# 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.

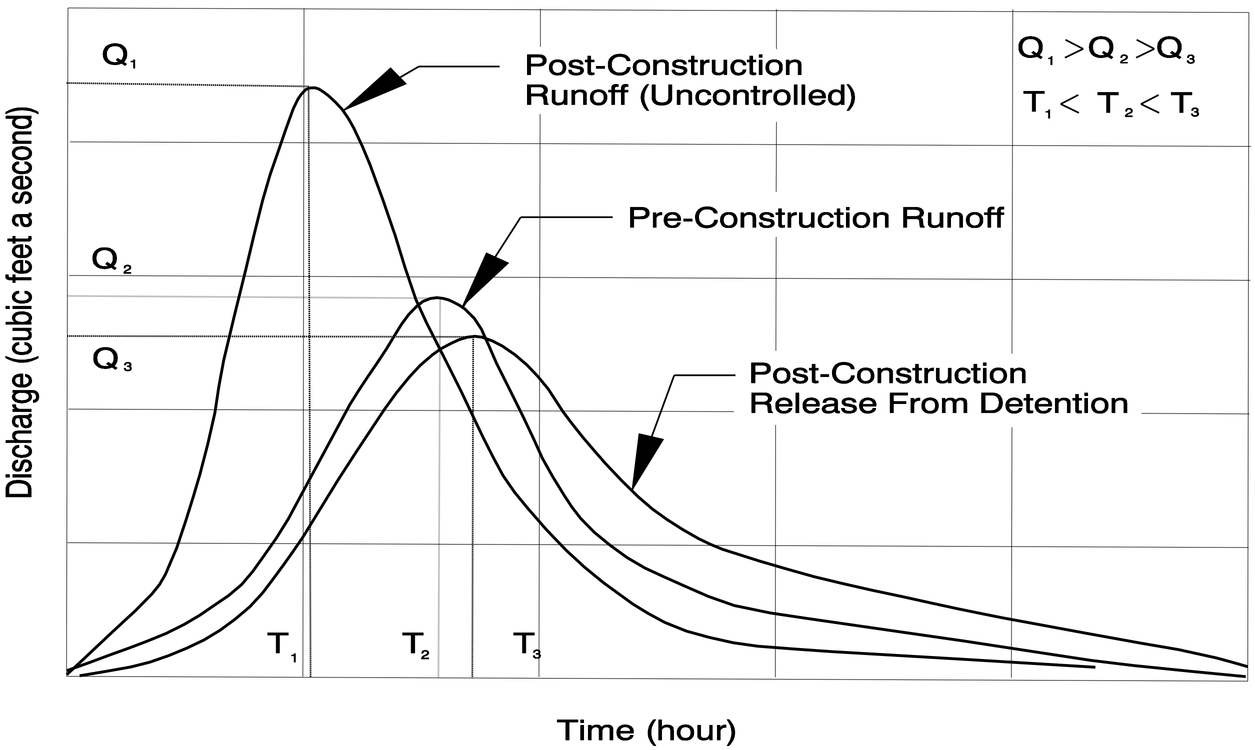



**Figure 12-1 Hydrograph Schematic**

## 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)
    - o Western Oregon = 42% of the 2-year, 24-hour event
    - o 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:
    - o For incised streams (those with an Entrenchment Ratio>=2.2)
        - The 10-year, 24-hour event
    - o 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.

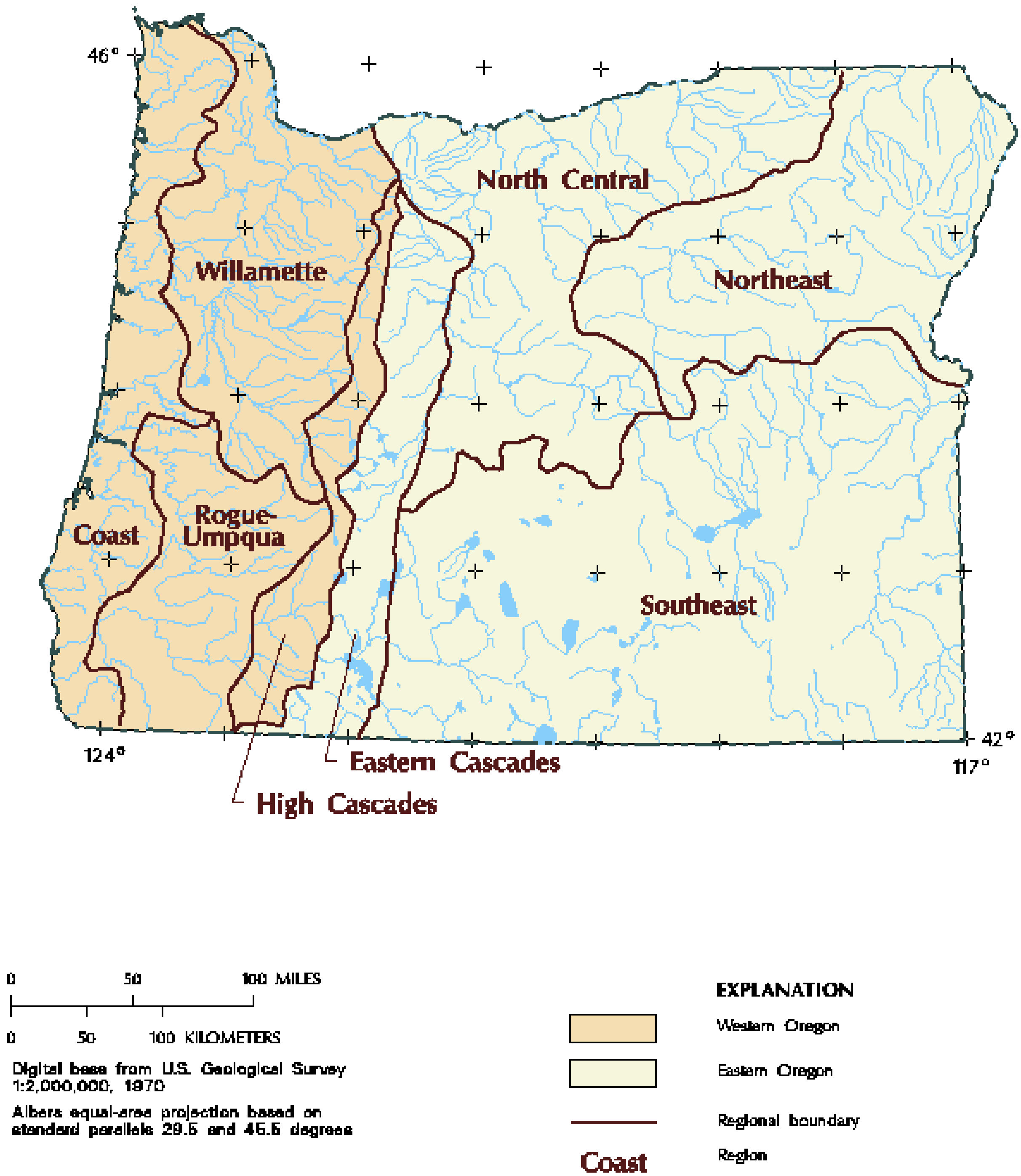



**Figure 12-2 USGS Flood-frequency region map for Oregon**

*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

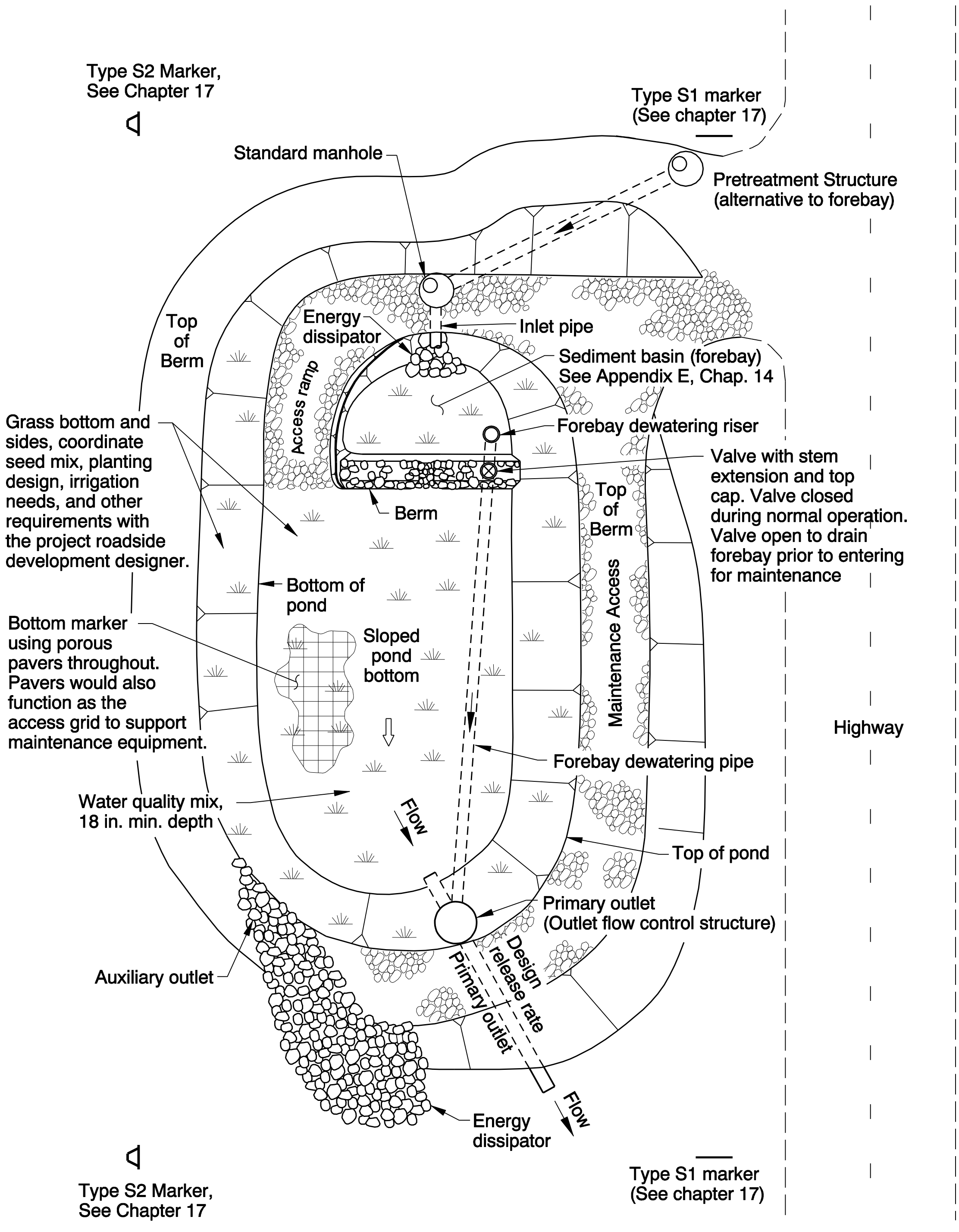



**Figure 12-3 Typical Detention Pond**

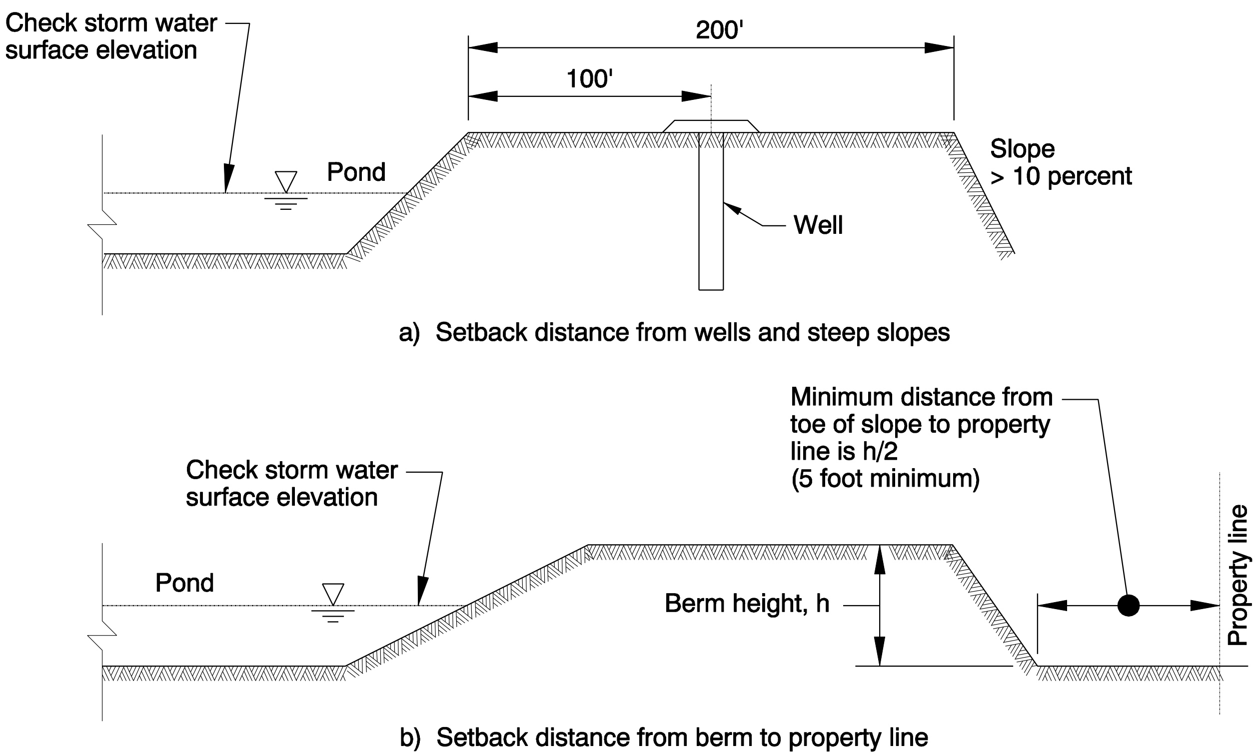



**Figure 12-4 Setback**

## 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.

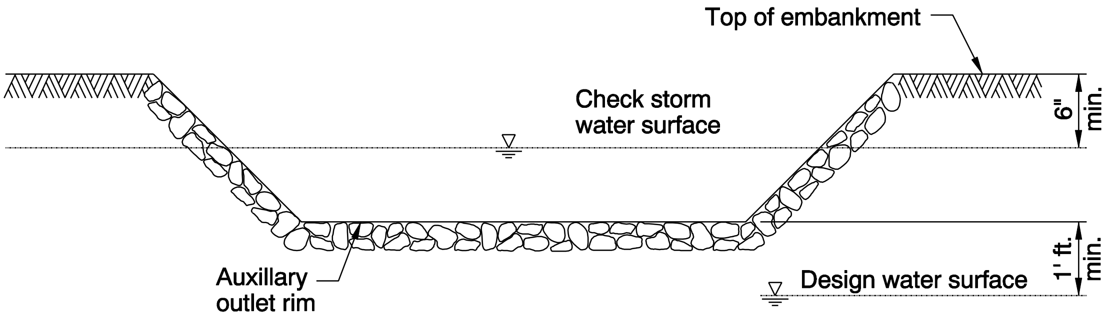



**Figure 12-5 Freeboard**

## 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.

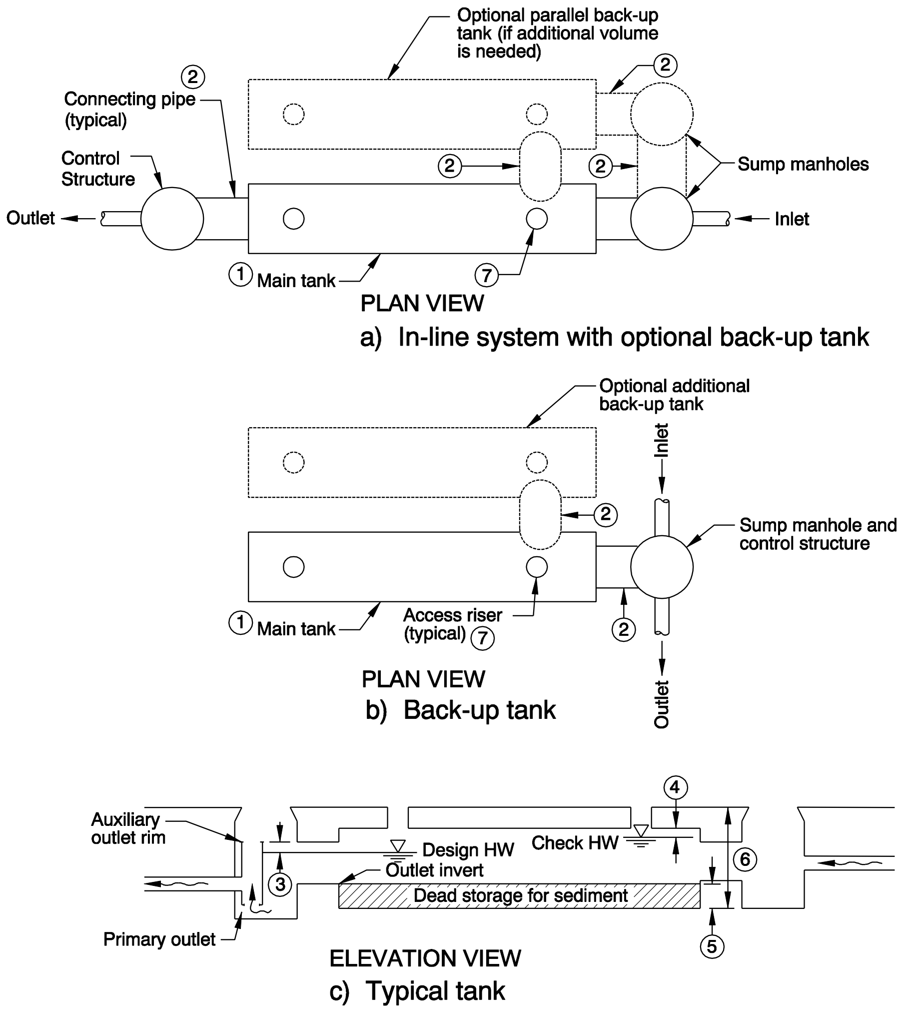
- **Check Storm –** 6 inches from the check storm high water elevation to the top of the tank. The water surface elevation for the check storm freeboard calculations is based on the entire flow passing through the auxiliary outlet and no flow through the primary outlet.

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**.

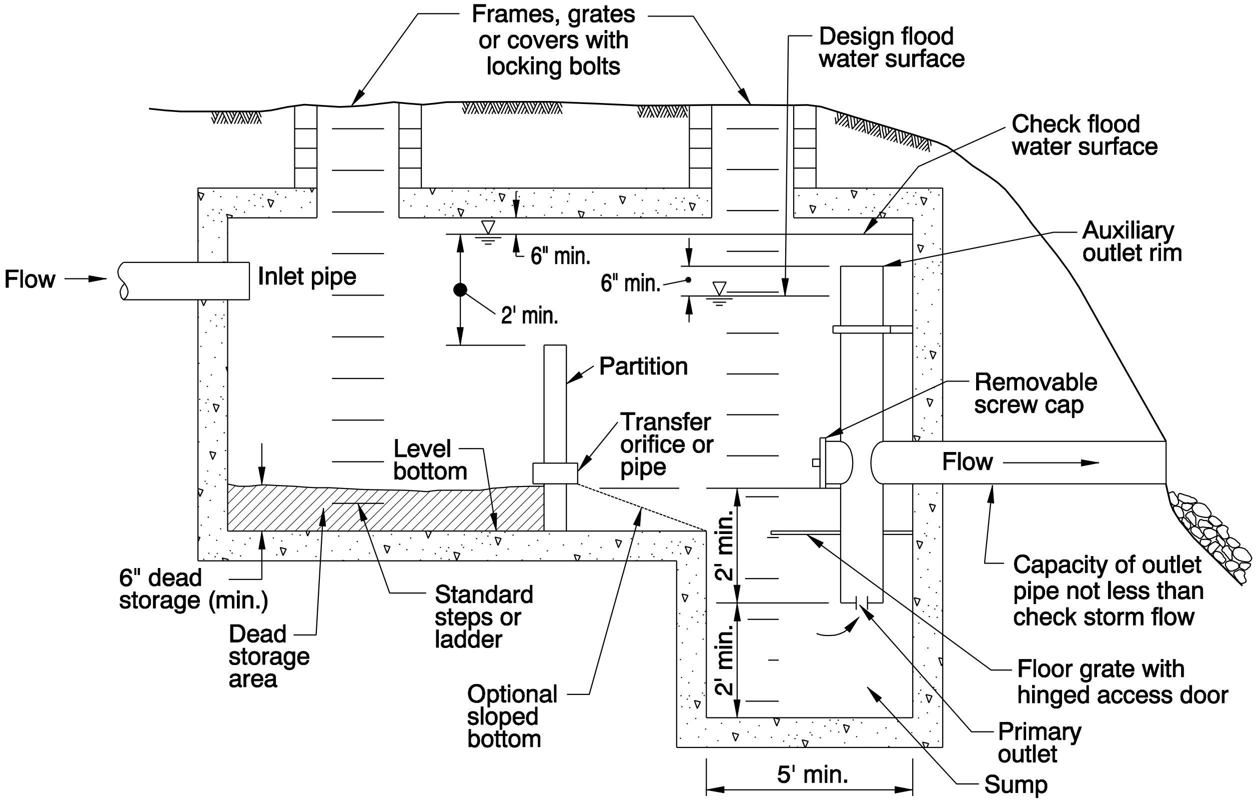



**Figure 12-7 Vault**

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

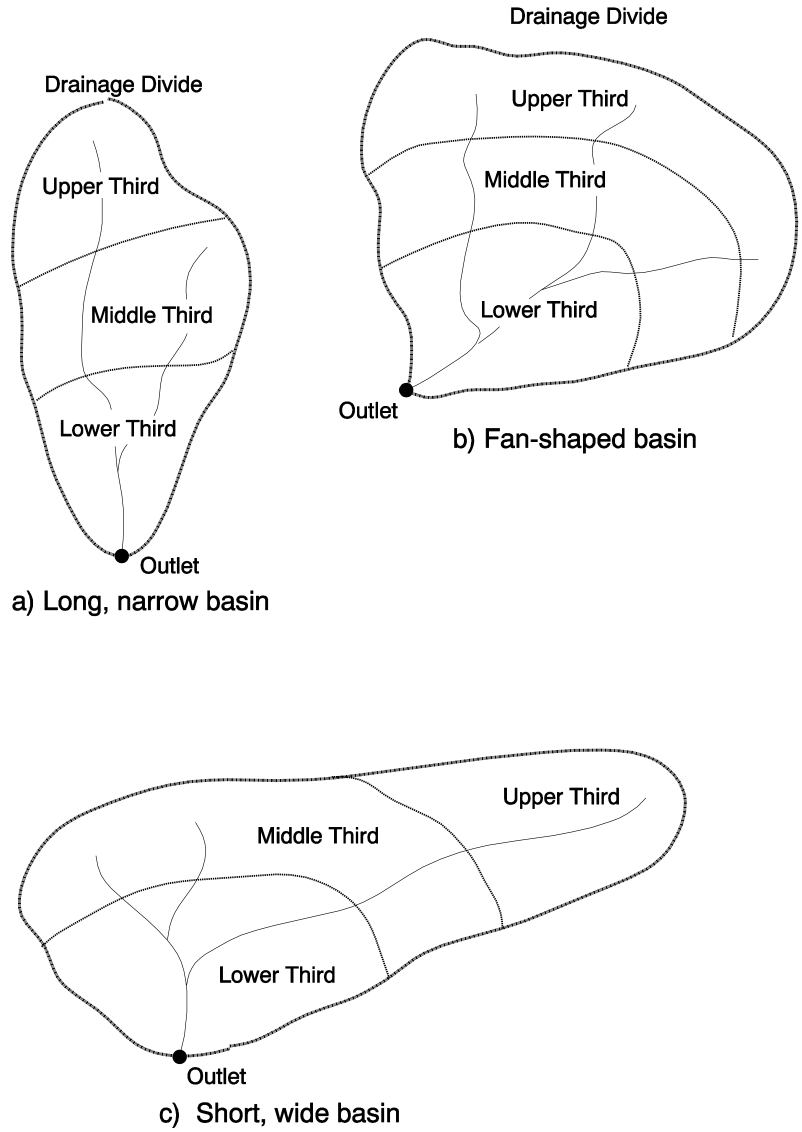
Resources Dam Safety program for review and approval prior to construction of the project works. The Water Resources Department should be consulted for further permitting, details and engineering requirements when preparing the design.

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.

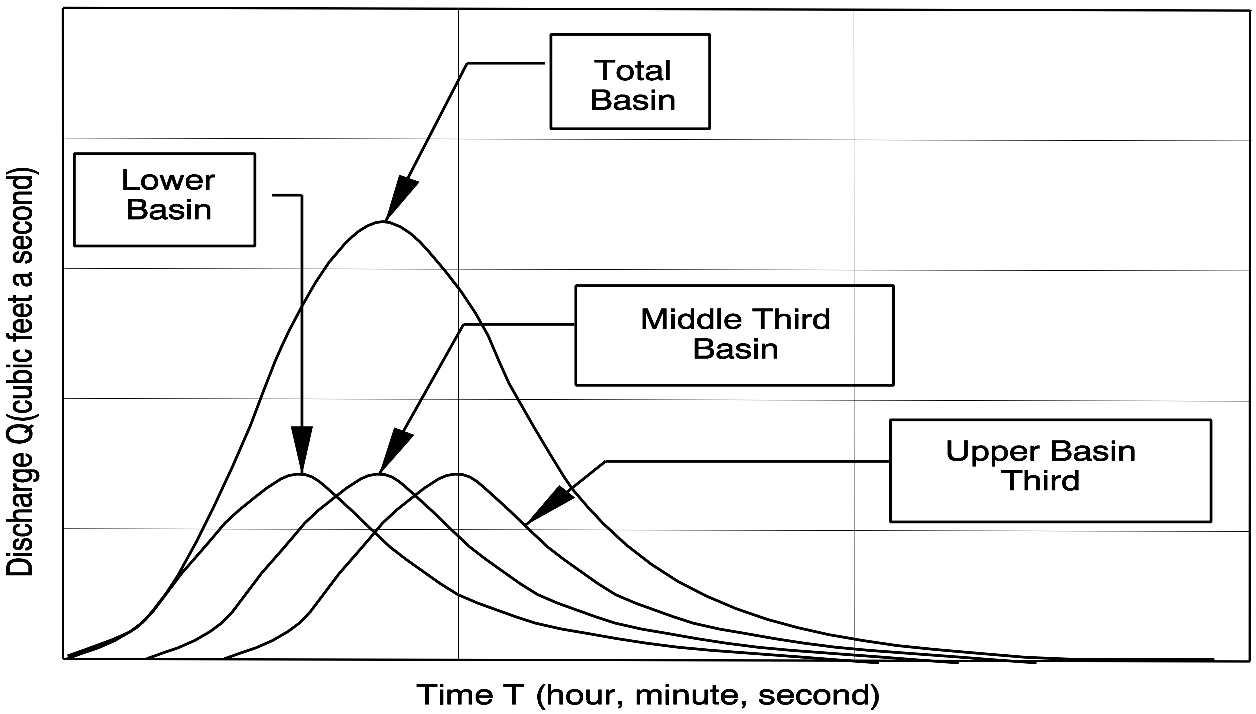
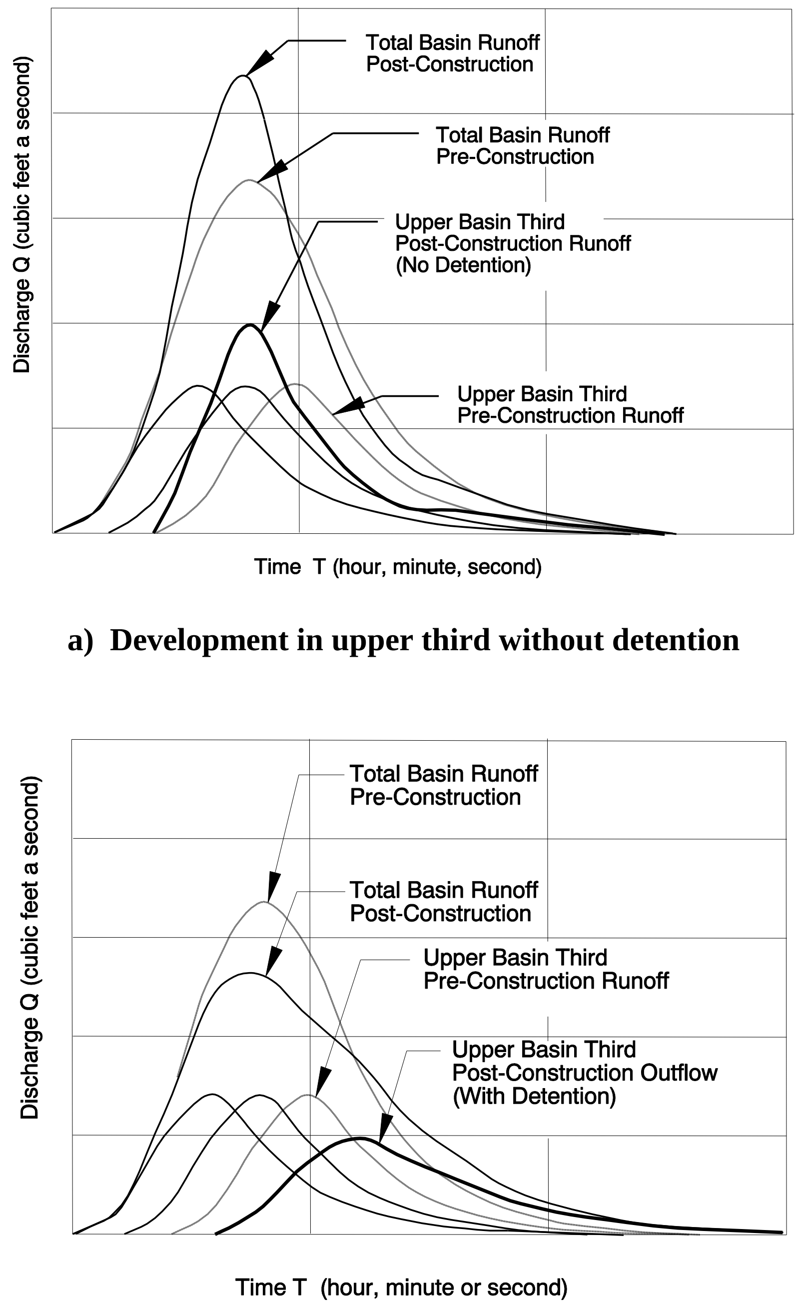



**Figure 12-9 Runoff Hydrographs of Undeveloped Basin**

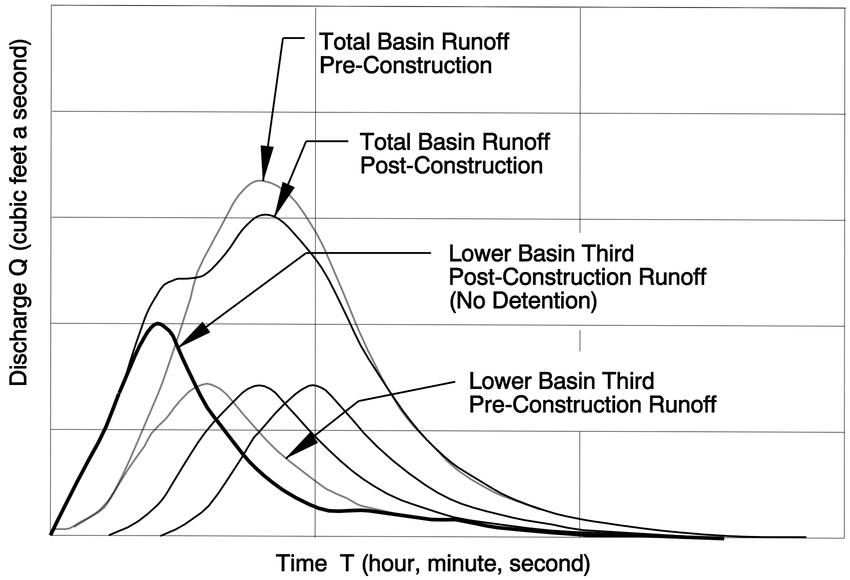



**a) Development in upper third without detention**

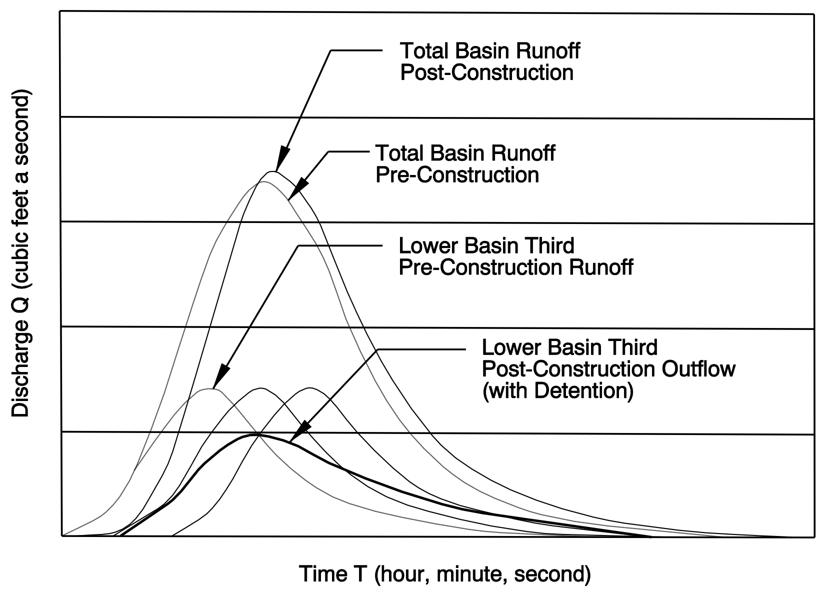


**b) Development in upper third with detention**

**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.

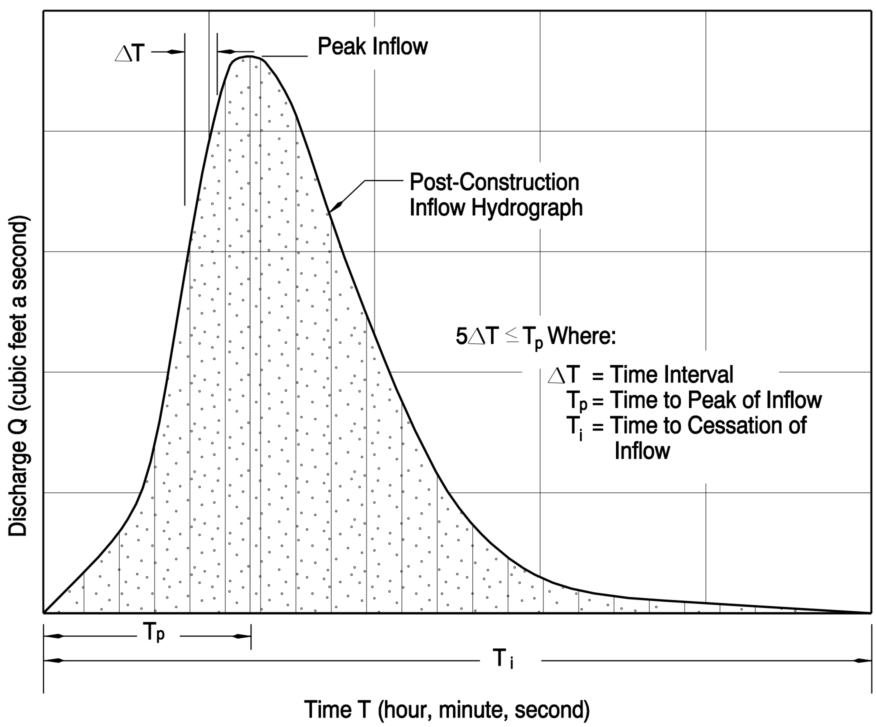



**Figure 12-12 Time Intervals and Period for Routing Analysis**

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.

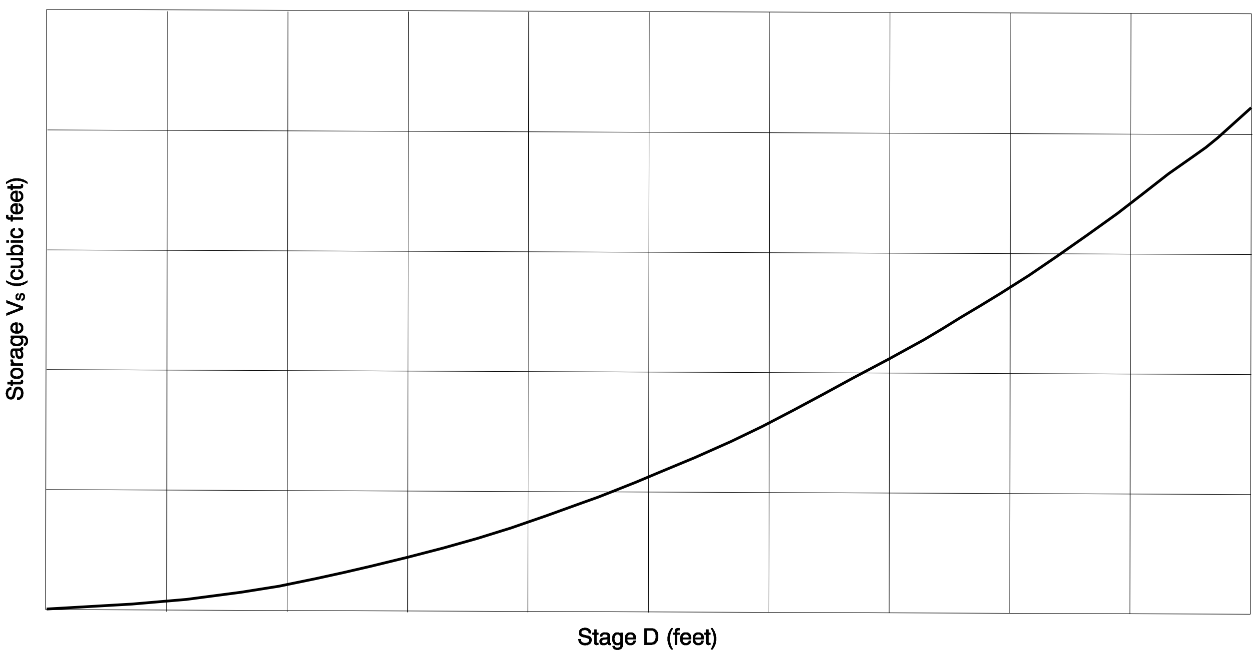



**Figure 12-13 Stage-Storage Curve**

#### 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: $V = \dfrac{D^2W}{2S}$

(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.*

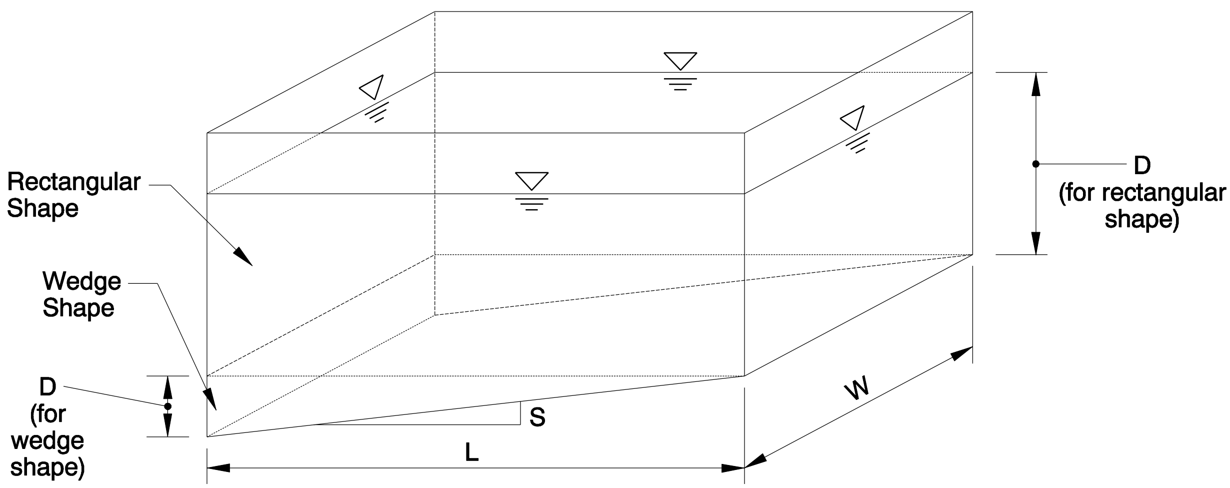



**Figure 12-14 Rectangular Basin**

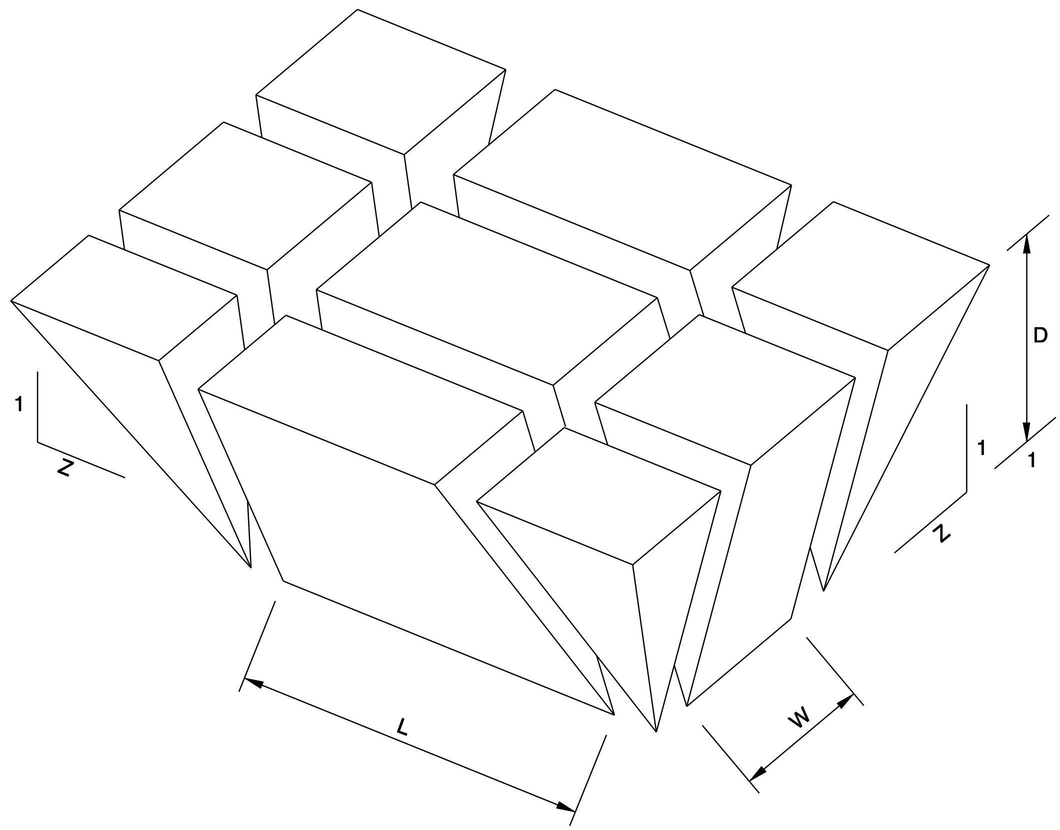



**Figure 12-15 Trapezoidal Basin**

### 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

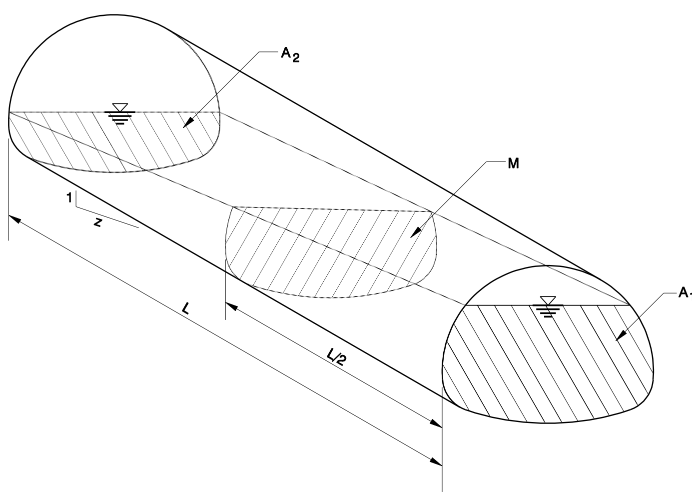



**Figure 12-16 Prismoidal Shape**

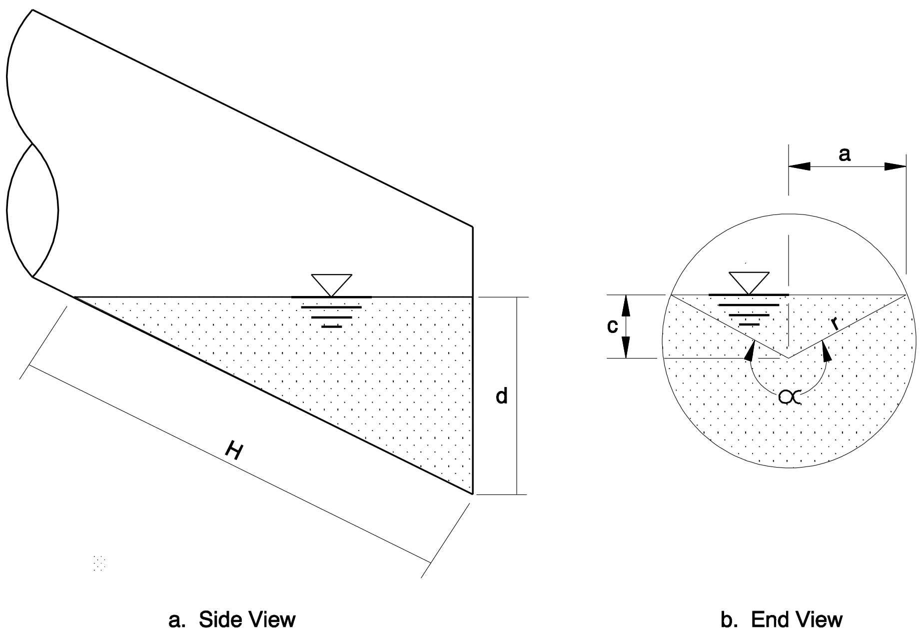



**Figure 12-17 Ungula of a Cone Shape**

**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.

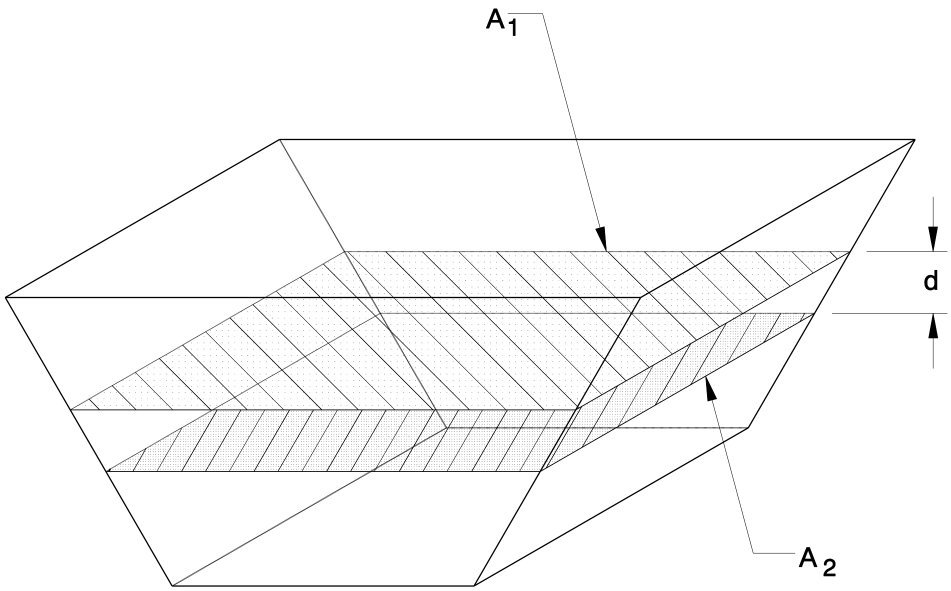



**Figure 12-18 Frustrum of a Pyramid**

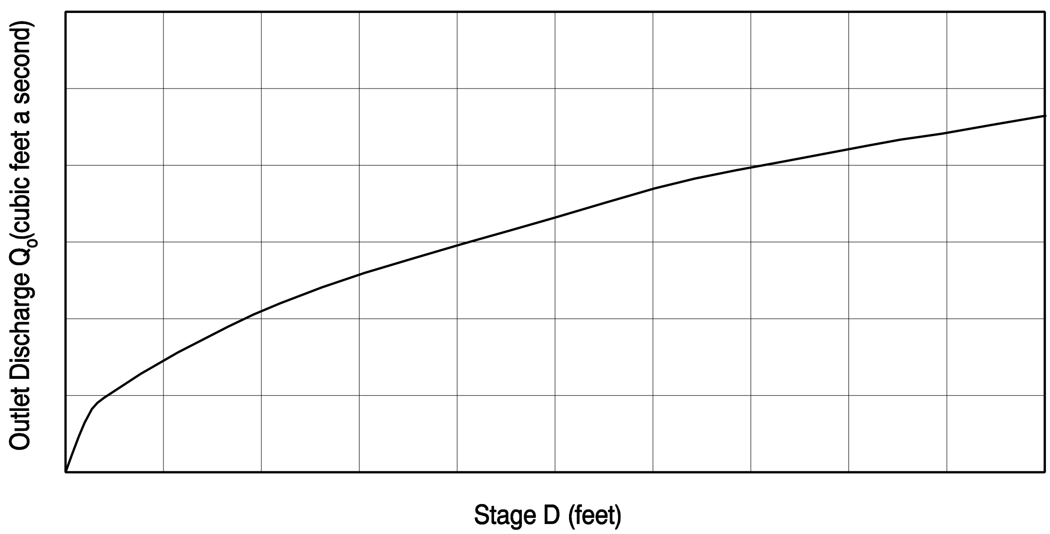



**Figure 12-19 Stage-Discharge Curve**

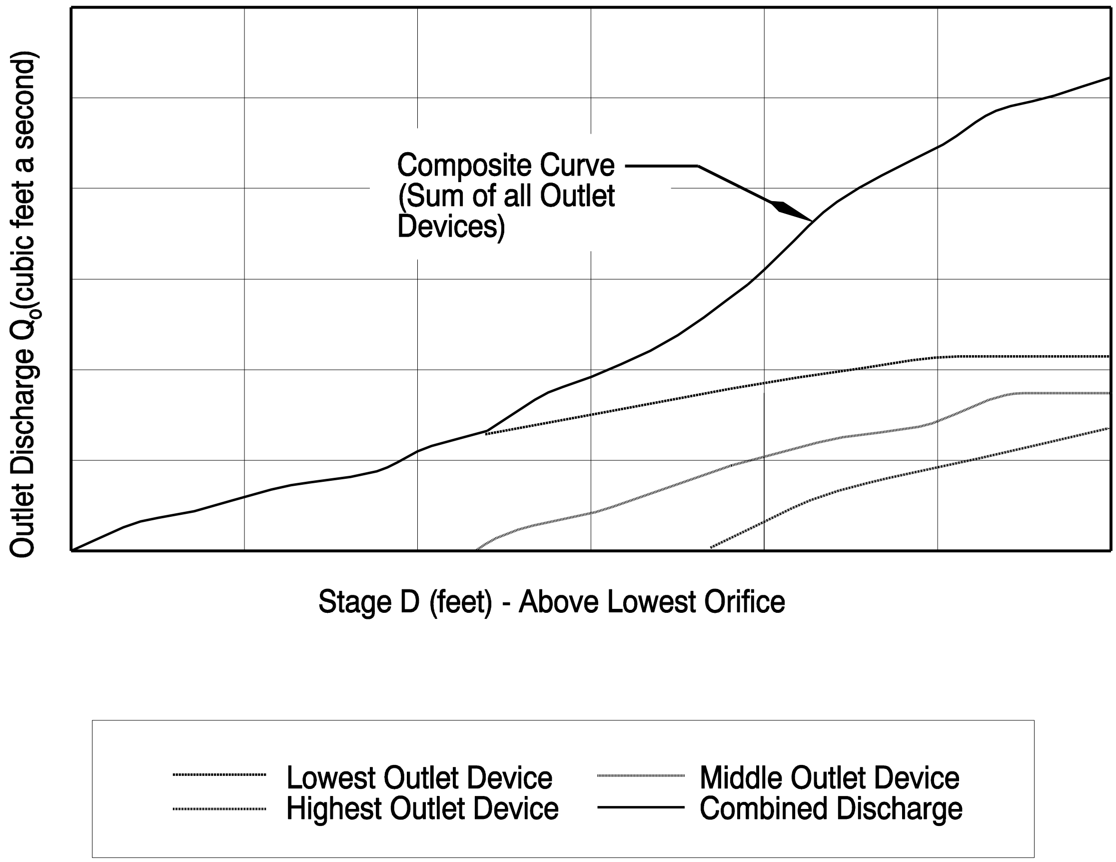



**Figure 12-20 Composite Stage-Discharge Curve**

### 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.

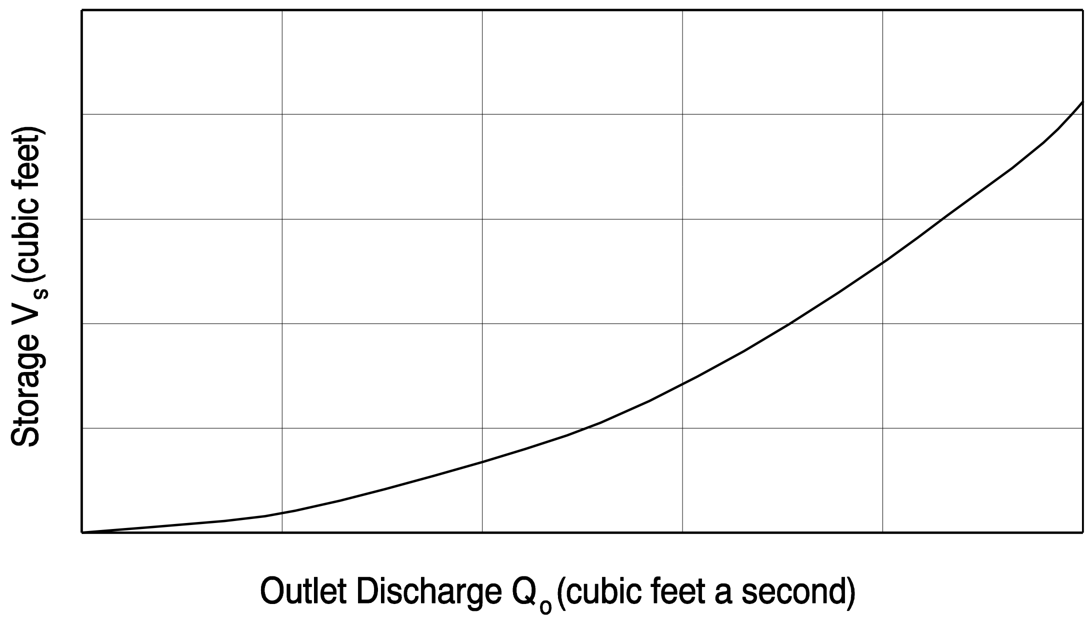



**Figure 12-21 Storage-Discharge Curve**

### 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 ($V_s$ + 1/2$Q_o\Delta T$) 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 ($V_s$ + 1/2$Q_o\Delta T$).

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 ($V_s$ + 1/2$Q_o\Delta T$) 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.

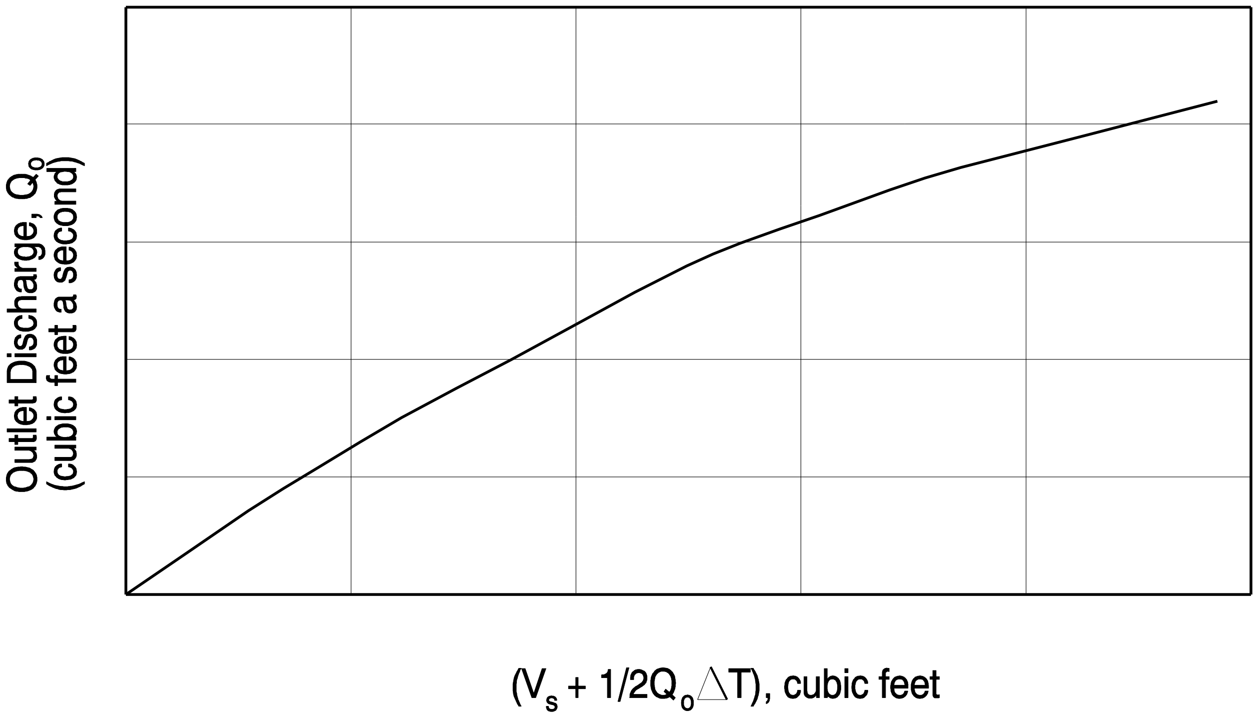



**Figure 12-22 Storage Indication Curve**

## 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.

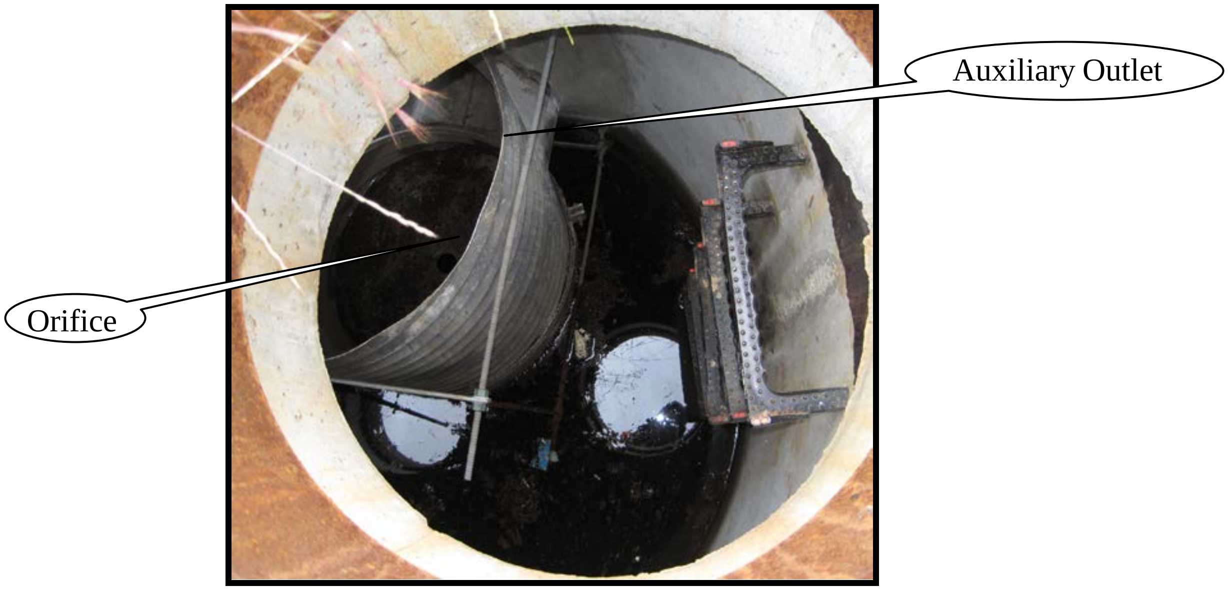



**Figure 12-23 Outlet Flow Control Structure I (Pipe Riser) Photo**

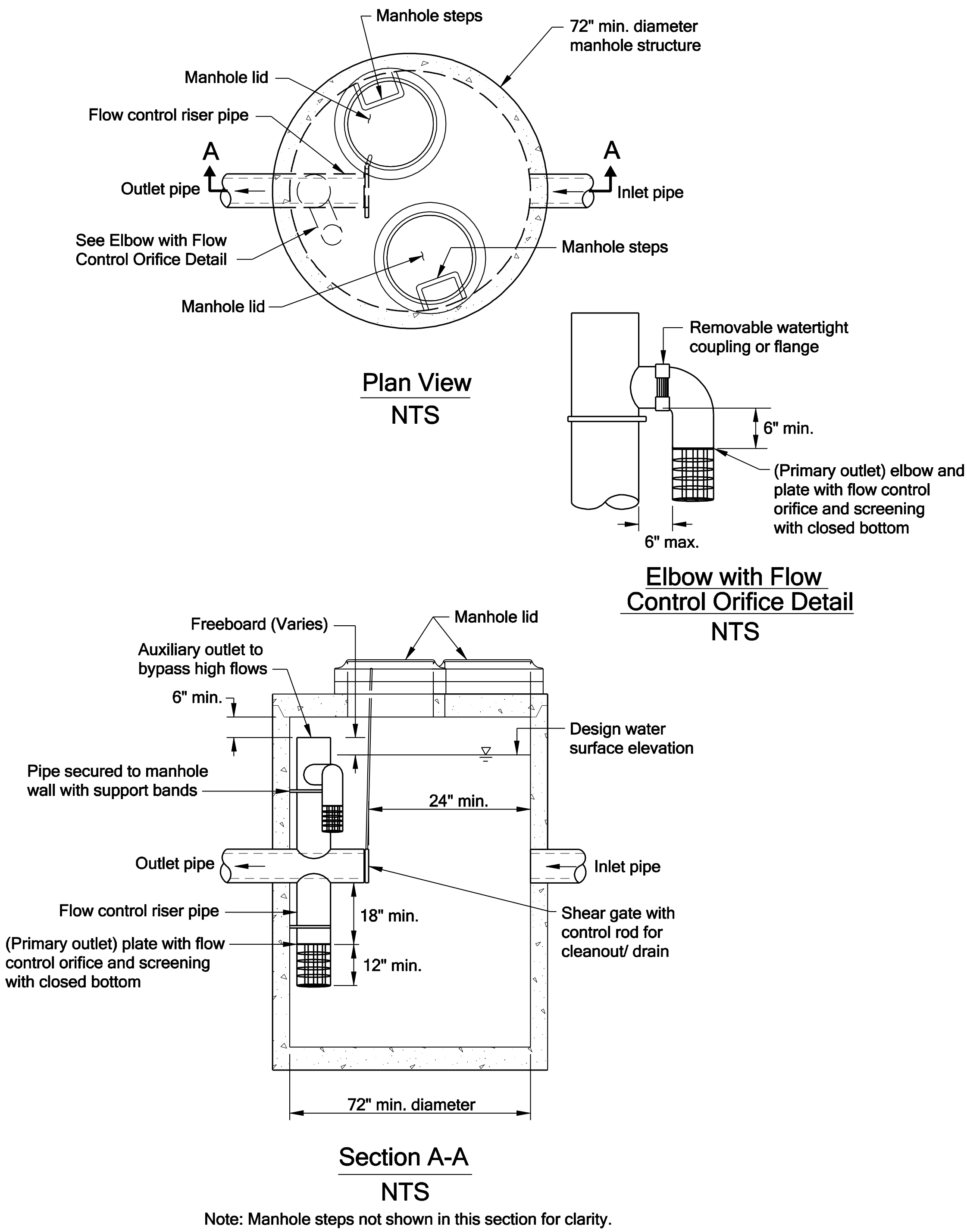



**Figure 12-24 Outlet Flow Control Structure I (Pipe Riser)**

#### 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.

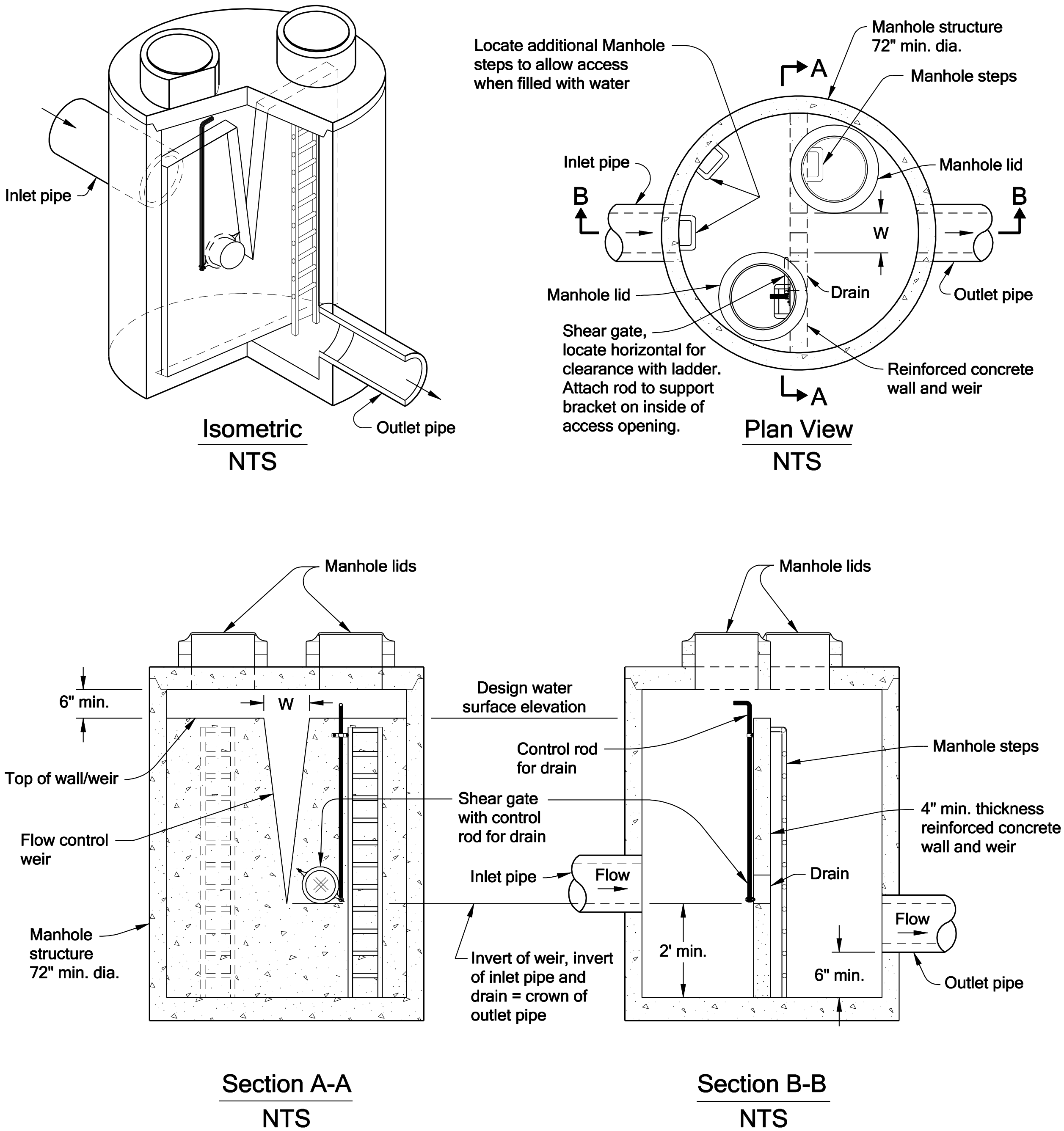



**Figure 12-25 Outlet Flow Control Structure II (V-Notch Weir)**

### 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.

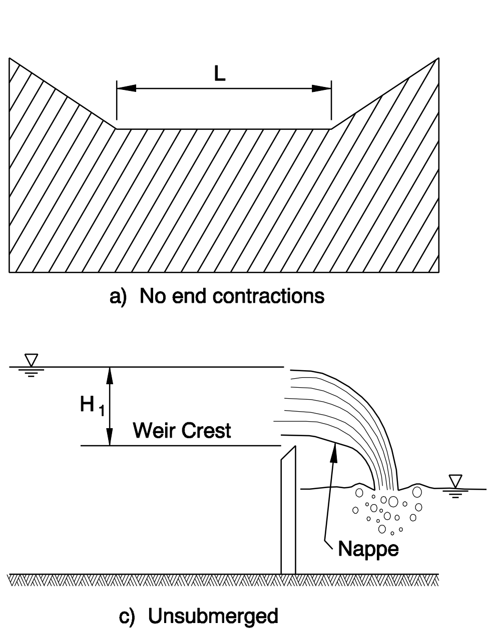



**Figure 12-26 Sharp-Crested Weirs**

**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.

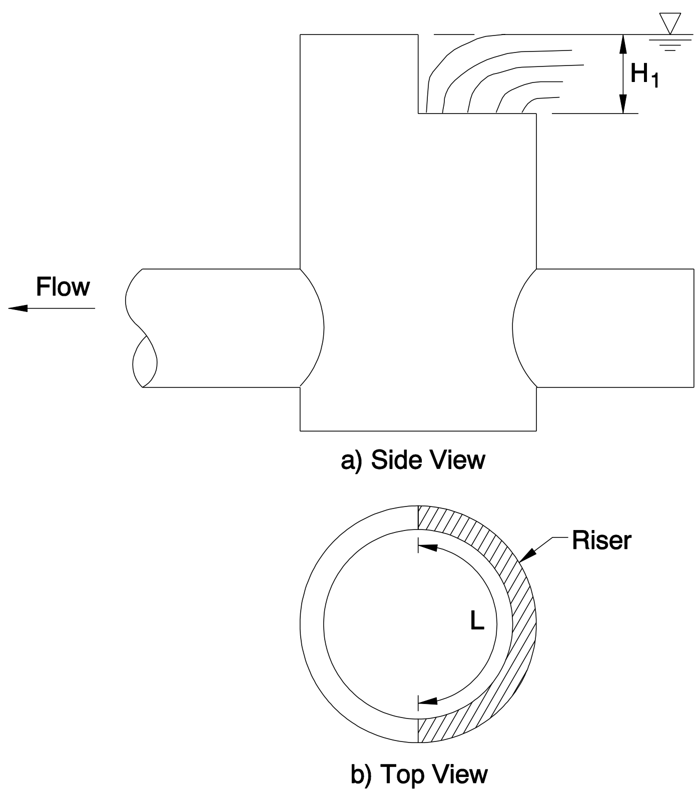



**Figure 12-27 Notch Weir**

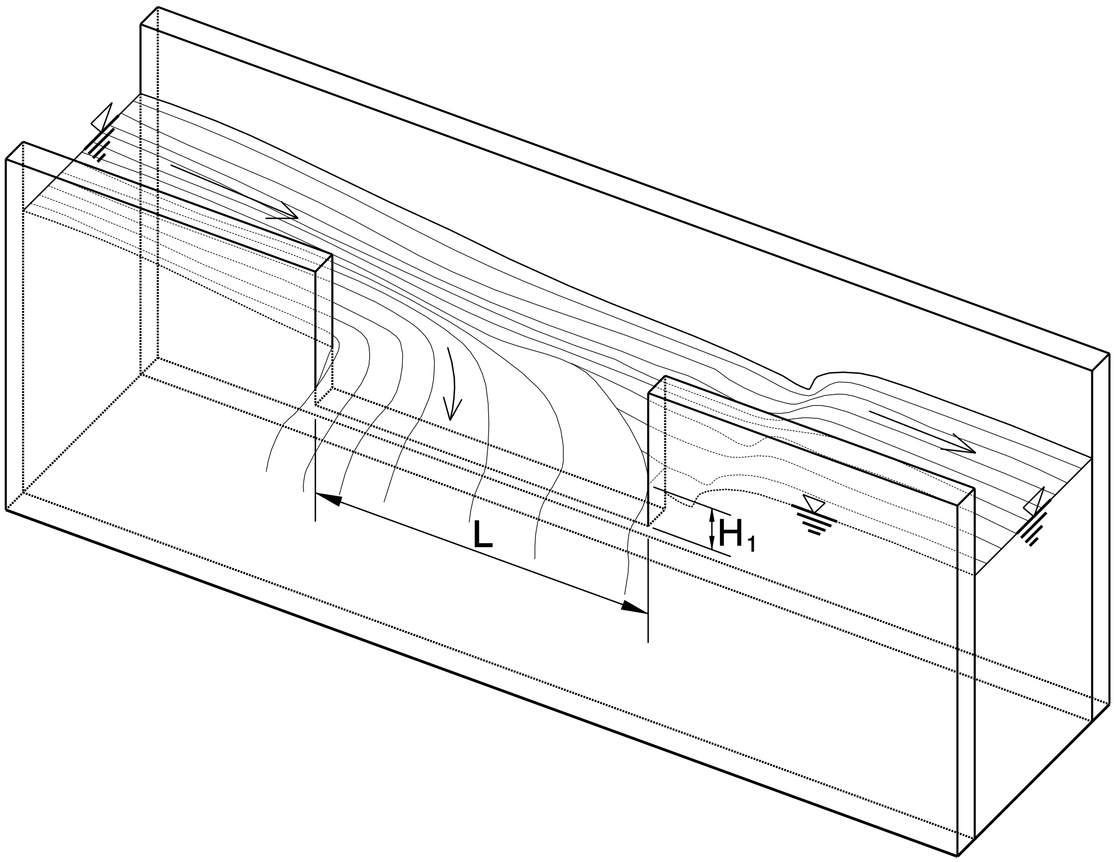



**Figure 12-28 Side Overflow Weir**

**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

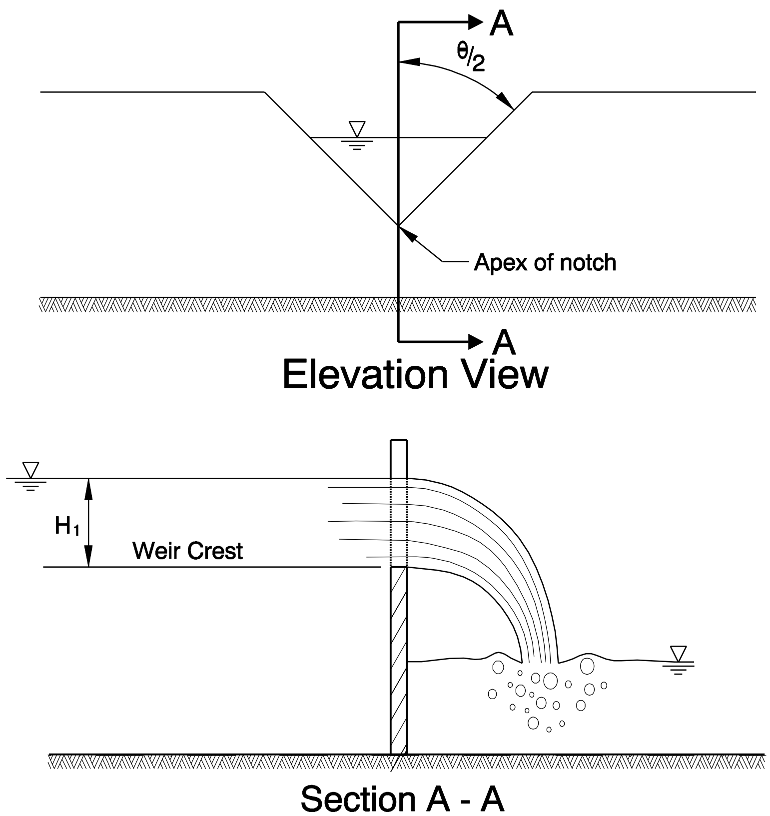



**Figure 12-29 V-Notch Weir**

**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.

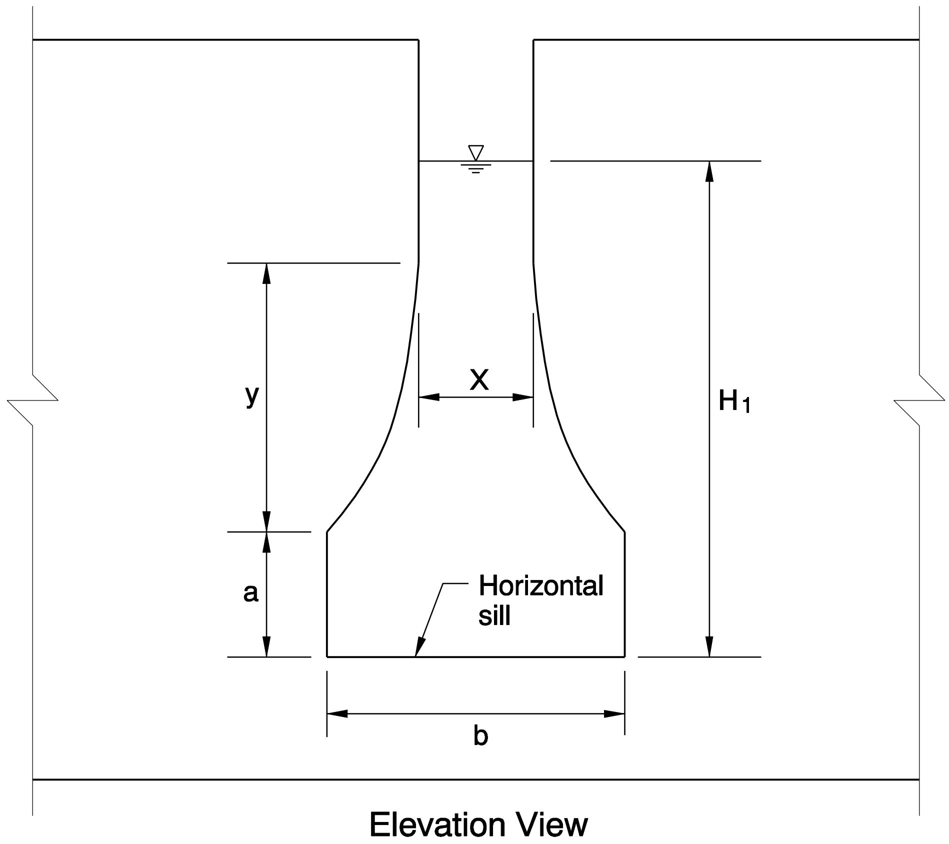
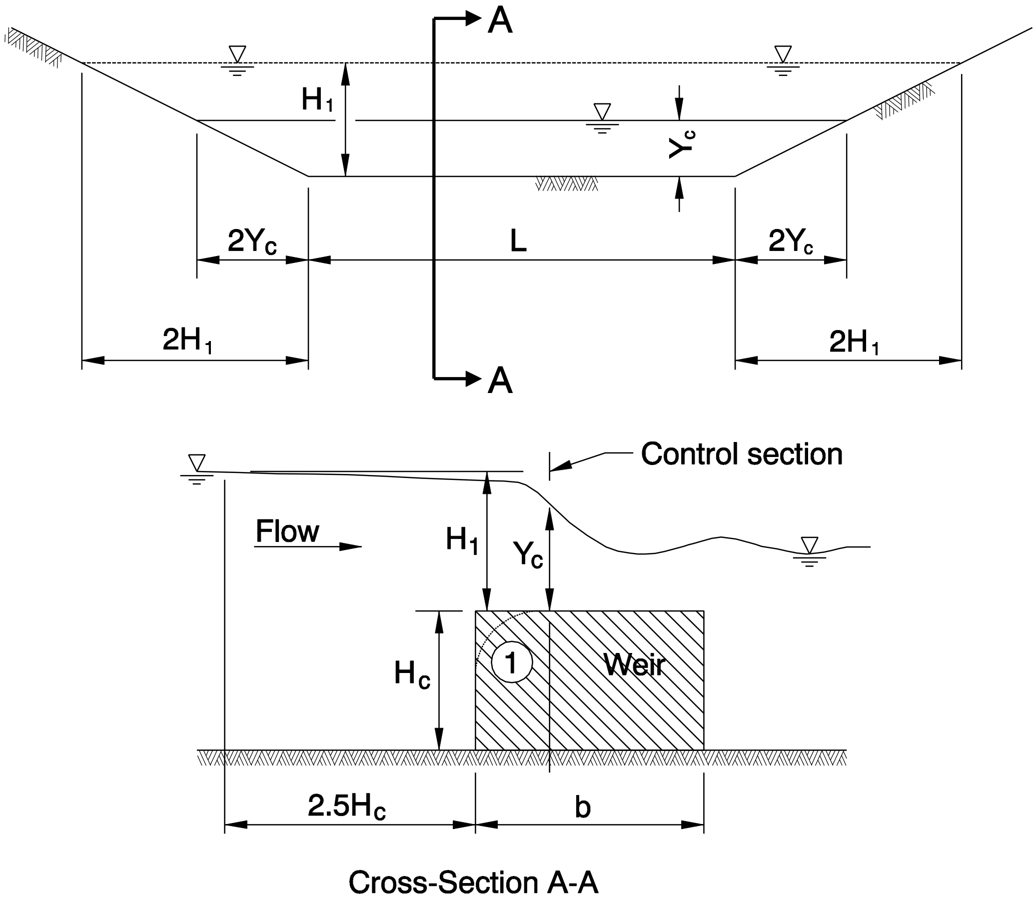



**Figure 12-30 Proportional Weir**




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).

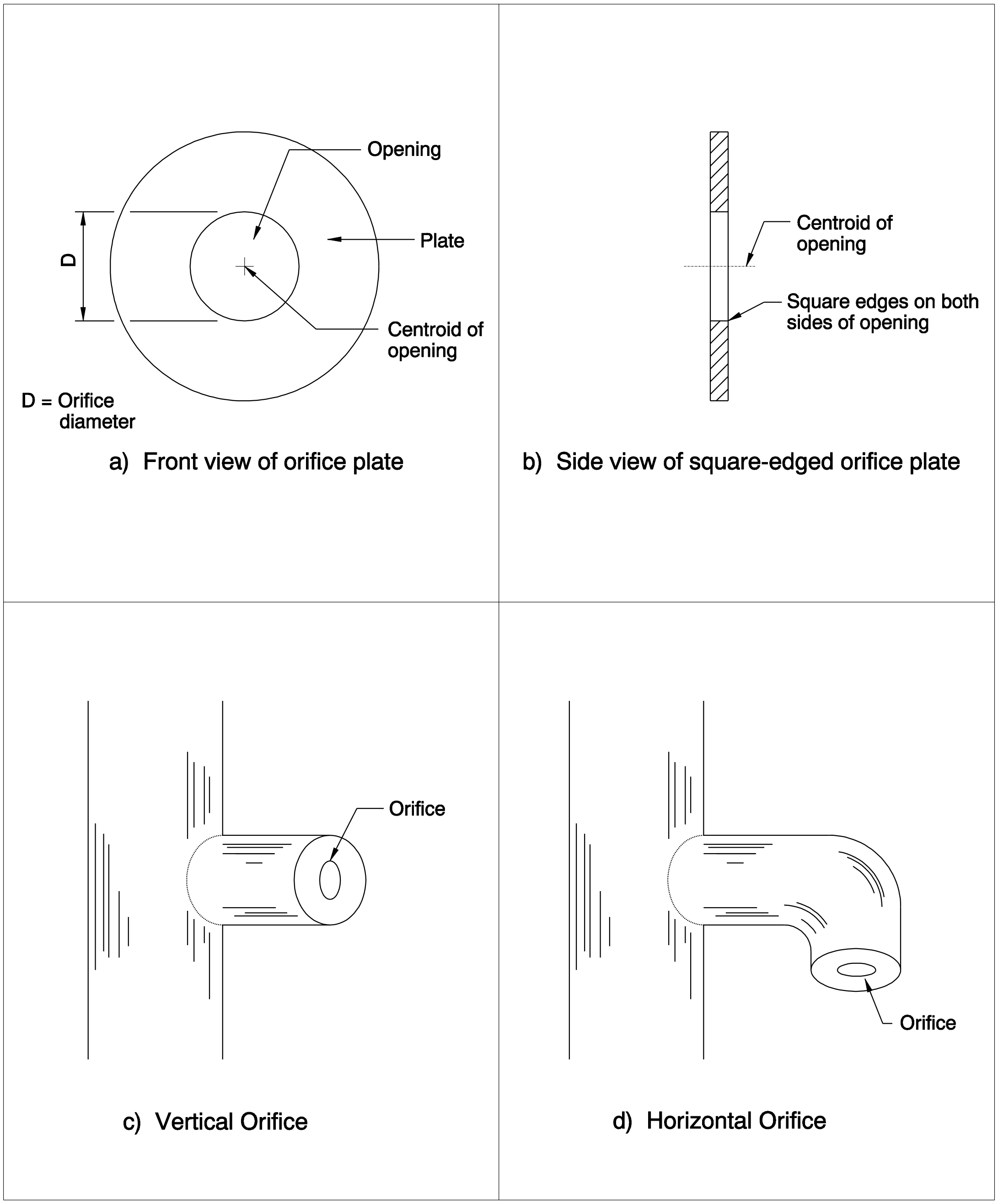



**Figure 12-32 Orifices**

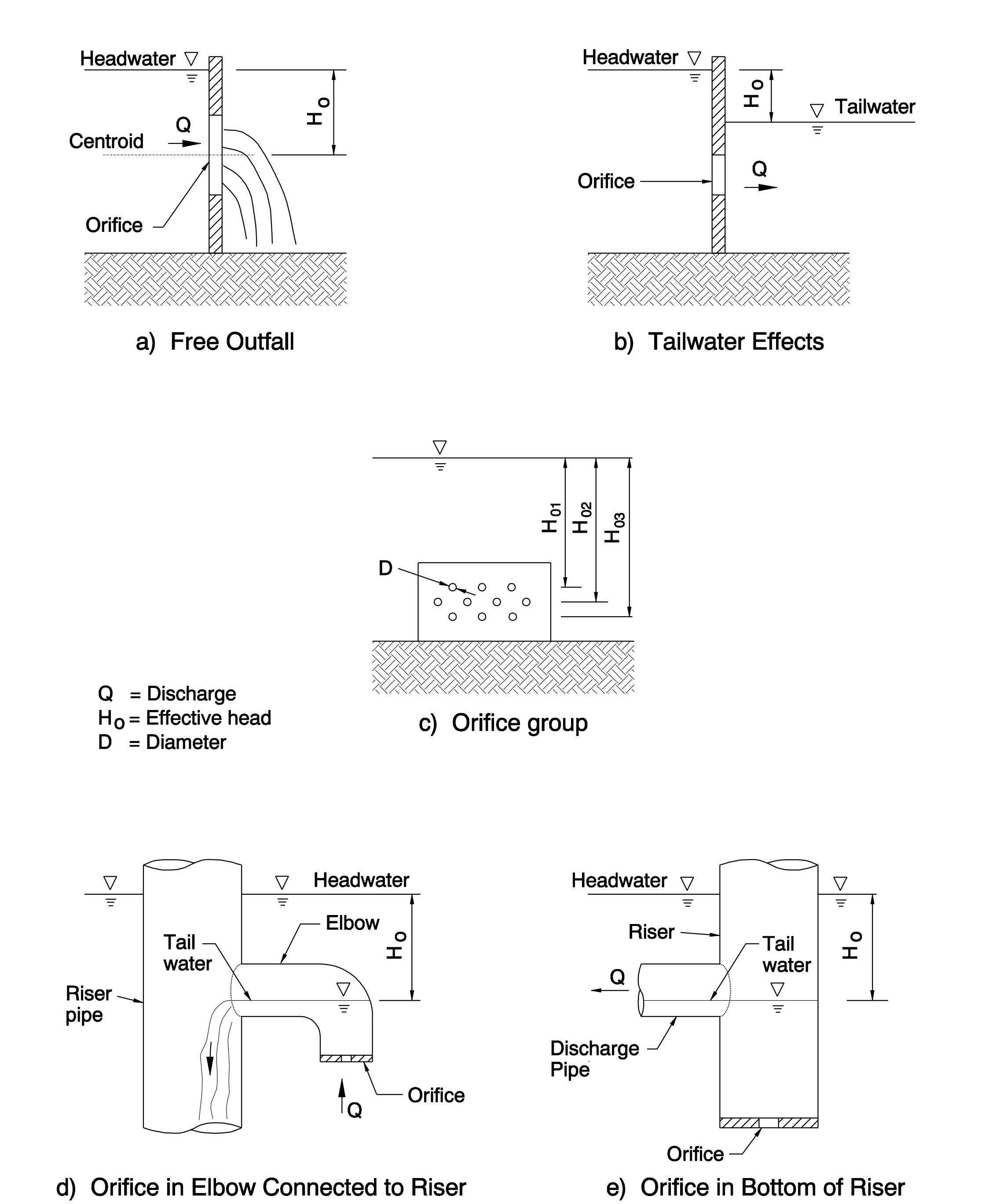



**Figure 12-33 Effective Heads for Orifices**

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.

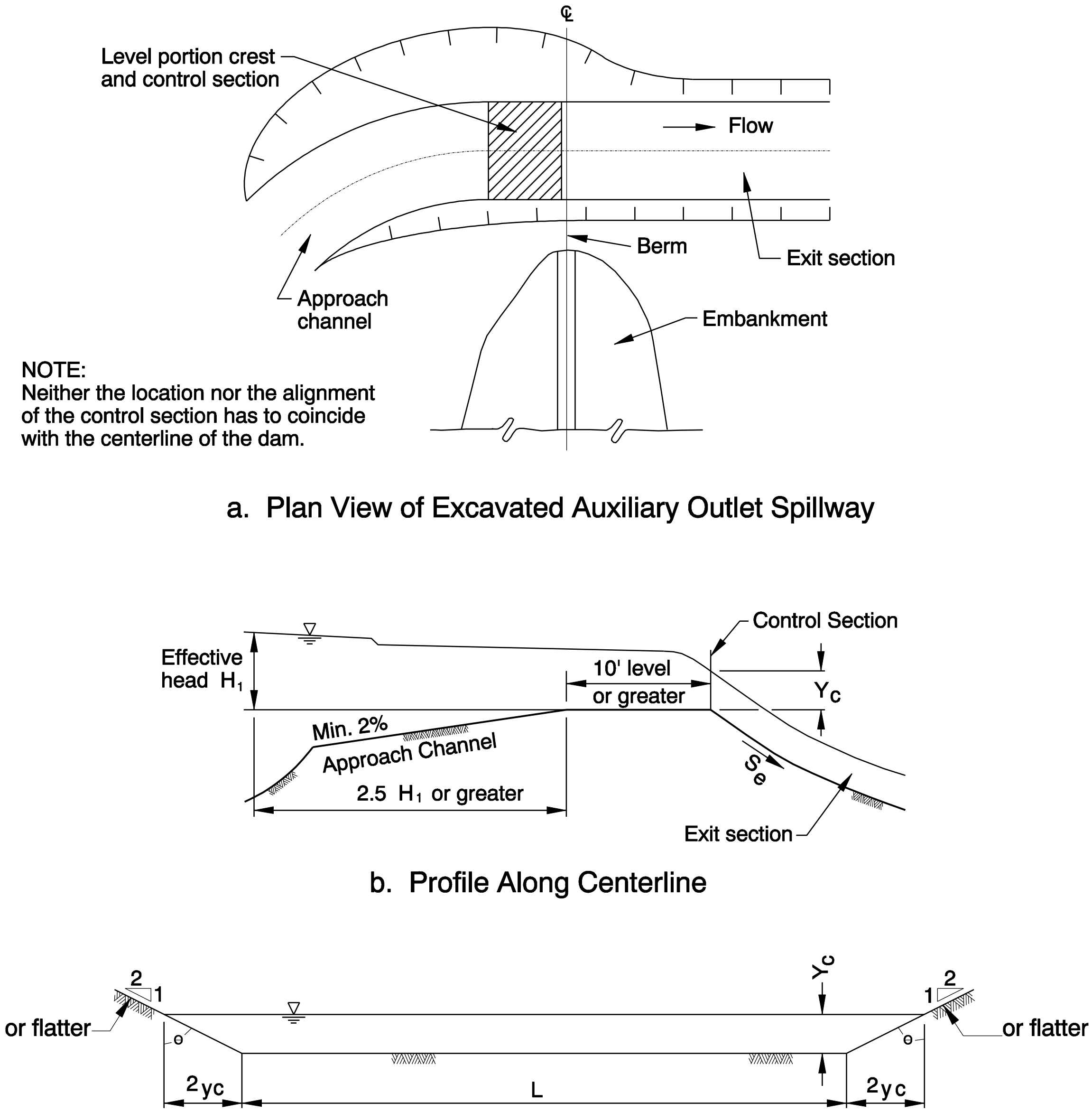
The hydraulic characteristics of spillway weirs are similar to the broad crested weirs described in Equation 12-24. One difference is the discharge coefficient. The coefficients for spillway weir applications are presented in this subsection, and they include adjustments for higher roughness coefficients 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.

$$Q = C_{sp}\,(2g)^{1.5}\left[\frac{2}{3}LH_1^{1.5} + \frac{8}{15}\left(\text{Tan}\,\theta\right)\,H_1^{2.5}\right] \qquad \text{(Equation 12 - 28)}$$

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,

$$L = \frac{\left\{\left(\frac{3}{16}\right)\left(\frac{Q}{C}\right) - \left[0.8\,(\mathrm{Tan}\,\theta)\,H^{2.5}\right]\right\}}{H^{1.5}} \qquad \text{(Equation 12 - 29)}$$

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.

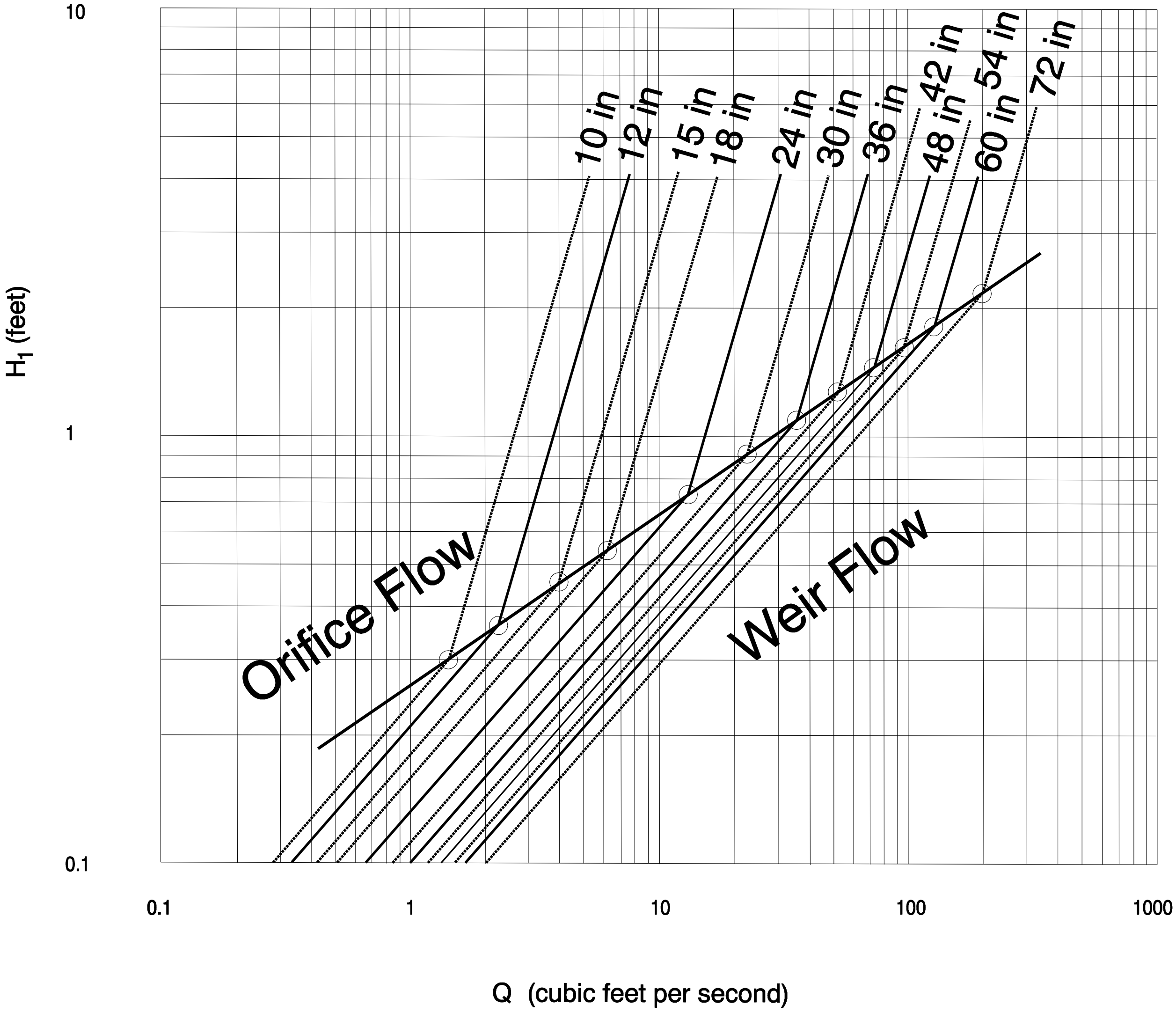



**Figure 12-35 Riser Overflow Hydraulics**

**Table 12-3 Weir Lengths for Pipe Risers**

| Pipe Diameter, D (inches) | Weir Length, L = πD (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.

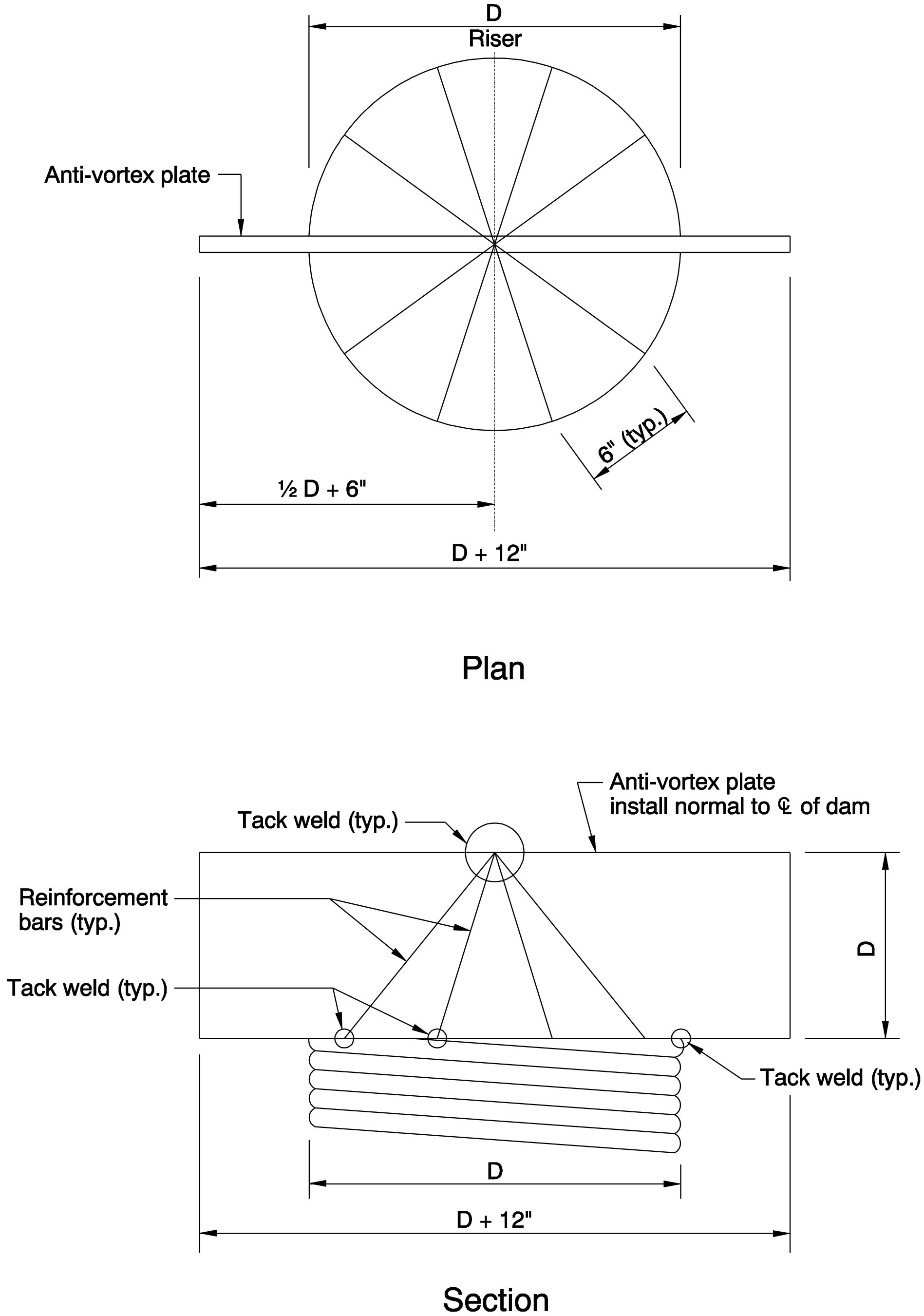



**Figure 12-36 Anti-Vortex Device and Trash Rack**

### 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.

$$\text{a) } L_{S\,(Revised)} = L_s + 2nV \qquad \text{or} \qquad \text{b) } n \geq \frac{(0.05\,L_S)}{V} \qquad \text{(Equation 12 - 30)}$$

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.

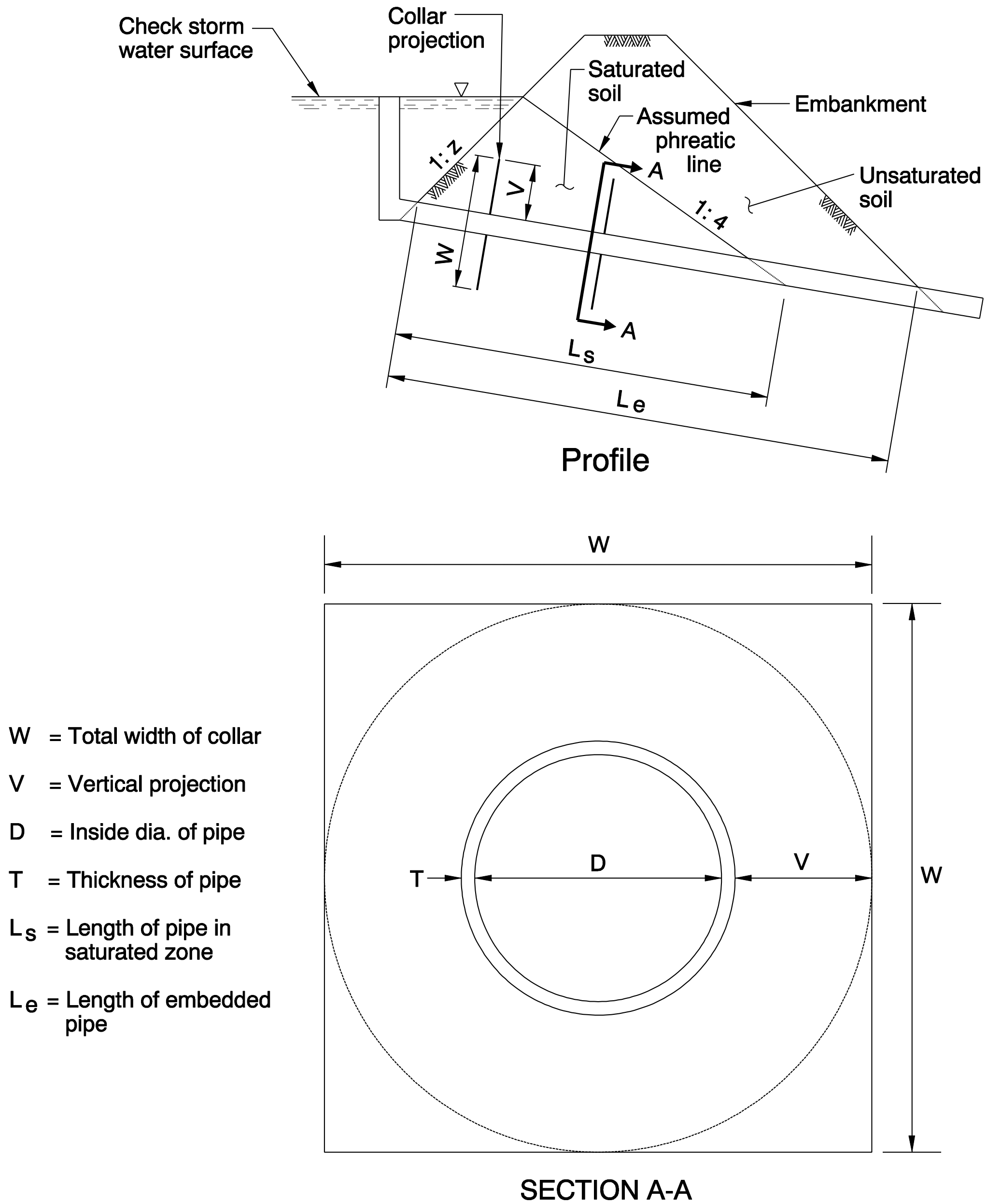



**Figure 12-37 Anti-Seep Collars**

## 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.

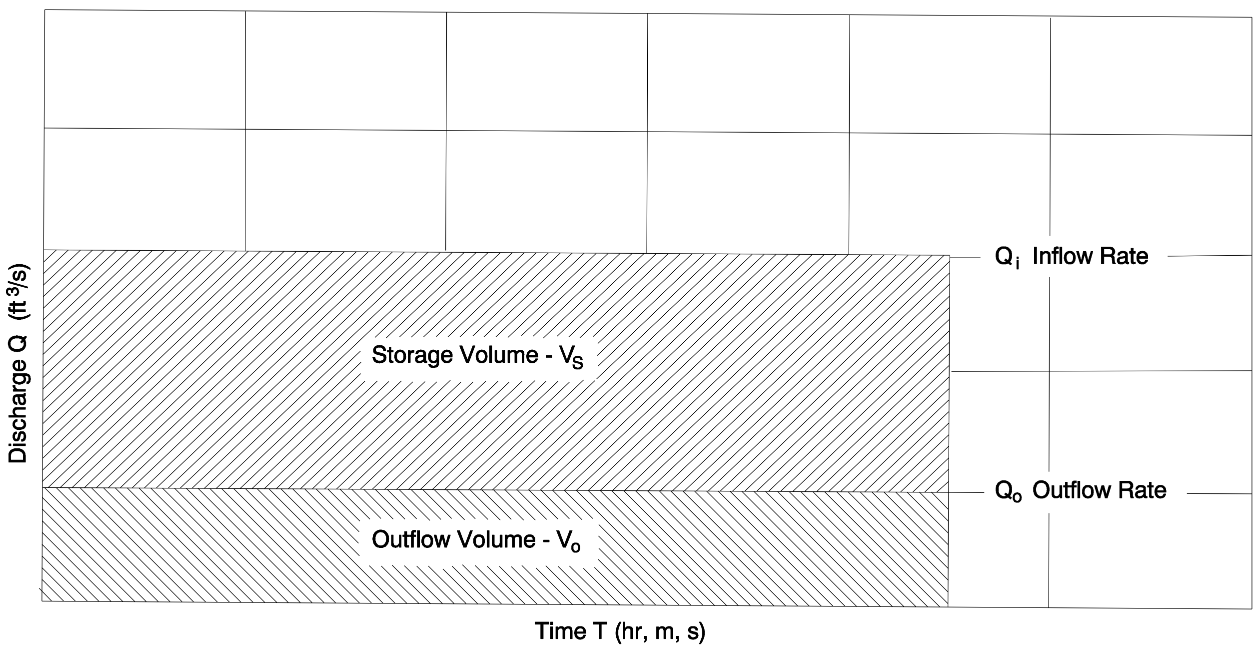



**Figure 12-38 Simplified Rational Method Storage Volume**

**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 .

$$\text{From Chapter 7, Appendix F, Equation 4}: \quad T_{osf} = \frac{0.93\,(300^{0.6}\ 0.4^{0.6})}{(i^{0.4}\ 0.05^{0.3})} = \frac{40.4}{i^{0.4}}$$

| 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: $T_{osf} = \dfrac{0.93\,(160^{0.6}\ 0.08^{0.6})}{(i^{0.4}\ 0.05^{0.3})} = \dfrac{10.5}{i^{0.4}}$

| 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:

$$T_{pf} = \frac{150}{(60)\,(10.2)} + \frac{450}{(60)\left[\dfrac{3.2 + 5.1}{2}\right]} = 2 \text{ minutes}$$

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

$$L = \frac{\left[(-\,2 \text{ x } 3)(1+1)\right] + \left\{\left[(2 \text{ x } 3)^2\,(1+1)^2\right] - \left[5.33\,(2 \text{ x } 3)^2 \text{ x } 1\right] + \left[\dfrac{(4 \text{ x } 1 \text{ x } 18{,}110)}{3}\right]\right\}^{0.5}}{(2 \text{ x } 1)}$$

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.

$$L = \frac{\left[(-3 \text{ x } 3)(2+1)\right] + \left\{\left[(3 \text{ x } 3)^2 (2+1)^2\right] - \left[5.33 (3 \text{ x } 3)^2 \text{ x } 2\right] + \left[\frac{(4 \text{ x } 2 \text{ x } 18{,}110)}{3}\right]\right\}^{0.5}}{(2 \text{ x } 2)}$$

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

$$D = \left( \frac{3.6}{(3.78)(3^{0.5})} \right)^{0.5} = 0.74 \text{ feet} = 8.9 \text{ inches} = 8\text{ - }7/8\text{ths inch diameter.}$$

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,

$$L = \frac{\left\{\dfrac{\left(\dfrac{3}{16}\right)28.6}{(0.4 - \left[(0.8)(2.83)(1)^{2.5}\right]}\right\}}{(1)^{1.5}} = 11.2 \text{ foot wide bottom width of weir is required}$$

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 <u>also</u> 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.

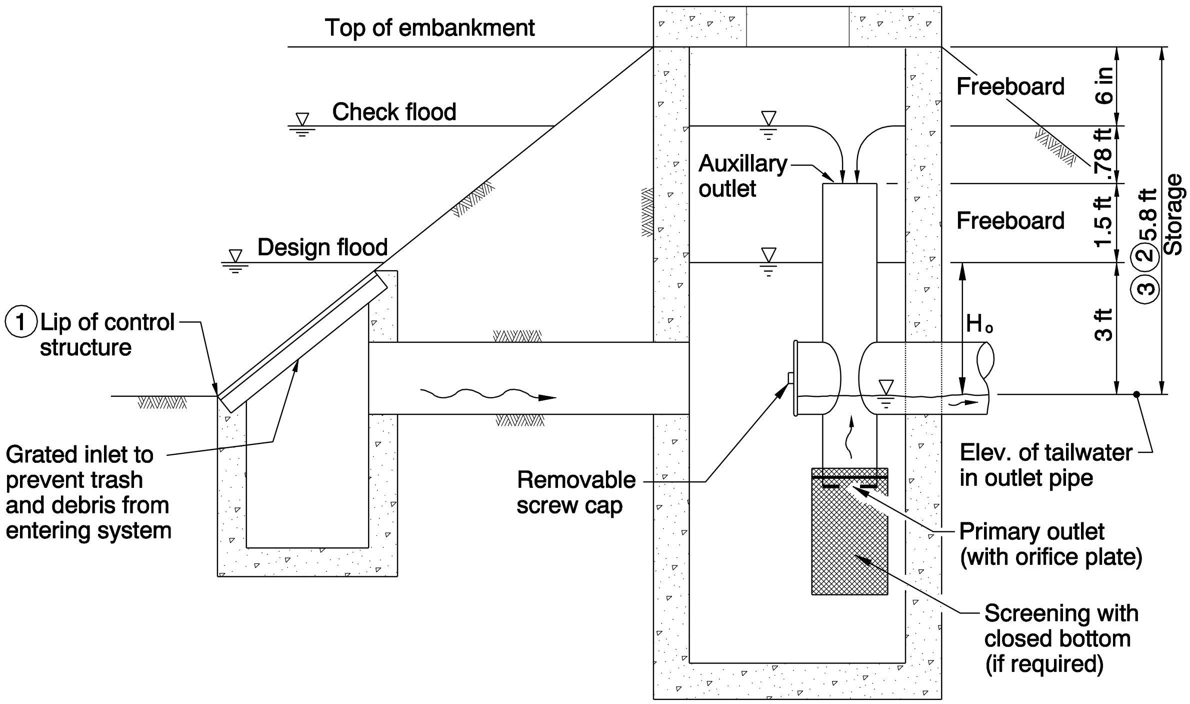
**Step 7 -** In this step the auxiliary outlet is sized. Size this outlet to convey the entire discharge from the check storm. It is assumed that the primary outlets are plugged. Check the basin dimensions determined in Step 4 and verify that there is adequate freeboard between the lowest part of the 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.




Notes

1. Lip of control structure and pond bottom structure must be equal to or lower than elevation of tailwater in outlet pipe.

2. Storage is the volume above the elevation of the tailwater in the outlet pipe.

3. Head water depths are based on riser pipe used for auxiliary outlet.

**Figure 12-39 Detention Outlet Structure for Example**

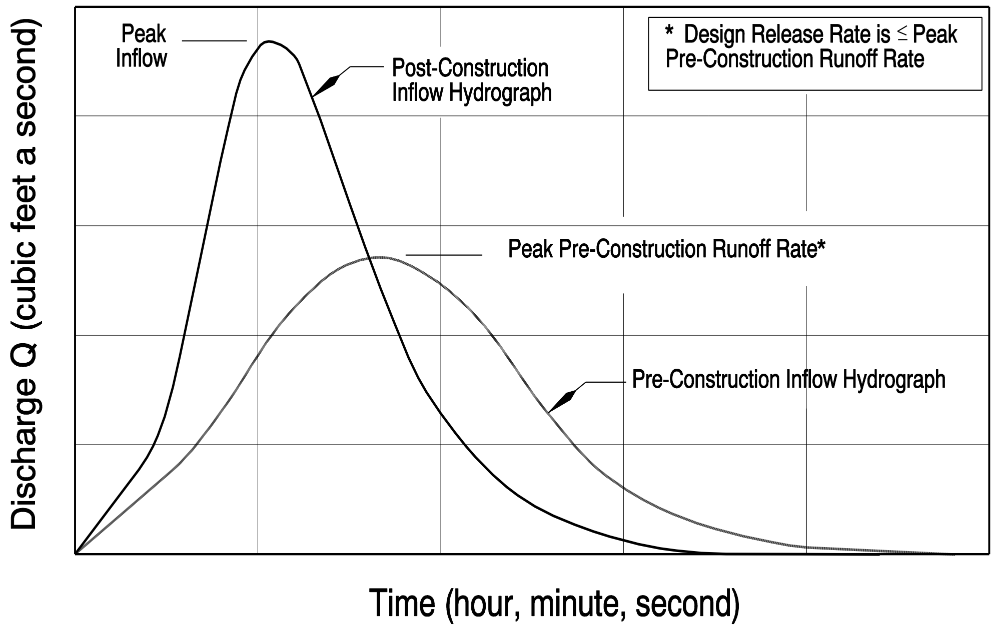



**Figure 12-40 Runoff and Inflow Hydrographs**

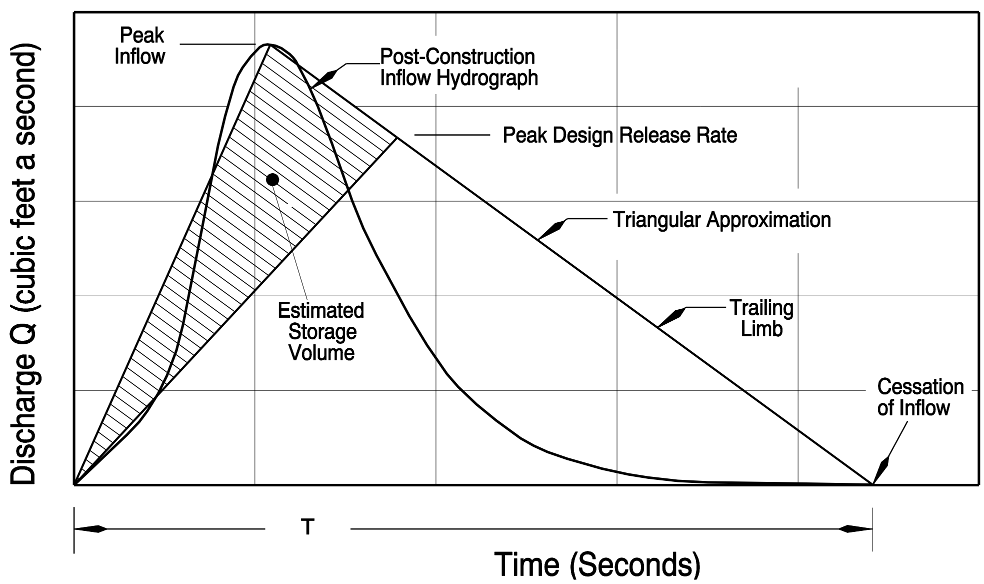



**Figure 12-41 Estimated Storage Volume**

### 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 $V_s + 1/2 Q_o \Delta T$).

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 "$\Delta T$," in cubic feet,
$Q_{o1}$ = Outflow rate at beginning of $\Delta T$ 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 "$\Delta T$" 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 "$\Delta T$." 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 "$\Delta T$." 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** | $T_2$ | $Q_{i2}$ | **Step 6b** | **Step 6c** | **Step 6d** | $Q_{o2}$ | $V_{s2}$ |
| Difference of values in Column 2 | **Step 1** Tabulate Values From Inflow Hydrograph | | Average of values in Column 3 Multiplied by Column 1 | Calculate using values from first row in Columns 7 & 8 | Add values from Columns 4 & 5 | **Step 6e** Use value from Column 6 and apply to storage-indication | **Step 6f** Use value from Column 7 and apply to storage-discharge |

**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 ($\underline{V}$)** | **($V_{s1} - {}^{1}/_{2}\, Q_{o1}\, \Delta T$)** | **($V_{s2} + {}^{1}/_{2} Q_{o2}\, \Delta T$)** | **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 $(V_{s1} - 1/2Q_{o1}\square\Delta T)$ is calculated for a time increment, entered into Column 5, and the calculations proceed to the next step.

**Step 6d -** In this step $(V_{s2} + 1/2Q_{o2}\square\Delta T)$ 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 $(V_{s2} + 1/2Q_{o2}\square\Delta T)$ 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** <br> **10-year** | **Time** | | **Post-Construction Inflow** <br> **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 |

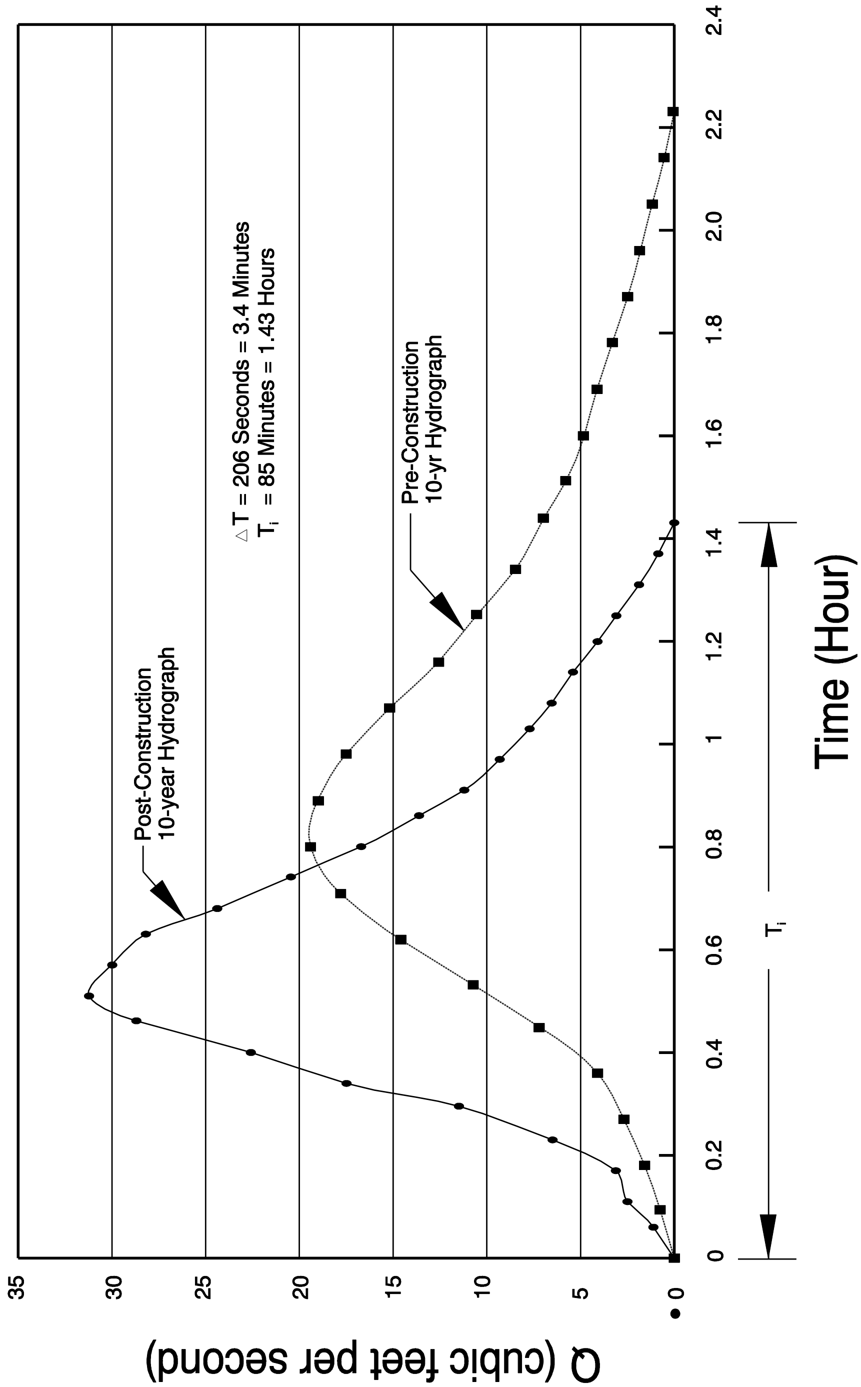


**Figure 12-42 Time Periods, Time Intervals, and Inflow Rates for Example**

**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 ($\bar{V}$)** | **$(V_{s1} - {}^1/_2 Q_{o1} \Delta T)$** | **$(V_{s2} + {}^1/_2 Q_{o2} \Delta T)$** | **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 |

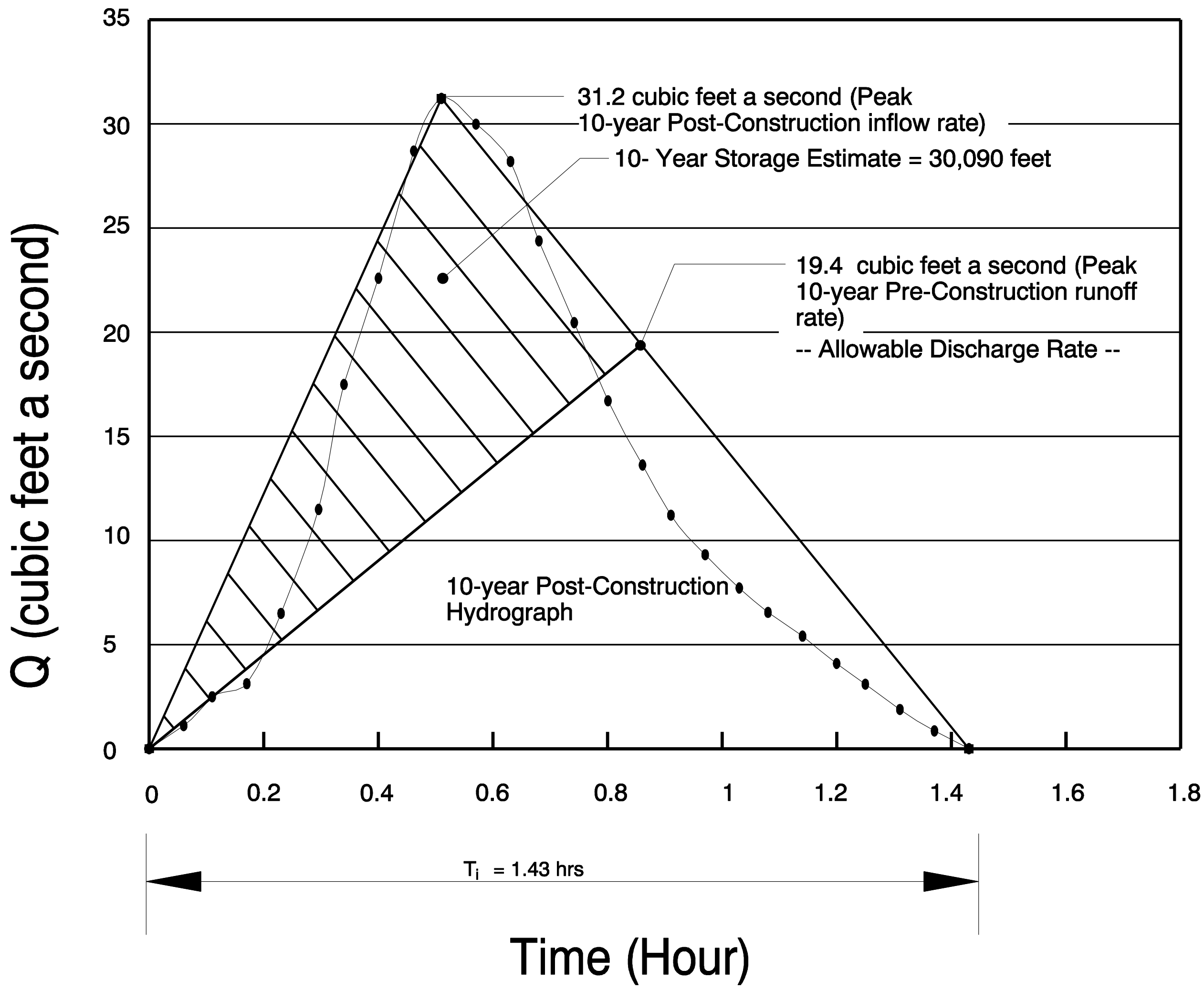


**Figure 12-43 Storage Volume Estimates for Example**

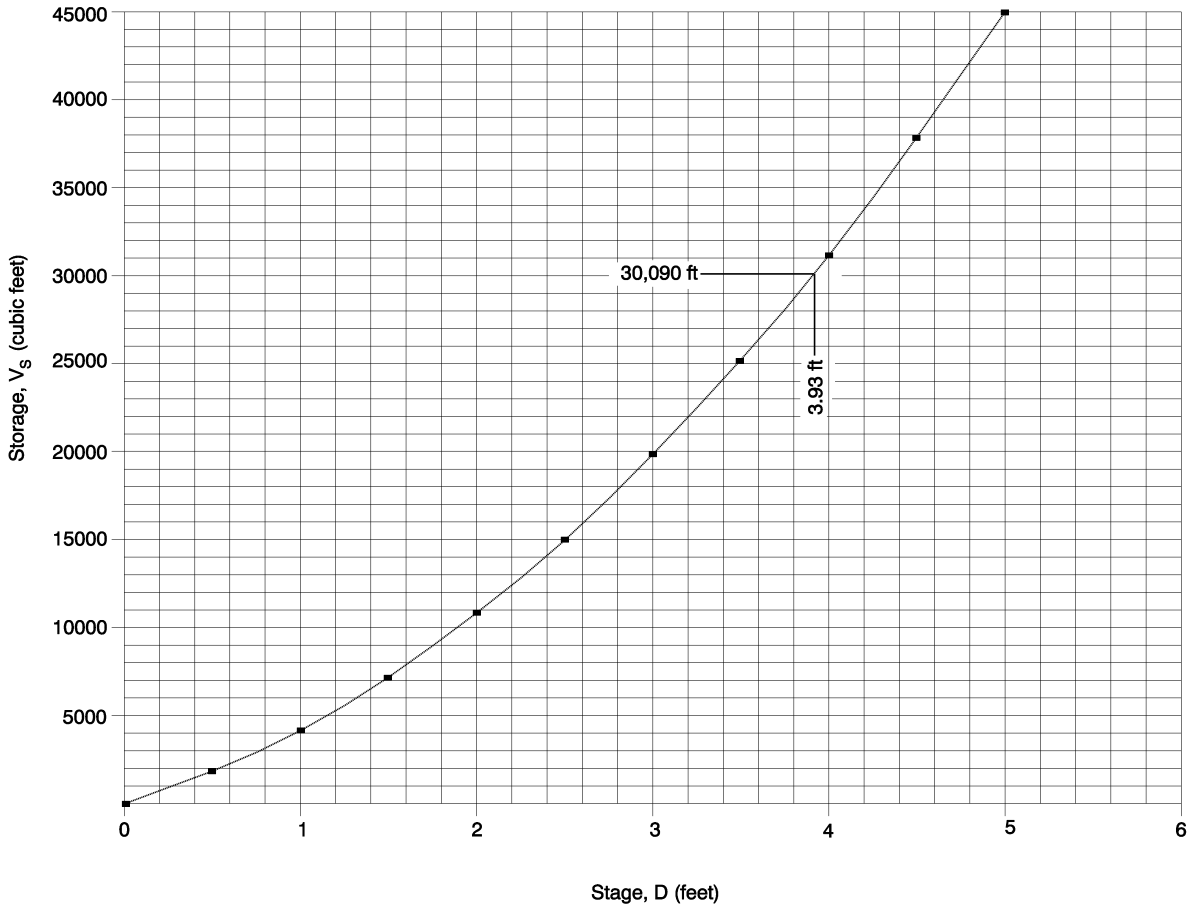


**Figure 12-44 Stage-Storage Curve for Example**

**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 |

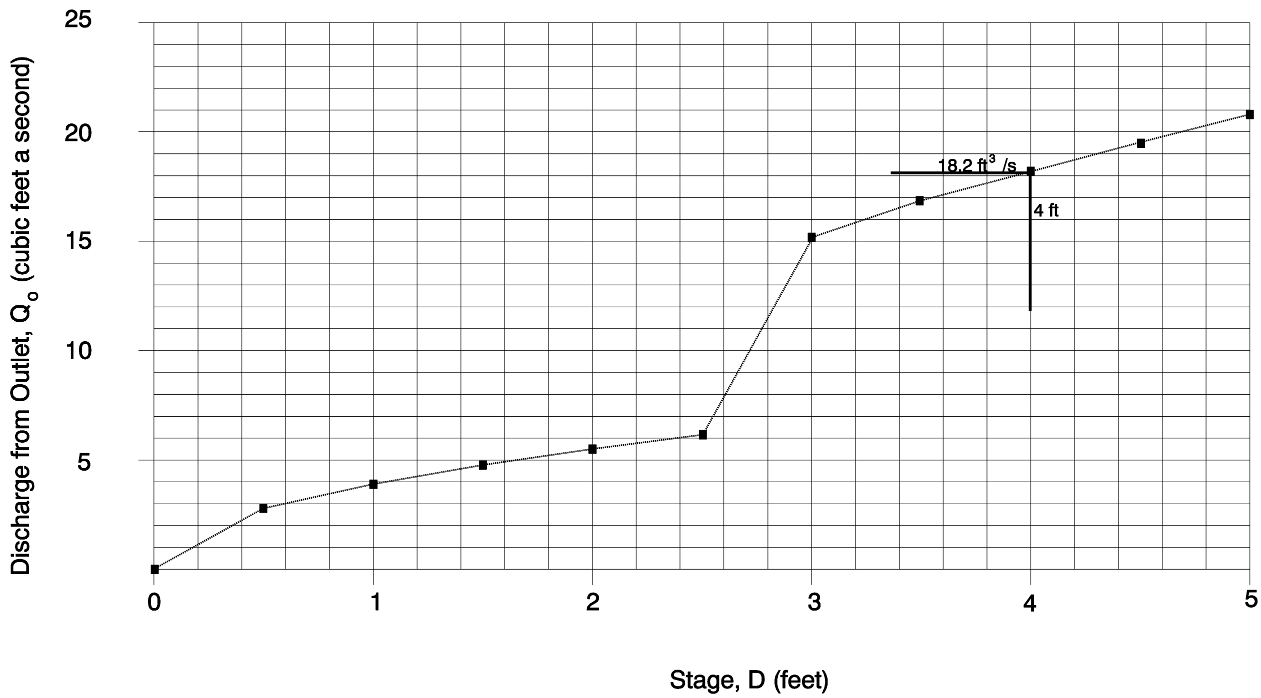


**Figure 12-45 Stage-Discharge Curve for Example**

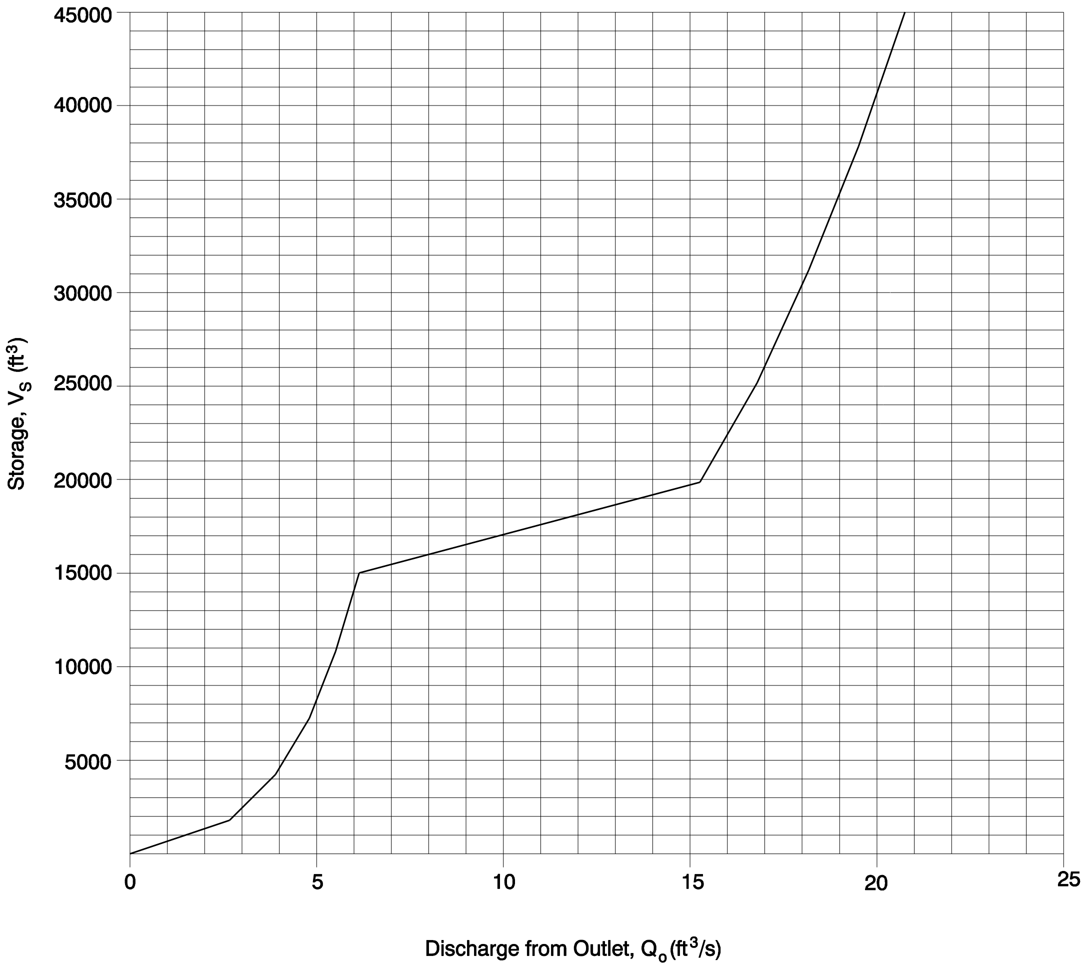


**Figure 12-46 Storage-Discharge Curve for Example**

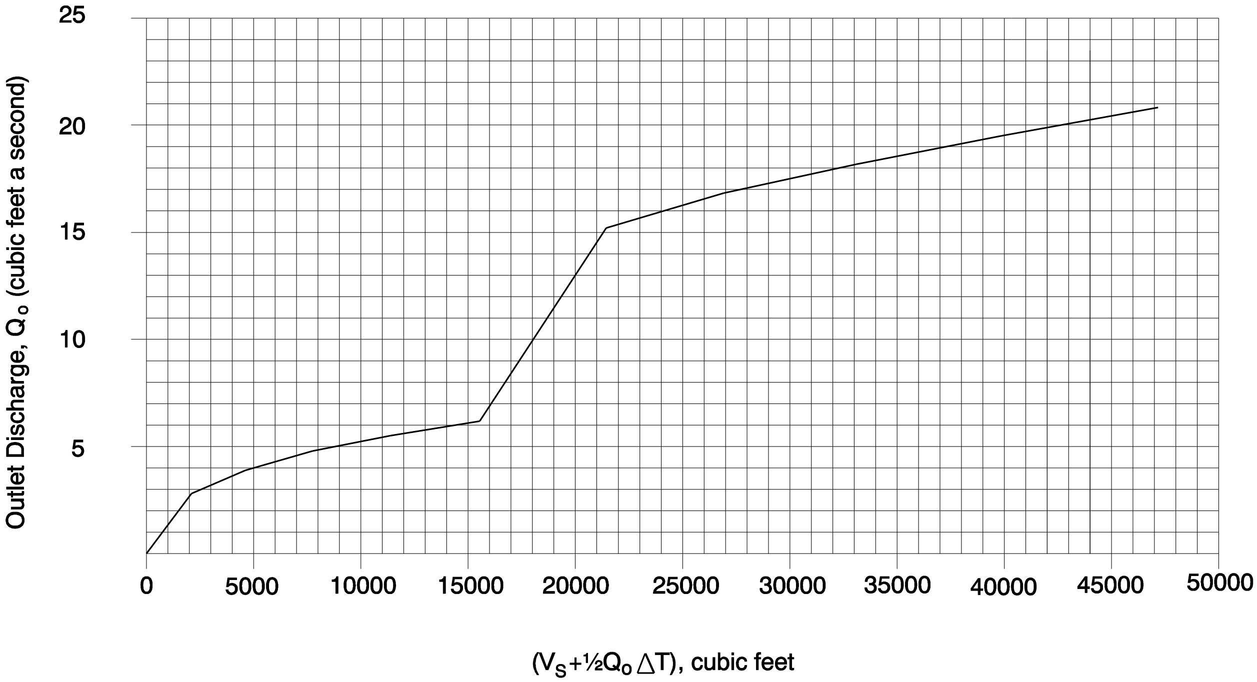


**Figure 12-47 Storage Indication Curve for Example**

## 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.

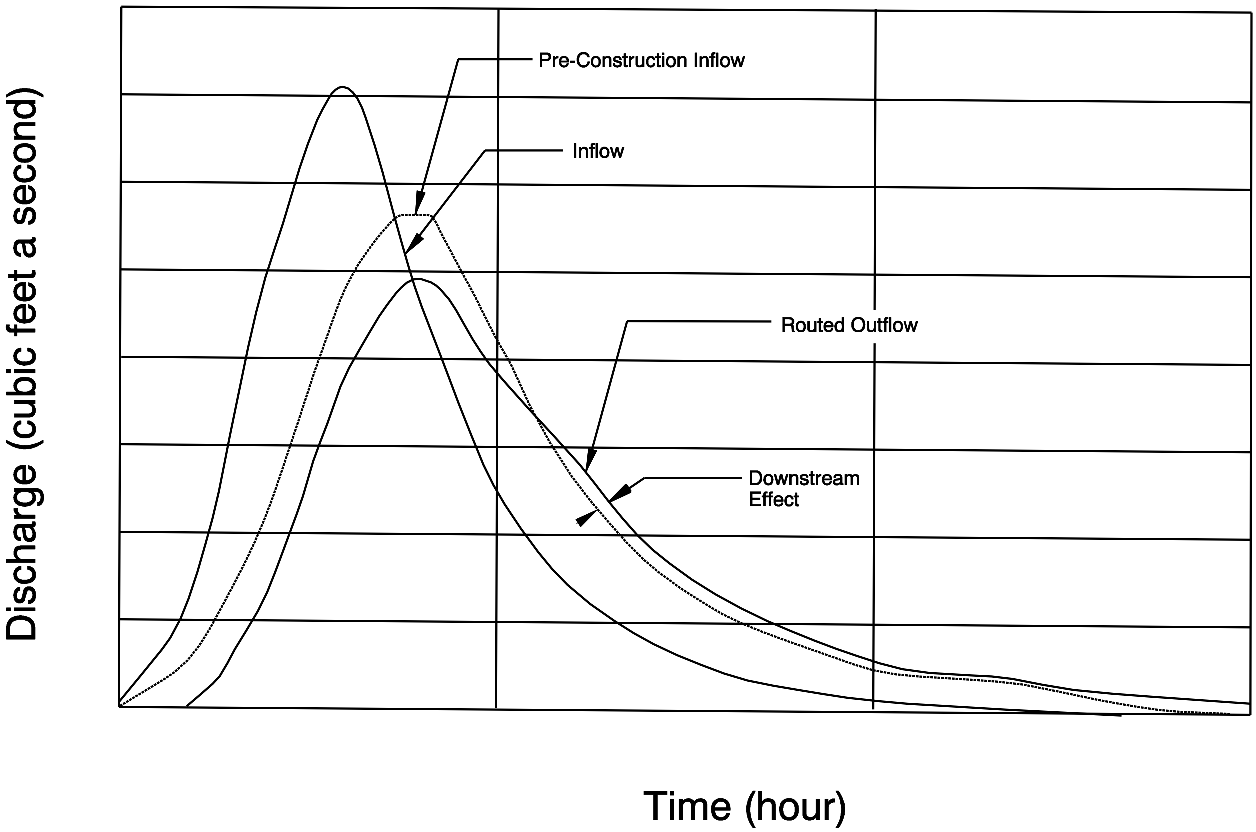


**Figure 12-48 Downstream Effects**