



calculating site hydrology (sub-basin delineation, land use types, runoff coefficients, time of concentration, and peak discharges) and hydraulic analysis (pipe flow capacity and hydraulic grade line). Hand calculations or computer programs and spreadsheets would be used to evaluate and document a storm drain system design.

*Note: Hand calculations are included in this appendix for full and partial full flow design. Hand calculations for surcharged-full flow design are discussed in [Appendix G](#). Step-by-step procedures have been included to aid the designer in understanding the analysis process, but the most efficient means of evaluating storm drain systems is with computer programs.*

## 2.0 Design Guidelines

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The following design guidelines to be used in storm drainage system design.

- Storm drainage systems shall not alter the existing drainage pattern without the written permission of the affected property owners. Right-of-way negotiations should cover this aspect of the sewer design and written releases obtained from the property owners where changes in drainage patterns will occur.
- Storm drainage systems shall be designed for the runoff from the level of development present at the time of design with the following exceptions:
  - A. Projects within the boundaries of an urban growth or master drainage plan will be designed for runoff from future development subject to the following agreement:
    - The cost of this system will be shared as follows:
      - The Highway Division will pay for a system designed for runoff from the level of development present at the time of design.
      - Additional cost for a system designed for future development is the responsibility of city, county or developer as appropriate.
  - B. If the system is not designed for future development, connections to the system will not be allowed unless the following conditions are met:
    - The connecting system collects runoff from areas which naturally drain to the highway storm system.
    - Runoff from the connecting system will be limited to the runoff the storm system was designed to accept from the areas drained by the connecting system.
    - Storm drainage systems shall be designed for future anticipated improvements to the highway which would increase the runoff to the system. Adding curbs to an uncurbed section of highway is an example of an improvement which would increase the runoff to the system.

- Storm drainage systems shall be designed to operate under full or partial-full flow when conditions permit. Surge flow shall be permitted only when back water from the receiving stream will submerge the storm sewer.
- Storm drainage systems shall be designed to operate at full or partial-full flow (open-channel flow) for the design storms described in **Chapter 3**. Open channel design only accounts for major losses (pipe friction losses) as demonstrated in the example problem at the end of this appendix. An increase to the pipe roughness coefficient should be applied to account for minor losses (manhole losses, bends in pipes, expansion and contraction losses, etc.). **Chapter 8, Appendix A** contains hydraulic roughness (Manning's n) values for conduits. When applicable, a minimum, normal, and maximum roughness coefficient is provided for various conduits. Use the maximum roughness coefficient to account for minor losses or the normal coefficient when minor losses will be calculated.
- Storm drainage systems operating under surcharge conditions (pressure flow) shall evaluate the hydraulic grade line and minor losses (manhole losses, bends in pipes, expansion and contraction losses, etc.) as discussed in [Appendix G](#). Adjust inlet sizes/locations and pipe size/location to correct hydraulic grade line problems. Pressure flow design must ensure the hydraulic grade line is 0.5 feet or more below the rim elevation of any drainage structure which may be affected. The energy grade line must also be at or below the rim elevation of any drainage structure which may be affected. A pressure flow example problem is provided in [Appendix G](#).
- Water quality – runoff collected by storm drain systems needs to address the most stringent local standards or refer to ODOT's water quality design guidelines (reference **Chapter 14**).
- Water quantity – runoff collected by storm drain systems needs to address the most stringent local standards or refer to ODOT's water quantity design guidelines (reference **Chapter 12**).

### 3.0 Storm Drain Design Flows

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Storm drain systems are designed to convey the discharge recurrence intervals listed in **Chapter 3**. Higher design discharges may be justified for critical installations where considerable danger to the public or expense could occur as a result of failure.

*Note: A higher design discharge is required for storm drains that convey runoff from highway sag points (reference **Chapter 3**).*

## 4.0 Storm Drain Outfalls

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All flow from a storm drain system is discharged to an outfall. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the storm water away from the highway. Outfall conditions are an important part of storm drain design as described in this section.

The following system features must be evaluated when designing outfalls:

- The flow line or invert elevation of the proposed storm drain outlet
- Tailwater elevations
- The need for energy dissipation
- The orientation of the outlet structure

The flow line or invert elevation of the proposed outlet should be equal to or higher than the flowline of the outfall. If this is not the case, there may be a need to pump the water to the elevation of the outfall.

*Note: Positive gravity flow is the desired method of discharge. All non-pumping options should be evaluated such as relocating the discharge point to an area that would allow positive gravity flow of stormwater before recommending the construction of a pumping system. Relocation of an outfall should not alter the existing drainage pattern without the written permission of the affected property owners.*

There may be instances in which excessive tailwater conditions cause the hydraulic gradeline to be above grade, which in turn pushes flow out of inlets and access holes, possibly creating hazardous flooding conditions. The potential for this hydraulic condition should be examined. Flap gates placed at the outlet can sometimes alleviate this condition; otherwise, it may be necessary to isolate the storm drain from the outfall by use of a pumping system.

Energy dissipators are used to reduce the velocity and, consequently, the erosion potential of flowing water. Protection is usually required at the outlet of storm drain systems that discharge onto embankments, into natural or unlined channels, or into drainage swales and ponds. Typical storm drain outfall energy dissipators meeting ODOT design guidelines are discussed in Chapter 11.

The orientation of the outfall is another important design consideration. Where practical, the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for excessive erosion. If the outfall structure can not be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of a receiving channel, care must be taken to avoid erosion on the opposite

channel bank. Armoring may be necessary along this scour critical area and should be protected by revetment or suitable biotechnical methods or both (reference **Chapter 15**).

Listed below are additional guidelines that need to be considered when designing outfalls:

- Runoff should discharge from a single outfall. Avoid multiple outlet pipes from a single outfall structure.
- When feasible, the outfall location selected should not be submerged.
- Avoid outfall locations that are subject to high velocities.
- Consideration should be given to erosion protection along the receiving stream and where storm drain outfall discharge occurs.
- Be aware of environmentally sensitive areas when selecting an outfall location.

## 5.0 Open Channel vs. Pressure Flow

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Two design approaches may be used for sizing storm drains under the steady uniform flow assumption. The first is referred to as open channel or gravity flow design. To maintain open channel flow, the segment must be sized so that the water surface within the conduit remains open to atmospheric pressure. For open channel flow, flow energy is derived from the flow velocity (kinetic energy), depth (pressure), and elevation (potential energy). If the water surface throughout the conduit is to be maintained at atmospheric pressure, the flow depth must be less than the height of the conduit.

Pressure flow design requires that the flow in the conduit be at a pressure greater than atmospheric. Under this condition, there is no exposed flow surface within the conduit. In pressure flow, flow energy is again derived from the flow velocity, depth, and elevation. The significant difference here is that the pressure head will be above the top of the conduit, and will not equal the depth of flow in the conduit. In this case, the pressure head rises to a level represented by the hydraulic grade line (see [Appendix G](#) for a discussion of the hydraulic grade line).

The question of whether open channel or pressure flow should control design has been debated. For a given flow rate, design based on open channel flow requires larger conduit sizes than those sized based on pressure flow. While it may be more expensive to construct storm drainage systems designed based on open channel flow, this design procedure provides a margin of safety by providing additional headroom in the conduit to accommodate an increase in flow above the design discharge. This factor of safety is often desirable since the methods of runoff estimation are not exact, and once placed, storm drains are difficult and expensive to replace.

There may be situations where pressure flow design is a desirable hydraulic condition. For example, on some projects, there may be adequate headroom between the conduit and inlet/access hole elevations to tolerate pressure flow. In this case, significant costs savings may be realized over the cost of a system designed to maintain open channel flow. Also, in some cases it may be necessary to use an existing system which must be placed under pressure flow to accommodate the proposed design flow rates. In instances such as these, there may be advantages in making a cursory hydraulic and economic analysis of a storm drain using both design methods before making a final selection.

Under most ordinary conditions, it is recommended that storm drains be sized based on a gravity flow criteria at flow full or near full. Designing for full flow is a conservative assumption since the peak flow actually occurs at 93 percent of full flow. However, the designer should maintain awareness that pressure flow design may be justified in certain instances. When pressure flow is allowed, special emphasis should be placed on the proper design of the joints so that they are able to withstand the pressure flow.

## 6.0 Hydraulic Capacity

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The most widely used formula to calculate the velocity and discharge in storm drains flowing full, but not under pressure flow is the Manning's equation.

The Manning's equation for full flow velocity is:

$$V_{\text{full}} = \left[ \frac{1.486}{n} \right] R^{2/3} S^{1/2} \quad (\text{Equation 1 and Chapter 8})$$

The Manning's equation for full flow discharge is:

$$Q_{\text{full}} = \left[ \frac{1.486}{n} \right] A R^{2/3} S^{1/2} \quad (\text{Equation 2 and Chapter 8})$$

Where:

- V = mean velocity in feet per second
- Q = rate of flow in cubic feet per second
- n = Manning roughness coefficient (**Chapter 8**)
- D = storm drain diameter in feet
- S = slope of the energy grade line in foot per foot (S = conduit slope for steady uniform flow)
- A = cross-sectional area in square feet
- R = hydraulic radius = A/P in feet
- P = wetted perimeter in feet

A complete discussion of closed conduit (storm drains) hydraulic properties was introduced in **Chapter 8**. This chapter contains nomographs to assist in the solution of the Manning's equation for full flow in storm drains. It also contains a hydraulic elements chart to assist in the solution of the Manning's equation for partial-full flow in storm drains.

## 7.0 Minimum Velocity and Grades

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Storm drains should be designed to maintain full-flow pipe velocities of 3.0 feet per second or greater to maintain a self-cleaning velocity in the storm drain to prevent deposition of sediments and subsequent loss of capacity. Minimum slopes required for a velocity of 3 feet per second can be computed using the form of Manning's equation given in Equation 3 or use values provided in Table A.

$$S = 2.9 \left[ \frac{nV}{D^{0.67}} \right]^2 \quad \text{(Equation 3 and Chapter 8)}$$

Where:

- S = slope of the energy grade line in feet per foot (S = conduit slope for steady uniform flow)
- n = Manning's coefficient
- V = mean velocity in feet per second
- D = diameter of pipe in feet

**Table A Minimum Pipe Slopes**  
*(To ensure 3 feet per second velocity in storm drains flowing full)*

Pipe Size Inches	Full Pipe Flow Cubic feet per second	Minimum Slopes, foot per foot		
		n	n	n
		0.013	0.024	0.027
6	0.59	0.0112	0.0381	0.0482
8	1.06	0.0076	0.0259	0.0328
10	1.65	0.0056	0.0192	0.0243
12	2.37	0.0044	0.0150	0.0190
16	4.21	0.0030	0.0102	0.0129
18	5.33	0.0026	0.0087	0.0111
21	7.25	0.0021	0.0071	0.0090
24	9.47	0.0017	0.0059	0.0075
30	14.78	0.0013	0.0044	0.0056
36	21.28	0.0010	0.0034	0.0044
48	37.79	0.0007	0.0023	0.0030
60	59.00	0.0005	0.0017	0.0022

## 8.0 Minimum Pipe Diameter

The minimum diameter for a storm drain pipe is 12 inches. Storm drain pipes should not decrease in size in a downstream direction regardless of the available pipe gradient.

## 9.0 Curved Alignment

Curved pipe alignment should only be used when it is not practical to change the pipe alignment with manholes. Check with the appropriate District Maintenance Supervisor before specifying a curved pipe alignment.

Long radius bend sections are available from many suppliers and are the preferable means of changing direction in pipes 48 inches and larger. Short radius bend sections are also available



and can be utilized if there is not room for the long radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. Utilizing large access holes solely for changing direction may not be cost effective on large size storm drains.

## 10.0 Storm Drains on Steep Slopes

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Storm drain systems that are designed for steep slopes need to take into consideration a number of design elements such as pipe materials to be used, pipe anchors, severe abrasion, high velocities, type of pipe joints, spacing of joints, use of armoring materials, and types of bed load materials flowing in the pipe system. Many of these design features are referenced in **Chapter 5**.

Listed below are additional guidelines that need to be considered when designing storm drains on steep slopes:

- Concrete pipes should not be used when the slope exceeds 20 percent.
- Differential settlement or movement of joints.
- Metal and plastic pipes which are used on steep slopes that exceed 20 percent should be anchored to prevent movement.
- Refer to ODOT Standard Drawing RD332 for pipe anchor details.
- Energy Dissipation is an important element for storm drain systems on steep slopes. Design elements related to the types and selection of energy dissipators is discussed in **Chapter 11**.

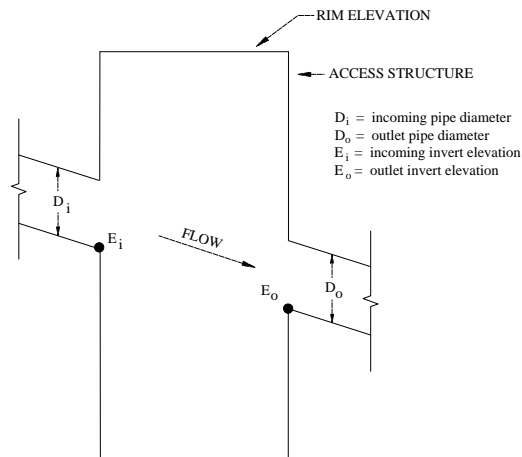
## 11.0 Cleanouts

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Permanent cleanouts are not allowed unless concurrence is obtained by the ODOT District maintaining the storm drain system. Temporary clean outs will be evaluated on a case by case basis.

## 12.0 Pipe Connections in a Junction

The crowns of all incoming pipes at a junction should be at the same elevation or higher than the crown of the outlet pipe when the outlet pipe is the same size or larger than the incoming pipes.



**Pipe Connections in a Junction**

$D_o$  is always greater than or equal to the diameter  $D_i$  and  $E_o$  is less than  $E_i$ . This is the recommended case for pipe connections to a junction. When the elevation difference in a junction exceeds 4 feet a drop manhole shall be used.

## 13.0 TV Pipe Inspections

After construction of storm drain systems a video inspection should be conducted. It is recommended to inspect 100 percent of the system. A minimum of 25 percent of the total storm drain system is required to be inspected with video equipment. It is recommended that the equipment include a way to record distances and can record the inspection activities onto video tape or DVD.

## 14.0 Utility Conflicts

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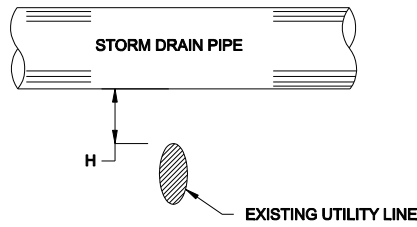
Storm drain alignment, both vertical and horizontal, is often affected by utilities located in close proximity or direct conflict with existing utilities, such as sanitary sewers, fiber optic cable, gas, water, etc. ODOT provides utilities a permit to install facilities within ODOT right-of-way. A condition of the permit requires the utility to relocate if there is a conflict with ODOT facilities. In some instances, ODOT may redesign its facilities to avoid utility disruption.

A utility facility is in conflict if a highway construction element is within two (2) feet of the outside diameter of the utility or is directly over or under for trenching, drilling or construction activities. The utility owner is required to provide information about these clearances as illustrated in the figure below (examples 1 and 2). The vertical distance (H) may vary depending on the utility type; therefore, early design coordination between ODOT and utility owner is necessary to minimize project delays caused by utility relocation work.

The clearances can be reduced, or sometimes eliminated, if casings or other protection measures are installed around the utility as illustrated in the figure below (examples 3 and 4). Listed below are additional guidelines that need to be considered when casing or proposing other protection measures:

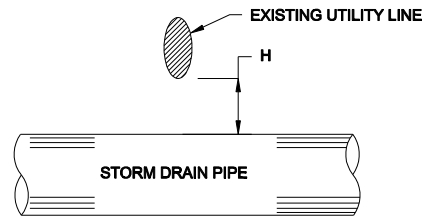
- The design capacity of the storm drain system must be maintained and the potential for blockage by debris must be minimized when a carry-through pipe is used in an access manhole.
- The carry-through pipe sleeve through an access manhole shall be ductile iron pipe. Reference ODOT standard drawing RD354.
- Coordinate casing or other protection measure with utility owner

In all cases, the final storm drain design should not be made until affected utilities are located. In some instances, pothole excavation and survey is needed to determine the exact utility location. This is an important factor in storm drain design because unanticipated utility conflicts are time consuming and expensive when encountered during construction. Furthermore, accidentally encountering gas lines, high voltage power lines or other utilities during construction could have deadly consequences.



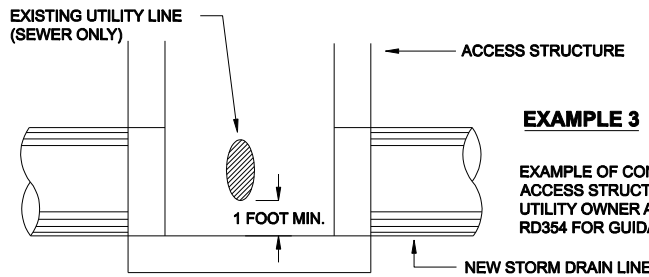
**EXAMPLE 1**

EXAMPLE OF STORM DRAIN CROSSING ABOVE AN EXISTING UTILITY. THE VERTICAL DISTANCE (H) MAY VARY DEPENDING ON THE TYPE OF UTILITY. NO PROTECTION MEASURES ARE REQUIRED WHEN (H) IS GREATER THAN OR EQUAL TO A MINIMUM CLEARANCE SPECIFIED BY THE UTILITY OWNER. CASING, ACCESS STRUCTURE OR OTHER PROTECTION MEASURES ARE REQUIRED WHEN VERTICAL DISTANCE (H) IS LESS THAN THE MINIMUM CLEARANCE SPECIFIED BY THE UTILITY OWNER.



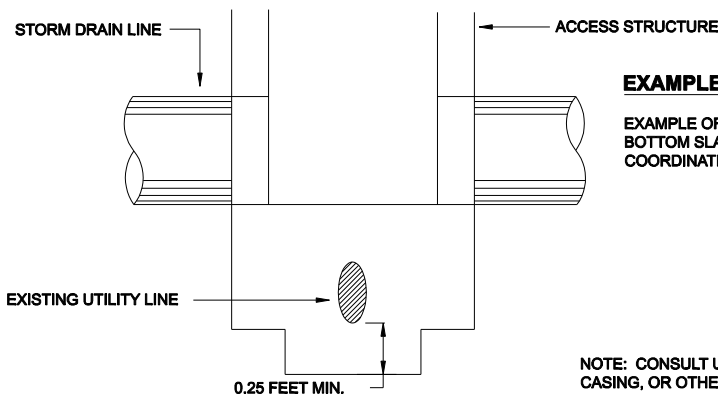
**EXAMPLE 2**

EXAMPLE OF STORM DRAIN CROSSING BELOW AN EXISTING UTILITY. THE VERTICAL DISTANCE (H) MAY VARY DEPENDING ON THE TYPE OF UTILITY. NO PROTECTION MEASURES ARE REQUIRED WHEN (H) IS GREATER THAN OR EQUAL TO A MINIMUM CLEARANCE SPECIFIED BY THE UTILITY OWNER. CASING, ACCESS STRUCTURE OR OTHER PROTECTION MEASURES ARE REQUIRED WHEN VERTICAL DISTANCE (H) IS LESS THAN THE MINIMUM CLEARANCE SPECIFIED BY THE UTILITY OWNER.



**EXAMPLE 3**

EXAMPLE OF CONFLICT UTILITY CROSSING INSIDE A STORM DRAIN ACCESS STRUCTURE BARREL. COORDINATE CONFLICT CROSSING WITH UTILITY OWNER AND/OR REFERENCE ODOT STANDARD DRAWING NO. RD354 FOR GUIDANCE.



**EXAMPLE 4**

EXAMPLE OF CONFLICT UTILITY CROSSING THROUGH THE BOTTOM SLAB OF A STORM DRAIN ACCESS STRUCTURE. COORDINATE CONFLICT CROSSING WITH UTILITY OWNER.

NOTE: CONSULT UTILITY OWNER FOR MINIMUM CLEARANCE, CASING, OR OTHER PROTECTION MEASURE INFORMATION.

**Storm Drain Conflicts**

## 15.0 Full and Partial-full Flow

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The following is the procedure to design storm drains for full and partial-full flow. The Storm Drain Design Sheet shown on Figure 1 should be used to tabulate the hydraulic data for the design.

- Step 1-** Prepare a map or maps (**Chapter 6**) showing the entire drainage basin contributing storm water to the system. Layout the proposed storm sewer indicating the location of each manhole and inlet.
- Step 2-** Delineate the basin drainage area contributing to each inlet, then divide these areas into subareas having similar RUNOFF COEFFICIENTS (ground cover and slope).

*Note: Layout of the storm drain system, drainage basin map and storm sewer design sheets must be documented as in **Chapter 4**.*

- Step 3-** Starting at the upper end of the system, label each of the subareas with an index number (1, 2, 3, etc.) and circle each.
- Step 4-** Enter in Column 1 of the STORM SEWER DESIGN SHEET (Figure 1) the station for the inlet of the pipe run under consideration.
- Step 5-** Enter in Column 2 the index number for each subarea.
- Step 6-** Enter in column 3 the area in acres of each subarea.
- Step 7-** Select the RUNOFF COEFFICIENT (**Chapter 7**) that best fits the ground cover condition and slope for each subarea. Enter this coefficient in Column 4 opposite the corresponding subarea.
- Step 8-** Multiply AREA in Column 3 by the RUNOFF COEFFICIENT in Column 4. Enter in Column 5.
- Step 9-** Sum all the EQUIVALENT AREAS in Column 5 for the pipe section under consideration. Enter sum in Column 6 opposite the last subarea in Column 5.
- Step 10-** Enter in Columns 1a and 1b (same forms) the station and node I.D. for the outlet of the pipe under consideration.
- Step 11-** Determine the TIME OF CONCENTRATION for the first pipe run only. For overland flow use the Kinematic Wave Equation (see **Chapter 7**) and for channel flow use the Manning Equation for open-channel flow (see **Chapter 8**). In the case of

- paved areas, the minimum time of concentration,  $T_c$  shall be 5 minutes while the minimum  $T_c$  for nonpaved areas shall be 10 minutes.
- Step 12-** Enter in Column 7 the value in minutes determined in Step 11.
- Step 13-** Add all TIME OF CONCENTRATION or FLOW TIME values in Column 7 for previous pipe runs on the same sewer branch. Enter in Column 8.
- Step 14-** Determine the RAINFALL INTENSITY from the appropriate Intensity-Duration Curve (see **Chapter 7**). Enter in Column 9. NOTE: RAINFALL DURATION is considered equal to the value in Column 8.
- Step 15-** Multiply the RAINFALL INTENSITY in Column 9 by the DRAINAGE AREA in Column 6 to determine the DESIGN DISCHARGE. Enter in Column 10.
- Step 16-** Enter in Column 11 a trial slope for the sewer pipe. Usually, the slope of the roadway can be used.
- Step 17-** Hydraulic pipe characteristics, roughness coefficients, and equations are located in **Chapter 8** to aid manual calculations. Also, nomographs for several types of pipes are located in **Chapter 8**. When the appropriate nomograph is available, determine the size of pipe by laying a straight edge between the DISCHARGE and SLOPE scales using the values in Column 10 and Column 11. The appropriate size pipe will be read directly above the straight edge on the PIPE DIAMETER scale. Enter this size in Column 12.
- Step 18-** Adjust the straight edge on the nomograph such that it lies on the SLOPE (Column 11) and the PIPE DIAMETER (Column 12).
- Step 19-** Read the pipe CAPACITY at full flow on the DISCHARGE scale. If this value is 10 percent larger than the DESIGN FLOW in Column 9, then the pipe slope should be flattened if feasible. The slope at which the pipe will just flow full can be found by laying the straight edge between the DESIGN DISCHARGE and CULVERT DIAMETER scales, and reading the new slope on the SLOPE scale.
- Step 20-** Re-enter in Column 11 the new pipe slope if an adjustment is made.
- Step 21-** Enter in Column 13 the pipe CAPACITY for the slope selected.
- Step 22-** With the straight edge on the DESIGN SLOPE and PIPE DIAMETER scales, read the DESIGN VELOCITY. The minimum DESIGN VELOCITY is 3 feet per second. Enter in Column 14.

- Step 23-** Enter in Column 15 the LENGTH of sewer pipe between stations listed in Column 1.
- Step 24-** Divide the LENGTH in Column 15 by the VELOCITY in Column 14, then divide the resultant by 60 to determine the FLOW TIME in minutes. Enter this value in Column 7.
- Step 25-** Multiply the INVERT SLOPE in Column 11 by the LENGTH in Column 15 to determine the FALL. Enter in Column 16.
- Step 26-** Enter in Column 17 the INVERT ELEVATIONS for both the inlet and outlet ends of the pipe.
- Step 27-** Enter in Column 18 the TOP OF MANHOLE ELEVATION at both the inlet and outlet ends of the pipe.
- Step 28-** Enter in Column 19 REMARKS pertinent to the pipe being considered.
- Step 29-** Repeat procedure for next run of pipe except that the TIME OF CONCENTRATION in Step 11 (Column 7) is replaced by FLOW TIME from Step 24 for the succeeding pipe runs.

## 16.0 Storm Drain Design (Single Branch Full Flow) Example

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The following example demonstrates the design process and calculations for a single branch storm drain flowing full. It will be designed to convey stormwater from the drainage basins delineated on Figure 2 (drainage map) to Mill Creek in Salem, Oregon.

### PART A

The following design steps outline the proposed storm drain system's characteristics (i.e. horizontal/vertical system alignment, delineating drainage basins, preparing a drainage map, collection structure selection, hydrology calculations, etc.) which are necessary prior to pipe sizing design:

- Step A.1-** Determine the inlet type (**Chapter 13**) and curb type (**Chapter 8**) which would provide the most economical method of collecting storm water from the highway and side areas. The CG-2 inlet was used for this design because of the considerable water and debris anticipated and also because it is a multiple lane highway. Calculate the inlet spacing using the methods shown in the inlet spacing and capacity appendix. The following table shows a comparison between curb type and the estimated number of CG-2 inlets from station 0+00 to station 10+00. Zero bypass was used as a design criterion for the storm drain.

Curb Type	Inlet Number CG-2
Standard Curb	13
Mountable Curb	50
Curb & Gutter	50

The **Standard Curb** would be the most economical based on the estimated number of inlets and will be used for this design.

- Step A.2-** Locate the inlets (see Figure 3) and delineate the drainage area to each storm water collection point. An alternative to the manhole with inlet would be a manhole and a CG-2 inlet located close by. This inlet would be connected into the manhole. The manhole and inlet alternate was not selected based on its additional cost. Numerically identify each delineated drainage area. Connect the inlets as shown on Figure 3 with lines to represent pipe alignment.
- Step A.3-** Locate and sketch the manhole, connecting the outfall pipe and the outfall as shown on Figure 3. The outfall should direct the storm water in the same general direction as the creek. The manhole should be located on the creek bank in a protected location. The outfall should be protected from potential flood damage.



- Step A.4-** Enter in Column 1a and 1b of the "Storm Sewer Design Sheet" (Figure 1) the station and node I.D.s of the inlets, the manhole with inlet grate, the manhole, and the outfall shown on Figure 3.
- Step A.5-** Enter the connecting pipe lengths from Figure 3 onto **Figures 4, 5 and 6** (Column 15).
- Step A.6-** Determine highwater elevation and average creek velocity at the outlet location shown on Figure 3 for the 50-, 25-, 10- and 5-year floods. The Hydraulics Engineer would be one source of obtaining the information listed in the table below.

Mill Creek Data

Recurrence Interval	Highwater Elevation at Outlet, feet	Average Velocity in Creek at Outlet, feet per second
50-Year	83.0	8.0
25-Year	81.5	7.5
10-Year	80.0	7.0
5-Year	77.0	6.0

- Step A.7-** The invert elevation of 77.85 feet was entered in Column 17 for the outlet end of the 90-foot long outfall pipe. From a field surveyed cross-section near the outfall, the bottom of the creek was estimated to be at elevation 77.85 feet. An outfall elevation of 77.85 feet was selected for this design and entered in Column 17. The outfall will be submerged by the creek; however, the rest of the system will not.
- Step A.8-** Enter the delineated upstream drainage area to each **trunk line** collection point in Columns 2, 3, 4, 5, and 6.

*Note: A common error in storm sewer design is to arbitrarily assume the limits of the contributing side drainage. The best way to determine the contributing drainage area is to walk the project and observe the limits of side drainage. This is particularly effective during a rain storm.*

- Step A.9-** Using information provided on Figure 2, determine the finish grade of the roadway above the trunk line and enter in Column 18 of **Figures 4, 5, and 6** using the relationship given below. The storm drain trunk line will be located 42 feet right of centerline.

$$\begin{aligned} \text{Top of Manhole or Inlet Grate Elevation} &= \text{Finish Grade at Center Line} - (\text{Cross} \\ &\quad \text{Slope feet/feet})(\text{Highway Width} / 2) \\ &= \text{Finish Grade at Center Line} - (0.02)(40 \text{ feet}) + (0.08)(2 \text{ feet}) \end{aligned}$$

Top of Manhole/Inlet Elevation = Finish Grade at Center Line - 1.0 feet

The results are shown in the table below:

Station	Finish Grade at Center Line, feet	Top of Inlets or Manhole Elevation, feet (Column 18)
0+00 Rt	110.1	--
3+00 Rt	105.9	104.9
4+50 Rt&Lt	103.8	102.8
6+00 Rt&Lt	101.7	100.7
7+00 Rt&Lt	100.3	99.3
8+00 Rt&Lt	98.9	97.9
9+00 Rt&Lt	97.5	96.5
10+00 Rt&Lt	96.1	95.1
		Elev. Natural Ground
10+00 (392 feet Rt)	--	95.5

**Step A.10-** Determine the minimum starting invert elevation (Column 17) at the beginning of **the trunk line and each lateral** using the following relationship. Assume 12-inch diameter pipe since it is the smallest allowable pipe size for storm drains. Smooth walled PVC pipe with a minimum allowable cover of one foot will be used with this design. To account for minor losses, a slightly higher roughness coefficient will be used. A roughness coefficient for concrete ( $n = 0.013$ ) will be used for this design.

Minimum Invert Elevation	=	Top of Manhole or Inlet Elevation	-	Pavement Thickness	-	Pipe Diameter	-	Shell Thickness of Pipe	-	Minimum Cover
Starting Invert Elevation	=	Top of Manhole or Inlet Elevation		1.0 feet	-	1.0 feet	-	2.5/12	-	1.0 feet
Starting Invert Elevation	=	Top of Manhole or Inlet Elevation		3.21 feet						

Enter the following minimum starting Invert Elevations in Column 17:

Station	Top of Manhole or Inlet Elevation, feet	Starting Invert Elevation, feet	Remarks	Comment
3+00 Rt	104.9	101.69	TRUNK	Fig. 4, NODE 2
4+50 Lt	102.8	99.59	LATERAL	Fig. 5, NODE 1
6+00 Lt	100.7	97.49	LATERAL	Fig. 5, NODE 4
7+00 Lt	99.3	96.09	LATERAL	Fig. 5, NODE 6
8+00 Lt	97.9	94.69	LATERAL	Fig. 6, NODE 8
9+00 Lt	96.5	93.29	LATERAL	Fig. 6, NODE 10
10+00 Lt	95.1	91.89	LATERAL	Fig. 6, NODE 12

**Step A.11-** Enter a 5-minute time of concentration in Columns 7 and 8 for the beginning of the trunk line (**Figure 4**) and for each lateral (**Figures 5 and 6**). Calculations for the 5-minute time of concentration are shown below for overland and gutter flow over pavement. The gutter is classified as **Standard Curb** (**Chapter 8**).

Figure 7 shows the estimated drainage paths for overland and gutter flow. There are two drainage paths which need to be investigated. The first path is the sum of  $L_o$  and  $L$ . The second path is the sum of  $L_1$  and  $L_2$ .

Given:

$$\begin{aligned}
 n &= 0.016 \text{ (Manning's coefficient for asphalt pavement)} \\
 S &= 0.014 \text{ feet/feet (roadway longitudinal slope)} \\
 S_{x3} &= 0.02 \text{ feet/feet (roadway cross slope)} \\
 S_w &= 0.083 \text{ feet/feet (gutter cross slope)} \\
 W &= 2 \text{ feet (depressed gutter width)} \\
 I &= 2.12 \text{ inches/hour (Zone 7, Chapter 7)} \\
 Q &= \text{CIA (Chapter 7) in cubic feet per second} \\
 Q &= (0.9)(2.12)(42 \times 300 + 100 \times 110) / 43,560 \\
 &= 1.03 \text{ cubic feet per seconds}
 \end{aligned}$$

(The gutter flow variables are defined in **Chapter 8**)

Time of concentration overland flow:

$$T_c = 0.93 \left[ \frac{L^{0.6} n^{0.6}}{I^{0.4} S^{0.3}} \right] \quad \text{(Chapter 7)}$$

$$\begin{aligned}
 T_c &= 0.93 \left[ \frac{(42)^{0.6} (0.016)^{0.6}}{(2.12)^{0.4} (0.02)^{0.3}} \right] \\
 &= 1.75 \text{ minutes for overland path } L_o
 \end{aligned}$$

$$T_c = 0.93 \left[ \frac{(110)^{0.6} (0.016)^{0.6}}{(2.12)^{0.4} (0.01)^{0.3}} \right]$$

$$= 3.85 \text{ minutes for overland path } L_1$$

Estimate flow widths with **Standard Curb** (Chapter 8).

By trial and error using the equations from Chapter 8 for a **Standard Curb**:

$$Q = \frac{(0.56)S^{0.5}}{n} \left[ \frac{d_2^{2.67}}{S_{x3}} + \frac{(d^{2.67} - d_2^{2.67})}{S_w} \right] \quad (\text{Chapter 8})$$

$$d_2 = S_{x3}T_3$$

$$d = S_{x3}T_3 + WS_w$$

$$Q = \frac{(0.56)(0.014)^{0.5}}{0.016} \left[ \frac{d_2^{2.67}}{S_{x3}} + \frac{(d^{2.67} - d_2^{2.67})}{S_w} \right]$$

$$Q = 4.733 \left[ \frac{d_2^{2.67}}{S_{x3}} + \frac{(d^{2.67} - d_2^{2.67})}{S_w} \right]$$

$$d_2 = 0.02T_3$$

$$d = d_2 + (2)(0.083) = d_2 + 0.166$$

Assume:  $T_3 = 2.5$  feet

$$d_2 = 0.02(2.5) = 0.05 \text{ feet}$$

$$d = 0.05 + 0.166 = 0.216 \text{ feet}$$

$$Q = 4.733 \left[ \frac{0.05^{2.67}}{0.02} + \frac{(0.216^{2.67} - 0.05^{2.67})}{0.083} \right]$$

$$Q = 1.01 \text{ cubic feet per second -- close enough}$$

$$\text{Total flow width} = T_3 + W = 2.5 \text{ feet} + 2 \text{ feet} = 4.5 \text{ feet}$$

$$A = \frac{1}{2} \left[ w(d + d_2) + \frac{d_2^2}{S_{x3}} \right]$$

$$Q = VA, \text{ discharge, cubic feet per second}$$

$$V = Q / A, \text{ velocity, feet per second}$$

$$V = \frac{2(1.03)}{\left[ 2(0.216 + 0.05) + \frac{0.05^2}{0.02} \right]}$$

$$V = 3.13 \text{ feet per second}$$

$$T_c = 300 / (3.13)(60) = 1.60 \text{ minutes for channel flow path } L$$

$$T_c = 100 / (3.13)(60) = 0.53 \text{ minutes for channel flow path } L_2$$

Determine longest time of concentration

$$\text{Path } L_o + L: T_c = 1.75 + 1.60 = 3.35 \text{ minutes}$$

$$\text{Path } L_1 + L_2: T_c = 3.85 + 0.53 = 4.38 \text{ minutes}$$

Path  $L_1 + L_2$  yields the controlling time of concentration; however, the minimum allowable time of concentration is 5 minutes. Consequently the starting time of concentration is 5 minutes. A similar analysis should be done for all storm sewers to determine the starting time of concentration. When in doubt, use 5 minutes as this will yield the most conservative design.

*Note: Refer to **Chapter 7** for additional discussion on determining time of concentration.*

## PART B

The following design steps outline the procedure for sizing storm drains. The design analysis is documented on Figures 4, 5, and 6.

**Step B.1-** Obtain the average 10-year rainfall intensity from **Chapter 7** for a 5.0 minute time of concentration. The 10-year rainfall intensity for Salem is 2.12 inches per hour and would be entered in Column 9 in **Figures 4, 5, and 6**.

**Step B.2-** Multiply Column 9 by Column 6 and enter the results in Column 10 in **Figures 5 and 6**.

**Step B.3-** Using Figure 8 by drawing a line through the points for a 12-inch pipe and 3.0 feet per second velocity on the nomograph, determine the pipe invert slope, capacity flowing full, and velocity flowing full and enter the results in Columns 11, 13 and 14 (**Figures 5 and 6**). Verify that the assumed 12-inch pipe has capacity flowing full which is at least equal to the calculated design discharge in Column 10. In this example all lateral pipes have sufficient capacity.

**Step B.4-** Determine and enter the fall in Column 16 for each of the **laterals** using the following relationship:

$$\boxed{\text{Fall in feet} = \text{Invert Slope in feet/feet} \times \text{length in feet}}$$

For station 4+50 Lt.

$$\text{Column 16} = \text{Column 11} / 100 \times \text{Column 15}$$

$$\text{Fall} = (0.45 / 100)(84) = 0.38 \text{ feet}$$

**Step B.5-** The outlet invert elevations for the upstream structures were determined in Step A.10. Now determine the inlet invert elevations of the 12-inch diameter laterals

at the downstream structures by subtracting Column 16 from Column 17. Enter the results in Column 17, **Figures 5 and 6**.

*Note: Steps B.6 through B.7 are to be completed for the outlet at 4+50Rt before completing the remainder of columns.*

- Step B.6-** Determine and enter the fall (Column 16) for the outlet of the **trunk line** at Station 4+50Rt. The fall also includes enough depth so that the crown of the lateral pipe will be at or above the crown of the trunk line using the following relationship. Enter the result in Column 16, **Figure 4**.

Minimum Fall (Column 16)	=	Longitudinal Highway Slope % (Figure 5)	x	<u>Length</u> (Column 15) 100	+	Lateral Fall (Column 16)
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$$\text{Minimum fall (Column 16)} = 1.4 \text{ percent} \times 150 \text{ feet} / 100 + 0.38 = 2.48 \text{ feet}$$

*Note: The addition of the lateral fall value is only for the outlet at 4+50Rt.*

- Step B.7-** Subtract Column 16 from Column 17 (previous station/node) in **Figure 4** and enter results in Column 17 for the present station/node.
- Step B.8-** Calculate the Invert Slope and enter the results in Column 11 in **Figure 4** using the following relationship:

$$\begin{aligned} \text{Invert Slope} &= (\text{Fall, Column 16})(100)/(\text{Length, Column 15}) \\ &= 2.48 \times 100 / 150 = 1.653 \text{ percent} \end{aligned}$$

- Step B.9-** Using Figure 8 by drawing a line between the pipe size selected (Column 12) and slope (Column 11) in **Figure 4**, determine the pipe capacity flowing full and velocity flowing full and enter the results in Columns 13 and 14 for **station 4+50Rt**.

Be sure to select a pipe size which has capacity flowing full which is at least equal to the estimated design discharge in Column 10. The 12-inch pipe has capacity (4.55 cubic feet per second) which is greater than the design discharge (1.04 cubic feet per second). Also check to make sure the velocity is greater than or equal to 3.0 feet per second.

- Step B.10-** Calculate and enter in Column 7 the flow time through the **trunk line** using the following relationship.

$$T = L/V(60) \quad (\text{Chapter 7})$$

$$T = 150/(5.84)(60) = 0.4 \text{ minutes}$$

**Step B.11-** Add the flow time calculated above to the previous total time of concentration and enter the result in Column 8, **Figure 4**.

$$0.4 + 5.0 = 5.4 \text{ minutes}$$

**Step B.12-** Obtain the 10-year rainfall intensity corresponding to a duration of 5.4 minutes using graphs in **Chapter 7**. Enter 2.08 inches per hour in Column 9, (**Figure 4**).

**Step B.13-** Multiply Column 9 by Column 6 in **Figure 4** and enter 2.81 cubic feet per second in Column 10 of Figure 4.

**Step B.14-** Assume the **trunk line** is parallel to the profile grade of the highway and enter 1.4 percent in Column 11 from the outlet side of the inlet at station 4+50Rt to the inlet side of the manhole with inlet at station 10+00Rt.

**Step B.15-** Estimate and enter in Column 16 in **Figure 4** the fall from station 4+50Rt (out) to station 6+00Rt (in) as shown below:

$$\text{Fall} = (1.40)(150) / 100 = 2.10 \text{ feet}$$

**Step B.16-** Using Figure 8 by drawing a line between pipe size selected (Column 12) and slope (Column 11) in **Figure 4**, select a pipe size that has capacity flowing full which is at least equal to the estimated design discharge (2.81 cubic feet per second) in Column 10. The 12-inch pipe has capacity of 4.25 cubic feet per second which is greater than the design discharge of 2.81 cubic feet per second. Enter 4.25 and 5.40 in Columns 13 and 14, respectively. Also check the velocity to ensure a minimum of 3.0 feet per second.

**Step B.17-** Repeat steps 10 through 16 for the remainder of the system.

**Step B.18-** The available fall of 11 feet between station 10+00Rt and 10+00 (350 feet Rt) was approximated by subtracting elevation 80.0 from elevation 91.0. This pipe could, therefore, be laid on a maximum grade of approximately 2.5 percent. The natural bank elevation where the proposed manhole would be located is higher (95.5 feet) than the top of manhole with inlet at station 10+00Rt (94.7 feet). This pipe must have a minimum flowing full capacity of 13.84 cubic feet per second.

Using Figure 8, an 18-inch pipe with a slope of 1.7 percent would have a capacity of 13.84 cubic feet per second. Full flow velocity is 7.9 feet per second.

**Step B.19-** Enter the design information for the 18-inch pipe into Columns 11, 12, 13, 14, 16 and 17 of **Figure 4** for station 10+00 Rt.

**Step B.20-** Enter in Column 14 of **Figure 4** for station 10+00 (350 feet) Rt. an outlet velocity of 3.0 feet per second for the 90-foot outfall. This velocity will retain the

solid material in motion (see Section 7) and still prevent an erosion problem at the outlet of this outfall.

**Step B.21-** Determine the pipe size between station **10+00 (350 feet) Rt** to the outfall. Using Figure 8, determine the slope and pipe size necessary to convey the design discharge, 13.8 cubic feet per second, with a velocity of 3 feet per second. This requires a 30-inch pipe laid on a 0.13 percent slope. Enter the invert slope (0.13 percent), pipe size (30-inch) and capacity flowing full (14.6 cubic feet per second) into Columns 11, 12 and 13, respectively in **Figure 4**.

**Step B.22-** Determine and enter the Fall, Column 6 in **Figure 4**, for the outfall using the following relationship.

$$\text{Fall} = 0.13/100 (90 \text{ feet}) = 0.12 \text{ feet}$$

**Step B.23-** Add and enter the invert elevation, Column 17, **Figure 4**, for the inlet end of the outfall using the following relationship.

$$\begin{aligned} \text{Invert Elevation} &= 77.85 \text{ feet} + 0.12 \text{ feet} \\ &= 77.97 \text{ feet (invert elevation out at node 14)} \end{aligned}$$







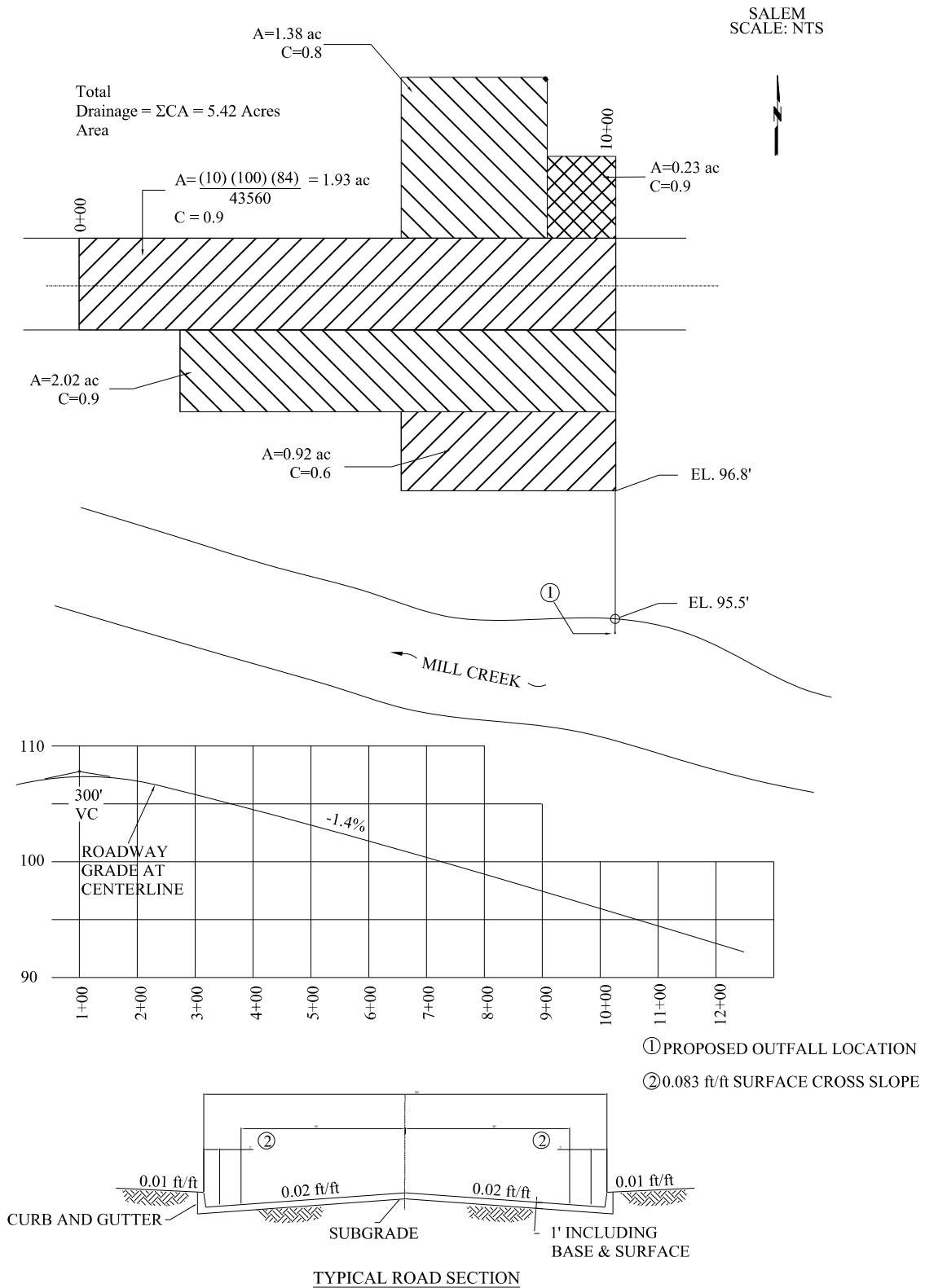
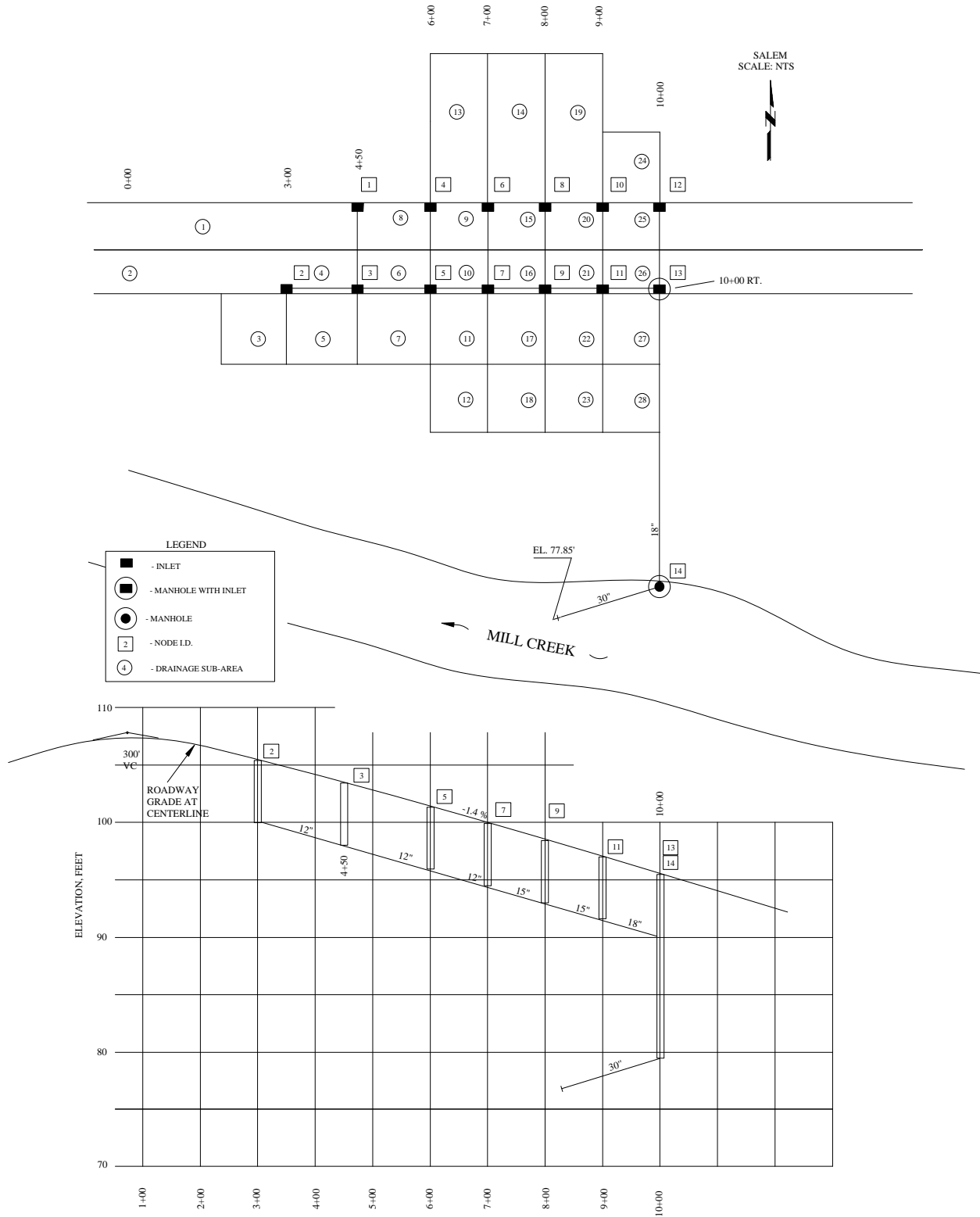


FIGURE #2  
DRAINAGE MAP AND ROADWAY GRADE INFORMATION



- (a) = 1 4 5
- (b) = 6 7 8
- (c) = 9 10 11 12 13
- (d) = 14 15 16 17 18
- (e) = 19 20 21 22 23
- (f) = 24 25 26 27 28

**FIGURE #4 (TRUNK)  
STORM DRAIN DESIGN SHEET  
FULL AND PARTIAL-FULL FLOW**

PROJECT: Example      DESIGN FREQUENCY: 10-Year; 50-Year\*      DESIGNED BY: JOE SMITH      DATE: 2005

Station	Node I.D.	Drainage Area		Runoff Coeff. C	Equiv. Area for 100% Runoff CA (3) x (4) Acres	Accumulative Impervious Drainage Area Σ CA Acres	Time of Concent. or Flow Time T <sub>c</sub> Minutes	Total Time of Concent. T <sub>c</sub> Minutes	Average Rainfall Intensity I in/hr	Design discharge Q (6) x (9) cfs	Invert Slope S %	Pipe Size D Inches	Capacity Flowing Full Q <sub>f</sub> cfs	Velocity Flowing Full V <sub>f</sub> fps	Length L feet	Fall feet	Invert Elevation feet	Top of Manhole or Inlet Elevation feet	Remarks
		Index No.	Area A Acres																
1a	1b	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
0+00		[2]	(2) (5) 0.54	0.90	0.49	0.49													
							5.0	5.0	2.12										
3+00 Rt		[3]	(a) 0.95	0.90	0.86	1.35				1.04							101.69	104.9	IE OUT AND TOP OF MH AT NODE 2
							0.4	5.4	2.08							2.48			
4+50 Rt		[5]	(b) 0.67	0.90	0.60	1.95				2.81							99.21	102.8	IE IN AND OUT AND TOP OF MH AT NODE 3
							0.5	5.9	2.00		1.40	12	4.25	5.40	150				
6+00 Rt		[7]	(c) 1.14	0.81	0.92	2.87				3.90							97.11	100.7	IE IN AND OUT AND TOP OF MH AT NODE 5
							0.3	6.2	1.98		1.40	12	4.25	5.40	100				
7+00 Rt		[9]	(d) 1.14	0.81	0.92	3.79				5.68							95.71	99.3	IE IN AT NODE 7
							0.3	6.5	1.92		1.40	15	7.70	6.30	100		95.46		IE OUT AT NODE 7
8+00 Rt		[11]	(e) 1.14	0.81	0.92	4.71				7.28							94.06	97.9	IE IN AND OUT AND TOP OF MH AT NODE 9
							0.3	6.8	1.90		1.40	15	7.70	6.30	100				
9+00 Rt		[13]	(f) 0.91	0.84	0.76	5.47				8.95							92.66	96.5	IE IN AT NODE 11
							0.2	7.0	2.53 *		1.40	18	12.50	7.30	100		92.41		IE OUT AT NODE 11
10+00 Rt										13.84							91.01		IE IN AT NODE 13
											1.70	18	13.84	7.90	350		88.97	95.1	IE OUT AT NODE 13
10+00 (350') Rt		[14]														5.95	83.02		IE IN AT NODE 14
											0.13	30	14.60	3.00	90		77.97	95.5	IE OUT AT NODE 14
Outfall																0.12			
																	77.85		

**FIGURE #5 (LATERALS)  
STORM DRAIN DESIGN SHEET  
FULL AND PARTIAL-FULL FLOW**

PROJECT: Example Problem      DESIGN FREQUENCY: 10-Year; 50-Year\*      DESIGNED BY: JOE SMITH      DATE: 2005

Station	Node I.D.	Drainage Area		Runoff Coeff. C	Equiv. Area for 100% Runoff CA (3) x (4) Acres	Accumulative Impervious Drainage Area CA Acres	Time of Concent. or Flow Time T <sub>c</sub> Minutes	Total Time of Concent. T <sub>c</sub> Minutes	Average Rainfall Intensity I in/hr	Design discharge Q (6) x (9) cfs	Invert Slope S %	Pipe Size D Inches	Capacity Flowing Full Q <sub>f</sub> cfs	Velocity Flowing Full V <sub>f</sub> fps	Length L feet	Fall feet	Invert Elevation feet	Top of Manhole or Inlet Elevation feet	Remarks
		Index No	Area A Acres																
1a	1b	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
0+00 Lt	[1]	①	0.43	.9	0.39	0.39													
4+50 Lt							5.0	5.0	2.12	0.83							99.59	102.8	IE OUT AND TOP OF MH AT NODE 1
4+50 Rt											0.45	12	2.40	3.00	84	0.38	99.21	102.8	IE IN AND TOP OF MH AT NODE 3
4+50 Lt	[4]	⑧	0.14	.9	0.13	0.13													
6+00 Lt							5.0	5.0	2.12	0.28							97.49	100.7	IE OUT AND TOP OF MH AT NODE 4
6+00 Rt											0.45	12	2.40	3.00	84	0.38	97.11	100.7	IE IN AND TOP OF MH AT NODE 5
6+00 Lt	[6]	⑬ ⑨	0.56	.82	0.46	0.46													
7+00 Lt							5.0	5.0	2.12	0.98							96.09	99.3	IE OUT AND TOP OF MH AT NODE 6
7+00 Rt											0.45	12	2.40	3.00	84	0.38	95.71	99.3	IE IN AND TOP OF MH AT NODE 7

FIGURE #6 (LATERALS)  
STORM DRAIN DESIGN SHEET  
FULL AND PARTIAL-FULL FLOW

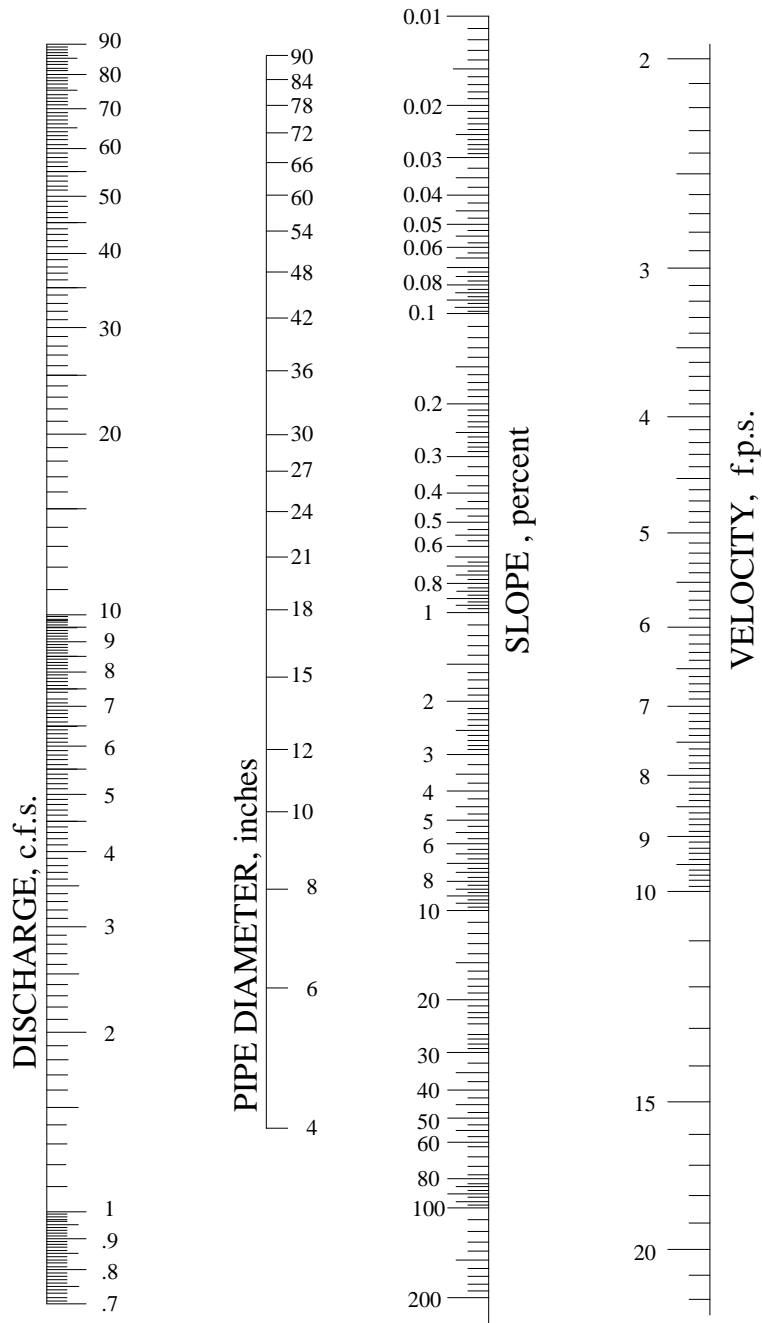
PROJECT: Example      DESIGN FREQUENCY: 10-Year; 50-Year\*      DESIGNED BY: JOE SMITH      DATE: 2005

Station	Node I.D.	Drainage Area		Runoff Coeff. C	Equiv. Area for 100% Runoff CA (3) x (4) Acres	Accumulative Impervious Drainage Area Σ CA Acres	Time of Concent. or Flow Time T <sub>c</sub> Minutes	Total Time of Concent. T <sub>c</sub> Minutes	Average Rainfall Intensity I in/hr	Design discharge Q (6) x (9) cfs	Invert Slope S %	Pipe Size D Inches	Capacity Flowing Full Q <sub>f</sub> cfs	Velocity Flowing Full V <sub>f</sub> fps	Length L feet	Fall feet	Invert Elevation feet	Top of Manhole or Inlet Elevation feet	Remarks
		Index No.	Area A Acres																
1a	1b	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
7+00 Lt	[8]	(14) (5)	0.56	.82	0.46	0.46												99.3	
8+00 Lt							5.0	5.0	2.12								94.69	97.9	IE OUT AND TOP OF MH AT NODE 8
8+00 Rt										0.98	0.45	12	2.40	3.00	84	0.38	94.31	97.9	IE IN AND TOP OF MH AT NODE 9
8+00 Lt	[10]	(19) (20)	0.56	.82	0.46	0.46												97.9	
9+00 Lt							5.0	5.0	2.12								93.29	96.5	IE OUT AND TOP OF MH AT NODE 10
9+00 Rt										0.98	0.45	12	2.40	3.00	84	0.38	92.91	96.5	IE IN AND TOP OF MH AT NODE 11
9+00 Lt	[12]	(24) (25)	0.33	.9	0.30	0.30													
10+00 Lt							5.0	5.0	2.12								91.89	95.1	IE OUT AND TOP OF MH AT NODE 12
10+00 Rt										0.64	0.45	12	2.40	3.00	84	0.38	91.51	95.1	IE IN AND TOP OF MH AT NODE 13





FIGURE #8



CAPACITY AND VELOCITY  
 NOMOGRAPH FOR  
 CIRCULAR CONCRETE PIPES  
 PIPE FLOWING FULL  
 $n = 0.013$