually measured at a relatively low stage.

**RIGHT BANK OF A CHANNEL** The right-hand bank of a channel when the observer is looking downstream. Compare with Left Bank of a Channel.

**RILL** Very small brook or channel made by a small stream, according to *Webster's New Collegiate Dictionary.* A small trickle flowing well within the low flow channel or thalweg.

**RIGHT-OF-WAY** A general term denoting land, property, or interest acquired for, or devoted to highway purposes.

**RIPARIAN** Pertaining to the banks of a stream *(37).* Of, on, or pertaining to the bank of a channel or the shore of a pond or a lake. Pertaining to anything connected with or adjacent to the banks of a channel or other body of water; a riparian owner is one who owns the banks.

**RIPARIAN DOCTRINE OR RULE** A doctrine that holds that the property owner adjacent to a surface water body has first right to withdraw and use the water *(20).* This doctrine may be set aside by a state's statutory law that holds that all surface waters are the property of the state. See Water Right, Flood Right, and Statutory Law.

**RIPARIAN OWNER** A riparian proprietor who owns land on the bank of a river, lake, channel or other body of water. An owner of land, in part bounded generally by a stream or river of water and having a qualified property in the soil to the [thalweg] thread of the channel with the privileges annexed thereto by law. Compare with Riparian Doctrine or Rule and Water Right.

**RIPARIAN PROPRIETOR** See Riparian Owner.

**RIPARIAN RIGHTS** The rights of the owners of lands along a watercourse, relating to such things as water, its use, ownership of soil under the stream or river and accretions. The legal right of a riparian owner to use the water on his riparian land originated in the common law, which

permitted him to require that the waters of a stream or river reach his land undiminished in quantity and unaffected in quality except for minor domestic uses. Compare with Riparian Doctrine or Rule and Water Right.

**RIPARIAN SPUR DIKE** See Dike, Riparian Spur.

**RIPARIAN WATER** Water which is below the highest line of normal flow of a river or stream, as distinguished from floodwaters. Compare with Floodwaters.

**RIPPLE** Two definitions may apply: 1. the light fretting or ruffling of a water surface caused by a breeze; 2. undulating ridges and furrows, or crests and troughs formed by action of the flow.

**RIPRAP** Stones, masonry, or similar constructed material such as broken concrete placed in a loose assemblage along such things as the banks and bed of a channel or the shore of a lake, pond, gulf, bay, or ocean to inhibit erosion and scour. Broken stone or concrete placed on earth surfaces for their protection against the erosive action of water; also, sometimes applied to brush or pole mattresses, or brush and stone, or other similar materials used for protection. In the restricted sense, layer or facing of broken rock or concrete dumped or placed to protect a structure or embankment from erosion and scour; also, the broken rock or concrete suitable for such use. The term "riprap" has also been applied to almost all kinds of armor, including such things as wire-enclosed riprap, grouted riprap, sacked concrete and concrete slabs. Compare with Riprap, Stone; Riprap, Wire-Enclosed; and Riprap, Plated.

**RIPRAP, PLATED** A dense layer of riprap which results from the successive dropping of a heavy, steel plate onto the riprap from a position above the riprap. The plate is raised into dropping position by a crane or similar machine.

**RIPRAP, STONE** Natural cobbles, boulders, or rock preferably of a specified gradation dumped or placed on a channel bank and/or channel bed (commonly with an underlying filter) as protection against erosion. Compare with Riprap, Blanket, and Mattress.

**RIPRAP, WIRE-ENCLOSED** Relatively flat mattresses, cylindrical wire cages, or baskets filled with stone or other suitable material of a specified gradation and placed on a stream bank or bed, often with an underlying filter and used as protection against erosion and scour. Sometimes staked in-place with steel pipe or rail that passes through the mattresses, cages or baskets into the underlying embankment. Usually used as a mattress (see Mattress) rather than in combinations to construction stream and river structures using gabions. Compare with Gabion.

**RISE** Terminology used with culverts (and similar type openings) as the vertical height dimension of a box, pipe-arch, and arch structure, as in Span X Rise. Compare with Span. Can also mean the increase in river stage; a flood; a peak.

**RISING LIMB** The rising portion of a hydrograph preceding a crest *(33).*

**RIVER** Natural stream of water of considerable volume. Depending on local usage, a larger form of a stream. See Channel.

**RIVER, CONTROLLED** A river and attendant floodplain that is regulated by such constructed devices as dams, flood control measures, navigational locks and diversions. Occasionally natural controls may exist such as natural diversions, sinks, deep sand beds, porous geology strata, etc. that also exert some measure of control or regulation of a river. Sometimes a controlled river is referred to as a regulated river. Compare with River, Regulated.

**RIVER FORECAST CENTERS** The organizational units within the National Weather Service that forecast floods *(33).*

**RIVER MECHANICS** Term for the practices that relate the physical laws governing channels, streams, and rivers to practicable engineering applications. Compare with Potomology.

**RIVER, REGULATED** A river and attendant floodplain that is subject to such things as governmental regulations and/or interstate compacts. May also be a river that is controlled by constructed or natural measures. Compare with River, Controlled.

**RIVER TRAINING** The practice of employing structural measures to try and force a stream or river channel to perform in a specified manner. See Channelization and River Training Structures. Compare with Non-Structural Measures.

**RIVER TRAINING STRUCTURE** Any configuration of structural measures constructed in a channel or placed on, adjacent to, or in the vicinity of a channel bank that is intended to deflect currents, induce sediment deposition, induce scour, or in some other way alter the flow and sediment regimes of a stream, or river channel.

**RIVER TRAINING WORKS** See River Training Structures.

**ROAD** A general term denoting a public way for purposes primarily of vehicular travel and includes the entire area within the right of way; generally with emergency parking areas adjacent to the traveled ways (shoulders), but without curb and gutters. The portion of a road within the limits of construction. Compare with Street.

**ROADBED** The graded portion of a highway within top and side slopes, prepared as a foundation for the pavement structure and shoulder.

**ROADWAY CROSS-SLOPE** Transverse slope and/or superelevation described by the roadway section geometry. Usually provided to facilitate drainage and/or resist the centrifugal force of a moving vehicle.

**ROADSIDE** A general term denoting the area adjoining the outer edge of the roadway. Extensive area between the roadways of a divided highway (not necessarily the median) may also be considered roadside.

**ROADSIDE DEVELOPMENT** Those items necessary to complete a highway which provide for: 1. preservation of landscape materials and features; 2. the rehabilitation and protection against erosion of all areas disturbed by construction, seeding, sodding, mulching and the placing of other ground covers; and 3. such suitable planting and other improvements as may increase the effectiveness and enhance the appearance of the highway.

**ROCK-AND-WIRE MATTRESS** See Riprap, Wire-Enclosed.

**ROD-FLOAT** A rod or staff designed to float in a practically vertical position for the purpose of observing velocities.

**ROOT ZONE** The zone from the land surface to the depth penetrated by plant roots. The root zone may contain part or all of the unsaturated zone, depending upon the depth of the roots and the thickness of the unsaturated zone *(20).*

**ROUGHNESS** See Roughness Coefficient.

**ROUGHNESS COEFFICIENT** The estimated measure of texture at the perimeters of channels and conduits. Usually represented by the "n value" coefficient used in Manning's channel flow equation. Numerical measure of the frictional resistance to flow in a channel, as in the Manning or Strickler equations. See Channel Coefficient, Manning's Equation, Manning's n, and Chezy Equation.

**RUBBLE** Rough, irregular fragments of random size placed on a channel bank or shore to retard erosion. The fragments may consist of broken concrete slabs, masonry, or other suitable refuse.

**RULE** An established guide for action. A rule may not be authoritatively enforced, but is generally observed in the interest of such things as order and uniformity. A formal decree, as by court decision. Such rules constitute a large portion of the body of common law. Compare with Rule of Law and Doctrine.

**RULE OF LAW** A legal principle of general application, sanctioned by the recognition of authorities and usually expressed as a maxim or a logical proposition. Called a "rule" because in doubtful or unforeseen cases it is a guide or norm for making decisions. Compare with Rule and Doctrine.

**RUN** Term of regional geographic use. See Channel.

**RUNOFF** Surface Water, Stream Water and Floodwater as defined in Volume V of the *Highway Drainage Guidelines.* That part of the precipitation which runs off the surface of a drainage area after accounting for all abstractions. The portion of precipitation that appears as flow in streams; total volume of flow of a stream during a specified time.

That part of the precipitation that appears in surface streams. It is the same as streamflow unaffected by artificial diversions, storage, or other works of man in or on the stream channels *(37).*

The total amount of water flowing in a stream. It includes Overland Flow, Return Flow, Interflow and Base Flow *(20).*

Runoff may be classified as to:

speed of appearance after rainfall or snow melting:

· direct runoff and
· base runoff; and

source:

· surface runoff (see Overland Flow),
· storm seepage, and
· groundwater runoff.

Compare with Discharge; Runoff; and Runoff, Base.

**RUNOFF, ANNUAL** The total natural discharge of a stream for a year, usually expressed in inches of depth or acre feet.

The total amount of water obtained in a year from such things as a stream, spring, or artesian well. Usually expressed in inches or inches of depth; acre feet; millions of gallons or liters; cubic feet or cubic meters *(51).*

The runoff from the drainage basin, including groundwater outflow that appears in the stream plus groundwater outflow that bypasses the gaging station and leaves the basin underground. Water yield is the precipitation minus the evapotranspiration *(37).*

The actual streamflow, at a given place, from a watershed. This is natural annual runoff that may be affected by irrigation uses, reservoir losses, diversions into or out of the watershed, etc. *(51).*

**RUNOFF, BASE** Sustained or fair weather runoff. In most streams, base runoff is composed largely on groundwater effluent. [Other terms] are often used in the same sense as base runoff groundwater runoff, direct runoff, base discharge and base flow. However, the distinction is the same as that between streamflow and runoff. When the concept in the terms base flow and base runoff is that of the natural flow in a stream, base runoff is the logical term *(37).*

That part of the stream discharge that is not attributable to direct runoff from precipitation or melting snow; it is usually sustained by groundwater discharge *(22).* That part of a stream

discharge derived from groundwater seeping into the stream *(20).* Contribution of groundwater to runoff *(1).*

Stream discharge derived from groundwater sources. Sometimes considered to include flows from regulated lakes or reservoirs. Fluctuates much less than storm runoff *(51).* See Runoff. Compare with Runoff, Direct.

**RUNOFF COEFFICIENT** A factor representing that portion of runoff which results from a unit of rainfall. Dependent on terrain and topography. The rate of runoff to precipitation.

**RUNOFF, DIRECT** The runoff entering stream channels promptly after rainfall or snowmelt. Superposed on base runoff, it forms the bulk of the hydrograph of a flood. See Surface Runoff. The terms base runoff and direct runoff are time classifications of runoff. The term surface runoff is classified according to source *(37).*

The water that enters the stream channels during a storm or soon after, forming a runoff hydrograph. May consist of rainfall on the stream surface, surface runoff and seepage of infiltrated water (rapid subsurface flow) *(51).*

See Runoff. Compare with Runoff, Base.

**RUNOFF, "FIRST FLUSH"** The condition, often occurring, in which a disproportionately high pollution load is carried in the first portion of urban runoff *(1).*

**RUNOFF IN INCHES** The depth to which the drainage area would be covered if all the runoff for a given time period were uniformly distributed on it *(52).*

**RUNOFF, SUBSURFACE** Water that infiltrates the soil and reappears as seepage or spring flow and forms part of the flood hydrograph for that storm. Difficult to determine in practice and seldom worked with separately. See Direct Runoff *(51).* See Storm Seepage.

**RUNOFF, SURFACE** That part of the runoff which travels over the soil surface to the nearest stream channel. It is also defined as that part of the runoff of a drainage basin that has not passed beneath the surface since precipitation. The term is misused when applied in the sense of direct runoff. See also Runoff, Overland Flow, Direct Runoff, Groundwater Runoff, and Surface Water *(37).*

Total rainfall minus interception, evaporation, infiltration and surface storage and which moves across the ground surface to a stream or depression *(51).*

**RUNOFF, URBAN** Storm-generated surface runoff from an urban drainage area. The term may relate to either the quantity or quality of the runoff or both, depending upon its application *(1).*

**RUN-UP, WAVE** See Wave Run-up.

**S.D.** (s.d.) Acronym for standard deviation *(51).*

**SACK REVETMENT** Stream bank protection consisting of sacks (e.g., burlap, paper, or nylon) filled with such things as mortar, concrete, sand, stone, or other available material placed on a bank to serve as protection against erosion.

**SALTATING** Bed load particles that skip along the bed while being transported by streamflows. See also Bed Load.

**SALTATION LOAD** That portion of the total sediment load bounced along the stream bed by energy and turbulence of flow and by other moving particles. Compare with Bed Load and Sediment Load.

**SAMPLE** An element, part, or fragment of a population. Every hydrologic record is a sample of a much longer record *(34).* Compare with Population.

**SAMPLING** See Gaging/Sampling.

**SAND** Soil material that can pass the No. 4 (4.76 mm) U.S. Standard Sieve and be retained on the No. 200 (0.074 mm) sieve. FHWA, HIRE, 1987. Granular material that is smaller than 2.0 mm and coarser than 0.062 mm. FHWA, HIRE, 1990.

**SATURATED SOIL** Soil that has its interstices, pores, or void spaces filled with water to the point at which runoff occurs.

**SATURATED ZONE** See Zone of Saturation.

**SCOUR** The displacement and removal of channel bed material due to flowing water; usually considered as being localized as opposed to general bed degradation or headcutting. The result of the erosive action of running water which excavates and carries away material from a channel bed. Compare with Erosion, Abrasion, Lateral Erosion, Mass Wasting, and Sloughing.

**SCOUR, BENDWAY** That component of natural scour consisting of the removal of material from the channel bed or banks which occurs along the concave (outside) bank in a channel bendway located, generally, across the channel from any point bar.

**SCOUR, CONTRACTION** The response of a river or drainage facility (such as bridge) to the change in its bed load requirement as a result of a natural or constructed contraction of flow; i.e., the flow contraction is due to an encroachment of either the main channel or the floodplain by a natural constriction or the highway embankment. Compare with Scour, General.

**SCOUR, GENERAL** Scour in a channel or on a floodplain that is not localized at a pier, abutment, bendway, or other obstruction to flow. In a channel, general scour usually affects all or most of the channel width; i.e., general scour involves the removal of material from the bed across

all or most of the width of a channel as a result of a natural flow contraction which causes increased velocities and bed shear stress. Compare with Scour, Contraction.

**SCOUR, LOCAL** Removal of material from the channel bed or banks which is restricted to a relatively minor part of the width of a channel. Scour in a channel or on a floodplain that is localized at a pier, abutment, or other obstruction to flow. Local scour is caused by the acceleration of the flow and the development of a vortex system induced by the obstruction to the flow. Does not include the additional scour caused by any contraction, natural channel degradation, or bendway.

**SCOUR, MEASURED** The measured depth to which a surface is lowered by scour below a reference elevation.

**SCOUR, NATURAL** Removal of material from the channel bed or banks which occurs in channels due to the migration of bed forms, shifting of the thalweg, and at bendways and natural contractions. Scour which occurs along a channel reach due to natural causes or an otherwise unstable stream; i.e., no external, constructed causes.

**SCOURED DEPTH** Total depth of the water from water surface to a scoured channel bed. Compare with Depth of Scour.

**SCUPPER** A device and/or vertical hole through such things as bridge decks or roofs for the purpose of deck or roof drainage. Sometimes a horizontal opening in the curb or barrier is called a scupper.

**SEASONAL STREAM** See Stream, Intermittent.

**SECCHI DISK** A disk-type device that, when lowered into surface waters, measures their clarity.

**SECOND LAW OF LAPLACE** See Probability Distribution.

**SECOND-FOOT** Same as cubic foot per second. Generally, this term is no longer used in published reports of Federal agencies *(37).*

See cfs in the *National Engineering Handbook (51).* A term, although incorrect, which is sometimes used in verbal discussions in place of "cubic feet per second." Compare with Cubic Feet Per Second.

**SEDIMENT** Fragmental material that originates from weathering of rocks and is transported by, suspended in, or deposited by water or air or is accumulated in beds by other natural agencies *(15).* Compare with Sediment, Fluvial.

**SEDIMENT CONCENTRATION** Weight or volume of sediment relative to quantity of transporting or suspending fluid or fluid-sediment mixture.

**SEDIMENT DISCHARGE, SUSPENDED** The rate at which the dry weight of sediment passes a section of a stream [or river,] or is the quantity of sediment (as measured by dry weight, or by volume) that is discharged in a given time *(15).* The quantity of sediment that is carried past any cross section of a stream or river above the bed layer in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section. Compare with Bed Load Discharge; Sediment Load, Suspended; Sediment Yield; and Wash Load.

**SEDIMENT DISCHARGE, TOTAL** The sum of suspended sediment discharge and bed load discharge or the sum of bed material discharge and wash load discharge of a stream or river. See Wash Load.

**SEDIMENT, FLUVIAL** Fragmental material transported, suspended, or deposited by water. Compare with Sediment.

**SEDIMENT LOAD, SUSPENDED** Sediment that is supported by the upward components of turbulent currents in a stream and that stays in suspension for an appreciable length of time. Compare with Sediment Discharge, Suspended; and Wash Load.

**SEDIMENT POOL** [That portion of the total] reservoir storage [expressedly] provided for sediment, thus prolonging the usefulness of floodwater or irrigation pools *(51).*

**SEDIMENT YIELD** The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area. Compare with Sediment Discharge, Suspended.

**SEDIMENTATION** The process involving the deposition of soil particles which have been carried by floodwaters. Sometimes erroneously termed "silting" by those unfamiliar with sediment transport and deposition as the deposited material does not necessarily contain much, if any material classified as silt. Compare with Aggradation.

**SEDIMENTATION BASIN** See Basin, Sedimentation.

**SEEPAGE** The slow movement of water through small cracks and pores of the bank material.

**SEICHE** The free oscillation of the bulk of water in a lake and the motion caused by it on the surface of the lake *(7).*

Long-period oscillation of a lake or similar body of water. An oscillation of the water surface of a lake or other large land-locked body of water due to unequal atmospheric pressure, wind, landslides, earthquakes, or other causes, which sets the surface in vibration. Waves that oscillate in lakes, bays, and gulfs from a few minutes to a few hours. Also associated with hurricanes on waters which are not landlocked.

**SEISMIC WAVE** A gravity wave caused by an earthquake.

**SELF-REPORTING GAGE** See Gage, Self-Reporting.

**SEMI-ARID** Geographic areas characterized by light rainfall. More specifically, having from about 10 inches (250 mm) to about 20 inches (500 mm) of annual precipitation. Compare with Arid and Desert.

**SEMILOG PAPER** Short for "semilogarithmic graph paper," which is graph paper having an arithmetic scale along one axis and a logarithmic scale along the other. Either scale is used for the independent variable, as the data require. Commercially available paper has various divisions for the arithmetic scale and various cycles for the logarithmic side [scale] *(51).* Compare with Log Paper and Log-Normal Paper.

**SENSITIVE SURFACE WATERS** See Surface Waters, Sensitive.

**SET-DOWN** See Wind Set-Down.

**SET-UP** See Wind Set-Up.

**SEWER** A conduit for conveying sanitary waste flows. Compare with Storm Sewer; Sewer, Combined; and Drain, Storm.

**SEWER, COMBINED** A sewer that conveys stormwater and, at times, sanitary sewage. See Sewer; and Drain, Storm.

**SEWER, STORM** Principally a drain for conveying stormwater, but at least part of the time, a drain which also conveys raw sewage is termed a storm sewer. Compare with Design Flood (or Storm) System; Sewer; and Sewer, Combined.

**SHALLOW WATER WAVE** See Wave, Shallow Water.

**SHARP-CRESTED WEIR** See Weir, Sharp-Crested.

**SHEAR FORCE** See Tractive Force.

**SHEET FLOW** See Flow, Overland.

**SHIFTING CONTROL** See Control *(37).*

**SHOAL** A place in any body of water where the water is especially shallow. A sandbank or sandbar. Compare with Shoaling.

**SHOALING** The process of deposition of alluvial material resulting in areas with relatively shallow depth (shoals). Compare with Shoal.

**SHOOTING FLOW** See Flow, Supercritical.

**SHORE** Interface between the land and the waters of a lake or pond. See Coast Line.

**SHOULDER** The portion of the roadway contiguous with the traveled way for accommodating stopped vehicles, for emergency use, and for lateral support of the road's base and surface courses.

**SHORE LINE** See Coast Line and Shore.

**SHORT-TERM DEGRADATION** A regulatory term for adverse surface water impacts or losses (generally associated with surface water quality) which essentially disappear within some arbitrary period of time (usually one year) following the disturbance from a highway action.

**SIDE SLOPES** The slope of the sides of a canal, dam, or embankment. Currently sanctioned the naming of the vertical distance first as 1 to 2 (or, frequently, 1:2) meaning a vertical distance of 1 m to 2 m horizontal. Another form, not as subject to misinterpretation by thoughtless transposition, is 1V on 2H.

For slopes less than 45□, the vertical component should be unitary (for example, 1:20). For slopes over 45□, the horizontal component should be unitary (for example, 5:1)

**SIDE-CHANNEL SPILLWAY** See Spillway, Side-Channel.

**SIDEWALK WIDTH** Unobstructed space for exclusive pedestrian use between barriers or between a curb and a barrier.

**SIGNIFICANCE** See Significant.

**SIGNIFICANCE TEST** See Test of Significance.

**SIGNIFICANT** Determined by relating an effect to such things as risk (probability or frequency of occurrence) and cost. Included in environmental determinations are such things as past or expected future accumulative effects (often termed "nibble" effect). See Effect.

**SIGNIFICANT ENCROACHMENT** An encroachment or any direct support of likely base floodplain development by a highway action which: 1. causes a potentially significant flood hazard by adversely impacting natural and beneficial floodplain values; 2. interrupts or terminates the use of a transportation facility needed for emergency vehicles or a community's only evacuation route; 3. is in environmentally sensitive surface waters; or 4. is in regulatory waters.

**SIGNIFICANT FLOOD HAZARD** Where it can be shown with reasonable hydrologic certainty that the 1. expected floodplain limits; or 2. roadway encroachment of the review flood (or storm) caused by an existing or proposed drainage facility will measurably exceed existing or natural floodplain limits or, in the case of guttered stormwaters, encroach on a traveled way (see Spread) to where either prudent judgment, regulatory criteria, an economic analysis, or political considerations indicate an unacceptable flood hazard. The base flood (or storm) will be used to review the design flood (or storm) system to judge if there might be an

unacceptable flood hazard. The 500-year flood is used for this review flood when evaluating bridge scour rather than the 100-year flood.

**SIGNIFICANT WAVE** See Wave, Significant.

**SILL** A grade control structure. See Grade Control Structure. Also a device placed across a channel, or generally transverse to flow in a steep culvert to provide a fishway.

**SILT** Material passing the No. 200 (0.074 mm) U.S. Standard Sieve that is nonplastic or very slightly plastic and exhibits little or no strength when air-dried (Unified Soil Classification System) according to the FHWA's "Highways in the River Environment Manual," January 1987. According to the FHWA's "Highways in the River Environment Manual" (February 1990), material finer than 0.062 mm and coarser than 0.004 mm...

Two other definitions may have some application: 1. waterborne sediment which refers to material that is generally confined to fine earth, sand, or mud, but is sometimes broadened to include all material carried, including both suspended and bed load; 2. deposits of waterborne material, as in a reservoir, on a delta, or on overflowed lands.

**SILTING** An incorrect term for sedimentation, or sediment deposition. See Sedimentation.

**SINKS** Geological provinces [areas] characterized by carbonate rocks or evaporite rocks [that] are susceptible to solution by groundwater, which results eventually in the formation of rock basins called sinks *(14).*

**SINK HOLES** See Sinks.

**SINUOUS FLOW** See Flow, Turbulent.

**SINUOSITY** The ratio of the length of the river thalweg to the length of the valley proper.

**SIPHON, OR INVERTED SIPHON** See Culvert, Sag.

**SKEW** The measure of the angle of intersection between a line normal to the roadway centerline and the direction of the flow in a channel at flood stage in the lineal direction of the main channel.

**SKEW COEFFICIENT** A numerical measure [statistical] or index of the lack of symmetry in a frequency distribution. Function of the third moment of magnitudes about their mean, a measure of a symmetry. Also called coefficient of skew or skew coefficient *(34).*

**SKEWNESS** When data plot in a curve on log-normal paper, the curvature is an indication of data skewness *(51).* See Coefficient of Skewness. Compare with Skew Coefficient.

**SLIPLAW** The initial and separate publication in brochure form of a statute upon enactment, prior to its formal publication in the *U.S. Statutes at Large.*

**SLOPE, ADVERSE** A conduit is on an adverse slope when its slope is positive (slopes upward) in the downstream direction. Compare with Slope, Mild; and Slope, Steep.

**SLOPE, CHANNEL** See Channel Slope.

**SLOPE, CRITICAL** See Critical Slope.

**SLOPE, FILL** See Fill Slope.

**SLOPE GAGE** See Gage, Slope.

**SLOPE, MILD** A conduit is on mild slope when its slope is less than the critical slope for a particular value of discharge. Whether a conduit slope is steep or mild depends on the discharge under consideration as well as the conduit slope—the conduit may be steep for one discharge and mild for another. A conduit is on mild slope if, for a given discharge, the normal depth is greater than critical depth. Compare with Slope Steep; and Slope, Adverse.

**SLOPE, STEEP** A conduit is on steep slope when its slope is greater than the critical slope for a particular value of discharge. Whether a conduit slope is steep or mild, depends on the discharge under consideration as well as the conduit slope—the conduit may be steep for one discharge and mild for another. A conduit is on steep slope if, for a given discharge, the normal depth is less than critical depth. See Flow, Supercritical; Flow, Subcritical; and Critical Flow. Compare with Slope, Mild; and Slope, Adverse.

**SLOPE, SURFACE** The inclination of the water surface expressed as change of elevation per unit of slope length; the sine of the angle which the water surface makes with the horizontal. The tangent of that angle is ordinarily used, with no appreciable error resulting except for the steeper slopes.

**SLOPE-AREA METHOD** A method of estimating unmeasured flood discharges in a uniform channel reach using observed highwater levels. More explicitly a flow measurement method based on an indirect measurement of peak discharge by field survey of a reach of channel and highwater marks, usually after a flood has passed. Discharge is computed by the Manning equation, modified to account for non-uniform flow. See Indirect Methods of Flow Measurement.

**SLOTTED DRAIN INLET** See Inlet, Slotted Drain.

**SLOUGHING** Shallow, transverse movement of a soil mass down a stream bank as the result of an instability condition at or near the surface (also called slumping and sometimes, perhaps incorrectly, mass wasting). Conditions leading to sloughing are: bed degradation, attack at the bank toe, rapid drawdown and slope erosion to an angle greater than the angle of repose of the material. Compare with Mass Wasting, Erosion, Abrasion, and Scour.

**SLUICE FLOW** See Flow, Sluice.

**SLUMPING** See Sloughing.

**SNOW** A form or precipitation composed of ice crystals *(37).*

**SNOW COURSE** A line or series of connecting lines along which snow samples are taken at regularly spaced points *(37) (68).*

**SNOW DENSITY** Ratio between the volume of melt water derived from a sample of snow and the initial volume of the sample; this is numerically equal to the specific gravity of the snow *(37).* The waste content of snow expressed as a percentage by volume. In snow surveys it is the ratio of the scale reading (inches of water) to the length of the snow sample in inches.

**SNOWLINE** The general altitude to which the continuous snow cover of high mountains retreats in summer, chiefly controlled by the depth of the winter snowfall and by the temperature of the summer *(37).*

**SNOWLINE, TEMPORARY** A line sometimes drawn on a weather map during the winter showing the southern limit of the snow cover *(37).*

**SODDING** The act of covering the ground with a section of grass-covered soil held together by matted roots.

**SOFFIT** The inside top of a conduit such as a culvert or storm drain pipe. Compare with Invert.

**SOIL** Finely divided material composed of disintegrated rock mixed with organic matter; the loose surface material in which plants grow. Compare with Dirt. Dirt, an incorrect term, is not soil as it contains humus and other foreign material.

**SOIL GROUP** See Hydrologic Soil Group.

**SOIL-CEMENT** A designed mixture of soil and portland cement compacted at a proper water content to form a veneer or structure that can prevent the erosion of such things as a channel bank, dam face, or inlet and outlet of a drainage structure.

**SOIL COLLOIDS** Suspended, finely divided particles in a continuous medium. The particles do not settle out of the substance readily and are not readily filtered.

**SOIL EVAPORATION** Evaporation of water from moist soils.

**SOIL MOISTURE** Water diffused in the soil; the upper part of the zone of aeration from which water is discharged by the transpiration of plants or by soil evaporation *(37).* The water contained in the unsaturated zone *(20).* Subsurface liquid water in the unsaturated zone expressed as a fraction of the total porous medium volume occupied by water; it is less than or equal to the porosity *(22).*

**SOIL POROSITY** The percentage of the soil (or rock) volume that is not occupied by solid particles, including all pore space filled with air and water. See Porosity.

**SOIL WATER** See Soil Moisture *(22).*

**SOIL-COVER COMPLEX** See Hydrologic Soil-Cover Complex.

**SOIL-WATER-STORAGE** The amount of water the soils (including geologic formations) of a watershed will store at a given time. Amounts vary from watershed to watershed. The amount for a given watershed is continually varying as rainfall or evapotranspiration takes place *(51).*

**SPAN** Terminology used with culverts (and similar type openings) as the horizontal width dimension of such things as a box, pipe-arch, or arch structure, as in Span X Rise. May also be the horizontal distance between bridge piers or abutments. Compare with Rise.

**SPATIAL CONCENTRATION** The dry weight of sediment per unit volume of water-sediment mixture in place or the ratio of dry weight of sediment or total weight of water-sediment mixture in a sample or unit volume of the mixture.

**SPAWNING** The act of depositing eggs or producing spawn.

**SPECIFIC ENERGY** See Energy, Specific.

**SPF** Acronym for Standard Project Flood.

**SPS** Acronym for Standard Project Storm.

**SPILLTHROUGH ABUTMENT** A bridge abutment having a fill slope on the channel side. The term originally referred to the "spillthrough" of fill at an open abutment but is now applied to any abutment having such a slope. Compare with Embankment End Slope, Fill Slope, and Vertical Abutment. See Abutment.

**SPILLWAY** A passage for spilling surplus water; a wasteway.

See Spillway, Principal; and Spillway, Emergency *(51).*

**SPILLWAY, CONTROLLED** A reservoir outlet works wherein the outflow is controlled by gates, valves, or similar flow control devices.

**SPILLWAY DESIGN FLOOD** See Inflow Design Flood *(30).*

**SPILLWAY, EMERGENCY** A rock or vegetated earth waterway around a dam, built with its crest above the normally used principal spillway. Used to assist [or supplement] the principal spillway in conveying extreme amounts of runoff safely past the dam [so as to minimize damage and flood hazards] *(51).* Compare with Spillway, Principal. See Spillway, Uncontrolled.

**SPILLWAY, LATERAL-FLOW** See Spillway, Side Channel.

**SPILLWAY, PRINCIPAL** Conveys all ordinary discharges coming into a reservoir and that portion of an extreme discharge that does not pass through the emergency spillway or outlet works. See Spillway, Uncontrolled.

A concrete or metal pipe or conduit used with a drop inlet dam or floodwater retarding structure. It conveys, in a safe and nonerosive manner, all ordinary discharges coming into a reservoir and all of an extreme amount that does not pass through the emergency spillway *(51).* Compare with Spillway, Emergency.

**SPILLWAY, SIDE-CHAN**NEL A spillway in which the initial and final flow are approximately at right angles to each other; a side-channel spillway.

**SPILLWAY, UNCONTROLLED** A spillway for a reservoir at which floodwater discharge is governed only by the inflow and resulting head in the reservoir. Usually the emergency spillway is uncontrolled.

**SPLASH-OVER** That portion of frontal flow at a grate which splashes over the grate and is not intercepted. Compare with Flow, Frontal.

**SPREAD** The accumulated flow in and next to the roadway gutter. The transverse encroachment of stormwater onto a street. See Street. This water often represents an interruption to traffic flow, splash-related problems and a source of hydroplaning during rainstorms. The lateral distance, in feet or meters, of roadway ponding extending out from the curb or edge of the traveled way. See Encroachment. Compare with Significant Encroachment.

**SPREAD FOOTING** A pier or abutment footing that transfers load directly to the earth.

**SPRIGGING** The act of planting a woody vegetation such as bushes and trees by using sprigs (twigs cut from live vegetation) stuck into the prepared ground. Sprigging must be "greenside up."

**SPRING** A discrete place where groundwater flows naturally from a rock or the soil onto the land surface or into a body of surface waters *(22).*

**SPRING BOX** An enclosure constructed to protect or otherwise contain a flow of water emerging from the ground.

**SPS** Acronym for Standard Project Storm.

**SPUR OR SPUR DIKE** See Dike; Dike, Spur; Dike, Riparian Spur; Dike, Embankment Spur; and Guide Bank.

**SQUARE-EDGED ENTRANCE** See Inlet, Square-Edged.

**STABLE CHANNEL** See Channel, Stable.

**STAFF GAGE** See Gage, Staff.

**STAGE** Height of water surface above a specified datum. Water surface elevation of a channel with respect to a reference elevation. The elevation of a water surface above its minimum; also above or below an established "low-water" plane; hence, above or below any datum of reference; gage height.

The height of a water surface above an established datum plane *(37).* See also Gage Height. The depth of water in a river or stream above the gage datum, or 0.0 level *(33).*

**STAGE, FLOOD** See Flood Stage.

**STAGE-CAPACITY CURVE** A graph showing the relation between the surface elevation of the water in a reservoir, usually plotted as ordinate, against: 1. the volume below that elevation, plotted as abscissa; or 2. amount of water flowing in a channel, expressed as volume per unit of time, plotted as abscissa. Compare with Stage-Discharge Curve and Rating Curve.

**STAGE-DAMAGE GRAPHS** [Charts or graphs that] show the relation between flood depth or elevation and damages in the area *(33).*

**STAGE-DISCHARGE CURVE** A graph showing the relation between the gage height, usually plotted as ordinate and the amount of water flowing in a channel, expressed as volume per unit of time, plotted as abscissa *(37).*

The relation expressed by the Stage-Discharge Curve *(37).* The relation between gage height (stage) and the volume of water, per unit of time, flowing in a channel *(52).*

The relation between stage and the discharge of a stream or river at a particular site, usually presented in the form of a graph or table. Sometimes referred to as the rating curve of a channel cross section. A correlation between channel flow rates and corresponding water surface elevations. A rating curve showing the relation between stage and discharge of a channel.

The relation expressed by the stage-discharge curve. Note that stage-discharge (stage versus discharge) is hyphenated when referring to a curve or relationship; for a particular event, it is more appropriate to refer to the discharge stage (not hyphenated). Compare with Rating Curve and Stage-Capacity Curve.

**STANDARD DEVIATION(S)** Statistician's name for an important measure [statistic] of dispersion, abbreviated s.d. Data grouped closely about their mean have a small s.d.; grouped less closely, they have a larger s.d. *(51).* See S.D. (s.d.).

The standard deviation measures the spread of a probability distribution [i.e., the spread of a sample of data] *(50).*

A measure [statistic] of the dispersion or precision of a series of statistical values such as precipitation or streamflow. It is the square root of the sum of squares of the deviations from the arithmetic mean divided by the number of values or events in the series. It is now standard

practice in statistics to divide by the number of values minus one in order to get an unbiased estimate of the variance from the sample data *(34).*

**STANDARD ERROR** An estimate of the standard deviation of a statistic. Often calculated from a single set of observations. Calculated like the standard deviation but differing from it in meaning *(34).*

Two definitions are paraphrased from Reference 50. The standard error of estimate $\sigma$ or $S_e$ is a measure (statistic) of the vertical (Y axis) variance about a least squares line (a least squares line describing the relationship between a dependent and independent variable/parameter); 2. The standard error of the mean is a measure (statistic) of the reliability the mean (x) as measured by $S/(n)^{1/2}$.

**STANDARD PLANS** Supplementary drawings of standard construction details that are not unique to any one project and are included with the project plans for construction.

**STANDARD PROJECT FLOOD** A totally theoretical ("deterministic" is better term) flood. The magnitude of the flood is computed by taking the precipitation from the greatest storm in the hydrologic region and transposing it to the stream basin and hydraulically routing it through the point of interest. It is [a term] in common usage by the U.S. Army Corps of Engineers. It has no frequency associated with it, but studies indicate that it is usually about half the Probable Maximum Flood and usually has about a 200- to 500-year (COE uses 500- to 1000-year) recurrence interval *(16).*

A Standard Project Flood (SPF) is a relationship of discharge and/or water surface elevation versus time and is developed by applying Standard Project Storm (SPS) runoff (SPS rainfall minus observed infiltration and other losses) to a watershed model. A base flow is usually added and snowmelt runoff may be added to obtain the total flood hydrograph. Depending on the watershed and river channel physical characteristics, available data and study needs, the watershed model may be a simple unit hydrograph or a complex flood routing technique. Then, various techniques are used, ranging from stage versus discharge curves to step backwater calculations to unsteady flood routing, to convert the flood discharge to an SPF water surface elevation. The SPF is used as one convenient way to compare levels of protection between projects, calibrate watershed models and provide a deterministic check of statistical flood frequency estimates *(16) (30).*

**STANDARD PROJECT STORM** The Standard Project Storm (SPS) is a relationship of precipitation versus time that is intended to be reasonably characteristic of large storms that have occurred or could occur in the locality of concern. It is developed by studying the major storm events in the region, excluding the most extreme. For areas east of 105 longitude, the [findings] of SPS studies are published in [the COE] EM 1110-2-1411 as generalized regional relationships for depth, duration and area of precipitation. For areas west of 105 longitude, special studies are made to develop the appropriate SPS estimates. These special studies may

involve transportation and adjustment of a large storm from its observed location to the locality of concern *(30).*

**STANDARD RAIN GAGE** See Gage, Standard Rain.

**STANDING WAVE** A sudden, rise in the water surface, generally fixed in position, such as a hydraulic jump; a standing wave may exist, however, where the principles of the hydraulic jump are not involved. More specifically, a term which when used to describe the upper flow regime in alluvial channels, means a vertical oscillation of the water surface between fixed nodes without appreciable progression in either an upstream or downstream direction. To maintain the fixed position, the wave must have a celerity (velocity) equal to the approach velocity in the channel, but in the opposite direction. See Celerity, Wave; and Froude Number.

**STATIC HEAD** See Head, Static.

**STATION IDENTIFIER** Information transmitted with data to identify the sending station *(33).*

**STATISTIC** [In computing] a number which describes a sample (such a number, for example, as an arithmetic mean, a median, a standard deviation, or a coefficient of variation) we call the number a statistic *(69).* Compare with Parameter, Variable, and Statistics.

**STATISTICS** [The science that] deals with the computation of [a statistic from] sampled data *(14).* Compare with Statistic and Probability.

**STATUTE** See Statutory Law.

**STATUTORY LAW** Law established by a legislative body and set forth in a formal document. In its specific application, law implies prescription and enforcement by a ruling authority.

**STATUTORY LIABILITY** Liability by virtue of a legislative act as differentiated from liability based on the contract of the parties.

**STEADY FLOW** See Flow, Steady.

**STEEP SLOPE** See Slope, Steep.

**STEM, MAIN** See Main Stem.

**STEMFLOW** Rainfall or snowmelt led to the ground down the trunks or stems of plants *(24).* See Interception. Compare with Throughfall.

**STILLING BASIN** A device or structure placed at or near the outlet of a structure for the purpose of inducing energy dissipation where flow velocities are expected to cause unacceptable channel bed scour and bank erosion.

**STILLING WELL** A pipe, chamber, or some other type of compartment with closed sides and bottom except for a comparatively small inlet or inlets communicating with a main body of water. Its purpose is to dampen waves or surges while permitting the water level within the well to rise and fall with the major fluctuations of the main body.

**STOCHASTIC** See Hydrology, Stochastic. Compare with Hydrology, Deterministic.

**STONE RIPRAP** See Riprap, Stone.

**STOP-LOGS** Devices used for temporary closure of an opening in a hydraulic structure. A generic term which is not intended to imply wood logs are used exclusive of other material; i.e., a log, wood plank, cut timber, steel, or concrete slab, or beam of some synthetic material fitting into end guides between walls or piers to close or limit an opening to the passage of water.

**STORAGE** "General Introduction and Hydrologic Definitions" *(37)* suggests two definitions: 1. water artificially impounded in surface or underground reservoirs, for future use (the term regulation refers to the action of this storage in modifying streamflow; see also Conservation Storage, Total Storage, Dead Storage, and Usable Storage); 2. water naturally detained in a drainage basin, such as groundwater, channel storage and depression storage [where] the term "drainage basin storage" or simply "basin storage" is sometimes used to refer collectively to the amount of water in natural storage in a drainage basin.

**STORAGE, BANK** The water absorbed into the banks of a stream channel when the stages rise above the water table in the bank formations; then the water returns to the channel as effluent seepage when the stages fall below the water table *(27)*.

**STORAGE, BASIN** Term sometimes used to refer collectively to the amount of water in natural storage in a drainage basin. See Storage. Compare with Surface Storage.

**STORAGE, CHANNEL** The volume of water at a given time in the channel or over the floodplain of the streams in a drainage basin or river reach. Channel storage is [occurs] during the progress of a flood event [and thus is transient in nature] *(37)*.

**STORAGE, CONSERVATION** Storage of water for later release for useful purposes such as municipal water supply, power, or irrigation in contrast with storage capacity used for flood control *(37)*.

**STORAGE, DEAD** The volume in a reservoir below the lowest controllable level *(37) (64)*.

**STORAGE, DEPRESSION** The volume of water contained in natural depressions in the land surface, such as puddles *(25)*. Water from precipitation which collects in puddles at the land surface *(20)*. Rainfall which is temporarily stored in land surface depressions within a watershed.

**STORAGE RATIO** The net available storage divided by the mean flow for one year *(64)*.

**STORAGE, TOTAL** The volume of a reservoir below the maximum controllable level, including dead storage *(64).*

**STORAGE, USABLE** The volume normally available for release from a reservoir below the stage of the maximum controllable level *(64).*

**STORAGE-INDICATION METHOD** Name often given to a flood-routing method. Also often called the Puls method (after Lows G. Puls), though it is actually a variation of the method devised by Puls *(51).*

**STORAGE-REQUIRED FREQUENCY CURVE** A graph showing the frequency with which storage equal to or greater than selected amounts will be required to maintain selected rates of regulated flow.

**STORM** A disturbance of the ordinary average conditions of the atmosphere which, unless specifically qualified, may include any or all meteorological disturbances, such as wind, rain, snow, hail, or thunder *(37).*

**STORM, BASE** See Flood, Base.

**STORM, CATASTROPHIC** See Flood, Catastrophic.

**STORM DRAIN** See Drain, Storm.

**STORM DURATION** The period or length of storm.

**STORM HYDROGRAPH** See Hydrograph, Storm.

**STORM SEEPAGE** That part of precipitation which infiltrates the surface soil and moves toward the streams as ephemeral, shallow, perched groundwater above the main groundwater level. Storm seepage is usually part of the direct runoff *(37).*

**STORM SEWER** See Sewer, Storm.

**STORM SURGE** Oceanic tidelike phenomenon resulting from wind and barometric pressure changes.

**STORMFLOW** See Runoff, Direct *(37).*

**STORMWATER** See Runoff, Urban *(1).*

**STRATIFICATION** The act or process of becoming layered.

**STREAM** A general term for a body of flowing water. In hydrology the term is generally applied to the water flowing in a natural channel as distinct from a canal. More generally as in the term "stream gaging," it is applied to the water flowing in any channel, natural or artificial *(37).*

To flow in or as if in a stream. Depending on local or regional usage, a lesser form of a river. A body of water that may range in size from a large river to a small rill flowing in a channel. See Rill. By extension, the term is sometimes applied to a natural channel or drainage course formed by flowing water whether it is occupied by water or not. A body of flowing water, whether in an open or closed conduit, a jet of water as from a nozzle; the term is incorrectly used to designate the conduit in which the stream flows.

Streams in natural channels may be classified as follows *(48)* [these and other stream types are defined in more detail on subsequent pages]:

relation of a stream (or river) to time:

· perennial—one which flows continuously or

· ephemeral—one that flows only in direct response to precipitation and whose channel is at all times above the water table;

relation of a stream (or river) to space:

· continuous—one that does not have interruptions in space or

· interrupted—one which contains alternating reaches, that are either perennial, intermittent, or ephemeral; and

relation of a stream (or river) to groundwater:

· gaining—a reach that receives water from the zone of saturation,

· losing—a reach that contributes water to the zone of saturation,

· insulated—a reach that neither contributes water to the zone of saturation nor receives water from it; it is separated from the zones of saturation by an impermeable bed, or

· perched—a reach that is either a losing or insulated and is separated from the underlying groundwater by a zone of aeration.

See Channel and River. Compare with Canal and Ditch.

**STREAM, ANABRANCHED** A stream or river whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars; the width of individual islands or bars is greater than about three times the average bankfull water surface width, and the channels are more widely and distinctly separated than those of a braided stream or river. Compare with Stream, Braided; and Stream, Anastomosing. See Anabranch and Braid.

**STREAM, ANASTOMOSING** Successive division and rejoining (of riverflow) with accompanying islands is the important characteristic denoted by the synonymous terms, braided or anastomosing stream *(42).* Not a common term. A braided stream is composed of

anabranches. Compare with Stream, Braided; and Stream, Anabranched. See Anabranch and Braid.

**STREAM BANK** See Bank.

**STREAM BED** See Bed.

**STREAM, BRAIDED** A stream whose surface is divided at normal stage by small middle bars or small islands. The individual width of bars and islands is less than three times the water width. A braided stream has the aspect of a single large channel within which are subordinate channels. The visual signature of an unstable channel. Braided streams have multiple subordinate channels, which are within the main stream channel. Anabranched streams have more than one channel. Compare with Stream, Anabranched; and Stream, Anastomosing. See Anabranch and Braid.

**STREAM, CONTINUOUS** A stream that does not have interruptions in space *(37).*

**STREAM CONTRACTION** A narrowing of a natural channels waterway. Usually in reference to a drainage facility installed in the roadway embankment.

**STREAM CONSTRICTION** See Stream Contraction.

**STREAM, EFFLUENT** See Stream, Gaining.

**STREAM, ENTRENCHED** Stream or river cut into bedrock or consolidated deposits.

**STREAM, EPHEMERAL** A stream [or river] that flows only in direct response to precipitation and whose channel is at all times above the water table *(37).* A stream or reach of a stream that does not flow continuously for most of the year. A stream or reach of a stream that does not flow for parts of the year. As used here, the term includes intermittent streams or rivers whose flow is less than perennial but more then ephemeral.

**STREAM, FLASHY** Stream characterized by a rapidly rising and falling stage, as indicated by a sharply peaked hydrograph. Most flashy streams are ephemeral but some are perennial.

**STREAM FLOW DEPLETION** The amount of water that flows into a valley or onto a particular land area, minus the water that flows out the valley or off from the particular land area *(8).*

**STREAM GAGE** See Gage, Stream; and Stream-Gaging Station.

**STREAM GAGING** The process and art of measuring the depths, areas, velocities and rates of flow in natural or artificial channels *(17).*

**STREAM, GAINING** A stream or reach of a stream that receives water from the zone of saturation *(37).* A stream or reach of a stream whose flow is being increased by inflow of groundwater *(22).* Also known as an effluent stream *(20).*

**STREAM, GRADED** See Regime of a Stream.

**STREAM, INCISED** A stream or river that flows in an incised channel with high banks; say, banks that stand above the 50-year to 100-year water surface stage are arbitrarily regarded as high. Sometimes termed an entrenched stream or river. See Incised Channel and Incised Reach.

**STREAM, INFLUENT** See Stream, Losing.

**STREAM, INSULATED** A stream or reach of a stream that neither contributes water to the zone of saturation nor receives water from it. It is separated from the zones of saturation by an impermeable bed *(37).*

**STREAM, INTERMITTENT** A stream which flows only at certain times of the year when it receives water from springs or from some surface source such as melting snow in mountainous areas *(37).*

**STREAM, INTERRUPTED** A stream which contains alternating reaches, that are either perennial, intermittent, or ephemeral *(37).*

**STREAM, LOSING** A stream or reach of a stream that contributes water to the zone of saturation *(37).* A stream or reach of a stream in which water flows from the channel bed into the ground *(22).* A stream or reach of a stream that is losing water by seepage into the ground; also known as an Influent Stream.

**STREAM, MEANDERING** See Meandering Channel.

**STREAM ORDER** A method of numbering [ordering] streams as part of a drainage basin network. The smallest, unbranched, mapped tributary is called first order, the stream receiving the tributary is called second order and so on. It is usually necessary to specify the scale of the map used. A first-order stream on a 1:62 500 map, may be a third-order stream on a 1:12 000 map *(41).*

Tributaries which have no branches are designated as of the first order, streams which receive only first-order tributaries are of the second order, larger branches which receive only first-order and second-order tributaries are designated third order and so on. The main stream being always of the highest order *(26).*

**STREAM, PERCHED** A perched stream is either a losing stream or an insulated stream that is separated from the underlying groundwater by a zone of aeration: "General Introduction and Hydrologic Definitions" *(37).*

**STREAM, PERENNIAL** A stream or reach of a stream that flows continuously for all or most of the year.

**STREAM, POISED** A term used by river engineers as applying to a stream that over a period of time is neither degrading nor aggrading its channel. A stream nearly in equilibrium as to

sediment transport and supply. See Regime Theory; Regime Channel; Channel, Stable; and Stream, Graded.

**STREAM POWER** An expression used in predicting bed forms and hence bed load transport in alluvial channels—a parameter comprised of the mean velocity, the specific weight of the water-sediment mixture, the normal depth of flow and the channel slope. A parameter that reflects the ability of a stream to distort its bed to produce such things as bed forms, scour, or deposition.

**STREAM REACH** A length of stream channel selected for use in hydraulic or other computations *(51).*

**STREAM REGIME** See Regime of a Stream.

**STREAM RESPONSE** Changes in the dynamic equilibrium of a stream by any one, or combination of various causes.

**STREAM, STABLE** See Channel, Stable.

**STREAM, SEASONAL** See Stream, Intermittent.

**STREAM-GAGING STATION** A gaging station where a record of discharge of a stream [or river] is obtained. Generally, this term is used only for those gaging stations where a continuous record of discharge is obtained: "General Introduction and Hydrologic Definitions" *(37).*

**STREAM BANK EROSION** Removal of soil particles or a mass of particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions and land use changes may also directly or indirectly lead to stream bank erosion. Compare with Mass Wasting and Sloughing.

**STREAM BANK FAILURE** See Mass Wasting and Sloughing.

**STREAM BANK PROTECTION** Any technique used to prevent erosion or failure of a channel bank. See and compare with Armor, Apron, Blanket, Channel Lining, Mattress, and Revetment.

**STREAM BED** See Bed.

**STREAMFLOW** The discharge that occurs in a natural channel. Although the term discharge can be applied to the flow of a canal, the word streamflow uniquely describes the discharge in a surface stream course. The term "streamflow" is more general than runoff, as streamflow may be applied to discharge whether or not it is affected by diversion or regulation *(37) (52).* Compare with Discharge and Runoff.

**STREAMFLOW RECORD** A tabulation of the flow of a stream. Streamflow records are published annually by the United States Geological Survey in their Water-Supply Papers.

Such things as daily, monthly, annual and instantaneous extremes of discharge are shown therein, along with information about the stream gage.

**STREAMING FLOW** See Flow, Subcritical.

**STREAMLINE FLOW** See Flow, Laminar.

**STREET** A general term denoting a public way for purposes of vehicular travel, usually including curb and gutters, to include the entire area within the right-of-way. Compare with Road.

**STRUCTURES** Such things as bridges, culverts, catch basins, drop inlets, retaining walls, cribbing, access holes, endwalls, buildings, storm drains, service pipes under drains, foundation drains and other appurtenant features.

**STRUCTURAL MEASURES** Methods of reducing damage from floods include [such things as] dams and reservoirs, levees, dikes, floodwalls, diversion channels, bridge modifications, channel alterations, pumping stations and land treatment. [Also] for flood prevention work, any form of earthwork (dam, ditch, levee, etc.) or installation of concrete, masonry, metal, or other material (drop spillway, jetties, riprap, etc.); or installation for forest fire protection (firetowers, roads, firebreaks); or, in some cases, a special planting for nonfarm purposes (stabilization of critical sediment-producing area, etc.) *(33).*

Structural measures reduce the frequency of damaging overflows. Different types of structural flood damage reduction measures have different primary and secondary impacts on flooding. Plan formulation and impact assessment should take into account all impacts and residual flooding from all sources. (The dominant flooding may be from a different source without and with project conditions.) In project planning, both the primary beneficial effects and the secondary effects of the alternatives must be borne in mind and appropriately accommodated. Compare with Non-structural Measures.

**STRUCTURAL METHODS** See Structural Measures.

**STUDENT'S t DISTRIBUTION** See Probability Distribution.

**SUB-BASE** The layer or layers of specified or selected material of designed thickness placed on a subgrade to support a base course. Compare with Sub-Bed Material and Subgrade.

**SUBGRADE** The top surface of a roadbed upon which the pavement structure and shoulders are constructed. Compare with Sub-Bed Material and Sub-Base.

**SUB-BED MATERIAL** Material underlying that portion of the channel bed which is subject to the direct action of the flow. Compare with Subgrade and Sub-Base.

**SUB-SURFACE FLOAT** A submerged body which is attached by a line to and the movement of which is indicated by, a surface float; used for the purpose of observing velocities or the direction of flow.

**SUBCRITICAL FLOW** See Flow, Subcritical.

**SUBLIMATE** In the hydrology sense, the loss of moisture from snow before it can infiltrate due to evaporation and wind.

**SUBMEANDER** Small meander contained within the low flow banks of a main channel, associated with relatively low discharges and rills.

**SUBMERGED INLET** Inlets of culvert like structures having a headwater greater than about 1.2 D, where D is the culvert rise. See Rise.

**SUBMERGED ORIFICE** An orifice which in use is drowned by having the tailwater higher than all parts of the opening. See Orifice.

**SUBMERGED OUTLET** Submerged outlets are those culvert-like outlets having a tailwater elevation greater than the soffit of the culvert.

**SUBMERGED WEIR** See Weir, Submerged.

**SUBMERGENCE** The ratio of the tailwater elevation to the headwater elevation, when both are higher than the crest, the overflow crest of the structure being the datum of reference. The distances upstream or downstream from the crest at which headwater and tailwater elevations are measured are important, but have not been standardized. In culvert terminology, the condition where tailwater or headwater elevation are greater than elevation of the conduit top (soffit).

**SUBSECTION** Part of a cross section; for example, the left floodplain section of a total cross section. Commonly defined as those contiguous portions of the total cross section having the same conveyance characteristics.

**SUBSOIL** The material lying below the surface soil, generally devoid of humus or organic matter. See Soil and Dirt.

**SUBSTITUTE WETLANDS** See Wetlands, Substitute.

**SUBSTRATE** A surface (such as a bed or bank) within surface waters, or beds, banks or shores which are periodically covered by surface waters on which a plant or animal may live, grow, or be attached. Compare with Bed.

**SUBSTRUCTURE** All of the structure below the bearings of simple and continuous spans, skewbacks of arches and tops of footings or rigid frames, together with the backwalls, wingwalls and wing protection railings.

**SUBSURFACE RUNOFF** See Runoff, Subsurface.

**SUBSURFACE WATER** See Water, Subsurface.

**SUBWATERSHED** A watershed that is part of a larger watershed [or drainage area]. It is worked on [analyzed] separately when necessary in order to improve computational accuracy for results on a whole watershed basis or to get results for that [sub] area only *(51)*.

**SUMP INLET** See Inlet, Sump.

**SUMP, WET WELL** See Wet Well Sump.

**SUPERELEVATION** The increase in water surface elevation at the outside of open channel bendways. May also be a transverse tilting of the channel bed (in lined channels with predominately supercritical flow) or the increase in the elevation at the outside edge of a road or traveled way located in a horizontal curve.

**SUPERFLOOD** Any flood or tidal flow with a flow rate greater than that of the 100-year flood. Flood used to evaluate the effects of a rare flow event; a flow exceeding the 100-year flood. It is recommended that the superflood be on the order of the 500-year event or a flood 1.7 times the magnitude of the 100-year flood if the magnitude of the 500-year flood is not known or predictable with available hydrology methods. Compare with Flood, Review; Flood, Catastrophic Flood; Five-hundred Year Flood; Flood, One-Thousand Year; Flood, Probable Maximum; and Design Discharge.

**SUPERSTRUCTURE** The portion of a structure above the substructure. See Substructure.

**SUPPLEMENTAL IRRIGATION** See Irrigation, Supplemental.

**SUPPLEMENTAL SOURCES** See Irrigation, Supplemental Sources.

**SUPPRESSED WEIR** See Weir, Suppressed.

**SURF ZONE** See Breakers.

**SURFACE AREA (LAKE)** See Lake Surface Area.

**SURFACE CURVE** See Water Surface Profile.

**SURFACE FLOAT** A float on a water surface used to indicate velocity or direction of flow.

**SURFACE RUNOFF** See Runoff, Surface.

**SURFACE SLOPE** See Slope, Surface.

**SURFACE STORAGE** Natural or constructed roughness of a land surface, which stores some or all of the surface runoff of a storm. [Such things as] natural depressions, contour furrows and terraces are usually considered as producing surface storage, but stock ponds, reservoirs, stream channel storage, etc. are generally excluded *(51)*.

Stormwater that is contained in surface depressions or basins. Compare with Storage, Basin.

**SURFACE WATER COURSE, NATURAL WATERWAY** See Natural Watercourse.

**SURFACE WATER ENHANCEMENT** Improving existing or new surface water functions and values with practicable measures.

**SURFACE WATER FUNCTION** See Function of Surface Waters.

**SURFACE WATER MITIGATION** The on-site and/ or off-site construction of new surface waters, enhancement of existing surface waters, acquisition and enhancement of existing surface waters, or combinations thereof.

**SURFACE WATER QUALITY** The findings from an evaluation of the importance and degree of excellence of surface water functions, values and features.

**SURFACE WATER VALUE** The various essential and nonessential aesthetics, products and services of sometimes definable value which surface waters provide to society, including such things as fish and wildlife habitat, water supply, improvement of water quality, flood control, bank erosion and shoreline protection, outdoor recreation opportunities, education and research and beauty.

The value, economic or environmental, of a surface water function. Compare with Function of Surface Waters.

**SURFACE WATERS** Water on the surface of the earth *(51).* Any stream, river, lake, pond, or reservoir. Some include wetlands in surface waters.

For regulatory purposes, navigable waters of the U.S. as currently defined by the U.S. Army Corps of Engineers (COE). See Navigable Waters.

A more legally based description might be, depending on the context, water appearing on the land surface in a diffused state for a considerable time, with no permanent source of supply or regular course; as distinguished from water appearing in watercourses, lakes, or ponds. Sometimes considered as overland flow or surface flow. See Overland Flow and Surface Flow. Compare with Streamflow, Discharge, and Runoff.

**SURFACE WATERS, JURISDICTIONAL** See Navigable Waters.

**SURFACE WATERS, RARE** Surface waters or wetlands having features, functions, values, or quality that are uncommon, unique, or seldom occur in the ecoregion. Compare with Surface Waters, Sensitive.

**SURFACE WATERS, SENSITIVE** Those surface waters or wetlands which, by their nature and setting, are inherently important, unique, or rare due to such things as their environment, public use and flood control function. Waters that, without mitigation measures, would be threatened by a highway action. Compare with Surface Waters, Rare.

**SURFACE WATERS THREAT** The likelihood that surface waters or wetlands, or a portion thereof, will be destroyed, degraded, or otherwise adversely impacted, directly or indirectly, through a highway action.

**SURFACE WATERS, UNIQUE** See Surface Waters, Rare.

**SURFICIAL BED MATERIAL** The part (1.2 inches to 2.4 inches) (30 mm to 60 mm) [in dimension]) of the bed material that is sampled using U.S. Series Bed Material Samplers *(52).*

**SUSPENDED** Used in tables of chemical analyses, this term refers to the amount (concentration) of undissolved material in a water-sediment mixture. It is associated with the material retained on a 0.45-micrometer filter *(52).*

**SUSPENDED SEDIMENT DISCHARGE** See Sediment Discharge, Suspended.

**SUSPENDED SEDIMENT LOAD** See Sediment Load, Suspended.

**SWALE** A wide, shallow ditch usually grassed or paved and without well-defined bed and banks. A slight depression in the ground surface where water collects and which may be transported as a stream. Often vegetated and shaped so as not to provide a visual signature of a bank or shore.

**SYNTHETIC FILTER** Fabric of synthetic material that serves the same purpose as a granular filter blanket.

**SYNTHETIC HYDROGRAPH** See Hydrograph, Synthetic.

**SYNTHETIC MATTRESS, MATTING, OR TUBING** A grout, or sand-filled, manufactured, semiflexible casing placed on a channel bank to prevent erosion. See Blanket and Mattress. Compare with Apron.

**SYNTHETIC SERIES** A storm or flood series obtained by taking selected values from a frequency line based on historical data according to the National Engineering Council.

**t DISTRIBUTION** See Probability Distribution.

**TAILWATER** Tailwater (TW) is the depth of flow in the channel directly downstream of a drainage facility. Often calculated for the discharge flowing in the natural stream without the highway effect (but may include other local effects from development), unless there is a significant amount of temporary storage that will be (or is) caused by the highway facility; in which case, a flood routing analysis may be required. The tailwater is usually used in such things as culvert and storm drain design and is the depth measured from the downstream flow line of the culvert or storm drain to the water surface. May also be the depth of flow in a channel directly downstream of a drainage facility as influenced by the backwater curve from an existing downstream drainage facility. With such things as releases from a dam, the water just downstream from a structure.

**TALUS** Rock debris collecting at the base of a cliff.

**TALUS SLOPE** Slope (talus slope) formed by an accumulation of rock debris at the base of a cliff.

**TANK** An artificial reservoir for stock water; local usage generally in the Southwest U.S. *(37).*

**TAPERED ENTRANCE** See Inlet, Tapered.

**TEMPORARY SNOWLINE** See Snowline, Temporary.

**TERRACE** A berm or discontinuous segments of a berm, in a valley at some height above the floodplain, representing a former abandoned floodplain of the stream *(37).*

**TEST OF SIGNIFICANCE** A [statistical] test made to determine the probability that a result is accidental or that a result differs from another result. For all the many types of tests there are standard formulas and tables. In making a test it is necessary to choose a "level of significance," the choice being arbitrary but generally not less than the low level of ten percent nor more than the high level of one percent *(34).*

**TETRAHEDRON** Component of river training works made of six steel or concrete struts fabricated in the shape of a pyramid.

**TETRAPOD** Bank protection component of precast concrete consisting of four legs joined at a central joint, with each leg making an angle of 109.5□ with the other three.

**TEXAS CROSSING** A low class road crossing of a channel designed to pass low, frequently occurring flows through a relatively small culvert type opening with large, more rare floods being conveyed over the road with little or no road damage; such damage often being precluded by: 1. an erosion protection blanket on the downstream fillslope of the road (keyed into the floodplain and channel bottom); and 2. the blanket being monolithic with the shoulder.

**THALWEG** The line or path (such as a rill) connecting the lowest flow points along the bed of a channel. The line does not include local depressions. The path very low flows would follow in proceeding down a stream, river, swale, or channel. The line extending along a channel profile that follows the lowest elevation of the bed.

**THALWEG, WANDERING** A thalweg whose position in the channel shifts during floods and typically serves as an inset channel (or rill) that transmits all or most of the channel flow at normal or lower stages.

**THERMAL STRATIFICATION** Vertical temperature stratification [in a lake or pond] that shows the following: 1. the upper layer of the lake, known as the epilimnion, in which the water temperature is virtually uniform; 2. a stratum next below, known as the thermocline, in which there is a marked drop in temperature per unit of depth; and 3. the lowermost region or

stratum, known as the hypolimnion, in which the temperature from its upper limit to the bottom is nearly uniform *(70).*

**THREAT, SURFACE WATERS** See Surface Waters Threat.

**THERMOCLINE** A central layer of rapid temperature transition that is located between the epilimnion and the hypolimnion of a lake or pond. See Thermal Stratification.

**THRESHOLD VALUE** The value beyond which a significant, adverse effect or impact would probably occur; generally a term applicable to the quality of surface waters.

**THROUGHFALL** In a vegetated area, the precipitation that falls directly to the ground or the rainwater or snowmelt that drops from twigs or leaves *(24).* See Interception. Compare with Stemflow.

**THROUGHFLOW** The lateral movement of water in an unsaturated zone during and immediately after a precipitation event. The water from throughflow seeps out at the base of slopes and then flows across the ground surface as return flow, ultimately reaching a stream or lake *(20).*

**TIDAL AMPLITUDE** Generally, half of the tidal range.

**TIDAL CYCLE** One complete rise and fall of the tide.

**TIDAL INLET** A body of water with an opening to the sea, but otherwise enclosed.

**TIDAL PASSAGE** A tidal channel connecting with the sea at both ends.

**TIDAL PERIOD** Duration of one complete tidal cycle.

**TIDAL PRISM** Volume of water contained in a tidal inlet or estuary, between low and high tide levels.

**TIDAL RANGE** Vertical difference between specified low and high tide levels.

**TIDES, ASTRONOMICAL** Variations in sea level due to the motion of heavenly bodies.

**TIEBACK** Structure connected to such things as a retard, or revetment paralleling a bank, or a blanket located on a bank. Tiebacks extend into or are otherwise tied to the bank to prevent flanking by streamflow.

**TIMBER OR BRUSH MATTRESS** Such things as a revetment, blanket, or armor made of such things as brush, poles, logs, or lumber interwoven or otherwise lashed together. The completed mattress is then placed on the bank of a stream or river and weighted with ballast. See Blanket and Mattress.

**TIME OF CONCENTRATION** The time [$T_c$] required for water to flow from the farthest point on the watershed to the gaging station *(55).* The time that it takes for water to flow from the most distant part of the drainage basin to the measuring point *(20).*

The time ($T_c$) it takes water from the most distant point (hydraulically) to reach a watershed outlet. $T_c$ varies, but [is] often used as a constant *(51).*

The estimated time required for runoff to flow from the most remote section of the drainage area to the point at which the discharge is to be determined. Stated another way, the time it takes water from the most distant point (hydraulically) to reach a watershed outlet.

Not synonymous with Travel Time. Compare with Travel Time.

**TIPPING BUCKET** See Gage, Tipping Bucket.

TOE That portion of a stream cross section where the lower bank terminates and the channel bottom or the opposite lower bank begins.

**TOE DIKE** See Dike, Toe.

**TOPSOIL** Surface soil which is suitable for the germination of seeds and the support of vegetative growth.

**TORRENTIAL FLOW** See Flow, Supercritical.

**TORT** A private or civil wrong committed upon the person or property independent of contract. The elements of every tort action are: 1. existence of legal duty from defendant to plaintiff; 2. breach of duty; and 3. damage as proximate result.

**TORT ACTION** See Tort.

**TORTUOUS CHANNEL** See Channel, Tortuous.

**TORTUOUS FLOW** See Flow, Turbulent.

**TOTAL ENERGY** See Energy.

**TOTAL FISH LENGTH** The greatest possible length of a fish between the mouth with mouth closed and compressed (i.e., squeezed together) and the tail fin to give the maximum overall measurement. See Fish; and Fish, Design.

**TOTAL HEAD** See Head, Total.

**TOTAL HEAD LINE** See Energy Grade Line.

**TOTAL SEDIMENT DISCHARGE** See Sediment Discharge, Total.

**TOTAL STORAGE** See Storage, Total.

**TOTALLY BOUNDED DISTRIBUTION** See Probability Distribution.

**TRACTIVE FORCE** The drag on a stream bank caused by passing water which tends to pull soil particles along with the streamflow. The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually expressed in units of stress; i.e., force per unit area. The force per unit area on a stationary boundary exerted by a fluid flowing past that boundary. Compare with Critical Shear Stress.

**TRAINING DIKE** See Dike, Training.

**TRANQUIL FLOW** See Flow, Subcritical.

**TRANSFORMATION** The change of numerical values of data to make later computations easier, to linearize a plot or to normalize a skewed distribution by making it more nearly a normal distribution. The most common transformation are those changing ordinary numerical values into their logarithms, square roots, or cube roots; many others are possible *(34).*

**TRANSITION** A short conduit and/or channel uniting two other conduits and/or channels having different hydraulic elements; a conversion; a variable conduit or channel section connecting one uniform conduit or channel to another of different cross section form.

**TRANSMISSION LOSS** A reduction in volume of flow in a stream, canal, [channel,] or other waterway, due to infiltration or seepage into the channel bed and banks. Evaporation is also a transmission loss, but it is ordinarily neglected under the assumption that it is small *(51).*

**TRANSPIRATION** The quantity of water absorbed and transpired and used directly in the building of plant tissue in a specified time. It does not include soil evaporation *(8).*

The process by which water vapor escapes from the living plant, principally the leaves and enters the atmosphere. The process by which plants give off water vapor through their leaves *(20).*

**TRAPEZOIDAL WEIR** See Weir, Trapezoidal.

**TRASH RACK** A device used to capture debris, either floating, suspended, or rolling and saltating along the bed, before it enters a drainage facility. A grid or screen across a stream or entrance to a drainage facility designed to catch debris.

**TRAVEL TIME** The average time for water to flow through a reach or other stream or valley length that is less than the total [stream or valley] length. A travel time is part of a $T_C$ [Time of Concentration] but never the whole $T_C$ *(51).*

The average time for water to flow through a reach or other stream or valley length.

Not synonymous with Time of Concentration. Compare with Time of Concentration.

**TRAVELED WAY** That portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes (such as turning lanes and parking lanes).

**TRENCH-FILLED REVETMENT** Stone, concrete, or masonry material placed in a trench dug behind and parallel to an eroding stream bank. When the erosive action of the stream reaches the trench, the material placed in the trench armors the bank and thus retards further erosion. Compare with Windrow Revetment and Revetment.

**TREND** A statistical term referring to the direction or rate of increase or decrease in magnitude of the individual members of a time series of data when random fluctuations of individual members are disregarded *(37).* Compare with Probability Distribution and Cycle.

**TRIANGULAR WEIR** See Weir, Triangular.

**TRIBUTARIES** Lesser branches of the watershed stream system. Compare with Main Stem.

**TROUGH** The long, narrow depression between waves.

**TSUNAMI** A gravity wave caused by an underwater seismic disturbance (such as sudden faulting, landsliding, or volcanic activity). Long period, ocean wave resulting from earthquake or other seismic disturbance. Waves created by earthquakes or other tectonic disturbance on the ocean bottom.

**TUKEY DISTRIBUTION** See Probability Distribution.

**TURBIDITY** Muddy water, having sediment or foreign particles stirred up or suspended. Measured by the Jackson Turbidity Unit (JTU) or Nephelometric Turbidity Unit (NTU). NTU is current (1992) practice.

**TURBULENCE** Motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where all particles of the fluid move in distinct and separate streamlines. A state of flow wherein the water is agitated by cross-currents and eddies; opposed to a condition of flow that is quiet or quiescent.

**TURBULENT FLOW** See Flow, Turbulent; and Turbulence.

**TURBULENT VELOCITY** That velocity above which, in a particular conduit, turbulent flow will always exist and below which the flow may be either turbulent or laminar, depending on circumstances.

**TURNOVER** See Lake Turnover.

**TW** Acronym for Tailwater (depth). [Also an acronym for] Actual Warning Time which is something less than potential warning time (TWP) *(33).*

**TWO-DIMENSIONAL WATER SURFACE PROFILE** An estimated water surface profile with recognizes flow only in the upstream-downstream and transverse direction; vertical velocity vector components are ignored. Compare with One-Dimensional Water Surface Profile.

**TWP** [Acronym for] The maximum time available for warning local communities prior to impending flooding *(33).*

**UNCONTROLLED CROSSING** A bridge crossing that imposes no constraints on the natural width of the stream or on its ability to shift its channel.

**UNCONTROLLED SPILLWAY** See Spillway, Uncontrolled.

**UNDERFLOW** The downstream flow of water through the permeable deposits that underlie a stream and that are more or less limited by rocks of low permeability. See Discharge.

**UNDERGROUND WATERCOURSE** See Watercourse, Underground.

**UNDEVELOPED FLOODPLAIN** See Floodplain, Undeveloped.

**UNGAGED STREAM SITES** Locations at which no systematic records are available regarding actual stream flows or water quality information.

**UNIFORM CHANNEL** See Channel, Uniform.

**UNIFORM FLOW** See Flow, Uniform.

**UNIQUE SURFACE WATERS** See Surface Waters, Rare.

**UNIT DISCHARGE** See Discharge, Unit.

**UNIT HYDROGRAPH** See Hydrograph, Unit.

**UNIT SHEAR FORCE** See Tractive Force.

**UNITGRAPH** See Hydrograph, Unit.

**UNSATURATED ZONE** See Zone, Unsaturated.

**UNSTEADY FLOW** See Flow, Unsteady.

**UPLIFT** The upward water pressure force on the base of a structure.

**UPPER BANK** See Bank, Upper.

**URBAN** As used in this [Glossary] ...this term includes suburban areas *(1).*

**URBAN RUNOFF** See Runoff, Urban.

**USABLE STORAGE** See Storage, Usable.

**USFS** Acronym for U.S. Forest Service.

**USGS** [Acronym for] United States Department of the Interior, Geological Survey *(51)*.

**USWB** [Acronym for] United States Department of Commerce, Weather Bureau. [Weather Bureau changed to National Oceanic and Atmospheric Administration, National Weather Service (see NWS) *(51)*].

**V-NOTCH WEIR** See Weir, Triangular.

**VADOSE ZONE** See Zone, Unsaturated *(22)*.

**VALUE OF SURFACE WATERS** See Surface Water Value.

**VARIABLE** A quantity susceptible of fluctuating in value or magnitude under different conditions according to the *Reader's Digest Great Encyclopedic Dictionary*. Given an equation or relation involving two or more variables (X, Y, Z ... n), the variable(s) which are assigned values {y = f(x, z...n)—in this case x, z...n} are called independent variables and the variable which takes on values (in this case y) is called the dependent variable.

Compare with Parameter.

**VARIABLE, DEPENDENT, OR INDEPENDENT** See Variable.

**VARIANCE** A [statistical] measure of the amount of spread or dispersion of a set of values around their mean, obtained by calculation of the mean value of the squares of the deviations from the mean and hence equal to the square of the standard deviation *(34)*.

**VARIED FLOW** See Flow, Unsteady.

**VEGETATION** Woody or nonwoody plants. Such vegetation is commonly used to stabilize a channel bank, floodplains and other ground areas exposed during highway construction in order to retard erosion.

**VELOCITY** The rate of motion of a stream or river or of the objects or particles transported therein, usually expressed in distance per time. Rate of travel; distance per unit of time.

**VELOCITY, AVERAGE** Velocity at a given cross section determined by dividing the total discharge at that point by the total cross section area.

**VELOCITY, CROSS SECTION AVERAGE** See Velocity, Average.

**VELOCITY, FALL** The velocity of a particle falling alone in quiescent, distilled water of infinite extent.

**VELOCITY HEAD** See Head, Velocity.

**VELOCITY HEAD COEFFICIENT** ($\alpha$) A correction factor, $\alpha$, (alpha) applied to the velocity head for the mean velocity, to correct for non-uniformity of velocity in a cross section. The factor is 1.0 where velocities are identical across a section and greater than 1.0 where velocities vary across a section. Regular channels have coefficients as low as 1.10 whereas overflooded river valleys may have coefficients as high as 2 *(13).* Compare with Momentum Coefficient, $\beta$ (beta).

**VELOCITY, LOCAL AVERAGE** Local discharge intensity divided by depth of flow.

**VELOCITY, MEAN** See Velocity, Average.

**VELOCITY OF APPROACH** The mean velocity in the conduit or channel immediately upstream from a weir, dam, Venturi throat orifice, or other structure; the mean velocity in the approach section.

**VELOCITY, PERMISSIBLE** The highest velocity at which water may be carried safely in a canal or other conduit without channel bed scour or bank erosion.

**VELOCITY-WEIGHTED SEDIMENT CONCENTRATION** The dry weight of sediment discharged through a cross section during a unit of time.

**VENA CONTRACTA** The most contracted section area of a stream, jet, or nappe beyond the plane of the constriction through which it issues.

**VENTURI FLUME** See Flume, Venturi.

**VERIFICATION** The process of testing a model to an observed set of data using the model parameters derived during calibration. Model calibration and verification must be performed on separate sets of data *(1).*

**VERTICAL ABUTMENT** An abutment, usually with wingwalls, that has no fill slope on its channel side. See Abutment. Compare with Spillthrough Abutment.

**VERTICAL-VELOCITY CURVE** A graph of the relation between depth and velocity along a vertical line in a stream, as determined by a set of observations or estimated by computations.

**VICE** "Instead of," or "in lieu of."

**VISCOUS FLOW** See Flow, Laminar.

**VISTAS** The passage framing the approach to such things as a scene, scenic landscape, or surface waterscape.

**WANDERING CHANNEL** See Channel, Wandering.

**WANDERING THALWEG** See Thalweg, Wandering.

**WASH** Term of local geographic usage. See Channel.

**WASH LOAD** That part of the total sediment discharge which is composed of particle sizes finer than those found in appreciable quantities in the bed material. Large quantities of fine materials that could be carried easily by stream flow. That part of the total sediment load that is composed of particle sizes finer than those represented in the bed. Normally the fine-sediment load is finer than 0.062 mm for a sand bed channel. Silts, clays, and sand could be considered as wash load in coarse gravel and cobble bed channels. Wash load sediments commonly originate on uplands and are carried into a channel by overland flows. Compare with Bed Load; Sediment Load, Suspended; Sediment Discharge, Total; and Sediment Discharge, Suspended.

**WATER BALANCE** See Hydrologic Budget.

**WATER BUDGET** An evaluation of all the sources of supply and the corresponding discharges with respect to an aquifer or a drainage basin *(20).* See Hydrologic Budget.

**WATER CONTENT** The amount of water lost from the soil after drying it to constant weight at 105□C, expressed either as the weight of water per unit weight of dry soil or as the volume of water per unit bulk volume of soil *(22).*

The ratio of the volume of soil moisture to the total volume of the soil. This is the volumetric water content; also called volume wetness *(20).*

**WATER CONTENT OF SNOW** See Water Equivalent of Snow.

**WATER CROP** See Water Yield.

**WATER CUSHION** A pool of water maintained to take the impact of water overflowing a dam, chute, drop, or other spillway structure.

**WATER DRAINAGE RIGHTS** The right which a land owner has, under the law, to dispose of excess or surplus water which accumulates upon his land, over the lands of others. Such rights are of several classes—see Common Enemy Doctrine or Rule; Civil Law Doctrine or Rule; Natural Drainage Doctrine or Rule; and Reasonable Use Doctrine or Rule.

Not to be confused with water right. Compare with Water Right and Flood Right.

**WATER EQUIVALENT OF SNOW** Amount of water that would be obtained if the snow should be completely melted. Water content may be merely the amount of liquid water in the snow at the time of observation *(37).*

The depth of water obtained by melting a given thickness of snow *(20).* The depth of water, in inches, that results from melting a given depth of snow *(51).*

**WATER LAW** See Doctrine, Rule, Riparian Doctrine or Rule, Water Right, Flood Right, Common Law, Civil Law, Common Enemy Doctrine or Rule, Civil Law Doctrine or Rule, Natural Drainage Doctrine or Rule, Reasonable Use Doctrine or Rule, Rule of Law, and Water Drainage Rights.

**WATER LEVEL** A water surface; also, its elevation above any datum; gage height; stage.

**WATER LOSS** The difference between the average precipitation over a drainage basin and the water yield from the basin for a given period. The basic concept is that water loss is equal to evapotranspiration, that is water that returns to the atmosphere and thus is no longer available for use. However, the term is also applied to differences between measured inflow and outflow even where part of the difference may be seepage *(37).*

Variable meaning, depending on personal interest of water users. Farmers and ranchers usually think of flood runoff as a water loss; many river engineers think of infiltration as a water loss *(37).* Compare with Loss, Hydrology.

**WATER REQUIREMENT** The quantity of water, regardless of its source, required by a crop in a given period of time, for its normal growth under field conditions. It includes surface evaporation and other economically unavoidable wastes *(8).*

**WATER, RETURN** See Flow, Return.

**WATER RIGHT** An adjudication of waters (usually by a State agency) to a specified user for a beneficial purpose. Compare with Flood Right. See Beneficial Use.

**WATER, SUBSURFACE** All water that occurs below the land surface *(22).*

**WATER SUPPLY PAPER (WSP)** An annual publication of the USGS, in which streamflow for the water year is given for all gaged streams in a geographical region of the United States.

**WATER TABLE** The upper surface of a zone of saturation. No water table exists where that surface is formed by an impermeable body *(47) (37).*

The upper surface of a zone of saturation except where that surface is formed by a confining unit. The upper surface of the zone of saturation on which the water pressure in the porous medium equals atmospheric pressure. Means that surface in a groundwater body at which the water pressure is atmospheric. Upper surface of a zone of saturation, where the body of groundwater is not confined by an overlying impermeable zone.

The surface in an unconfined aquifer or confining bed at which the pore water pressure is atmospheric. It can be measured by installing shallow wells extending a few feet into the zone of saturation and then measuring the water level in those wells *(20).*

The upper surface of groundwater *(51).*

The upper surface of a zone of saturation in soil or in permeable strata or beds. The upper surface of the zone of saturation, except where that surface is formed by an impermeable body. Compare with Groundwater, Perched.

**WATER YEAR** In the Federal agency reports dealing with surface-water supply, the 12-month period, October 1 through September 30. The water year is designated by the calendar year in which it ends and which includes 9 of the 12 months. Thus, the year ended September 30, 1959, is called the "1959 water year" *(37).*

The year taken as beginning October 1. Often used for a convenience in streamflow work, since in many areas streamflow is at its lowest at that time. Used by the U.S. Geological Survey in their WSP National Engineering Handbook *(51).*

**WATER YIELD** See Annual Runoff.

**WATER-LEVEL (STAGE) RECORDER** A device for producing a graphic record of the rise and fall of a water surface with respect to time. Compare with Gage and Crest-Stage.

**WATER SURFACE PROFILE** A graph of water levels plotted against stream distance at a particular time or for a particular condition, such as for a flood peak or for a low-flow period. Two other definitions are appropriate: 1. the longitudinal profile assumed by the surface of a stream of water flowing in an open conduit (the surface curve of a stream of water is the curve of equilibrium of all forces acting on the flowing water); 2. the hydraulic grade line. See Hydraulic Grade Line. Water-surface curves or profiles are generally catalogued into twelve classifications, three of which are designated strictly as backwater curves. The classifications are accounted for by the different bottom slopes and relative values of normal and critical depth. The curves are classified by the nomenclature; M1, M2, and M3 for mild slope (backwater curves); C1 and C3 for critical slope; H2 and H3 for horizontal (zero slope); S1, S2, and S3 for steep slope; and A2 and A3 for adverse slope. Compare with Backwater Curve.

**WATERCOURSE** A stream, river, or channel in which a flow of water occurs, either continuously or intermittently, with some degree of regularity. See Channel, River, and Stream.

**WATERCOURSE, NATURAL** See Channel, Natural.

**WATERCOURSE, UNDERGROUND** A geological formation which contains water flowing in a known and defined channel. A water right for water in underground watercourses are in most States similar to the water right for water in natural surface watercourses.

**WATERS OF THE UNITED STATES** See Navigable Waters.

**WATERSHED** See Drainage Area.

**WATERSHED MEASURES** Any vegetative or structural means (including earthwork) of directly improving or conserving the soil and water resources of a watershed. See Land Treatment Measure and Structural Measure *(51).*

**WATERWAY** Any stream, river, lake, pond, or ocean that can be traversed for purposes of commerce or recreation. May also refer to a channel. See Channel and Watercourse.

**WATERWAY OPENING** See Waterway Opening Area.

**WATERWAY OPENING AREA** Area of bridge opening at (below) a specified stage, measured normal to principal direction of flow.

**WATERWAY OPENING DEPTH** Depth corresponding to a waterway opening area. Requires clarification in some cases as to whether this is the average depth, maximum depth, minimum depth, normal depth, or some other measure of depth.

**WATERWAY OPENING WIDTH** Width corresponding to a waterway opening area. Requires clarification in some cases as to whether this is a top width, bottom width, average width, or some other measure of width.

**WAVE ATTACK** Impact of waves on a channel bank or shore.

**WAVE CELERITY** See Celerity, Wave.

**WAVE FREQUENCY** Wave frequency, n, is the number of waves passing a point in the liquid per unit time *(14).*

**WAVE PERIOD** Time period between arrivals of successive wave crests at a point.

**WAVE RUN-UP** Height to which water rises above still-water level when waves meet such things as a beach, wall, embankment, or causeway. Compare with Wave Set-Up and Wind Set-Up.

**WAVE SET-UP** The creation of waves from wind. Compare with Set-Up, Wind Set-Up, Wind Set-Down, and Wave Run-Up.

**WAVE, SHALLOW WATER** Water of such a depth that waves are noticeably affected by bottom conditions; customarily, water shallower than half the wavelength.

**WAVE, SIGNIFICANT** A statistical finding for denoting surface water waves with the average height and period of the one-third highest wave of a given wave group.

**WEEPHOLE** An opening left in such things as an impermeable wall, revetment, apron, lining, or foundation, to relieve the neutral stress or pore water pressure and permit drainage.

**WEIBULL DISTRIBUTION** See Probability Distribution.

**WEIGHTED MEAN** [Statistical] value [or statistic] obtained by multiplying each of a series of values by its assigned weight and dividing the sum of those products by the sum of the weights *(34).*

**WEIR** A dam across a channel for diverting flows or for measuring the flow.

**WEIR, BROAD-CRESTED** An overflow structure on which the nappe is supported for an appreciable length; a weir with a significant dimension in the direction of the stream. Highways generally function as broad-crested weirs when overtopped by floodwaters.

**WEIR, CIPOLLETTI** A contracted measuring weir, in which each side of the notch has a slope of 1 horizontal to 4 vertical, to compensate for end contractions; named after Caesar Cipolletti, an Italian engineer.

**WEIR FLOW** Free surface flow over a control surface which has a defined discharge versus depth relationship.

**WEIR, MEASURING** A device for measuring the flow of water. It generally consists of rectangular, trapezoidal, triangular, or other shaped notch in a thin plate in a vertical plane through which the water flows. The weir head is an index of the rate of flow. Unless a suppressed weir is specified the term may be taken to mean a contracted weir. See Weir, Cipolletti; Weir, Rectangular; and Weir, Triangular.

**WEIR, RECTANGULAR** A contracted measuring weir with a rectangular notch.

**WEIR, SHARP-CRESTED** A contracted measuring weir with its crest at the upstream edge or corner of a relatively thin plate, generally of metal.

**WEIR, SUBMERGED** A weir which in use has the tailwater level equal to, or higher than the weir crest. See Weir, Measuring.

**WEIR, TRAPEZOIDAL** A contracted measuring weir with a trapezoidal notch. See Weir, Cipolletti.

**WEIR, TRIANGULAR** A contracted measuring weir notch with sides that form an angle with its apex downward; the crest is the apex of the angle; a V-notch weir.

**WELLS** Shallow to deep vertical excavations, generally with perforated or slotted pipe backfilled with selected aggregate. The bottom of the excavation terminates in pervious strata below the water table.

**WET WELL SUMP** The feature in a pump station in which runoff waters are temporarily stored.

**WETLAND ACQUISITION** As used in the National Wetlands Priority Conservation Plan, any purchase of complete or partial interest in a wetland site obtained with total or partial government funding.

**WETLAND BANK** That document maintained to reflect wetland credit that may be available for use in the mitigation of a wetland impacted by a future highway action.

**WETLAND BANKING** Although not required at the time of a highway action, the process of creating, restoring or enhancing a wetland in conjunction with the highway action to

compensate for unavoidable and at the time unknown wetland impacts from some future highway action and then obtaining approval from the cognizant resource and regulatory agencies for this work as a wetland credit to be entered into the wetland bank.

**WETLAND CREDIT** The credited value in the wetland bank for substitute wetlands in a particular geographic area and biologic region as approved by the cognizant resource and regulatory agencies.

**WETLAND LOSS INDEX** Measure of loss of a wetland type within an ecoregion expressed by the equation:

$$\left[\frac{(Y - X)\,(100)}{N}\right]\left[\frac{(Y - X)(100)}{Y}\right]$$

$$\left[\frac{(\text{Unit Loss})(100)}{(\text{Net National Loss})}\right]\left[\frac{(\text{Unit Loss})(100)}{(1954\ \text{Unit Base})}\right]$$

Where

| | | |
|---|---|---|
| Y | = | 1954 Unit Base $km^2$ per wetland type and unit area; |
| X | = | 1974 Remaining $km^2$ per wetland type and unit area; |
| Y - X | = | Unit Loss (e.g., 1954-74 State loss per wetland type); |
| N | = | 1954-74 Net National Loss per wetland type; |
| Unit | = | Area of comparison (e.g., ecoregion, State); and |
| Base | = | $km^2$ of wetlands in 1954 for the unit. |

**WETLAND LOSSES, HISTORIC** The losses of wetlands from a particular site or loss of a specific type of wetlands within a region from the time of European settlement of the United States through the present.

**WETLANDS** Those lands having: wetland hydrology; hydric soils; and hydrophyte type vegetation as delineated by current editions of the *Federal Manual for Identifying and Delineating Jurisdictional Wetlands.* These include wetlands subject to Federal law regardless of whether they involve Federal, State, or private lands.

More generally, an area that is inundated or saturated by surface waters or groundwater at a frequency and duration sufficient to support and under normal circumstances does support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs and similar areas. Mudflats, sand flats, rocky shores, gravel beaches and sand bars, although they often do not support vegetation, can also be considered wetlands. Wetlands typically have hydric soils, phreatic vegetation and wetland hydrology.

Some regulatory agencies may prefer land that has a predominance of hydric soils that is inundated or saturated by surface or groundwater at a frequency and duration sufficient to support "and that under normal circumstances does support," a prevalence of hydrophytic vegetation typically adapted for life in saturated soil conditions.

**WETLANDS, JURISDICTIONAL** See Jurisdictional Surface Waters.

**WETLANDS, REPLACEMENT** Replacement wetlands provided through mitigation methods to replace wetlands to be destroyed or adversely impacted on a specific highway project. Not synonymous with substitute wetlands. Compare with Wetlands, Substitute.

**WETLANDS, SUBSTITUTE** Wetlands that were created or acquired and developed (enhanced) in perpetuity through application of wetland mitigation methods on previously constructed highway projects. Not synonymous with replacement wetlands. Compare with Wetlands, Replacement.

**WETTED PERIMETER** The boundary over which water flows in a channel, stream, river, swale, or drainage facility such as a culvert or storm drain. The boundary is taken normal to the flow direction of the discharge in question. The length of the wetted contact between a stream of water and its containing conduit, measured along a plane at right angles to the flow in question; that part of the periphery of the cross section area of a stream in contact with its container. See Hydraulic Radius.

**WIDTH CONSTRICTION** A constriction whereby the flow is contracted in the horizontal plane only; no bottom contraction; a bridge with approach embankments across a floodplain is an example of a width constriction.

**WILDLIFE** Living things that are neither human or domesticated; mammals, birds and fishes [that may be] hunted by man according to the *Webster's New Collegiate Dictionary.* Compare with Livestock and Aquatic Life.

**WIND SET-DOWN** Lowering of the level of surface waters due to wind action. Corresponding fall in level at the windward side of a surface water body due to wind stresses on the surface. Compare with Wind Set-Up.

**WIND SET-UP** Raising of the level of surface waters due to wind action. Rise in level at the leeward side of a surface water body due to wind stresses on the surface. See Set-up. Compare with Wind Set-Down, Wave Run-Up, and Wave Set-Up.

**WINDROW REVETMENT** A row of stone (called a windrow) placed on top of the bank landward of an eroding stream bank. As bank erosion and mass wasting continues the windrow is eventually undercut, launching the stone downslope, thus armoring the bank face. Compare with Trench-Filled Revetment and Revetment.

**WIRE-ENCLOSED RIPRAP** See Riprap, Wire-Enclosed.

**WIRE MESH** Wire woven to form a mesh, the openings of which are of a suitable size, shape and durability to serve as an enclose for such things as rock, rubble, or broken concrete used as a mattress, gabion, or wire enclosed riprap. Also used as fencing on riparian spur dikes and retards.

**WITHDRAWAL USE OF WATER** The water removed from the ground or diverted from a stream or lake for use *(45).*

**WSP** Acronym for Water Supply Paper. See Water Supply Paper or Water Surface Profile.

**X-PERCENT CHANCE FLOOD** A flood magnitude that has X chances in 100 of being exceeded in any 1-year period. The occurrence of floods is assumed to be random in time; no schedule or regularity of occurrence is implied. The exceedance of a 1-percent chance flood is no guarantee, therefore, that a similar size flood will not occur next week or next year. Over time periods longer than 1 year, the risks of experiencing large floods increase in a nonadditive fashion. For example, the risk of exceeding a 1-percent chance flood one or more times during a 3-year period is 25 percent and during a 70-year period is 50 percent *(31).* Compare with Flood, One-Hundred Year; Flood, One-Thousand Year; and Flood-Frequency. See Probability Distribution.

**YEAR** See Climatic Year and Water Year *(37).*

**ZONE OF AERATION** The zone above the water table. Water in the zone of aeration does not flow into a well *(37).*

**ZONE OF BREAKUP** See Breakers.

**ZONE, COASTAL** See Coastal Zone.

**ZONE, FLOOD** See Flood Zone.

**ZONE OF SATURATION** The zone in which the functional permeable rocks are saturated with water under hydrostatic pressure *(47).* Water in the zone of saturation will flow into a well and is called groundwater *(37).* The Groundwater Subcommittee offers three definitions: 1. those parts of the earth's crust in which all voids are filled with water under pressure greater

than atmospheric; 2. that part of the earth's crust beneath the regional water table in which all voids, large and small, are filled with water under pressure greater than atmospheric; 3. means that part of the earth's crust beneath the regional water table in which all voids, large and small, are ideally filled with water under pressure greater than atmospheric.

The zone in which the voids in the rock or soil are filled with water at a pressure greater than atmospheric. The water table is the top of the saturated zone in an unconfined aquifer *(20).*

**ZONE, LIMNETIC** See Limnetic Zone.

**ZONE, LITTORAL** See Littoral Zone.

**ZONE, PROFUNDAL** See Profundal Zone.

**ZONE, RECOVERY** See Recovery Zone.

**ZONE, UNSATURATED** The Groundwater Subcommittee offers five definitions: 1. the zone between the land surface and the water table; 2. the zone between the land surface and the deepest water table which includes the capillary fringe (generally water in this zone is under less than atmospheric pressure and some of the voids may contain air or other gases at atmospheric pressure; also, beneath flooded areas or in perched water bodies, the water pressure locally may be greater than atmospheric); 3. the zone between the land surface and the regional water table (generally, water in this zone is under less than atmospheric pressure and some of the voids may contain air or other gases at atmospheric pressure; also, beneath flooded areas or in perched water bodies, the water pressure locally may be greater than atmospheric); 4. the zone between the land surface and the regional water table (generally, fluid pressure in this zone is less than atmospheric pressure and some of the voids may contain air or other gases at atmospheric pressure; also, beneath flooded areas or in perched water bodies the fluid pressure locally may be greater than atmospheric); 5. the zone between the land surface and the water table (generally, water in this zone is under less than atmospheric pressure and some of the voids may contain air and other gases at atmospheric pressure; also, beneath flooded areas or in perched water bodies, the water pressure locally may be greater than atmospheric).

The zone between the land surface and the water table. It includes the root zone, intermediate zone and capillary fringe. The pore spaces contain water at less than atmospheric pressure, as well as air and other gases. Saturated bodies, such as perched groundwater, may exist in the unsaturated zone *(20).*

## G.3 References

References include two types:

- cited and
- uncited.

Cited References

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