# CHAPTER 6: STORM DRAINS

**6. STORM DRAINS ........ 6-1**

**6.1 FDOT Storm Drain Tabulation Form ........ 6-1**

**6.2 Hydrology ........ 6-6**
6.2.1 Design Frequency ........ 6-6
6.2.1.1 Storms of Greater Magnitude ........ 6-6
6.2.2 Time of Concentration ........ 6-8
6.2.2.1 Peak Flow from Reduced Area ........ 6-9
6.2.2.2 Ignoring Time of Flow in Section ........ 6-13

**6.3 Inlets and Pavement Hydraulics ........ 6-14**
6.3.1 Inlets ........ 6-14
6.3.1.1 Apparent Locations ........ 6-14
6.3.1.2 Sags ........ 6-14
6.3.1.3 Continuous Grades ........ 6-15
6.3.1.4 Back of Sidewalk ........ 6-15
6.3.1.5 Inlet Capacity ........ 6-16
6.3.2 Pavement Hydraulics ........ 6-17
6.3.2.1 Gutter Grades ........ 6-17
6.3.2.2 Cross Slope ........ 6-20
6.3.2.3 Shoulder Gutter ........ 6-20
6.3.2.4 Determining the Spread ........ 6-22

**6.4 Pipe System Placement ........ 6-34**
6.4.1 Plan Layout ........ 6-34
6.4.1.1 Retaining Wall Drainage ........ 6-35
6.4.2 Profile Placement ........ 6-35
6.4.2.1 Slopes ........ 6-35
6.4.2.2 Minimum Pipe Depth ........ 6-37
6.4.3 Utility Coordination ........ 6-38
6.4.4 Pipe-to-Structure Connections ........ 6-38

**6.5 Pipe Hydraulics ........ 6-48**
6.5.1 Pressure Flow ........ 6-48
6.5.2 Partially Full Flow ........ 6-49
6.5.3 Lower-End Hydraulic Gradient ........ 6-49
6.5.3.1 Design Tailwater (DTW) ........ 6-52
6.5.4 Upper-End Hydraulic Gradient ........ 6-52
6.5.5 Flow Velocity in the Pipe Section ........ 6-54

6.5.6 Utility Conflict Box Losses ........ 6-56
6.5.7 Minor Losses ........ 6-57

**6.6 Procedure ........ 6-58**

# 6. STORM DRAINS

## 6.1 FDOT STORM DRAIN TABULATION FORM

The primary means of documenting storm drain design is the Department's storm drain tabulation form shown in Figure 6.1-1. On this form, record items identified by numbers in parentheses on Figure 6.1-1. These items are discussed in the description following the form. This information also is available on the FDOT Drainage web site.

For projects that utilize the 900 Series of the FDM, use alternative Flex tables to represent the same information that is shown below in Figure 6.1-1 and provided in the Drainage Manual, Section 3.13. For guidance to analyze storm sewer systems and develop the appropriate flex tables for the Drainage Report documentation is provided in FDOT CADD Office publication, FDOTConnect Drainage Design & 3D Modeling with Plans Development Training Guide. https://www.fdot.gov/cadd/main/FDOTCaddTraining.shtm

**FLORIDA DEPARTMENT OF TRANSPORTATION**
**STORM DRAIN TABULATION FORM**

**Financial Project Identification:** **County:** **Network:** **Prepared:** **Date:**
**Description:** **Organization:** **State Road:** **Checked:** **Date:**

| LOCATION OF UPPER END | | | STRUCTURE NO. | TYPE OF STRUCTURE | LENGTH (ft.) | DRAINAGE AREA (Acres) | | SUB-TOTAL (C*A) | TIME OF CONCENTRATION (min) | TIME OF FLOW IN SECTION (min) | INTENSITY (in/hr) | TOTAL (C*A) | BASE FLOW (cfs) * | TOTAL FLOW (cfs) | MINOR LOSSES (ft.) * | INLET ELEVATION (ft.) | HGL CLEARANCE (ft.) | HYDRAULIC GRADIENT | | | NUMBER OF BARRELS * | PIPE SIZE (in.) | SLOPE (%) | ACTUAL VELOCITY (fps) | FULL FLOW CAPACITY (cfs) * | NOTES AND REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | C= (1) ** | | | | | | | | | | | | CROWN | | | | RISE | HYD. GRAD. | | | ZONE: (38) |
| | | | | | | C= (1) ** | | | | | | | | | | | | FLOWLINE | | | | | | | | FREQUENCY (Yrs): (39) |
| ALIGNMENT NAME | | | UPPER | | | C= (1) ** | | | | | | | | | | | | | | | | | PHYSICAL | | | MANNING'S "n": (40) |
| STATION | DISTANCE (ft.) | SIDE | LOWER | | | INCREMENT | TOTAL | | | | | | | | | | | UPPER END ELEVATION (ft.) | LOWER END ELEVATION (ft.) | FALL (ft.) | | SPAN | MIN. PHYS. | PHYSICAL VELOCITY (fps) | | TAILWATER EL (ft): (41) |
| (2) | | | (6) | | | (9) | (10) | (11) | | | | | | | | | | (21) | (22) | (27) | | (30) | (32) | (35) | | |
| (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) | (23) | (24) | (28) | (29) | (31) | (33) | (36) | (37) | (42) |
| | | | | | | (9) | (10) | (11) | | | | | | | | | | (25) | (26) | | | | (34) | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |

* Denotes optional information.
** A composite runoff coefficient may be shown in lieu of individual C-values, provided the composite C calculations are included in the drainage documentation.

**Figure 6.1-1: Storm Drain Tabulation Form**

**Tabulation Form Description:**

1. Runoff Coefficients (C): You will be limited to three runoff coefficients. For most projects, this provides sufficient flexibility.

2. Alignment Name: The name of the alignment that the structure's station and offset references.

3. Station: The survey station number for the structure being used.

4. Distance (ft): The offset distance, in feet, from reference point of the structure to the reference station.

5. Side: The side, Right (Rt) or Left (Lt), of the reference station.

6. Structure Number: The structure number at the upper end is shown above the structure number at the lower end. Each major row (three minor rows) of the form identifies an inlet and the downstream pipe from that inlet.

7. Type of Structure: Usually shown with abbreviations such as Type P-3 or P-5 for inlets; Type C or E for ditch bottom inlets (DBI); Type P-8 or J-7 (MH) for manholes; and Type J-7 (Junct) for junction boxes.

8. Length (ft): The length, in feet, from the hydraulic center of the structure to the hydraulic center of the next downstream structure.

9. Increment: The incremental drainage areas, in acres, corresponding to the runoff coefficients being used. It is normally only the area that drains overland to an inlet, but it can include areas that drain to structures through existing pipes. If so, note it in the Remarks Column (42) or use the optional Base Flow Column.

   Manholes usually do not have incremental areas as they are handling areas already tabulated. If the incremental drainage area does not fit one of the three runoff coefficients selected, mathematically adjust the size of the area to fit one of the selected runoff coefficients. Note the adjustment in the Remarks Column (42).

   $Area_{ADJ} = (C_{ACT}/C_{SELECT}) \times Area_{ACT}$

10. Total: The total area, in acres, associated with each runoff coefficient and passing through the structure. Identify all the areas that drain to the structure through pipes from upstream structures. Add these "upstream areas" to the incremental drainage areas for the structure (9).

11. Sub-Total (C*A): The result of multiplying the total area associated with each runoff coefficient (10) by the corresponding runoff coefficient.

12. Time of Concentration (min): Usually, the time required for the runoff to travel from the most hydraulically remote point of the area drained to the point of the storm drain system under consideration. This time consists of overland flow, gutter flow, and flow time within the pipe system. Occasionally, this time is associated with a reduced area that creates a peak flow. If so, note it in the Remarks Column (42). Show this number in minutes.

13. Time of Flow in Section (min): The time, in minutes, it takes the runoff to pass through the section of pipe.

$T_{SECT}$=Hydraulic Length (8)/Actual Velocity (35)

14. Intensity (in/hr): Determined from NOAA Atlas 14 data. Intensity depends on the design frequency and the time of concentration. Intensity at the pipe segment for the given time of concentration (*T*) can be interpolated from NOAA's Atlas 14 tabular intensity (*i*) table via a linear interpolation and verification the interpolated intensity is within the 90% confidence limit bounding values. Show this number in inches per hour.

15. Total (C*A): The sum of the sub-total C*A values (11).

16. Base Flow (cfs): This is an optional column to account for known flows from underdrains, offsite pipe connections, etc. Show this number in cubic feet per second.

17. Total Flow (cfs): The product of the intensity (14) and the Total C*A (15) plus Base Flows (16). Show this number in cubic feet per second.

18 Minor Losses (ft): This is an optional column to account for minor losses according to Section 3.6.2 of the *Drainage Manual*. Show this number to one hundredth of a foot.

19. Inlet Elevation (ft): The elevation of the edge of pavement for curb inlets (Standard Plans, Indexes 425-020 through 425-025 and 425-061). The elevation of the theoretical grade point for barrier wall inlets of Standard Plans, Indexes 425-030 and 425-032. The grate elevation as shown in the Indexes for barrier wall inlet (Standard Plans, Index 425-031) and gutter inlets (Standard Plans, Indexes 425-040 and 425-041). The grate elevation for ditch bottom inlets (Standard Plans, Indexes 425-050 through 425-055). The elevation of the manhole cover for manholes. Show this number to one hundredth of a foot.

20. HGL Clearance (ft): This value is determined by calculating the difference between the Inlet Elevation (19) and the Upper End Hydraulic Gradient Elevation (21). Show this number to one hundredth of a foot.

21. Upper End Elevation (ft) (Hydraulic Gradient): The elevation of the hydraulic gradient at the upper end of the pipe section. The elevation, under design conditions, to which water will rise in the various inlets and manholes. Show this number to one hundredth of a foot.

22. Lower End Elevation (ft) (Hydraulic Gradient): The elevation of the hydraulic gradient at the lower end of the pipe section. Show to one hundredth of a foot.

23. Upper End Elevation (ft) (Crown): The inside crown elevation at the upper end of the pipe section. Show this number to one hundredth of a foot.

24. Lower End Elevation (ft) (Crown): The inside crown elevation at the lower end of the pipe section. Show to one hundredth of a foot.

25. Upper End Elevation (ft) (Flowline): The flowline at the upper end of the pipe section. Show this number to one hundredth of a foot.

26. Lower End Elevation (ft) (Flowline): The flowline at the lower end of the pipe section. Show this number to one hundredth of a foot.

27. Fall (ft) (Hydraulic Gradient): The elevation change of the hydraulic grade line from the upper end to lower end of the pipe section. Show this number to one hundredth of a foot.

28. Fall (ft) (Crown/Flowline): The physical fall of the pipe section. Show this number to one hundredth of a foot.

29. Number of Barrels: This optional column should be used for systems with pipe segments that have multiple barrels.

30. Pipe Size (Rise) (in): The vertical distance between the Flowline (25) and the Crown (23) in inches.

31. Pipe Size (Span) (in): The horizontal distance of the inside of a pipe at its widest point in inches.

32. Slope (%) (Hydraulic Gradient): For pipes under pressure flow, this is the full-flow friction slope. For pipes flowing partially full, this is: [Upper End HG (21) – Lower End HG (22)] /Hydraulic Length (8). Show this number to one hundredth of a percent.

33. Slope (%) (Physical): Determined from Physical Fall (28)/Hydraulic Length (8). Show this number to one hundredth of a percent.

34. Slope (%) (Minimum Physical): The flattest physical slope to maintain a velocity of 2.5 FPS flowing full, obtained from rearranging Manning's equation:

    $S_{MIN} = [Vn / (1.49R^{2/3})]^2$

    Show this number to one hundredth of a percent.

35. Actual Velocity (fps): Determined by Total Flow (17) divided by the average cross-sectional flow area. See discussion in Section 6.5. Show this number to a minimum of one tenth of a foot per second.

36. Physical Velocity (fps): The velocity produced when the pipe is flowing full based on the Physical Slope (33). Show this number to a minimum of one tenth of a foot per second.

    $V = (1.49/n)R^{2/3}S_{PHYSICAL}^{1/2}$

37. Full-Flow Capacity (cfs): This optional column is the product of the Physical Velocity (36) and the cross-sectional area of the pipe. Show this number in cubic feet per second.

38. Frequency (Yrs): The Storm Drain Design Frequency according to Section 3.3 of the *Drainage Manual*.

39. Manning's "n": For storm drains, this value should be 0.012 according to Section 3.6.4 of the *Drainage Manual*. Document any other Manning's "n" values used in the Remarks Column (42).

40. Tailwater EL (ft): The water elevation coincident with the outlet pipe and established by Section 3.4 of the *Drainage Manual*. Some districts may have more stringent criteria.

41. Remarks: Include such things as: area adjustments, partial flow depths, existing pipe connections, or anything unusual.

## 6.2 HYDROLOGY

The rational method is used for pipe sizing, inlet capacity, and spread calculations.

$$Q = C\ i\ A$$

where:

Q = Runoff, in cubic feet per second (cfs)
C = Runoff Coefficient (see Table B-4, Appendix B)
i = Rainfall intensity, in inches per hour (NOAA Atlas 14)
A = Area, in acres

### 6.2.1 Design Frequency

The *Drainage Manual* states the design frequency for storm drains. For the Department's facilities, the frequencies range from NOAA Atlas 14 five-year to 50-year, with five-year as the most common design frequency. These frequencies apply to pipe hydraulics, not inlet capacity or spread within the roadway. Section 6.3, below, discusses the criteria for inlet capacity and spread. If a storm drain system includes both curb inlets and ditch bottom inlets, check the ditch bottom inlets for a 10-year design frequency and all structures in the mixed system should meet the five-year design frequency.

#### 6.2.1.1 Storms of Greater Magnitude

You should always consider the intent of the Department's criteria regarding the flooding of properties upstream or downstream of the Department's right of way. In several chapters of the Drainage Manual, it says that any increases over pre-development stages must not significantly change land use values. So, you should consider the impacts of storm events that are more severe than the standard design frequency of the storm drain. Initially, this should be a qualitative evaluation. Realize there are several reasons why urban typical sections with storm drains can handle storms of greater magnitude.

The first reason is conservatism within the storm drain design procedure. The flow rate calculated for each pipe section is the peak flow rate. This is conservative because we calculate the hydraulic gradient assuming that peak flow rates exist in all of the pipe sections simultaneously. In reality, when one pipe section is at peak flow, usually one or more of the other pipe sections have flow rates less than peak. This is most evident when considering the differences between the upper and lower parts of a long system. For example, consider a system where the outlet pipe's flow is calculated based on a Time of Concentration of 35 minutes. The flow rates of the first several pipes were based on Times of Concentration of 10 minutes to 15 minutes. If a 35-minute storm and its associated intensity is applied to the entire system, the flow rates in the first several pipes would be less than the flow rate we calculated based on Times of Concentration = 10 minutes to 15 minutes. Therefore, the friction losses in these pipes actually are less than we calculated. Conversely, during short, intense storms, the upper pipes could reach their design flow rates, but the downstream portion of the system does not have the entire area contributing, so downstream pipes do not see the design flows. This conservatism exists to some degree throughout all pipe systems, but has a minimal effect on short systems where the differences in Times of Concentration are small.

Another reason an urban typical section can handle storms of greater magnitude is that the roadway itself can convey substantial flow. A standard pavement section of 0.02 ft/ft cross slope on a 0.3-percent longitudinal grade can convey approximately 7 cfs[1] with the depth of the flow at the top of the curb.

The last reason, although less significant, is that when the flow in the road reaches the height of the curb, there is more pressure on the piping system, thus forcing more flow through the pipes.

Considering these reasons, look at the system to see if there are any places where the water elevations or discharge rates could be increased.

- Are there sags in the profile? If so, could the pond water leave the right of way at these locations? Would water have gone that direction in the pre-developed condition?

- Is the roadway blocking overland flow in any areas? If so, would the blocked water substantially change land use values?

- Where back-of-sidewalk inlets are used, should check valves or flap gates be used to prevent the water in the pipes from backing off of the right of way?

- Would the inlets at the ends of the system bypass flow during a more severe storm event? If so, would water have gone that direction in the pre-developed condition?

If you have concerns after considering these factors, it may be appropriate to do a more detailed evaluation. Perhaps check the operation of the storm drain system with higher frequency (less frequent) storm. Perhaps the storage in the road and the pipes could be modeled. You may need a more detailed model of the pre-developed conditions.

If, after evaluating these situations, it is evident there would be increased discharge or

1 $Q = (0.56/n) \cdot S_x^{1.67} \cdot S^{0.5} \cdot T^{8/3} = (0.56/0.016) \cdot 0.02^{1.67} \cdot 0.003^{0.5} \cdot 18.75^{8/3} \approx 7$ cfs where T = curb height / cross slope = (4.5/12)/0.02 = 18.75’

increases over pre-developed stages that would significantly change land use values, document this conclusion. Then change the storm drain design as necessary to bring the stages down or to reduce the discharge. This is not saying use a higher design frequency for the storm drain system. Instead, use larger pipes where necessary. Increasing pipe size to prevent the adverse impacts to adjacent properties is different than using a higher design frequency and maintaining the standard hydraulic gradient clearance.

## 6.2.2 Time of Concentration

The Time of Concentration ($t_c$) is the time required for the runoff to travel from the most remote point in the drainage basin to the point of the storm drain system under consideration. This will be the longer of: (a) the overland flow time to the inlet, or (b) the sum of the $t_c$ to the inlet immediately upstream in the piping system plus the time of flow through the upstream pipe section. For inlets that have more than one upstream pipe, you will need to compare the $t_c$ and Time of Flow through Section of all the upstream inlets and pipes with the overland travel time to the subject inlet. Use the longest of these as the $t_c$. See Figure 6.2-1. For pipe segments that do not have upstream pipes, the $t_c$ will be simply the overland flow time.

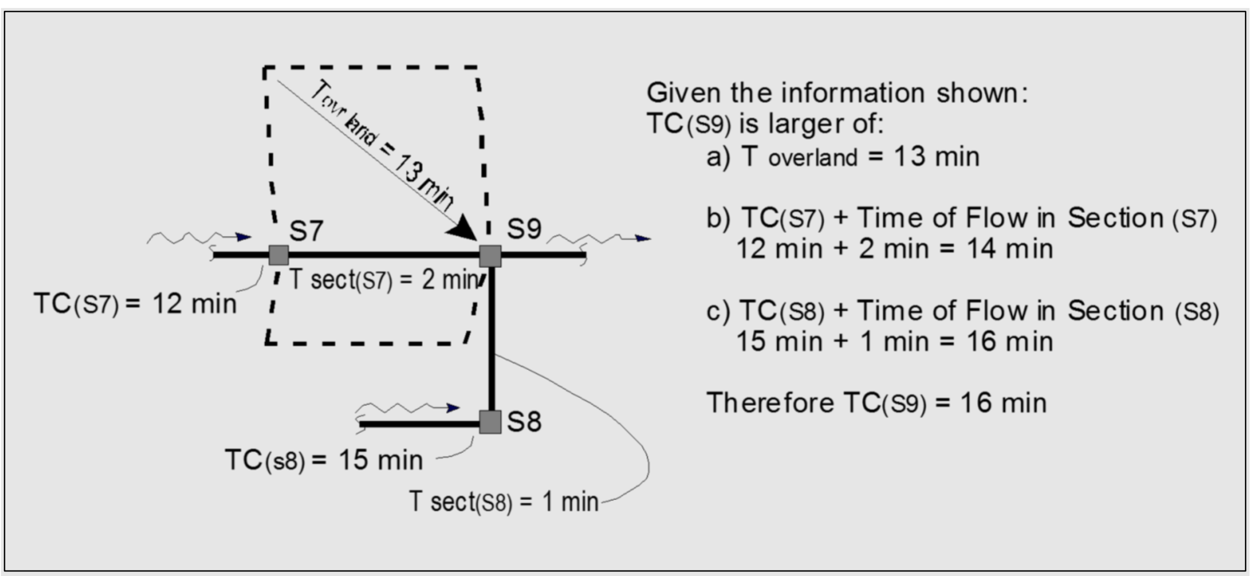



**Figure 6.2-1: Determining Time of Concentration**

### 6.2.2.1 Peak Flow from Reduced Area

Check to see if a portion of the drainage area will produce a larger flow rate than the entire area. This could occur where a larger portion of the drainage area exists toward the bottom or outlet, as in Figure 6.2-2. This is even more likely if the land cover of the area toward the outlet is more impervious than the upstream area. Mathematically, you will observe this where the reduction in area is more than offset by an increased intensity and possibly an increased runoff coefficient.

The Department encourages that this check be made at apparent junctions or inlets in a storm drain system. It is acceptable but not necessary to check every pipe section for peak flow from reduced area. Some computer programs may do this automatically.

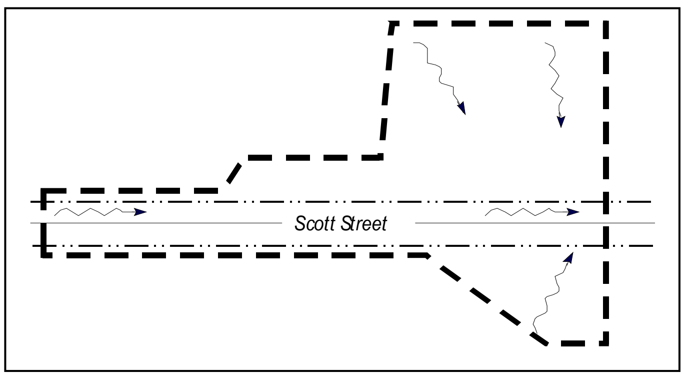



**Figure 6.2-2: Drainage Basin with Larger Portion of Area Near Outlet**

**Example 6.2-1—Peak Flow from a Reduced Area**

Given:

- The partial storm drain system shown in Figure 6.2-3
- The design standard is a 10-year return period

Find:

- The design flow rate for pipe section $P_{31-32}$.

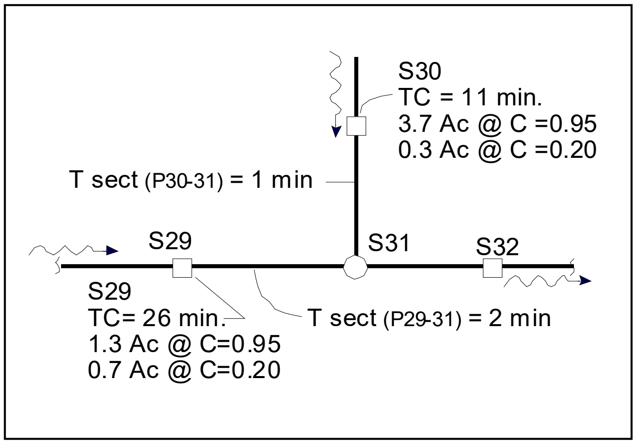



**Figure 6.2-3: Example 6.2.1**

First, calculate the flow rate using the total drainage area (maximum $t_c$).

1. Add the product of C*A for the upstream areas. Note, S31 is a manhole, therefore, there is no additional surface drainage area to this structure and flow is contributed from upstream pipes only.

   Total $C*A_{S-29}$ = 0.95 x 1.3 ac + 0.2 x 0.70 ac = 1.38
   Total $C*A_{S-30}$ = 0.95 x 3.7 ac + 0.2 x 0.30 ac = <u>3.58</u>
   Total $C*A_{S-31}$ = 4.96

2. Determine the time of concentration.

   The $t_c$ is the time it takes for the entire drainage area to contribute. It is the longer of:
   $(t_c)_{S-29}$ + Time of Flow in Section $P_{29-31}$ = 26 + 2 = 28 min
   $(t_c)_{S-30}$ + Time of Flow in Section $P_{30-31}$ = 11 + 1 = 12 min
   Therefore, $(t_c)_{S-31}$ = 28 min.

3. Determine the design rainfall intensity from the 10 year storm frequency having a duration equal to the time of concentration.

   NOAA Atlas 14 data for the site has tabulated intensities for a 15-minute duration and 30-minute duration. The intensity for 28-minute time of concentration can be estimated from NOAA’s IDF curve (under PF Graphical tab) or can be interpolated from NOAA’s IDF Precipitation Frequency (PF) tabulation table.

$$i_x = \left(\frac{T_x - T_1}{T_2 - T_1}\right) * (i_2 - i_1) + i_1$$

For the purposes of this example, assume NOAA’s 15-minute intensity is 4.32 in/hr and 30-minute intensity is 3.97 in/hr.

$$i_x = \left(\frac{28 - 15}{30 - 15}\right) * (3.97 - 4.32) + 4.32$$
$$i_x = 4.0\ in/hr$$

4. Determine the flow.

   Q = (C*A) x i = 4.96 x 4.0 = 19.8 cfs

   Now, check for a larger flow from part of the drainage area (peak flow from reduced area).

5. Determine the intensity associated with the shorter $t_c$.

   The shorter system time is from S-30 and is 11 + 1 min = 12 min.
   For the purposes of this example, assume the intensity from NOAA’s webpage referenced above for 12 min = 6.0 in/hr.

6. Estimate the area that will contribute from S-29 during a 12-minute storm.

   One approach is to reduce the area from the pipes having long times of concentration by the ratio of the times of concentrations. Ratio = (Short $t_c$)/($t_c$ of the associated pipe).

   $A_{S\text{-}29}$ is reduced by 12 min/28 min = 0.43
   $A_{S\text{-}29\ REDUCED}$ @ C = 0.95 = 1.3 ac x 0.43 = 0.56 ac
   $A_{S\text{-}29\ REDUCED}$ @ C = 0.20 = 0.7 ac x 0.43 = 0.30 ac

7. Add the areas that will contribute to S-31 during a 12-minute storm.

   Area $_{TOTAL}$ = Area$_{S\text{-}29\ REDUCED}$ + Area$_{S\text{-}30}$
   @ 0.95 = 0.56 + 3.7 = 4.26
   @ 0.20 = 0.30 + 0.3 = 0.60

8. Add the product of C*A contributing to S-31 during a 12-minute storm.

Total C*A = 0.95 x 4.26 + 0.2 x 0.6
= 4.05 + 0.12 = 4.17

9. Determine the flow from the reduced area.

$Q_{\text{Reduced Area}}$ = (C x A) x $i_{12\ min}$
= 4.17 x 6.0 = 25.0 cfs

For pipe sections downstream of $P_{31\text{-}32}$, add the incremental drainage areas to the reduced areas recorded for $P_{31\text{-}32}$. Then, add the time of flow in downstream sections to the reduced time of concentration for $P_{31\text{-}32}$.

Table 6.2-1 shows a way of presenting these approaches on the Tabulation form.

**Table 6.2-1: Data from Example 6.2.1**

| STRUCTURE NUMBER (UPPER / LOWER) | TYPE OF STRUCTURE | DRAINAGE AREA (ac.) INCREMENT | DRAINAGE AREA (ac.) TOTAL | SUB-TOTAL C•A | TIME OF CONCENTRATION (MINUTES) | TIME OF FLOW IN SECTION (MINUTES) | INTENSITY (IN/HR ) | C•A—TOTAL | TOTAL FLOW (cfs ) | NOTES AND REMARKS | Example Approach |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | C = 0.95 | | | | | | | | | |
| UPPER | | C = | | | | | | | | Frequency (Yrs): | |
| | | C = 0.2 | | | | | | | | Manning's "n": | |
| LOWER | | INCREMENT | TOTAL | | | | | | | Tailwater EL (ft): | |
| | | 0.2 | 1.3 | 1.24 | | | | | | | Total Area – Max $t_c$ |
| 29 | J1 | | | | 26 | 2 | | 1.38 | | | |
| 31 | | 0 | 0.7 | 0.14 | | | | | | | |
| | | n/a | 3.7 | 3.52 | | | | | | | |
| 30 | J1 | | | | 11 | 1 | | 3.58 | | | |
| 31 | | n/a | 0.3 | 0.06 | | | | | | | |
| | | | 5.0 | 4.75 | | | | | | | |
| 31 | MH | | | | 28 | - | 4.0 | 4.95 | 19.8 | | |
| 32 | | | 1.0 | 0.20 | | | | | | | |
| | | 0.2 | 1.3 | 1.24 | | | | | | | Peak Flow from Reduced Area |
| 29 | J1 | | | | 26 | 2 | | 1.38 | | | |
| 31 | | 0 | 0.7 | 0.14 | | | | | | | |
| | | n/a | 3.7 | 3.52 | | | | | | | |
| 30 | J1 | | | | 11 | 1 | | 3.58 | | | |
| 31 | | n/a | 0.3 | 0.06 | | | | | | | |
| | | 0.2 | 4.26 | 4.05 | | | | | | Area from S-29 reduced by 12/28. Intensity based on System Time from S-30 | |
| 31 | MH | | 0.0 | | 12 | - | 6.0 | 4.17 | 25.0 | | |
| 32 | | | 0.6 | 0.12 | | | | | | | |

## 6.2.2.2 Ignoring Time of Flow in Section

For systems where the pipes are full without a storm event because of normal tailwater conditions, the time of flow in the pipe section is meaningless. For the runoff to get into the pipe, the water that is in the pipe has to move out. Since water under the pressures we are dealing with is essentially incompressible, what goes in the inlet must be coming out the outlet at the same time. In these situations, it is realistic to ignore the travel time through pipes submerged by normal tailwater. Note that you should use normal tailwater (perhaps the control elevation of a wet pond), not the design tailwater, to determine if a pipe segment is submerged.

The Department realizes that current design software does not use the approach of ignoring time of flow in section. As such, some districts may not require that time of flow be ignored through submerged pipes.

# 6.3 INLETS AND PAVEMENT HYDRAULICS

## 6.3.1 Inlets

Factors controlling the selection of an inlet type include hydraulics, utility conflicts, right-of-way limits, bicycle and pedestrian safety, etc. Guidelines for selecting inlets are located in Section 3.7.4 of the *Drainage Manual*.

### 6.3.1.1 Apparent Locations

- **At low points in the gutter.** Double-throated inlets—such as Type 2, 4, and 6—are symmetrical about the centerline and are intended to accept flow from both sides. Use these where the minor gutter flow exceeds 50 feet in length or 0.5 cubic feet per second.

- **Upstream of pedestrian cross walks.**

- **Upstream of curb returns**.

- **10 feet outside the flat cross sections in super elevation transitions.** Although the flow may be small, the cross slope is nearly flat so the spread potential is high.

- **Outside of driveway turnouts.** If the adjacent property is under development or redevelopment, try to obtain the site plans to identify future driveway locations.

### 6.3.1.2 Sags

Normally, one inlet at the low point in combination with inlets on each of the approaching grades is sufficient to meet spread criteria.

Use flanking inlets for sags that have no outlet other than the storm drain system—for instance, underpasses, barrier wall sections, or depressed sections where the roadway is much lower than the surrounding ground. Flanking inlets are those placed on one or both sides of and fairly close to the sag inlet. They provide backup capacity for the sag inlet if it becomes clogged. The flanking inlets must be located to satisfy spread criteria when the sag inlet is blocked. Figure 6.3-1 shows a representation of this location pattern. Figure 6.3-9 provides vertical curve formulae to help determine the flanking inlet locations.

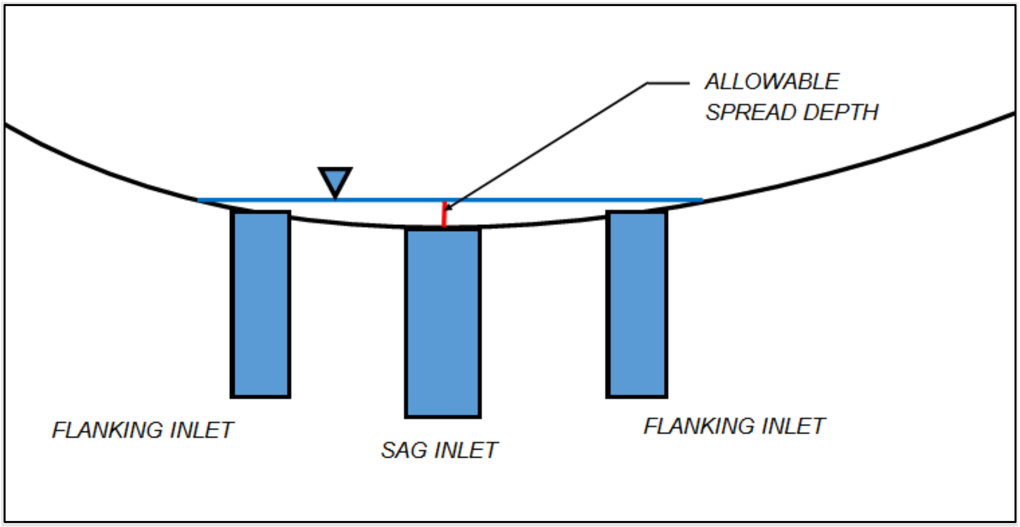



**Figure 6.3-1: Inlet Location Satisfying the Sag Requirements**

#### 6.3.1.3 Continuous Grades

When deciding about the initial placement of curb and gutter inlets on a continuous grade, base placement on the 300-foot maximum spacing for an 18-inch pipe (Drainage Manual, Section 3.10.1). After the initial placement of inlets, check the spread and add or move inlets as necessary to meet the spread standards.

The piping system layout may affect the locations of curb and gutter inlets. As you lay out the piping system, you may need a manhole to redirect the flow, to provide maintenance access, or merely to connect stub pipes. If you use an inlet rather than a manhole, you get the benefit of an additional hydraulic opening for little or no additional cost. Section 6.4, below, discusses piping system layout.

#### 6.3.1.4 Back of Sidewalk

The Field Review is Critical to Designing Back-of-Sidewalk Drainage

Locate back-of-sidewalk inlets where concentrated flows drain toward the road and where the proposed sidewalk would block overland flow. Often, you can identify these areas from the survey, the back-of-sidewalk profiles, and the proposed cross sections. <u>Do not rely on these alone!</u> Walk the entire project area looking for places where concentrated runoff flows to the road and for localized depressed areas that were not identified in the survey. Development may have changed the existing ground line since the time the survey was done. Your field review with the back-of-sidewalk profiles and proposed cross sections will identify areas where you need back-of-sidewalk inlets.

In instances where you may need numerous back-of-sidewalk inlets, check with the roadway designer about modifying the roadway profile grade to better accommodate overland flow.

Standard Plans, Index 425-060 contains the standard back-of-sidewalk drainage inlets. Use yard drains and the double four-inch pipes under the sidewalk to correct small existing flooding problems. For any other back-of-sidewalk drainage, obtain right of way as necessary to construct a ditch bottom inlet or other substantial back-of-sidewalk drainage conveyance.

Where the Department's storm drain system connects to back-of-sidewalk inlets, check the hydraulic grade line elevation at these inlets to see if water would back up or cause the system to create adverse impacts to adjacent properties. If so, first consider increasing the size of some downstream pipe sections. If avoiding adverse impacts by increasing pipe sizes is not feasible, consider using check valves or flap gates in the pipe connected to the back-of-sidewalk inlet (see Figure 6.3-2). Flap gates and check valves are not ideal because they require maintenance; nevertheless, they may be the most practical option for some situations.

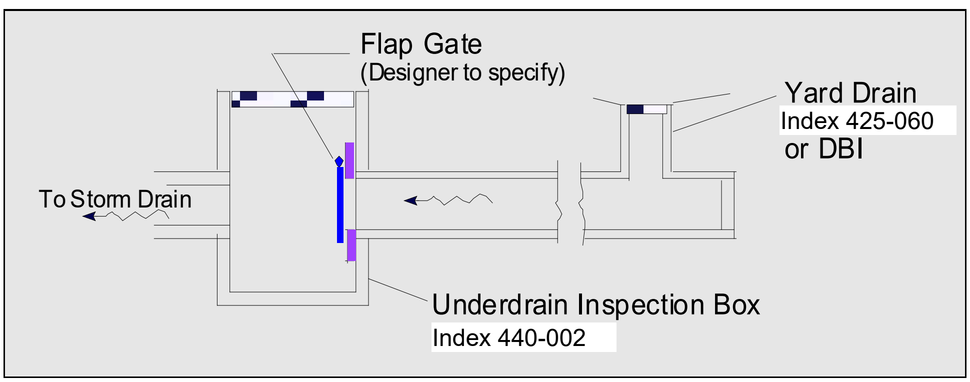



**Figure 6.3-2: Flap Gate Connected to the Back-of-Sidewalk Inlet**

## 6.3.1.5 Inlet Capacity

Capacity data for most of the Department's inlets were developed by laboratory studies done at the University of South Florida (Anderson, 1972). A graphical presentation of these data is given in Appendix I. Separate curb inlet capacity charts are presented for various cross slopes. You also can use methods described in FHWA's *Hydraulic Engineering Circular* HEC 12 or HEC 22 to evaluate the interception capacity of the Department's inlets.

## 6.3.2 Pavement Hydraulics

The Department uses driver visibility as a basis for the spread standards. There is a rainfall intensity that reduces the driver's sight distance to less than the minimum stopping sight distance. Removing the water from the road for intensities greater than this serves no purpose. If a driver's sight distance is less than the minimum stopping sight distance when the driver sees an object, the driver cannot stop in time regardless of how much water is on the road. So removing the water from the road for intensities greater than this is over-design from a vehicle standpoint.

The Department uses four inches per hour (in/hr) as the intensity that reduces the driver's sight distance to less than the minimum stopping sight distance. This is based on information summarized in FHWA HEC 21.

Use the integrated form of Manning's equation to calculate spread in gutters.

$$Q = \frac{0.56}{n} S_x^{5/3} S_L^{1/2} T^{8/3}$$

where:

Q = Gutter flow rate, in cubic feet per second (cfs)
n = Manning's roughness coefficient (see Table B-2, Appendix B)
$S_x$ = Pavement Cross Slope, in feet per feet (ft/ft)
$S_L$ = Longitudinal Slope, in feet per feet (ft/ft)
T = Spread, in feet (ft)

Figure 6.3-8 provides a nomograph for solving this equation, which is intended for use with triangular gutter sections. The standard Type F curb forms a composite section when combined with the pavement cross slope. In most cases, it is reasonable to ignore the gutter depression and treat the flow section as a simple gutter formed by the cross slope of the road and the curb. Ignoring the gutter depression is conservative[2], but allows for debris buildup in the gutter. If you need to determine the additional capacity of the gutter depression, use Figure 6.3-10 or the procedures provided in FHWA's HEC 12 or HEC 22.

### 6.3.2.1 Gutter Grades

Standard gutter grades should not be less than 0.3 percent. Some District Drainage Engineers will approve a 0.2-percent gutter grade in very flat terrain. Use of a saw tooth

2 The gutter depression can add approximately 31% to the conveyance of the flow section in cases where the pavement cross slope is 0.02 and the spread width is 7.5 feet.

profile can maintain minimum grades in very flat terrain.

To provide adequate drainage in sag vertical curves, maintain a minimum gutter grade of 0.3 percent down to the inlet at the low point. Without this minimum grade, the flat longitudinal grade near the low point would cause the spread to be greater than allowable. Maintaining the minimum gutter grade up to the inlet increases the cross slope at the low point, thus providing additional drainage. To maintain the minimum gutter grade, develop and show special gutter grades in the plans.

**Example 6.3-1—Special Gutter Grade**

Given:

- The sag vertical curve described in the figure below.

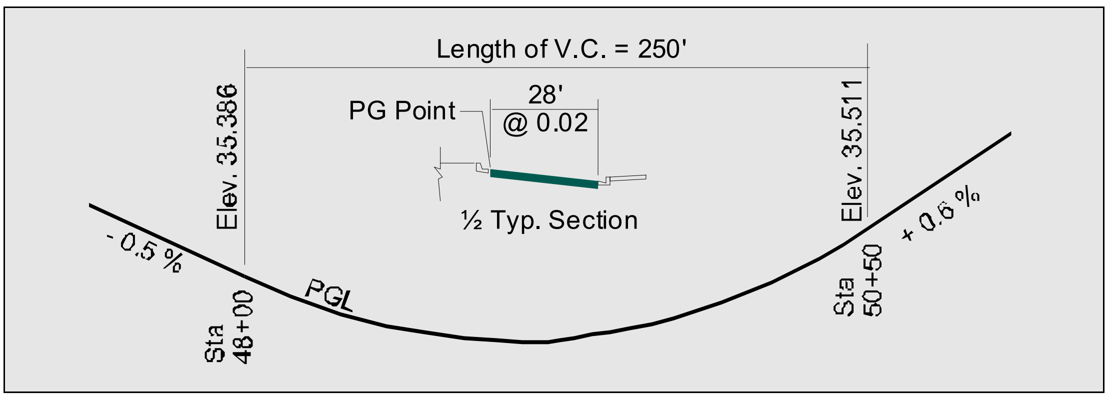



**Figure 6.3-3: Example 6.3-1 Given Information**

Find:

- The limits of the special gutter grade
- The theoretical gutter elevation at the low point
- The cross slope at the low point

1. Determine the rate of change of longitudinal slope. (Formula from Fig. 6.3-9)

   Rate of change = $r = (g_2 - g_1)/L = 0.6 - (-0.5)/2.5 = 0.44$

2. Find the location of the low point and the location where the longitudinal slope on the curve is -0.3 percent and +0.3 percent. Use the equation for longitudinal slope at any point and rearrange to solve for X.

   $X_{-0.3\%} = (S_L - g_1)/r$
   $= [-0.3 - (-0.5)]/0.44 = 0.4545$ stations

\ Station = *48+00* + *0+45.45* = 48+45.45

$X_{+0.3\%}$ = $(S_L – g_1)/r$
= [0.3 – (-0.5)]/0.44 = 1.8182 stations
∴ Station = *48+00* + 1+*81.82* = 49+81.82

Using the equation for the station of the turning point:

$X_{LOW\ POINT}$ = $(g_1 \times L)/(g_1 – g_2)$
= (-0.5 x 2.5)/(-0.5 – 0.6)
= 1.1364 stations
∴ Station = *48+00* + 1+*13.64* = 49+13.64

So, a special gutter grade of -0.3 percent is needed from Sta. 48+45.45 to Sta. 49+13.64 and a special gutter grade of +0.3 percent is needed from Sta. 49+13.64 to Sta. 49+81.82.

3. Find the elevation of the profile grade line at Sta. 48+45.45 and Sta. 49+81.82. Both are equal distance from the center, so we only need to find one elevation.

$Elev_{48+45.45}$ = $Elev_{48+00} + g_1 X + ½ r X^2$
= 35.386 ft. + (-0.5)(0.4545) + ½ (0.44)$(0.4545)^2$
= 35.204 ft.

4. Find the elevation at the gutter at Sta. 48+45.45. (This equals the elevation of the gutter at Sta. 49+81.82.)

The edge of pavement is 0.56 ft. (28 ft. x 0.02) lower than profile grade line and the gutter is 1.5 inches (0.125 ft.) below the edge of pavement, so:

$Elev_{GUTTER}$ = Elev $PGL_{Sta.\ 48+45.45}$ – 0.56 – 0.125 = 35.204 ft. – 0.56 ft. – 0.125 ft. = 34.519'

5. Find the theoretical gutter elevation at the low point.

Elev = $Elev_{Sta.\ 48+45.45}$ – (special gutter grade x length of special gutter)
Elev = 34.519 ft. – [0.3 x (49.1364 – 48.4545)] = 34.314 ft.

This elevation would be used to check the hydraulic grade line clearance below the sag inlet.

6. Find the cross slope at the low point.

The elevation of the profile grade line at the low point is:

PGL $Elev_{49+13.64}$ = $Elev_{48+00} + g_1 X + ½ r X^2$

$= 35.386 \text{ ft.} + (-0.5)(1.1364) + \frac{1}{2}(0.44)(1.1364)^2$
$= 35.102 \text{ ft.}$

The elevation at the edge of pavement at the low point is:
EOP $Elev_{49+13.64}$ = $Elev_{GUTTER}$ + 1.5 inches
= 34.314 ft. + 0.125 ft. = 34.439 ft.

Cross Slope = (35.102 ft. – 34.439 ft.)/28 ft. = 0.024 ft./ft.

This would be used to check the spread of the inlet at the low. Interpolate between the values in Appendix I, Figures I-17 through I-19, where the cross slope value is between the values of the figures.

### 6.3.2.2 Cross Slope

FDM 210 and 211 gives the standard cross slopes.

### 6.3.2.3 Shoulder Gutter

Use shoulder gutter on fill slopes and at bridge ends to protect the slopes from erosion caused by water from the roadway and bridge. Use shoulder gutter in accordance with Section 3.7.3 of the *Drainage Manual*. When placed at bridge ends, the gutter should be long enough to construct the gutter transitions shown on Standard Plans, Indexes 536-001 and 425-040. The terminal shoulder gutter inlet should intercept all of the flow coming to it for a 10-year storm.

The *Drainage Manual* gives two spread criteria for sections with shoulder gutter. One is related to driver visibility (rainfall intensity of four inches per hour) and the other is related to erosion protection of the fill slope (10-year design storm). Both criteria need to be met. Consider the potential for future additional lanes in the median when determining the flow rates in shoulder gutter.

In a typical situation where standard cross slopes and shoulder widths exist, the criterion for protecting the fill slope has a higher intensity and less allowable spread than the criterion for driver safety. Thus, the criterion for protecting the fill slope will set the inlet spacing.

Given the typical situation where both the shoulder and the miscellaneous asphalt behind the gutter slope upward at 0.06 ft/ft from the gutter, the spread across the gutter and pavement section should not exceed six feet for the 10-year storm. This section has a conveyance of approximately 28 cubic feet per second [$K = Q/S_L^{1/2}$ = 28 cfs]. You can use the conveyance to determine maximum allowable flow rates for various longitudinal slopes. Another approach is to treat the shoulder gutter and pavement section as a triangular gutter with a cross slope of 0.05 ft/ft, designing for 10-year flows, and limiting the spread to six feet (see Figure 6.3-4).

1. The maximum shoulder gutter design conveyance should be K = 28 cfs adjacent to guardrail, and K = 15 cfs with no guardrail for the 10-year storm. K = 28 cfs is derived from the flow area being limited to 15 inches outside the shoulder gutter and n = 0.016. K = 15 cfs is derived from limiting the flow area to the shoulder gutter section.

2. The maximum shoulder gutter design conveyance approaching a terminal gutter inlet should be K = 15 cfs to intercept 100 percent of the design storm flow.

3. Consider placing two gutter inlets at the down gradient shoulder gutter terminus to provide 100-percent interception, unless 100-percent interception by one inlet (K = 15 cfs) is demonstrated by appropriate calculation.

4. Inlet spacing must meet spread criteria (*Drainage Manual*, Section 3.9), maximum pipe length criteria (*Drainage Manual*, Section 3.10.1), and 10-year frequency gutter capacity criteria. In most cases, the 10-year frequency storm may govern inlet spacing.

5. Where applicable, design inlet spacing to accommodate the additional runoff from future widening.

6. Place gutter inlet(s) at the down gradient end of all shoulder gutter, in lieu of concrete spillways or flumes, to reduce the potential for erosion.

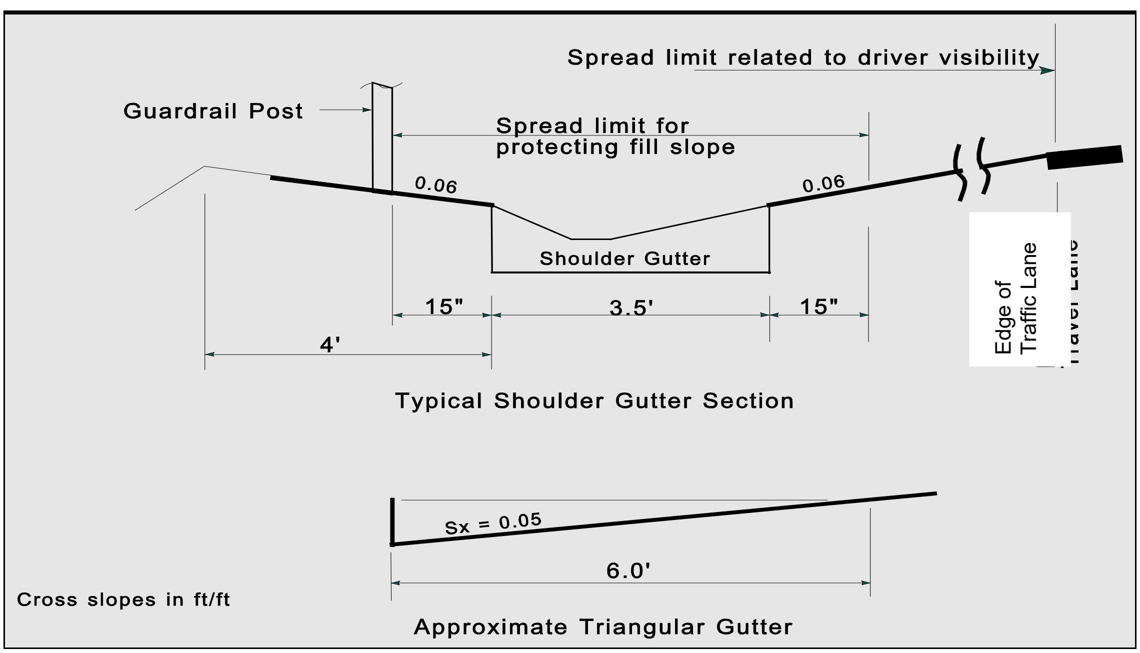



**Figure 6.3-4: Determining the Spread for a Shoulder Gutter**

### 6.3.2.4 Determining the Spread

For roads that have uniform longitudinal grades and cross slopes, the spread calculations may be as simple as calculating the spread and bypass for the inlets with the largest overland flow. For these projects, you usually can make a reasonable assumption that if the inlets with the largest overland runoff do not exceed the spread standards and do not have any bypass, the other inlets will not exceed the spread standards and will not have any bypass. If you cannot comfortably make this assumption, you can determine the spread by the following procedure used with Table 6.3-1. In general, the information in Table 6.3-1 is the minimum required for spread calculations. You may need additional information in certain situations.

Start at the upper-most inlet and work to the low point, then start at the opposite high side and work back to the low.

1. Determine the drainage area and runoff coefficient of the overland runoff. Record the product of the area and runoff coefficient (C*A) in column 2.

2. Calculate the overland runoff by multiplying the product of C*A in column 2 by the appropriate intensity (four inches per hour or 10-year storm design).
   $Q = C \cdot A \cdot i$.

3. Calculate the total flow to the inlet by adding the overland runoff in column 3 to the bypass from the upstream inlets.

4. Record the cross slope and longitudinal slope in column 6 and column 7, respectively.

5. Calculate the spread and compare it to the allowable spread, keeping in mind that allowable spread can vary along the project due to super-elevation slope toward the median, turn lanes, and design speed. If it is within the standards, record the number in column 8 and go to the next step. If it is not, move the inlet (and add and move inlets as necessary) to make the spread acceptable and repeat Step 1 through Step 5.

6. Calculate intercepted flow and bypass flow. (The figures in Appendix I can be used in lieu of calculations to determine intercepted flow. Record these numbers in column 9 and column 10, respectively.

7. Proceed to the next downstream inlet and repeat Step 1 through Step 6.

## Table 6.3-1: Spread Calculations

FLORIDA DEPARTMENT OF TRANSPORTATION
SPREAD CALCULATIONS

Road:________________________________ Sheet ______of _______
County:______________________________ Prepared by:_____Date _____
Financial Project ID: _____________________ Checked by:_____Date _____
System Description: ____________________________________________________

| | | | | | | | | | Manning's n = ______ | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Inlet No. or Location (1) | C • A (2) | Overland Runoff (3) | Previous By-pass (4) | Total Flow (5) | Cross Slope (ft/ft) (6) | Long Slope (%) (7) | Spread (8) | Allowable Spread (8a) | Intercepted Flow (9) | Bypass Flow (10) | Bypass to Inlet No. or to Inlet @ (11) |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

**Example 6.3-2—Sag Vertical Curve Given**

Given:

- The sag vertical curve and associated approach grades shown below.
- Four-lane curb and gutter section with 12-foot lanes, 12-foot continuous two-way left-turn lane, four-foot bike lane, and six-foot sidewalk.
- Offsite drains to the road from 65 feet beyond the sidewalk.
- Offsite area draining to the road is impervious (C = 0.95).
- Type 1 and Type 2 inlets are preferred by the District.
- Inlet location is not restricted by driveways or side streets.
- Design speed = 45 mph, then allowable spread is 11.5 feet [1.5-foot gutter + four-foot bike lane + six feet (half of a travel lane)].
- A minimum gutter grade of 0.3 percent is used approaching the sag.

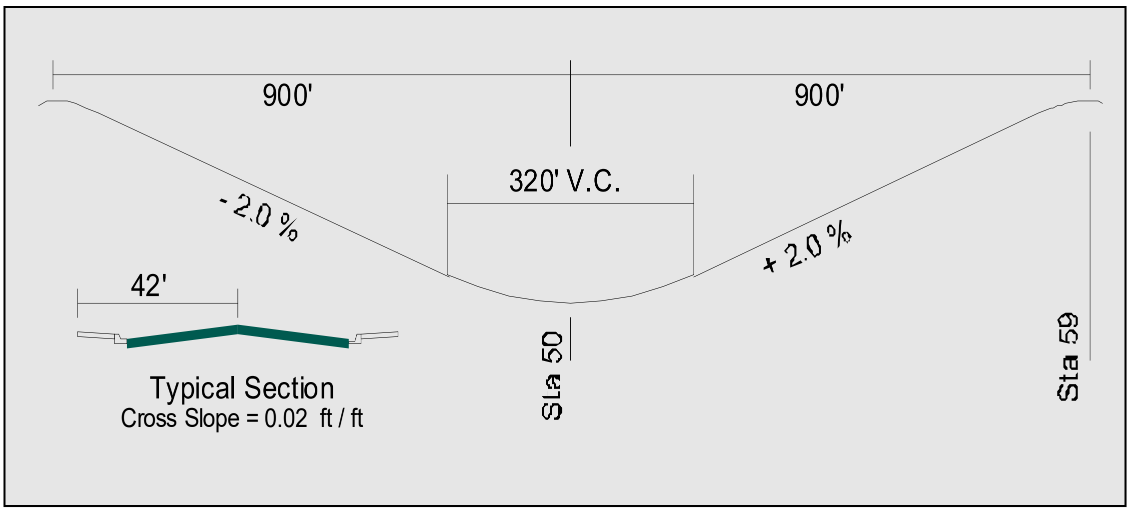



**Figure 6.3-5: Example 6.3-2 Given Information**

Find:

- Inlet spacing necessary to meet the spread criterion

1. For the first try, place the inlets at the maximum 300-foot spacing out from the low point. So, the inlets will be placed at Station 44+00, Station 47+00, Station 50+00, Station 53+00, and Station 56+00.

   The area to each inlet on the approach grades is:

   Area = (½ Rdwy Width + 65 ft.) x 300 ft.
   Area = (42 ft.+ 65 ft.) x 300 ft./43,560 = 0.74 ac @ C = 0.95
   C x A = 0.95 x 0.74 = 0.70
   $Q_{OVERLAND}$ = CAi
   = 0.95 x 0.74 x 4 = 2.8 cfs.

2. Determine the spread, the intercepted flow, and the bypass (if any) for the uppermost inlets. (Sta. 44 & 56)

Spread (T) $= [(Q \times n)/(0.56 \times S_X^{5/3} \times S_L^{½})]^{3/8}$
This conservatively ignores the 1.5-inch gutter depression.

$= [(2.8 \times 0.016)/(0.56 \times 0.02^{5/3} \times 0.02^{½})]^{3/8}$

= 9.3 ft. Acceptable (allowable spread is 11.5 ft.)
$Q_{INTRCEPT}$ ~ 2.1 cfs From Figure I-1, Appendix I
$Q_{BYPASS}$ = 2.8 – 2.1 = 0.7 cfs

3. Determine total flow to the next downstream inlets. (Sta. 47 & 53)

$Q_{TOTAL} = Q_{OVERLAND} + Q_{BYPASS}$
= 2.8 + 0.7 = 3.5 cfs

4. Determine the spread, the intercepted flow, and the bypass.

Spread = 10.2 ft. Still acceptable
$Q_{INTRCEPT}$ = 2.3 cfs From Figure I-1.
$Q_{BYPASS}$ = 3.5 – 2.3 = 1.2 cfs

5. Determine the spread approaching the sag inlet from either side.

$Q_{TOTAL} = Q_{OVERLAND} + Q_{BYPASS}$
= 2.8 + 1.2 = 4.0 cfs

6. Determine the spread approaching the sag inlet. The longitudinal slope is 0.3 percent approaching the sag. For this example, the cross slope at the low point is 0.021 ft./ft. due to maintaining 0.3-percent gutter grade to the sag inlet. This was calculated using the approach in Example 6.3-1.

T $= [(4.0 \times 0.016)/(0.56 \times 0.021^{5/3} \times 0.003^{½})]^{3/8}$
= 14.7 ft. Not acceptable (Allowable spread is 11.5 ft.)

The following table summarizes the above calculations.

Flow in cfs Allowable spread = 11.5 ft. Manning's n = 0.016

| 1 Inlet Location (Sta.) | 2 C • A | 3 Overland Runoff | 4 Previous Bypass | 5 Total Flow | 6 Cross Slope (ft/ft) | 7 Long Slope (%) | 8 Spread | 9 Intercepted Flow | 10 Bypass Flow | 11 Bypass to Inlet @ Station |
|---|---|---|---|---|---|---|---|---|---|---|
| 44+00 | 0.70 | 2.8 | -- | 2.8 | 0.02 | 2.0 | 9.3 | 2.1 | 0.7 | 47+00 |
| 47+00 | 0.70 | 2.8 | 0.7 | 3.5 | 0.02 | 2.0 | 10.2 | 2.3 | 1.2 | 50+00 |
| 56+00 | 0.70 | 2.8 | -- | 2.8 | 0.02 | 2.0 | 9.3 | 2.1 | 0.7 | 53+00 |
| 53+00 | 0.70 | 2.8 | 0.7 | 3.5 | 0.02 | 2.0 | 10.2 | 2.3 | 1.2 | 50+00 |
| 50+00 Approach | 0.70 | 2.8 | 1.2 | 4.0 | 0.021 | 0.3 | 14.7 | ← Exceeds Standard | | |

7. Add and adjust inlets.
   There is no direct solution. It is a trial-and-error process of moving inlets to reduce the spread. Adding an inlet to each side of the sag and adjusting the spacing of the inlets on the continuous grades should reduce the flow to the sag inlet and reduce the spread. Try placing the inlets at Stations 44+30, 46+80, 48+80, 50+00, 51+20, 53+20, and 55+70 as shown in Figure 6.3-6.

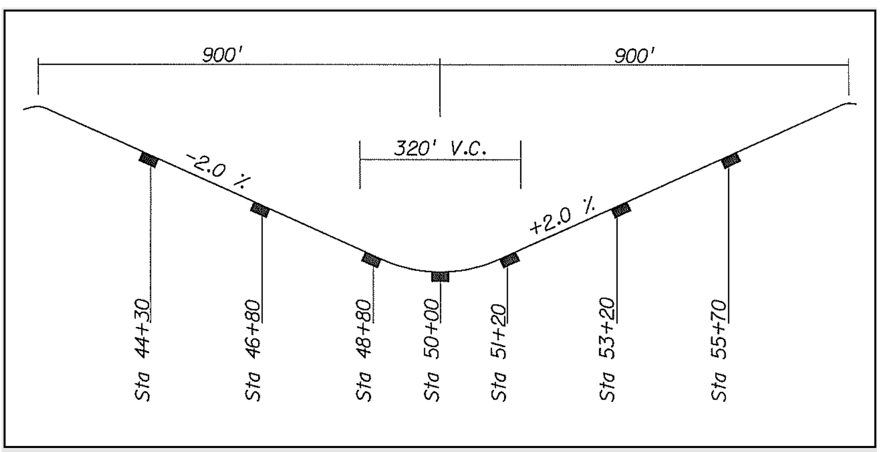



**Figure 6.3-6: Location of Inlets**

**Example 6-3.2—Second Iteration**

The drainage area to the first continuous grade inlets (43+30, 55+70) is:

Area = (½Rdwy Width + 65 ft.) x 250 ft.
Area = (42 ft. + 65 ft.) x 330 ft./43,560 = 0.81 ac @ C = 0.95

The drainage area to the next inlets (46+80, 53+20) is:

Area = (42 ft. + 65 ft.) x 250 ft./43,560 = 0.61 ac @ C = 0.95

The drainage area to the next inlets (48+80, 51+20) is:

Area = (42 ft. + 65 ft.) x 200 ft./43,560 = 0.49 ac @ C = 0.95

The drainage area to each side of the sag is:

Area = (42 ft. + 65 ft.) x 120 ft./43,560 = 0.29 ac @ C = 0.95

The inlets at stations 48+80 and 51+20 are on the vertical curve; therefore, the longitudinal slope is flatter than 2.0 percent. Using vertical curve formulae:

Rate of change of grade (r) = $(g_2 – g_1)/L$
= [2 – (-2)]/3.2 = 1.25 ft./station

Long slope = $g_1 + r X$ (X is distance along curve in Sta.)
= -2 + 1.25 (0.4)
= -1.5 percent

For this example, the cross slope at the low point is 0.021 ft./ft. due to maintaining 0.3-percent gutter grade down to the sag inlet. You can calculate this using the approach in Example 6.3-1. Using Figure I-17 (cross slope = 0.02) will provide a slight conservatism.

The following table shows the results of the change.

Flow in cfs    Allowable Spread = 11.5 ft    Manning's n = 0.016

| 1<br>Inlet<br>Location<br>(Sta) | 2<br>CxA | 3<br>Overland<br>Runoff | 4<br>Previous<br>By-Pass | 5<br>Total<br>Flow | 6<br>Cross<br>Slope<br>(ft/ft) | 7<br>Long<br>Slope<br>(%) | 8<br>Spread<br>(ft) | 9<br>Intercepted<br>Flow | 10<br>Bypass<br>Flow | 11<br>Bypass to<br>Inlet @ |
|---|---|---|---|---|---|---|---|---|---|---|
| 44+30 | 0.77 | 3.1 | 0 | 3.1 | 0.02 | 2 | 9.7 | 2.2 | 0.9 | 46+80 |
| 46+80 | 0.58 | 2.3 | 0.9 | 3.2 | 0.02 | 2 | 9.8 | 2.2 | 1.0 | 48+80 |
| 48+80 | 0.47 | 1.9 | 1.0 | 2.9 | 0.02 | 1.5 | 9.9 | 2.4 | 0.5 | 50+00 |
| 50+00 | 0.28 | 1.1 | 0.5 | 1.6 | 0.021 | 0.3 | 10.5 | n/a | n/a | n/a |
| | | | | | | | | | | |
| 55+70 | 0.77 | 3.1 | 0.0 | 3.1 | 0.02 | 2 | 9.7 | 2.2 | 0.9 | 53+20 |
| 53+20 | 0.58 | 2.3 | 0.9 | 3.2 | 0.02 | 2 | 9.8 | 2.2 | 1.0 | 51+20 |
| 51+20 | 0.47 | 1.9 | 1.0 | 2.9 | 0.02 | 1.5 | 9.9 | 2.4 | 0.5 | 50+00 |
| 50+00 | 0.28 | 1.1 | 0.5 | 1.6 | 0.021 | 0.3 | 10.5 | n/a | n/a | n/a |
| | | | | | | | | | | |
| 50+00 | 0.56 | 2.2 | 1.0 | 3.2 | 0.021 | n/a | 6.3 | n/a | n/a | n/a |

In an actual project, the inlet location is affected by driveways and side streets.

**Example 6.3-3—Shoulder Gutter**

Given:

- The bridge approach grades shown below.
- Four-lane rural divided highway, two 12-foot lanes, 10-foot paved outside shoulder, four-foot slope to gutter under guardrail (3-foot paved)
- Cross slope of shoulder and asphalt under guardrail = 0.06 ft./ft.
- Fill slope is 10 feet high at Station 67+00
- Using the project location with NOAA Atlas 14 data, the 10-year/10-minute intensity = 7.4 in/hr
- Additional lanes may be added in the future
- Runoff from bridge = 0.2 cfs (scuppers used on bridge)

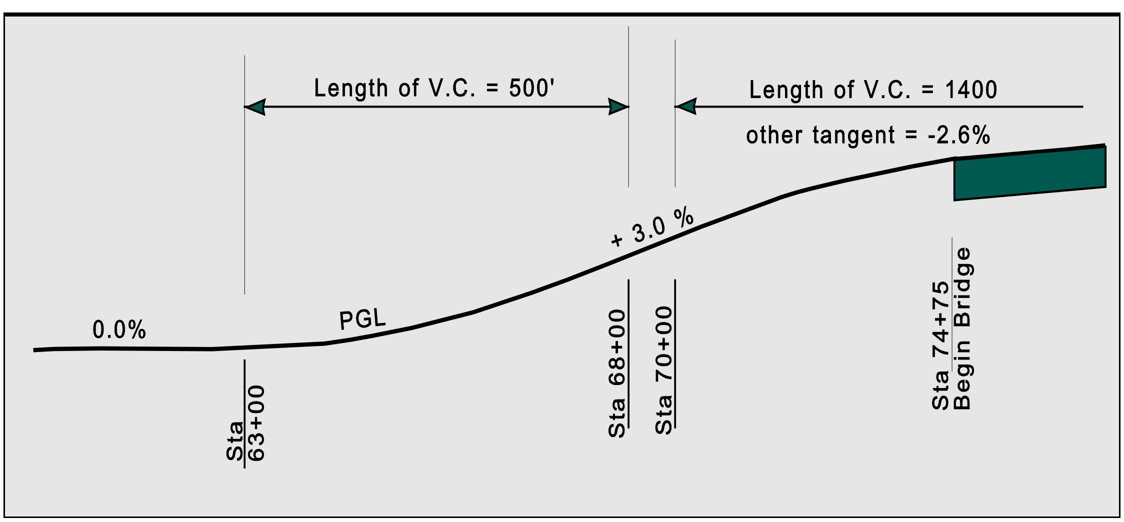


**Figure 6.3-7: Example 6.3-3 Given Information**

Find:

1. The location of the shoulder gutter inlets.

2. Determine the vertical curve geometry.

   Crest Curve:
   Rate of change of curve (r) = $(g_2 - g_1)/L$ = (-2.6 – 3.0)/14 = -0.4
   Long Slope at any point X = $g_1$ + r X = 3.0 – 0.4 X

   Sag Curve:
   Rate of change of curve (r) = $(g_2 - g_1)/L$ = (3.0 – 0.0)/5 = 0.6
   Long Slope at any point X = $g_1$ + r X = 0.0 + 0.6 X

3. Estimate the lowest point at which shoulder gutter is needed.

   You should use shoulder gutter on all fill slopes greater than 10 feet long if the roadway grade is greater than 2 percent. For this example, the fill is approximately 10 feet at Station 67+00. The longitudinal slope at this station 67+00 is: 0.6 x (67 – 63) = 2.4 percent. Since this is steeper than 2 percent, shoulder gutter should begin at or before Station 67+00.

4. For the first try at inlet spacing, divide the distance between Station 67+00 and the beginning of the bridge into equal distances that are less than 300 feet.
   Distance = 74+75 – 67+00 = 775 feet

   This equates to three spaces at approximately 258 feet. Rounding it to 260 feet, the first inlet will be located at 74+75 – 260 feet = 72+15. The other inlets are at 69+55 and 66+95.

5. Determine the longitudinal slope at these inlets:

   @ 72+15, the longitudinal slope = 3 – 0.4 (72+15 – 70+00) = 3 – 0.4 x 2.15 = 2.14 percent
   @ 69+55, the longitudinal slope = 3 percent
   @ 66+95, the longitudinal slope = 0.6 x (66+95 – 63+00) = 0.6 x 3.95 = 2.37 percent

6. Determine area and overland runoff to each inlet.

   An additional lane may be added toward the median in the future, so use 36 feet of pavement.

Width = travel lanes + shoulder + gutter + slope* under guardrail.

*Conservatively assume that all four feet sloping back to gutter is paved.

= 36 +10 + 3.5 + 4 = 53.5 ft.

Area = 260 ft. x 53.5 ft. = 0.32 ac.
C x A = 0.95 x 0.32 =0.30

The travel time for flow along 260 feet of pavement is small, so we will use the 10-minute intensity for the 10-year storm. i = 7.4 in/hr
Q = CiA = 0.95 x 7.4 x 0.32 = 2.2 cfs

7. Approximate the shoulder gutter as a triangular section with a cross slope of 0.05 ft./ft. and n = 0.016. The spread must be limited to six feet in this triangular section to match the capacity of the shoulder gutter section. See previous discussion of shoulder gutter.

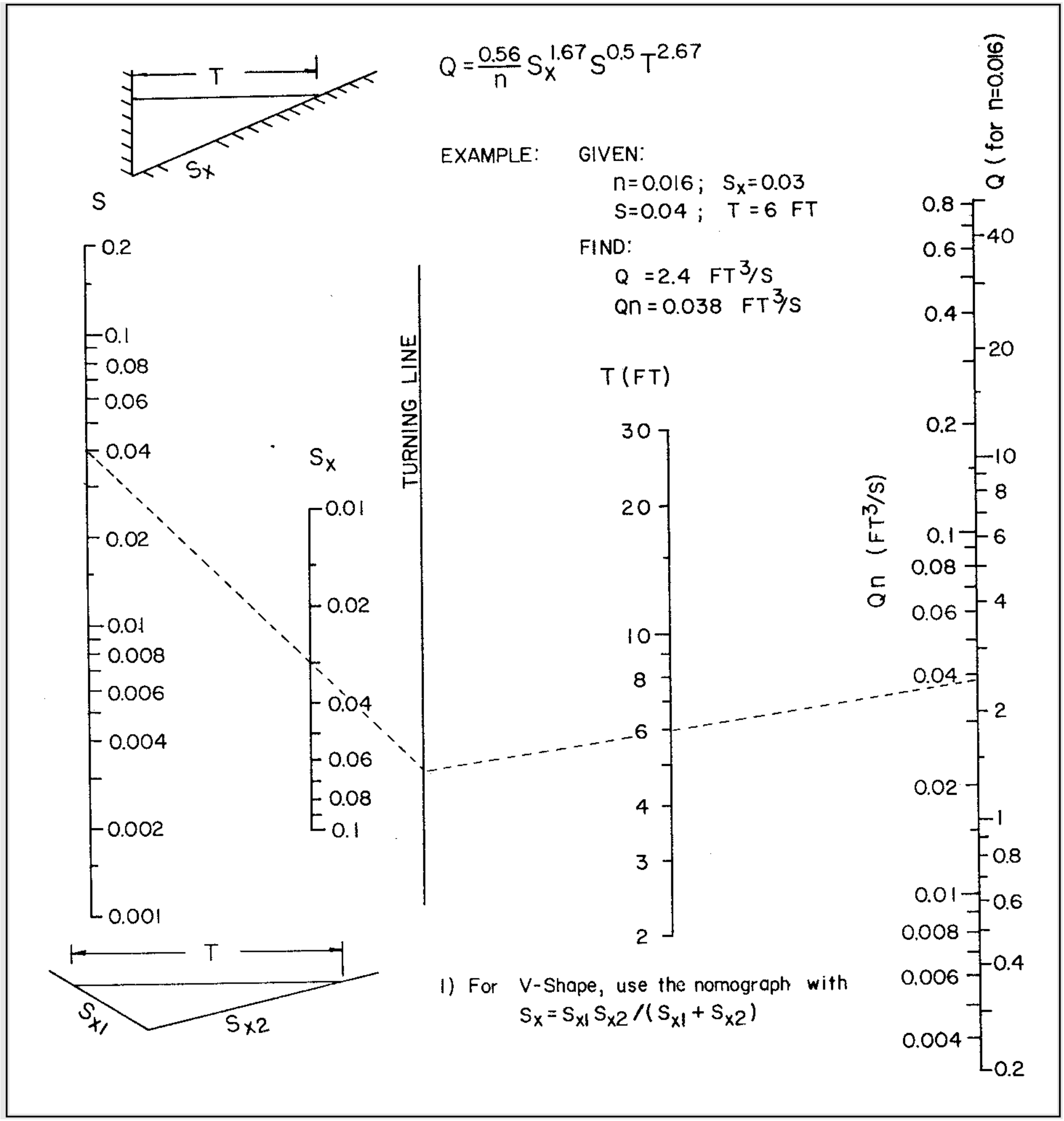
Spread (T) $= [(Q \times n)/(0.56 \times S_X^{5/3} \times S_L^{1/2})]^{3/8}$

The intercepted flow is determined from Figure I-16. The following table summarizes the calculations.

All flows (cfs) based on 10-year flow Allowable Spread = 6 ft Manning's n = 0.016

| 1 Inlet Location (Sta.) | 2 C • A | 3 Overland Runoff | 4 Previous By-pass | 5 Total Flow | 6 Cross Slope (ft/ft) | 7 Long Slope (%) | 8 Spread | 9 Intercepted Flow | 10 Bypass Flow | 11 Bypass to Inlet @ Station |
|---|---|---|---|---|---|---|---|---|---|---|
| 72+15 | 0.30 | 2.2 | 0.2 | 2.4 | 0.05 | 2.14 | 4.9 | 2.4 | --- | |
| 69+55 | 0.30 | 2.2 | --- | 2.2 | 0.05 | 3.0 | 4.5 | 2.2 | --- | |
| 66+95 | 0.30 | 2.2 | --- | 2.2 | 0.05 | 2.37 | 4.7 | 2.2 | --- | |

This inlet spacing meets the spread criterion for protecting the fill slopes and the last inlet captures all the runoff coming to it. Therefore, this design is acceptable. There is no need to check the four inches per hour criterion because the intensity would be less and the allowable spread would be greater.




Reference: FHWA Hec-12 (1984)

**Figure 6.3-8: Nomographic Solution for Spread in Gutters**

## VERTICAL CURVE FORMULAE

1. Rate of Change of Grade: $r = \frac{g_2 - g_1}{L}$

2. Offset from tangent to curve: $y = \frac{1}{2} r X^2$

3. Elevation for any point on curve: $E_x = E_{pvc} + g_1 X + (\frac{1}{2} r X^2)$

4. Grade (longitudinal slope) at any point: $\frac{\partial E_x}{\partial X} = g_1 + rX$

5. Station from PVC to turning point (local tangent horizontal) on a curve:

$$X = \frac{g_1 L}{g_1 - g_2}$$

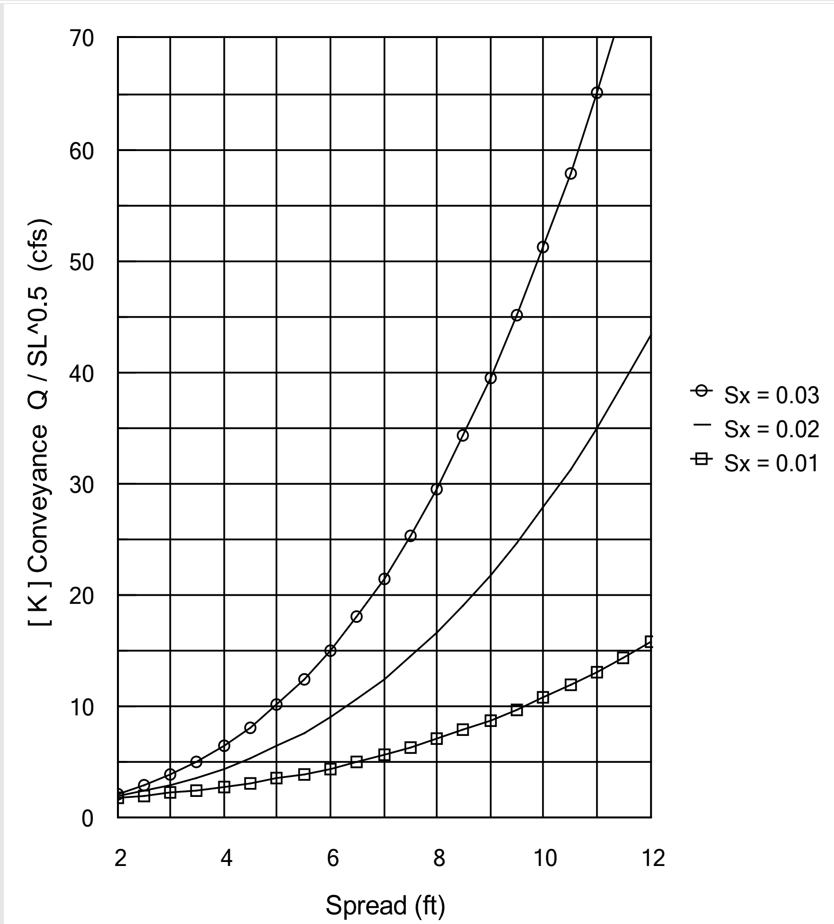
6. Elevation of turning point: $E_{TP} = E_{pvc} - \frac{1}{2}\left[\frac{L g_1^2}{g_2 - g_1}\right]$

---

Where:
All horizontal dimensions (X) are in Stations.
All vertical dimensions (E) are in Feet.
All grades are in percent.
L = Length of vertical curve in Stations.
$E_{pvc}$ = Elevation of the Point of Vertical Curve.

**Figure 6.3-9: Vertical Curve Formulae**

Conveyance vs. Spread
for Composite Gutter Sections with Type E or Type F Curb
Manning's n = 0.016




Based on FHWA HEC-12, App. C.

$S_L$ = Longitudinal Slope (ft/ft)
$S_x$ = Pavement Cross Slope (ft/ft)

Example
Given: Q = 3 cfs, $S_x$ = 0.03, $S_L$ = 0.04 ft/ft

Find Spread:
1. $K = Q/S_L^{0.5} = 3 / 0.04^{0.5} = 15$
2. From Chart at K = 15 & $S_x$ = 0.03, Spread = 6.0 ft

**Figure 6.3-10: Conveyance vs. Spread for Composite Gutter Sections with Type E or Type F Curb**

## 6.4 PIPE SYSTEM PLACEMENT

### 6.4.1 Plan Layout

After the inlets have been placed to drain the pavement adequately, lay out the piping system to connect the inlets. While laying out the system, you will add manholes as necessary to redirect the flow, or to provide maintenance access, or merely to connect stub pipes. At this stage, consider adding an inlet instead of a manhole. When an inlet is used instead of a manhole, you get the benefit of an additional hydraulic opening for little or no additional cost.

There are several items to consider that can influence the piping system plan layout. The most important issues are hydraulics, constructability, and utility conflicts.

- Avoid placing pipes that would oppose flows from other pipes, especially in high-velocity situations. Impinging flows can be avoided by staggering the elevations of the pipes entering a junction box.

- Consider right of way necessary to open the trench for the pipes. This is especially important for deep pipes. You might use temporary sheet piling during installation to reduce the trench width, but this is very costly, so you will want to explore other alternatives (e.g., moving the trunk line).

- Use either a manhole or an inlet at changes in flow direction. This will provide maintenance access where debris and sediment often collect.

- Preferably, place manholes in or behind the sidewalk. This allows access without closing the travel lanes and is much safer for maintenance personnel. If you must place manholes in the pavement, avoid putting the lids in the wheel path.

- Minimize interference with major utilities, such as fiber optic lines and sanitary and potable water lines greater than eight inches in diameter. See the discussion in Section 6.4.3.

- Where there is one main trunk line, place it on the side of the road constructed first. This prevents constructing stub lines that can't be drained.

- Where there is one main trunk line, locate it, if possible, on the low side of super-elevated roadway sections to minimize the depth of cut.

- Where there is one main trunk line, consider connecting several inlets along the opposite side of the road from the trunk line, and then running only one pipe laterally across the road. This will reduce the number of cuts across the road.

- Consider using two trunk lines to minimize the number of cuts across the road and thus simplify maintenance. In such cases, you should weight the gains in improved maintenance against the increased cost of the additional trunk line.

#### 6.4.1.1 Retaining Wall Drainage

Whenever possible, avoid placing piping within mechanically stabilized earth (MSE) retaining wall embankments. If placing pipes within the MSE wall section is your only option, please refer to Section 3.11 of the *Drainage Manual*.

### 6.4.2 Profile Placement

#### 6.4.2.1 Slopes

The *Drainage Manual* states that the minimum physical slope must produce a velocity of 2.5 feet per second flowing full. The slope is obtained from the velocity form of Manning's equation using the full cross sectional area of the pipe:

$$V = (1.49/n)\ R^{2/3}\ S^{1/2}$$

rearranging:

$$S = [V\ n\ /\ (1.49\ R^{2/3})]^2$$

where:
R is based on the full cross sectional area
V = 2.5 fps

Table 6.4-1 provides the minimum physical slope to produce 2.5 feet per second flowing full for various pipe sizes with Manning's roughness coefficient of 0.012. Note that the velocity of 2.5 feet per second is not necessarily the actual velocity in the pipe under design conditions. Refer to Section 6.5.5 for a discussion of flow velocity.

**Table 6.4-1: Minimum Slope to Produce Velocity of 2.5 Feet per Second**

| Minimum Physical Slope n = 0.012 | |
|---|---|
| Diameter (inches) | Slope (%) |
| 18 | 0.150 |
| 24 | 0.102 |
| 30 | 0.076 |
| 36 | 0.059 |
| 42 | 0.048 |
| 48 | 0.041 |
| 54 | 0.035 |
| 60 | 0.030 |
| 66 | 0.027 |
| 72 | 0.024 |
| 78 | 0.021 |
| 84 | 0.019 |

For very flat systems, the minimum physical slope may not be realistic. The overall fall across the system is based on outlet pipe depth and structural clearances at the upper end. Most District Drainage Engineers will approve deviation from the minimum pipe slope in these cases.

Where you cannot attain the minimum slope, try to design the system to avoid appreciable drops in the velocity. This will help to carry sediment through the system instead of dropping sediment at the velocity drop point in the system.

The minimum slope is 0.1 percent for systems under pressure flow.

Setting flow lines relates to the slope. Refer to *FDM 300* for the accuracy level to which you must display flow lines.

### 6.4.2.2 Minimum Pipe Depth

The minimum depth of the pipe is controlled either by the minimum pipe cover or by the need to have clearance above the top of the pipe to maintain strength in a precast structure. Minimum pipe cover requirements are given in Appendix C of the Drainage Manual.

The loads placed on precast structures during shipping and handling often are greater than the loads placed on them in their final location. Since contractors prefer precast drainage structures and they have become the industry standard, you should consider the potential for breakage during shipping and handling.

Where pipes are placed high in a structure, the structure has little, if any, strength above the pipe. This can result in breakage during shipping or handling. For strength reasons, it is best to maintain a minimum amount of precast concrete section above the pipe.

The ideal amount of precast section varies with the type of inlet and bottom configuration. Generally, where a pipe is placed in grated inlets or in structure bottoms, try to maintain a six-inch precast section that has full wall thickness above the pipe opening, as shown in Figure 6.4-1. For ditch bottom inlets placed on J bottoms, the recommended minimum precast riser section varies depending on if the unit has slots. Refer to Structure/Pipe configuration numbers 4 & 5 in Figure 6.4-5. For ditch bottom inlets without slots, maintain a 10-inch riser section below the grate seat. For ditch bottom inlets with slots, maintain a 12-inch riser section below the slot.

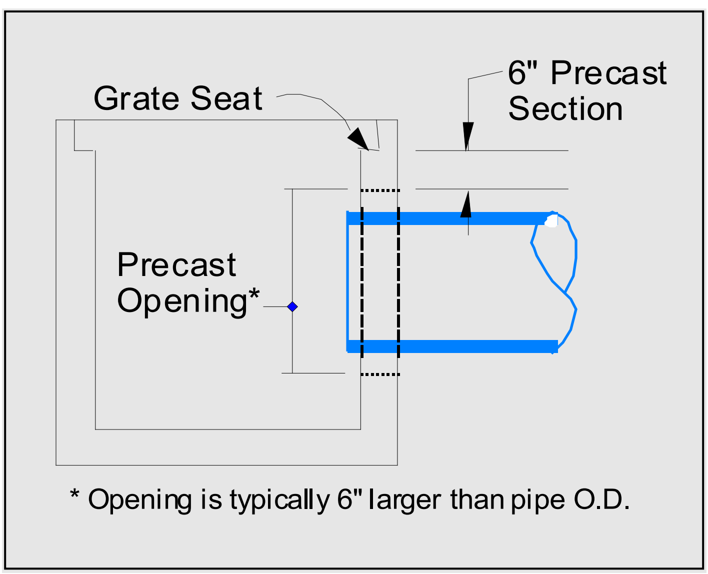



**Figure 6.4-1: Maintaining Six Inch From Grate Seat to Opening For Pipe**

Tables 6.4-4 and 6.4-5 give recommended minimum distances from inlets to pipe flow lines for most of the Department's standard inlets. These distances provide the precast section discussed above and are based on concrete pipe that is centered in the precast opening. The above discussion represents ideal values that you should try to achieve. On occasion, you will need to use less precast section than discussed above. This is acceptable because the contractor has the option to cast structures in place. When using less precast section, you must add all the appropriate dimensions to assure that no conflict will exist between pipes and the structure.

## 6.4.3 Utility Coordination

During the design process, avoid designing storm drain systems in the vicinity of utilities where practical unless it would substantially increase the cost of the system. Try to obtain information not only on the location of existing facilities, but on proposed locations as well. The utility companies (both private and public) will view the design proposed on the Phase II plans as part of the utility coordination process. You may be asked to attend utility coordination meetings, which can be very beneficial to the design effort because the concerned parties gather together to resolve utility placement conflicts. The utility companies are accustomed to meeting face-to-face with FDOT representatives. The Department and the utility companies usually negotiate final storm drain design and utility locations, with the goal to minimize the costs to the public.

Sometimes minor changes in the storm drain design can reduce the cost to a utility company and minimize the cost to the public. At other times, it may not be practical or cost effective to accommodate a utility company proposal. Utility companies often take the opportunity to upgrade their systems or add facilities during the Department's construction project. Do not assume they will relocate their systems in the process.

On projects with long storm drain systems in areas of many utilities, include one additional manhole in the quantities for unforeseen utility conflicts.

## 6.4.4 Pipe-to-Structure Connections

When a bridge deck piping system connects to a roadway structure, use a resilient connector to accommodate the expected thermal movement of the bridge and its piping system.

Check sizes of structure bottoms to make sure that the pipes fit. When doing so, use the outside diameter of concrete pipe[3]. It has the thickest wall of any of the optional pipe materials. Type P structure bottoms are either 4'-0" or smaller diameter round (Alternate

[3] An easy way to remember the wall thickness of the concrete pipe is to take the inside diameter in feet and add one (1). The result is the wall thickness in inches. Examples: 30" pipe, I.D. = 2.5 feet, Wall Thickness = 2.5 + 1 = 3.5".

A) or 3’-6” square (Alternate B). 30-inch pipe is the maximum size that will fit in Type P bottoms. The contractor has the option of using either Alternate A or Alternate B for Type P bottoms unless restricted by the plans. Type J structure bottoms are larger than Type P bottoms and come in various sizes, as described in Standard Plans, Index 425-010. The plans usually specify the alternate and the size of the J bottoms. Table 6.4-2 gives the minimal structure dimensions for various pipe sizes.

**Table 6.4-2: Minimum Structure Sizes for Single Pipe Connection Per Side**

MINIMUM STRUCTURE SIZES FOR SINGLE PIPE CONNECTION PER SIDE

| PIPE SIZE | RECTANGULAR Side Dimension (L) | | ROUND Diameter (D) | |
|---|---|---|---|---|
| | Single Pipe Per Side | Note Number | Single Pipe or θ=180° | 2 to 4 Pipes θ=90° |
| 18" | 3'-6" | | 3'-6" | 4'-0" |
| 24" | 3'-6" | | 3'-6" | 5'-0" |
| 30" | 3'-6"/4'-0" | 2 | 4'-0" | 6'-0" |
| 36" | 4'-0"/5'-0" | 3 | 5'-0" | 7'-0" |
| 42" | 5'-0" | | 6'-0" | 7'-0" |
| 48" | 6'-0" | | 6'-0" | 8'-0" |
| 54" | 6'-0" | | 7'-0" | 10'-0" |
| 60" | 7'-0" | | 7'-0" | 10'-0" |
| 66" | 7'-0"/8'-0" | 4 | 8'-0" | 12'-0" |
| 72" | 8'-0" | | 8'-0" | 12'-0" |
| 78" | 9'-0" | | 10'-0" | 12'-0" |
| 84" | 9'-0" | | 12'-0" | N/A |

TABLE NOTES:

1. For Round Structures sizes with variable angles between pipes and variable pipe sizes, refer to the FDOT Storm Drain Handbook.

2. For 3'-6" Precast Square Structure Bottoms, 30" Pipes with similar invert elevations are not permitted in adjacent walls. Use 4'-0" Side Dimensions when 30" pipe openings are required on adjacent walls and the difference in flow lines is less than 3'-0".

3. For 4'-0" Precast Square Structure Bottoms, 36" Pipes with similar invert elevations are not permitted in adjacent walls. Use 5'-0" Side Dimensions when 36" pipe openings are required on adjacent walls and the difference in flow lines is less than 3'-0".

4. For 7'-0" Precast Square Structure Bottoms, 66" Pipes with similar invert elevations are not permitted in adjacent walls. Use 8'-0" Side Dimensions when 66" pipe openings are required on adjacent walls and the difference in flow lines is less than 4'-0".

The skew at which a pipe enters a precast rectangular structure is limited by the precast pipe opening as shown in Figure 6.4-2. The maximum opening is six inches larger than the pipe outside diameter (Standard Plans, Index 425-001). The maximum pipe skew varies with the structure wall thickness and the pipe size. The maximum skew for various pipe sizes passing through eight-inch structure walls is shown in Table 6.4-3. Standard Plans, Index 425-010 provides skew values for six-inch structure walls and other pipe sizes. Use round structure bottoms (Alternate A) where the pipe enters the structure at a larger angle.

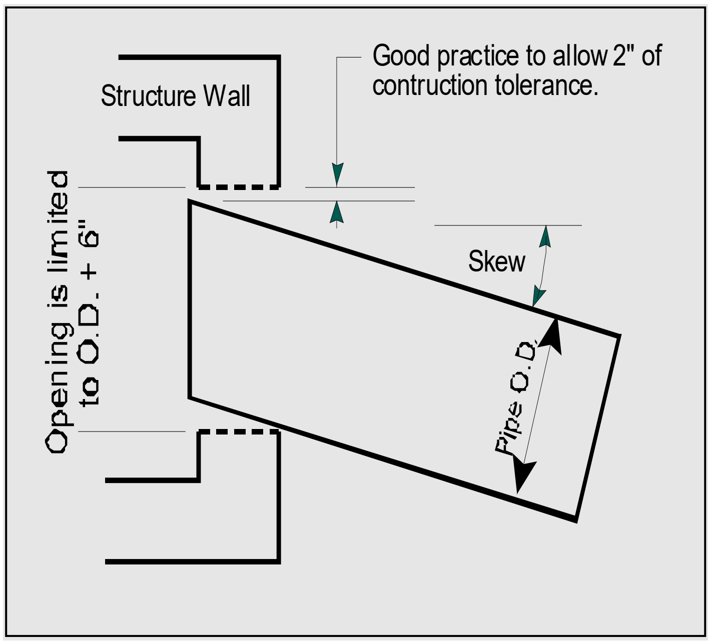



**Figure 6.4-2: Skew in Pipe Opening**

Standard Plans, Index 425-001 includes a detail of a pipe opening at a corner of a structure. Although a detail exists for this condition, restrict its use to situations where other alternatives do not exist. Make every attempt to ensure pipes do not enter the corner of rectangular structures ("corner-cutouts").

When placing pipes in existing rectangular structures, the maximum skew is limited by the dimension of the skewed pipe cut fitting between the walls.

**Table 6.4-3: Maximum Skew for Pipe Sizes Passing through an Eight Inch Wall**

| | Pipe Size | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 18" | 24" | 30" | 36" | 42" | 48" | 54" | 60" |
| Max. Skew | 19° | 17° | 16° | 16° | 15° | 14° | 14° | 13° |

These values are based on two inches of construction tolerance, precast structures with eight-inch walls, and concrete pipe dimensions.

When using round structure bottoms, consider the need to maintain a precast section between the openings of adjacent pipes. Try to maintain at least a two-inch section along the inside wall between adjacent pipe openings, as shown in Figure 6.4-3. Table 6.4-6 provides the minimum angle between adjacent pipe centerlines to maintain the two-inch precast section along the inside wall. The values in Table 6.4-6 are based on equal pipe centerline elevations and standard concrete pipe openings. Using these minimum angles for pipes with offset centerline elevations and other pipe materials is conservative and would yield more than two inches of precast section.

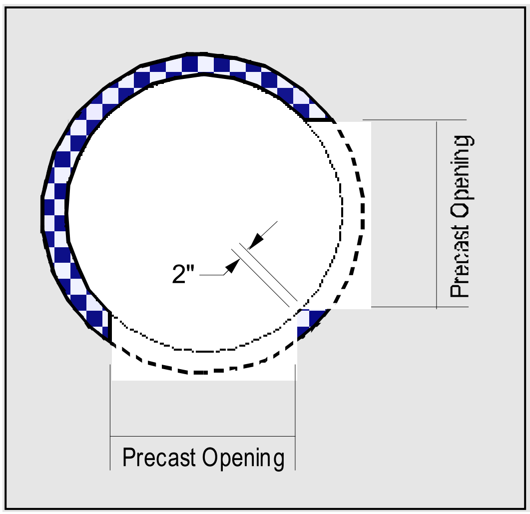



**Figure 6.4-3: Adjacent Pipe Openings**

Where large pipes are stubbed into the main line or a large main line pipe makes a 90-degree turn, rectangular structures can be smaller than round structures given the same pipe sizes. Figure 6.4-4 shows 48-inch pipes making a 90-degree turn at a structure. An eight-foot round structure is needed, while a six-foot rectangular structure would work.

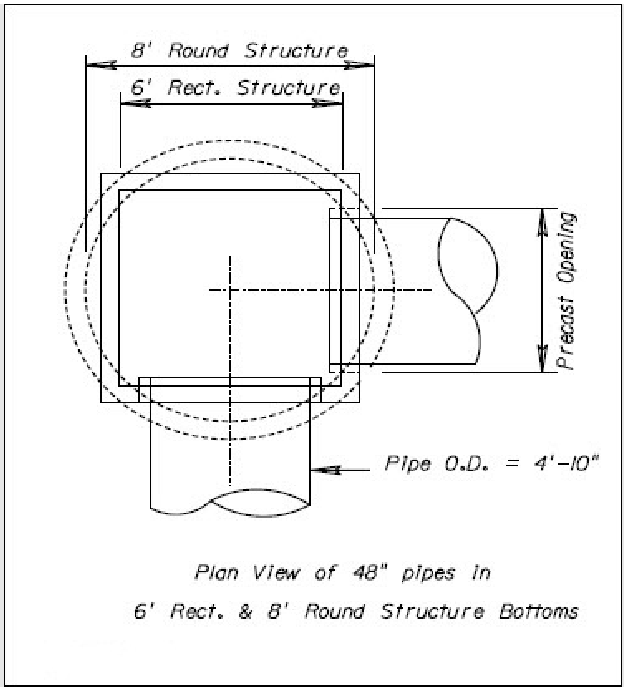



**Figure 6.4-4: 48” Pipe at 90 Degrees**

## Table 6.4-4: Recommended Min. Distance from Inlet Elevation to Pipe Flow Line

| INLET TYPE | SLOT TYPE | PIPE LOCATION | | RECOMMENDED MIN. DISTANCE (FT.) FROM GRATE (INLET) ELEVATION TO PIPE FLOW LINE | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall | Wall Dim. | 15" Pipe | 18" Pipe | 24" Pipe | 30" Pipe | 36" Pipe | 42" Pipe | 48" Pipe | 54" Pipe | 60" Pipe |
| Type A | | Short | 2'-0" | 2.2 (1) | 2.5 (1) | 4.8 (5) | 5.4 (5) | 5.9 (5) | 6.5 (5) | 7.0 (5) | 7.5 (5) | 8.1 (5) |
| | | Long | 3'-1" | 2.5 (2) | 2.8 (2) | 4.8 (5) | 5.4 (5) | 5.9 (5) | 6.5 (5) | 7.0 (5) | 7.5 (5) | 8.1 (5) |
| Type B (Note 3) | Travers | Short | No Slot | 2.6 (2) | 2.9 (2) | 3.4 (2) | 4.0 (2) | 6.9 (4) | 7.5 (4) | 8.0 (4) | 8.5 (4) | 9.1 (4) |
| | | | Under Slot | 3.4 (3) | 3.6 (3) | 4.2 (3) | 4.7 (3) | 6.9 (4) | 7.5 (4) | 8.0 (4) | 8.5 (4) | 9.1 (4) |
| | | Long | | 2.6 (2) | 2.9 (2) | 3.4 (2) | 4.0 (2) | 4.5 (2) | 7.5 (4) | 8.0 (4) | 8.5 (4) | 9.1 (4) |
| Type C (Note 3) | None | Short | 2'-0" | 2.2 (1) | 2.5 (1) | 4.7 (5) | 5.2 (5) | 5.8 (5) | 6.3 (5) | 6.8 (5) | 7.4 (5) | 7.9 (5) |
| | | Long | 3'-1" | 2.4 (2) | 2.6 (2) | 4.7 (5) | 5.2 (5) | 5.8 (5) | 6.3 (5) | 6.8 (5) | 7.4 (5) | 7.9 (5) |
| | Travers | Short | No Slot | 2.2 (1) | 2.5 (1) | 5.3 (4) | 5.8 (4) | 6.3 (4) | 6.9 (4) | 7.4 (4) | 8.0 (4) | 8.5 (4) |
| | | | Under Slot | 2.8 (3) | 3.0 (3) | 5.3 (4) | 5.8 (4) | 6.3 (4) | 6.9 (4) | 7.4 (4) | 8.0 (4) | 8.5 (4) |
| | | Long | | 2.4 (2) | 2.6 (2) | 5.3 (4) | 5.8 (4) | 6.3 (4) | 6.9 (4) | 7.4 (4) | 8.0 (4) | 8.5 (4) |
| | Non-Trav 12" Std. | Short | No Slot | 2.2 (1) | 2.5 (1) | 5.7 (4) | 6.2 (4) | 6.8 (4) | 7.3 (4) | 7.8 (4) | 8.4 (4) | 8.9 (4) |
| | | | Under Slot | 3.2 (3) | 3.5 (3) | 5.7 (4) | 6.2 (4) | 6.8 (4) | 7.3 (4) | 7.8 (4) | 8.4 (4) | 8.9 (4) |
| | | Long | | 2.4 (2) | 2.6 (2) | 5.7 (4) | 6.2 (4) | 6.8 (4) | 7.3 (4) | 7.8 (4) | 8.4 (4) | 8.9 (4) |
| Type D (Note 3) | None | Short | 3'-1" | 2.4 (2) | 2.6 (2) | 3.2 (2) | 5.2 (5) | 5.8 (5) | 6.3 (5) | 6.8 (5) | 7.4 (5) | 7.9 (5) |
| | | Long | 4'-1" | 2.2 (1) | 2.5 (1) | 3.0 (1) | 3.5 (1) | 4.1 (1) | 6.3 (5) | 6.8 (5) | 7.4 (5) | 7.9 (5) |
| | Travers | Short | | 2.4 (2) | 2.6 (2) | 3.2 (2) | 5.8 (4) | 6.3 (4) | 6.9 (4) | 7.4 (4) | 8.0 (4) | 8.5 (4) |
| | | Long | No Slot | 2.2 (1) | 2.5 (1) | 3.0 (1) | 3.5 (1) | 4.1 (1) | 6.9 (4) | 7.4 (4) | 8.0 (4) | 8.5 (4) |
| | | | Under Slot | 2.8 (3) | 3.0 (3) | 3.6 (3) | 4.1 (3) | 4.7 (3) | 6.9 (4) | 7.4 (4) | 8.0 (4) | 8.5 (4) |
| | Non-Trav 12" Std. | Short | | 2.4 (2) | 2.6 (2) | 3.2 (2) | 6.2 (4) | 6.8 (4) | 7.3 (4) | 7.8 (4) | 8.4 (4) | 8.9 (4) |
| | | Long | No Slot | 2.2 (1) | 2.5 (1) | 3.0 (1) | 3.5 (1) | 4.1 (1) | 7.3 (4) | 7.8 (4) | 8.4 (4) | 8.9 (4) |
| | | | Under Slot | 3.2 (3) | 3.5 (3) | 4.0 (3) | 4.5 (3) | 5.1 (3) | 7.3 (4) | 7.8 (4) | 8.4 (4) | 8.9 (4) |
| Type E (Note 3) | None | Short | 3'-0" | 2.2 (1) | 2.5 (1) | 3.0 (1) | 5.2 (5) | 5.8 (5) | 6.3 (5) | 6.8 (5) | 7.4 (5) | 7.9 (5) |
| | | Long | 4'-6" | 2.4 (2) | 2.6 (2) | 3.2 (2) | 3.7 (2) | 4.3 (2) | 6.3 (5) | 6.8 (5) | 7.4 (5) | 7.9 (5) |
| | Travers | Short | No Slot | 2.2 (1) | 2.5 (1) | 3.0 (1) | 5.8 (4) | 6.3 (4) | 6.9 (4) | 7.4 (4) | 8.0 (4) | 8.5 (4) |
| | | | Under Slot | 2.8 (3) | 3.0 (3) | 3.6 (3) | 5.8 (4) | 6.3 (4) | 6.9 (4) | 7.4 (4) | 8.0 (4) | 8.5 (4) |
| | | Long | | 2.4 (2) | 2.6 (2) | 3.2 (2) | 3.7 (2) | 4.3 (2) | 6.9 (4) | 7.4 (4) | 8.0 (4) | 8.5 (4) |
| | Non-Trav 12" Std. | Short | No Slot | 2.2 (1) | 2.5 (1) | 3.0 (1) | 6.2 (4) | 6.8 (4) | 7.3 (4) | 7.8 (4) | 8.4 (4) | 8.9 (4) |
| | | | Under Slot | 3.2 (3) | 3.5 (3) | 4.0 (3) | 6.2 (4) | 6.8 (4) | 7.3 (4) | 7.8 (4) | 8.4 (4) | 8.9 (4) |
| | | Long | | 2.4 (2) | 2.6 (2) | 3.2 (2) | 3.7 (2) | 4.3 (2) | 7.3 (4) | 7.8 (4) | 8.4 (4) | 8.9 (4) |

**Notes:**

1. The number in parentheses ( ) refers to one of the structure pipe combinations shown in Figure 6.4-5.
2. ***** CAUTION ***** Where multiple pipes enter a structure, needing a J-bottom because of one pipe could dictate greater distances than shown above for other pipes entering the structure.
3. The values shown for Type B, C, D, and E inlets are based on Alternate B Bottoms. Alternate A Bottoms have thicker slabs, so add two inches for up through six-foot diameter bottoms. Add four inches for eight-foot diameter bottoms.
4. The distances are based on precast structures and standard precast openings for concrete pipes.
5. The designer should check that the minimum cover requirements of *Drainage Manual*, Appendix C are met.

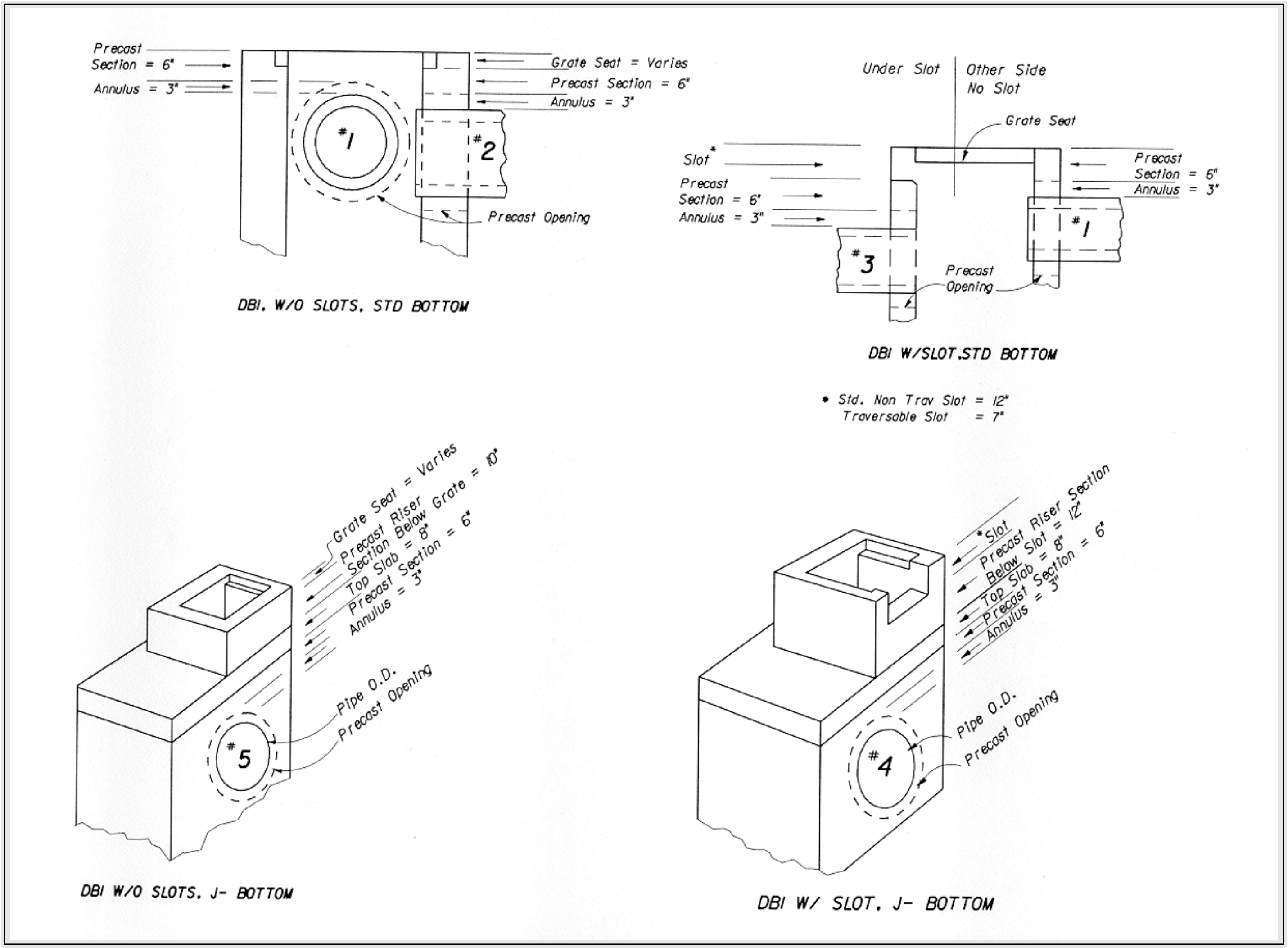

Precast Section = 6"
Annulus = 3"
Grate Seat = Varies
Precast Section = 6"
Annulus = 3"
#1
#2
Precast Opening
DBI, W/O SLOTS, STD BOTTOM
Under Slot
Other Side No Slot
Grate Seat
Slot*
Precast Section = 6"
Annulus = 3"
Precast Section = 6"
Annulus = 3"
#3
#1
Precast Opening
DBI W/SLOT,STD BOTTOM
* Std. Non Trav Slot = 12"
Traversable Slot = 7"
Grate Seat = Varies
Precast Riser Section Below Grate = 10"
Top Slab = 8"
Precast Section = 6"
Annulus = 3"
Pipe O.D.
Precast Opening
#5
DBI W/O SLOTS, J- BOTTOM
*Slot
Precast Riser Section Below Slot = 12"
Top Slab = 8"
Precast Section = 6"
Annulus = 3"
Pipe O.D.
Precast Opening
#4
DBI W/ SLOT, J- BOTTOM

**Figure 6.4-5**

## Table 6.4-5: Recommended Min. Distance from Inlet Elevation to Pipe Flow Line

| INLET TYPE | SLOT TYPE | PIPE LOCATION | | RECOMMENDED MIN. DISTANCE (FT.) FROM GRATE (INLET) ELEVATION TO PIPE FLOW LINE | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall | Wall Dim. | 15” Pipe | 18” Pipe | 24” Pipe | 30” Pipe | 36” Pipe | 42” Pipe | 48” Pipe | 54” Pipe | 60” Pipe |
| Type F | n/a | Short | 2’-6” | 2.2 (1) | 2.5 (1) | 4.8 (5) | 5.3 (5) | 5.8 (5) | 6.4 (5) | 6.9 (5) | 7.5 (5) | 8.0 (5) |
| | | Long | 4’-0” | 2.4 (2) | 2.7 (2) | 3.3 (2) | 3.8 (2) | 4.3 (2) | 6.4 (5) | 6.9 (5) | 7.5 (5) | 8.0 (5) |
| Type H | None | Short | 3’-0” | 2.2 (1) | 2.5 (1) | 3.0 (1) | n/a | n/a | n/a | n/a | n/a | n/a |
| | | Long | 6’-7” | 2.4 (2) | 2.6 (2) | 3.2 (2) | 3.7 (2) | 4.3 (2) | 4.8 (2) | 5.3 (2) | 5.9 (2) | 6.4 (2) |
| | Non-Trav 12” std. | Short | 3’-0” | 3.2 (3) | 3.5 (3) | 4.0 (3) | n/a | n/a | n/a | n/a | n/a | n/a |
| | | Long | 6’-7” | 2.4 (2) | 2.6 (2) | 3.2 (2) | 3.7 (2) | 4.3 (2) | 4.8 (2) | 5.3 (2) | 5.9 (2) | 6.4 (2) |
| Type J | n/a | Short | 3’-3” | 2.6 (2) | 2.9 (2) | 3.4 (2) | 5.5 (5) | 6.0 (5) | 6.5 (5) | 7.1 (5) | 7.6 (5) | 8.2 (5) |
| | | Long | 3’-10” | 2.4 (2) | 2.7 (2) | 3.3 (2) | 3.8 (2) | 6.0 (5) | 6.5 (5) | 7.1 (5) | 7.6 (5) | 8.2 (5) |
| Type S | n/a | Short | 3’-3” | 2.6 (2) | 2.9 (2) | 3.5 (2) | 5.5 (5) | 6.0 (5) | 6.6 (5) | 7.1 (5) | 7.7 (5) | 8.2 (5) |
| | | Long | 3’-10” | 2.3 (2) | 2.5 (2) | 3.1 (2) | 3.6 (2) | 6.0 (5) | 6.6 (5) | 7.1 (5) | 7.7 (5) | 8.2 (5) |
| Type V | n/a | Short | 3’-3” | 2.6 (2) | 2.9 (2) | 3.4 (2) | 5.5 (5) | 6.0 (5) | 6.5 (5) | 7.1 (5) | 7.6 (5) | 8.2 (5) |
| | | Long | 3’-10” | 2.4 (2) | 2.7 (2) | 3.3 (2) | 3.8 (2) | 6.0 (5) | 6.5 (5) | 7.1 (5) | 7.6 (5) | 8.2 (5) |
| Manhole Type 8 | n/a | n/a | | **RECOMMENDED MIN. DISTANCE3 (FT.) FROM TOP ELEVATION TO PIPE FLOW LINE** | | | | | | | | |
| | | | | 3.7 (10) | 4.0 (10) | 4.5 (10) | 5.0 (10) | 6.3 (11) | 6.8 (11) | 7.3 (11) | 7.9 (11) | 8.4 (11) |
| Barr-Wall 218 | n/a | | | **RECOMMENDED MIN. DISTANCE (FT.) FROM LOW POINT OF GRATE TO PIPE FLOW LINE** | | | | | | | | |
| | | Short | 3’-3” | 4.2 (8) | 4.5 (8) | 5.0 (8) | 6.2 (9) | 6.8 (9) | 7.3 (9) | 7.8 (9) | 8.4 (9) | 8.9 (9) |
| | | Long | 3’-8” | 4.2 (8) | 4.5 (8) | 5.0 (8) | 5.5 (8) | 6.8 (9) | 7.3 (9) | 7.8 (9) | 8.4 (9) | 8.9 (9) |
| Curb 1-9 | n/a | n/a | | **RECOMMENDED MIN. DISTANCE (FT.) FROM EDGE OF PAVEMENT TO PIPE FLOW LINE** | | | | | | | | |
| | | | | 3.9 (6) | 4.2 (6) | 4.7 (6) | 5.3 (6) | 6.5 (7) | 7.0 (7) | 7.5 (7) | 8.1 (7) | 8.6 (7) |

**Notes:**
1. The number in parentheses ( ) refers to one of the structure pipe combinations shown in Figure 6.4-6.
2. ***** CAUTION ***** Where multiple pipes enter a structure, needing a J-bottom because of one pipe could dictate greater distances than shown above for other pipes entering the structure.
3. ***** CAUTION ***** For curb inlets and manholes, where 30” pipes with similar inverts enter a structure at 90 degrees,a J-bottom is required, thus the minimum distance may be greater than shown above. This may apply to other inlets also.
4. The distances are based on precast structures and standard precast openings for concrete pipes.
5. The designer should check that the minimum cover requirements of *Drainage Manual*, Appendix C are met.

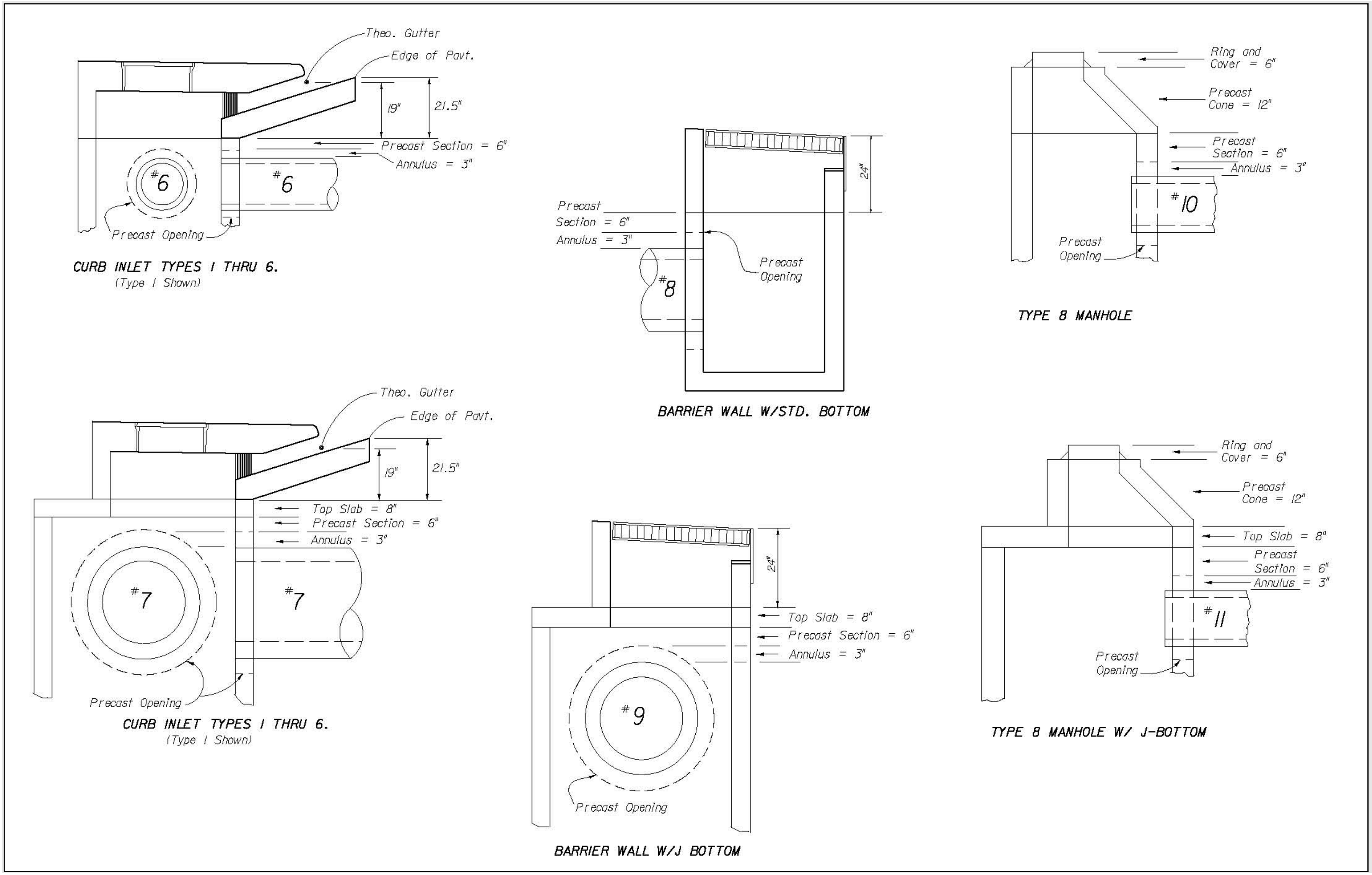

Theo. Gutter
Edge of Pavt.
19"
21.5"
Precast Section = 6"
Annulus = 3"
#6
#6
Precast Opening
CURB INLET TYPES 1 THRU 6.
(Type 1 Shown)
24"
Precast Section = 6"
Annulus = 3"
Precast Opening
#8
BARRIER WALL W/STD. BOTTOM
Ring and Cover = 6"
Precast Cone = 12"
Precast Section = 6"
Annulus = 3"
#10
Precast Opening
TYPE 8 MANHOLE
Theo. Gutter
Edge of Pavt.
19"
21.5"
Top Slab = 8"
Precast Section = 6"
Annulus = 3"
#7
#7
Precast Opening
CURB INLET TYPES 1 THRU 6.
(Type 1 Shown)
24"
Top Slab = 8"
Precast Section = 6"
Annulus = 3"
#9
Precast Opening
BARRIER WALL W/J BOTTOM
Ring and Cover = 6"
Precast Cone = 12"
Top Slab = 8"
Precast Section = 6"
Annulus = 3"
#11
Precast Opening
TYPE 8 MANHOLE W/ J-BOTTOM

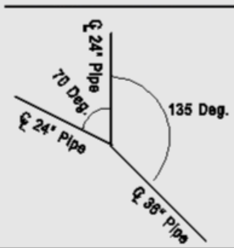
**Figure 6.4-6**

**Table 6.4-6:** The Minimum Angle Between Adjacent Pipe Centerlines to Maintain the Two-inch Precast Section along the Inside Wall

**RECOMMENDED MINIMUM ANGLE (in Degrees) BETWEEN ADJACENT PIPE CENTER LINES IN ROUND (Alt. A) STRUCTURE BOTTOMS**

| PIPE SIZE | ADJACENT PIPE SIZE | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 18" | | 24" | | 30" | | 36" | | 42" | | 48" | | 54" | | 60" | | 66" | | 72" | |
| 18" | 26 | *12'* | 28 | *12'* | 31 | *12'* | 33 | *12'* | 38 | *12'* | 41 | *12'* | 44 | *12'* | 46 | *12'* | 49 | *12'* | 52 | *12'* |
| | 31 | *10'* | 34 | *10'* | 37 | *10'* | 40 | *10'* | 46 | *10'* | 50 | *10'* | 53 | *10'* | 57 | *10'* | 61 | *10'* | 65 | *10'* |
| | 39 | *8'* | 43 | *8'* | 47 | *8'* | 51 | *8'* | 59 | *8'* | 64 | *8'* | 69 | *8'* | 75 | *8'* | 82 | *8'* | 90 | *8'* |
| | 45 | *7'* | 49 | *7'* | 54 | *7'* | 59 | *7'* | 69 | *7'* | 75 | *7'* | 83 | *7'* | 92 | *7'* | 114 | *7'* | | |
| | 52 | *6'* | 58 | *6'* | 63 | *6'* | 70 | *6'* | 84 | *6'* | 94 | *6'* | | | | | | | | |
| | 64 | *5'* | 71 | *5'* | 78 | *5'* | 87 | *5'* | | | | | | | | | | | | |
| | 82 | *4'* | 90 | *4'* | 105 | *4'* | | | | | | | | | | | | | | |
| 24" | | | 31 | *12'* | 33 | *12'* | 36 | *12'* | 41 | *12'* | 43 | *12'* | 46 | *12'* | 49 | *12'* | 52 | *12'* | 55 | *12'* |
| | | | 37 | *10'* | 40 | *10'* | 43 | *10'* | 49 | *10'* | 53 | *10'* | 56 | *10'* | 60 | *10'* | 64 | *10'* | 68 | *10'* |
| | | | 46 | *8'* | 50 | *8'* | 54 | *8'* | 63 | *8'* | 68 | *8'* | 73 | *8'* | 79 | *8'* | 85 | *8'* | 94 | *8'* |
| | | | 53 | *7'* | 58 | *7'* | 63 | *7'* | 74 | *7'* | 80 | *7'* | 87 | *7'* | 96 | *7'* | 118 | *7'* | | |
| | | | 63 | *6'* | 69 | *6'* | 75 | *6'* | 90 | *6'* | 100 | *6'* | | | | | | | | |
| | | | 78 | *5'* | 85 | *5'* | 94 | *5'* | | | | | | | | | | | | |
| | | | 102 | *4'* | 114 | *4'* | | | | | | | | | | | | | | |
| 30" | | | | | 36 | *12'* | 38 | *12'* | 43 | *12'* | 46 | *12'* | 49 | *12'* | 51 | *12'* | 54 | *12'* | 57 | *12'* |
| | | | | | 43 | *10'* | 46 | *10'* | 52 | *10'* | 56 | *10'* | 59 | *10'* | 63 | *10'* | 67 | *10'* | 71 | *10'* |
| | | | | | 54 | *8'* | 58 | *8'* | 67 | *8'* | 72 | *8'* | 77 | *8'* | 83 | *8'* | 89 | *8'* | 98 | *8'* |
| | | | | | 63 | *7'* | 68 | *7'* | 78 | *7'* | 85 | *7'* | 92 | *7'* | 101 | *7'* | 123 | *7'* | | |
| | | | | | 75 | *6'* | 81 | *6'* | 95 | *6'* | 105 | *6'* | | | | | | | | |
| | | | | | 93 | *5'* | 101 | *5'* | | | | | | | | | | | | |
| | | | | | 127 | *4'* | | | | | | | | | | | | | | |
| 36" | | | | | | | 41 | *12'* | 46 | *12'* | 48 | *12'* | 51 | *12'* | 54 | *12'* | 57 | *12'* | 60 | *12'* |
| | | | | | | | 49 | *10'* | 55 | *10'* | 59 | *10'* | 62 | *10'* | 66 | *10'* | 70 | *10'* | 74 | *10'* |
| | | | | | | | 62 | *8'* | 71 | *8'* | 76 | *8'* | 81 | *8'* | 87 | *8'* | 93 | *8'* | 102 | *8'* |
| | | | | | | | 72 | *7'* | 83 | *7'* | 89 | *7'* | 97 | *7'* | 106 | *7'* | 128 | *7'* | | |
| | | | | | | | 91 | *6'* | 101 | *6'* | 111 | *6'* | | | | | | | | |
| | | | | | | | 110 | *5'* | | | | | | | | | | | | |
| 42" | | | | | | | | | 51 | *12'* | 53 | *12'* | 56 | *12'* | 59 | *12'* | 62 | *12'* | 65 | *12'* |
| | | | | | | | | | 62 | *10'* | 65 | *10'* | 69 | *10'* | 72 | *10'* | 76 | *10'* | 81 | *10'* |
| | | | | | | | | | 80 | *8'* | 85 | *8'* | 90 | *8'* | 95 | *8'* | 102 | *8'* | 111 | *8'* |
| | | | | | | | | | 90 | *7'* | 100 | *7'* | 107 | *7'* | 117 | *7'* | 138 | *7'* | | |
| | | | | | | | | | 116 | *6'* | 126 | *6'* | | | | | | | | |
| 48" | | | | | | | | | | | 56 | *12'* | 59 | *12'* | 62 | *12'* | 65 | *12'* | 68 | *12'* |
| | | | | | | | | | | | 69 | *10'* | 72 | *10'* | 76 | *10'* | 80 | *10'* | 84 | *10'* |
| | | | | | | | | | | | 89 | *8'* | 94 | *8'* | 100 | *8'* | 107 | *8'* | 115 | *8'* |
| | | | | | | | | | | | 106 | *7'* | 114 | *7'* | 123 | *7'* | 145 | *7'* | | |
| | | | | | | | | | | | 136 | *6'* | | | | | | | | |
| 54" | | | | | | | | | | | | | 62 | *12'* | 64 | *12'* | 67 | *12'* | 70 | *12'* |
| | | | | | | | | | | | | | 76 | *10'* | 79 | *10'* | 83 | *10'* | 87 | *10'* |
| | | | | | | | | | | | | | 100 | *8'* | 105 | *8'* | 112 | *8'* | 121 | *8'* |
| | | | | | | | | | | | | | 121 | *7'* | 130 | *7'* | 152 | *7'* | | |
| 60" | | | | | | | | | | | | | | | 67 | *12'* | 70 | *12'* | 73 | *12'* |
| | | | | | | | | | | | | | | | 83 | *10'* | 87 | *10'* | 91 | *10'* |
| | | | | | | | | | | | | | | | 111 | *8'* | 118 | *8'* | 126 | *8'* |
| | | | | | | | | | | | | | | | 139 | *7'* | 161 | *7'* | | |
| 66" | | | | | | | | | | | | | | | | | 73 | *12'* | 76 | *12'* |
| | | | | | | | | | | | | | | | | | 91 | *10'* | 95 | *10'* |
| | | | | | | | | | | | | | | | | | 124 | *8'* | 133 | *8'* |
| 72" | | | | | | | | | | | | | | | | | | | 79 | *12'* |
| | | | | | | | | | | | | | | | | | | | 99 | *10'* |
| | | | | | | | | | | | | | | | | | | | 142 | *8'* |

Notes: 1. The italicized numbers to the right of the degree values are the structure bottom diameters.
2. The values are based on the pipe center lines being at equal elevation, a 2" precast section along the inside structure wall between adjacent pipes, and standard precast openings for concrete pipe. The sizes of the precast openings are those proposed by the Florida Precast Concrete Structures Association and are not always O.D. plus 6".
3. The value for two 36" pipes in a 6' diameter structure is adjusted to be consistent with Index no. 200. A similar change was made for two 42" pipes in a 7' diameter structure.




Example:
What size round bottom should be used for these pipes?

1. Looking at the 2-24" adjacent pipes, the minimum internal angle is 78° for a 5' dia. bottom and 63° for a 6' dia. bottom. Since these pipes enter at 70°, we need a 6' dia. bottom.

2. Checking the adjacent 24" and 36" pipes, 75° is needed between the pipes in a 6' dia. bottom. We have 135°, so the 6' dia. bottom works.

## 6.5 PIPE HYDRAULICS

The *Drainage Manual* states that you must consider friction losses in the computation of the design hydraulic gradient for all storm drain systems. Energy losses associated with pollution control structures (weirs and baffles) and utility conflict structures also must be considered when present in a system. When the hydraulic calculations consider only the above, the elevation of the hydraulic gradient must be at least one foot below the theoretical gutter elevation. This is equivalent to 13.5 inches (1.13 feet) below the edge of pavement for sections with Type E or Type F curb and gutter. For gutter inlets (Standard Plans, Indexes 425-040 and 425-041), ditch bottom inlets used within the roadway section (Standard Plans, Indexes 425-050 through 425-055), and barrier wall inlets (Standard Plans, Index 425-031), the one foot of clearance applies to the grate elevation. For barrier wall inlets (Standard Plans, Indexes 425-030 and 425-032), the one foot of clearance applies to the theoretical grade point.

If you calculate all minor energy losses, it is acceptable for the hydraulic grade line to reach the theoretical gutter elevation. Minor losses include all the losses at inlets, manholes, and junctions due to expansion, contraction, and changes in flow direction. Minor losses also include exit losses at the outlet of the system. The *Drainage Manual* states that minor losses must be calculated when the velocity is greater than 7.5 fps and for systems longer than 2,000 feet.

### 6.5.1 Pressure Flow

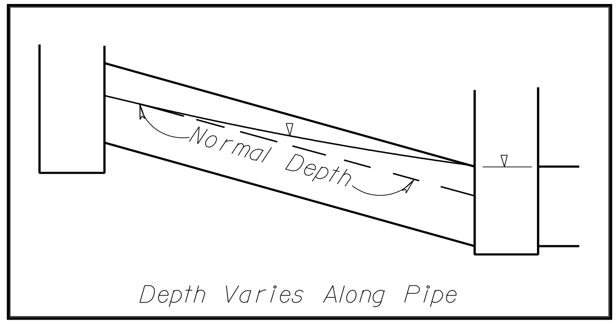
Under pressure flow conditions, the pipe section flows full throughout. Calculate friction losses using Manning's equation, with the flow area equal to the full cross sectional area of the pipe.

$$\text{Head loss [in feet]} = \frac{29n^2LV^2}{R^{1.33}\,2g} = \frac{4.61n^2LQ^2}{D^{5.33}}$$

where:

n = Roughness coefficient (refer to the *Drainage Manual*)
L = Pipe length, in feet (ft.)
V = Velocity, in feet per second (fps)
Q = Flow rate, in cubic feet per second (cfs)
R = Hydraulic radius, in feet (ft.) = Area/wetted perimeter
D = Pipe diameter, in feet (ft.)
g = Gravitational constant = 32.2 ft/s$^2$

## 6.5.2 Partially Full Flow




For pipes that are flowing partially full, the calculations are more complicated. The cross-sectional flow area actually changes as the flow goes through the pipe. For example, the flow area at the downstream end of the pipe shown here is the full cross section area, but at the upstream end the flow area is much less.

The most accurate approach to calculating this number is to do water surface profile calculations throughout the pipe section. Although acceptable, these calculations are tedious and not usually required. The Department accepts the following approach to calculating the hydraulics of partial full and pressure pipes.

Three values must be determined for each pipe section: (1) the lower-end hydraulic gradient (lower-end HG), (2) the upper-end hydraulic gradient (upper-end HG), and (3) the flow velocity.

## 6.5.3 Lower-End Hydraulic Gradient

Either the downstream hydraulic gradient or the flow conditions in the pipe controls the lower-end HG. So you must compare the water surface elevations associated with these numbers and use the higher of the two as the lower-end HG.

> The downstream hydraulic gradient elevation is the downstream pipe Upper End HG elevation plus junction losses, if they are calculated.

Where the downstream HG is above the lower-end crown of the subject pipe, the lower-end HG is the downstream HG. See Detail A of Figure 6.5-1. Pipe flow conditions will not control in this situation, and comparing water surface elevations is not necessary. If the downstream HG is below the lower-end crown, you will need to compare the downstream HG with the water elevation associated with the pipe flow conditions.

Where the downstream hydraulic gradient is low enough, one of two pipe flow conditions will control the lower-end HG. See Detail C & D of Figure 6.5-1. The appropriate flow condition is dependent on the relationship of the physical pipe slope and the full-flow friction slope. If the pipe is sloped steeper than the full-flow friction slope, it is reasonable to assume that normal depth flow exists at the lower end. Then the lower-end HG is the normal depth plus the lower-end flow line elevation. (Actual depth could be above normal depth because the pipe was not long enough to allow normal depth to be reached.)

Partial depth pipe flow graphs in Appendix E, or the Department's hydraulic calculator, can be used to get normal flow depth and associated velocity.

If the pipe slope is equal to or flatter than the full-flow friction slope, the pipe is flowing full over most of its length. Although the flow may be dropping through critical depth[4] near the outlet, assuming full flow at the outlet is reasonable and conservative. During very low flow rates, even flat pipes will not flow full, but such low rates are not typical for design conditions.

In short, use the higher of the following for the lower-end HG, as shown in Figure 6.5-1.

Condition 1: The downstream pipe upper-end HG (+ junction losses, if calculated)

OR

Condition 2: The normal depth + lower-end flow line elevation (for pipes sloped steeper than full-flow friction slope) or lower-end crown elevation (for pipes sloped equal to or flatter than full-flow friction slope).

For the outlet pipe of the system, the lower-end HG elevation is the Design Tailwater elevation.

4 - For a slightly more refined analysis in this situation, midway between critical depth and the crown of the pipe of [($D_c$+ D)/2} could be used as the Lower End HG.

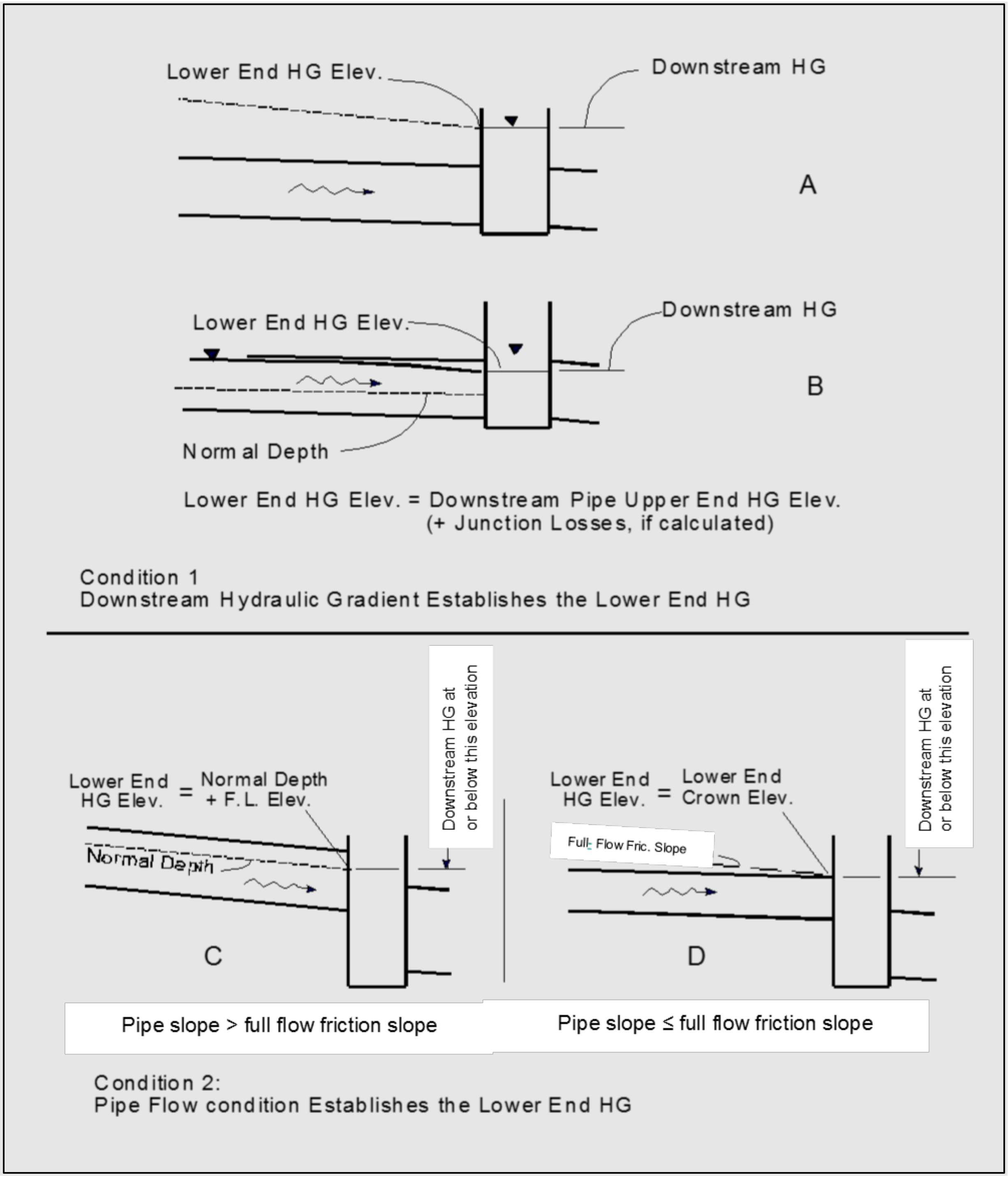



**Figure 6.5-1: Determining the Lower End Hydraulic Gradient Elevation**

#### 6.5.3.1 Design Tailwater (DTW)

The *Drainage Manual* gives the standard design tailwater conditions. In general, it says use the higher of the crown of the pipe or the downstream condition. Stormwater ponds are commonly constructed at the outlet of storm drains, so the pond stage may be the design tailwater. Some Districts may have more stringent criteria than shown in the *Drainage Manual.*

You can determine the pond stage by “routing” the storm drain design event (frequency) through the pond. “Routing” refers to the use of the storage indication method that is commonly used to simulate runoff hydrographs flowing through stormwater management facilities. HEC 22 contains a discussion and example of the storage indication method.

### 6.5.4 Upper-End Hydraulic Gradient

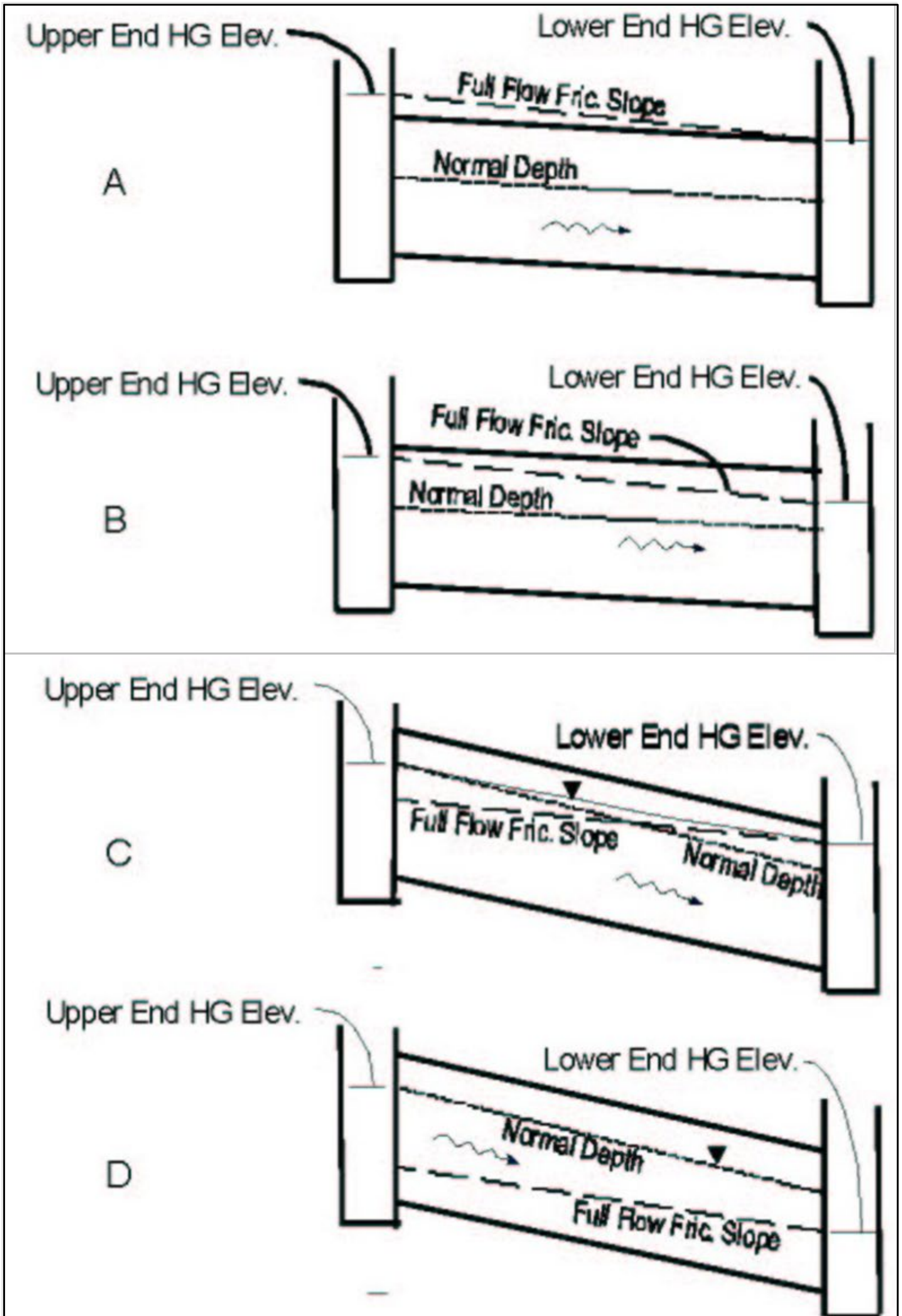
Use the higher of the following, as shown in Figure 6.5-2:

Condition 1: The lower-end HG plus the full-flow friction loss

OR

Condition 2: The elevation of normal depth in the pipe at the upper end

A comparison may not be necessary. First, add the full-flow friction loss to the lower-end HG. If this is above the upper-end crown, there is no need to calculate normal depth. The lower-end HG plus full-flow friction loss will control.




The Upper End HG is higher of the Lower End HG + Full Flow Friction loss (A & B) or Normal Depth + F.L. (C & D)

**Figure 6.5-2: Determining the Upper-End Hydraulic Elevation**

### 6.5.5 Flow Velocity in the Pipe Section

For pressure flow pipes, the velocity is based on the full cross section area.

> Where the tailwater conditions submerge the storm drain without stormwater flow, the travel time in the pipe can be ignored since the velocity is irrelevant. See the discussion of ignoring travel time in Section 6.2.

$$\text{Velocity(fps)} = \frac{Q}{A} = \frac{Q}{\frac{\pi D^2}{4}}$$

where:
Q = Flow rate, in cubic feet per second (cfs)
D = Diameter, in feet (ft.)

For pipes flowing partially full, it is more complicated to determine the velocity. There can be a water surface profile in the pipe, so the cross sectional flow area can change, thus changing the velocity along the pipe section. The most accurate velocity should represent the average velocity through the pipe section, assuming the velocity associated with normal depth is a conservative assumption. See Figure 6.5-3.

You can use partial depth pipe flow graphs in Appendix E, or the Department's hydraulic calculator, to get normal flow depth and associated velocity.

The flow velocity is referred to as the Actual Velocity in the Storm Drain Tabulation Form, Figure 6-1. The actual velocity is sometimes called the design velocity.

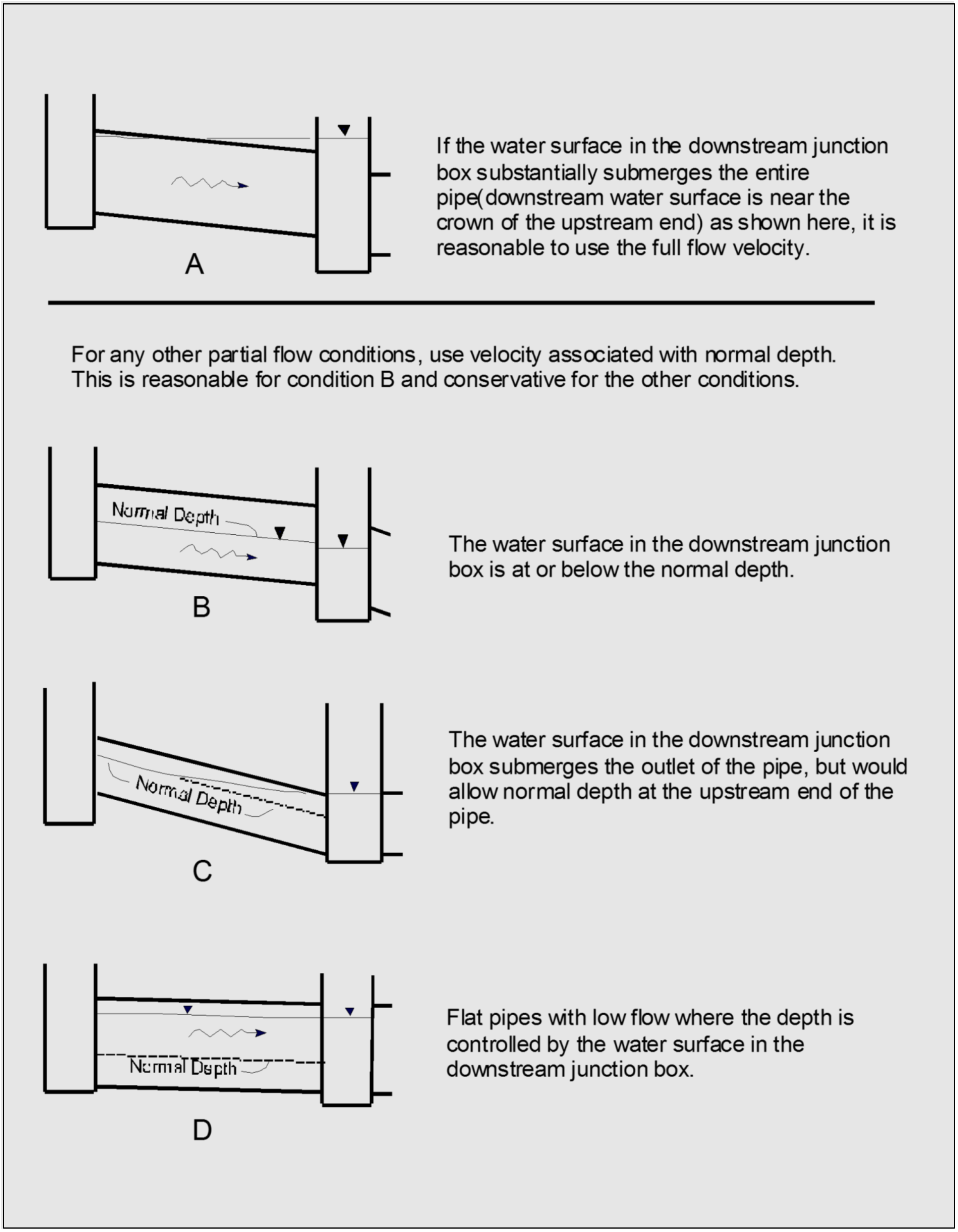



**Figure 6.5-3: Determining the Velocity in Partially Filled Pipes**

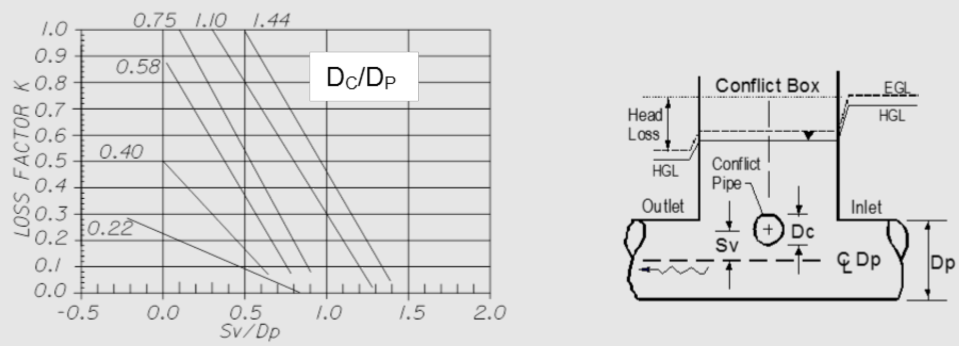
### 6.5.6 Utility Conflict Box Losses

Calculate the loss through a utility conflict box using the equation:

$$\text{Head Loss [in feet]} = K\frac{V^2}{2g}$$

Where:
K = Loss factor (or coefficient)
V = Flow velocity in the storm drain, in feet per second (ft/s)
g = gravitational constant = 32.2 ft/s$^2$

Use Figure 6.5-4 to determine the loss factor in conflict boxes where the pipes are flowing full.




Notes:

1. The loss factors were obtained under full-flow conditions, conflict centered between storm drain inlet and outlet.
2. Where two or more conflict pipes are closely spaced and one is above the other, treat the conflict as a single obstruction with an effective diameter equivalent to the sum of the two pipe diameters.
3. No correction factor is required for conflict pipes angled within the horizontal plane. Configurations were tested at a 45-degree angle.
4. This information is based on research by the University of South Florida and is documented in two reports: (1) *Hydraulic Performance of Conflict Junction Boxes,* July 1996; WPI no. 0510710; contract no. B-9080; (2) *Hydraulic Performance of Conflict Manholes,* November 1999; WPI no. 0510819; contract no. B-B304.
5. Contact the FDOT Research Center at (850) 414-4615 to obtain copies.

**Figure 6.5-4: Loss Factors for Conflict Manholes**

### 6.5.7 Minor Losses

Minor losses are all the losses that are not due to friction. Generally, these are energy losses due to changes or disturbances in the flow path. They include such things as entrance, exit, bend, and junction losses. The losses are calculated from the equation:

$$\text{Head loss [in feet]} = K\frac{V_o^2}{2g}$$

where:
K = Loss factor (or coefficient)
$V_o$ = Flow velocity in the outlet pipe of the junction box, in feet per second (ft/s)
g = Gravitational constant = 32.2 ft/s$^2$

FHWA has printed the latest information on computing minor losses in HEC 22, and they continue to do research on minor losses. A report titled *Junction Loss Experiments: Laboratory Report* summarizes work that has been done more recently than the information published in HEC 22. The report and HEC 22 are available on the Internet at:

http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm?sort=Publication_title&archived=0

It is important to calculate minor losses in high-velocity situations and in long systems. As the velocity approaches eight feet per second, the velocity head ($V^2$/2g) approaches one foot (64/64.4). The standard one foot of HGL clearance would be used up where the total loss coefficient, k, equals 1.0. For long systems, the 1.0 foot of clearance could be used up by numerous small individual junction losses, e.g., 10 junctions with 0.1 foot minor loss each.

## 6.6 PROCEDURE

The following is a basic procedure for designing a storm drain system. You can vary slightly from the procedure and still develop an adequate design. With experience, you will develop shortcuts and personal preference. The goal is to minimize pipe sizes while meeting the appropriate standards.

The FDOT CADD Office also has several training guides for storm system's layout and tabulation analysis within various software programs (OpenRoads Designer, Autodesk Civil 3D, and Bentley MicroStation SS10) that meet FDOT Drainage standards and guidelines that pair with plans production and design workflow. FDOT CADD Training publications can be found here: https://www.fdot.gov/cadd/downloads/documentation

The numbers in parentheses **(xx)** refer to a space on the Storm Drain Tabulation Form.

| | |
|---|---|
| **IDENTIFY INLET LOCATIONS** | |
| 1. Define the overall basin draining to the project. | Using the drainage map, identify the overall watershed that drains to the project. |
| 2. Determine the outfalls and divide the overall basin into sub-basins. | This is typically done as a part of the stormwater management design. |
| 3. For each sub-basin, select inlet locations. | |
| 4. Determine the drainage area to each inlet. | |
| 5. Calculate spread and revise inlet location, as necessary. | |
| **LAYOUT PIPES** | |
| 6. Connect pipes between the inlets to create a schematic of the piping system layout. | You will use the schematic of the piping system for the rest of the design procedure. |

| | |
|---|---|
| **DETERMINE THE TOTAL "C*A" PRODUCT FOR EACH PIPE SECTION**<br>Begin filling out the Storm Sewer Tabulation Form. Record the Inlet Types **(7)**, Inlet Locations **(3) (4) (5)**, Inlet Elevations **(19)**, Structure Numbers **(6)**, Incremental Areas **(9)**, C-Factors **(1)**, and Length **(8)** on the tabulation form. The incremental areas and C-factors are those used to calculate the spread. | |
| 7. Add the areas that contribute flow to the downstream pipe. | This involves checking for all the upstream areas. Refer to the piping system schematic to ensure that all the areas are included. Record these in the space **(10)** on the tabulation form. |
| 8. Multiply the subtotal areas by their respective C-factors. | Record the result in space **(11)** on the tabulation form. |
| 9. Add the sub-total (C*A) values. | Record the total in space **(15)** on the tabulation form. |
| 10. Repeat Step 7 through Step 9 for the entire system. | |
| **PRELIMINARY HYDRAULIC GRADE LINE (HGL) SLOPE** | |
| 11. Estimate a preliminary hydraulic grade line slope.<br><br>This slope will be used as a guide for selecting the trial pipe size only. It will not control the final design. | For flat terrain, estimate which inlet will be critical. The critical inlet usually will be the lowest inlet in the portion of the system farthest from the outlet. It may be simply the inlet farthest from the outlet. Use the following formula to calculate the slope.<br><br>$\text{Slope} = \dfrac{\text{Critical Inlet Elev - DTW - 1 foot}}{\text{System Length between Outlet \& Critical Inlet}}$<br>For moderately sloped terrain, an average slope of the ground line along the project usually is acceptable for a preliminary HGL slope.<br><br>For some systems, there may be two or more distinct sections of the system that have noticeably different slopes. For these, calculating a preliminary HGL slope for each section is advised. |

| **CALCULATE RUNOFF FLOW RATES**<br>The following is the beginning of an iterative process of calculating flow rates as you move down the system and calculating hydraulic grade line elevations as you move up the system. Step 12 through Step 18 are done on each pipe segment, beginning at the upper end of the system and working down toward the outlet. | |
|---|---|
| 12. Determine the $t_c$. | Record the value in space **(12)** on the tabulation form. |
| 13. Determine the intensity. | Determine the intensity from the NOAA Atlas 14 data using a storm duration equal to the $t_c$ previously computed. Record the value in space **(14)** on the tabulation form. |
| 14. Calculate total runoff for the pipe segment. | Multiply the total CA times the intensity. Record the value in space **(17)** on the tabulation form. |
| 15. Select a pipe size. | For the first pass through the system, select a diameter that has a full-flow friction slope close to the preliminary HGL slope. The minimum pipe diameter will probably control the pipe size of the first few pipe sections. You will probably not find a pipe diameter that matches the preliminary HGL exactly. The objective is to maintain the standard HGL clearance at each inlet. Matching the preliminary HGL is merely a technique to begin selecting pipe diameters. Some pipe diameters will likely be revised later.<br><br>Record the pipe size **(30) (31)** and associated Minimum Physical Slope **(34)** on the tabulation form. Record the full-flow friction slope as the hydraulic grade line slope **(32)** during the first pass down the system. Use the full-flow friction slope in the calculation of the hydraulic gradient. |
| 16. Determine the pipe flow lines, fall, and physical slope. | The flow lines usually will be controlled by such things as cover requirements, structure clearances, and minimum physical pipe slope. Record the Flow Line Elevations **(25) (26)**, Crown Elevations **(23) (24)**, Physical Slope **(33)**, and Pipe Fall **(28)** on the tabulation form. |
| 17. Calculate the flow velocity. | Actual Velocity: For the first pass through the system, assume full flow unless the pipe is obviously flowing part full. For subsequent passes through the system, use full-flow velocity or velocity associated with normal |

| | |
|---|---|
| | depth, as appropriate. See discussion in Section 6.5. Record the value in space **(35)** on the tabulation form.<br><br>Physical Velocity: Record the value in space **(36)** of the tabulation form. |
| 18. Calculate time of flow in pipe section. | Divide the pipe length by the actual flow velocity. Record the value in space **(13)** on the tabulation form. |
| 19. Repeat Step 12 through Step 18 for the entire system. | Check for peak flow from reduced area. See discussion in Section 6.2. |

<table>
<tr><td colspan="2">CALCULATE HYDRAULIC GRADE LINE (HGL) ELEVATION<br><br>Step 20 and Step 21 are done on each pipe segment, beginning at the outlet and working up the system toward the most remote inlet.</td></tr>
<tr><td>20. Determine the Lower-End Hydraulic Gradient Elevation.</td><td>The Lower-End HG for the outlet is the Design Tailwater (DTW). See the discussion in Section 6.5. Record the value in space (22) on the tabulation form for the outlet pipe and in space (41).</td></tr>
<tr><td>21. Determine the Upper-End HG Elevation, HGL Slope, and HGL Fall.</td><td>See the discussion in Section 6.5. Record the Upper-End HG Elevation (21). Record the HGL Clearance (20).<br><br>Where a pipe is flowing full, the full-flow friction slope recorded in Step 15 is the Hydraulic Grade Line Slope (32). The HG Fall (27) is calculated by multiplying the HGL Slope by the pipe length.<br><br>Where a pipe is flowing partially full and the Upper-End HG is based on normal depth, as in Figure 6.5-2 C and D, the HG Slope and Fall recorded in Step 15 are not correct. Here, the HG Slope and Fall are not critical to the design process, but their values can be recorded as:<br>HG Fall (27) = Upper-End HG – Lower-End HG<br>HG Slope (32) = HG Fall/pipe length</td></tr>
<tr><td colspan="2">Repeat Step 20 and Step 21 for the entire system. For the first pass through the system, you may want to calculate the HGL elevation only along the main line from the outlet to the critical inlet. The flow rates and friction losses in the stub lines usually are small. Calculating the HG through the entire system (i.e., all the stubs) for the first iteration is acceptable, but may result in extra effort. For subsequent passes, calculate the HGL elevation for the entire system.</td></tr>
</table>

| **COMPARE HYDRAULIC GRADE LINE (HGL) ELEVATION TO STANDARD** | |
|---|---|
| 22. Compare the HGL Elevation to the standard and adjust pipe diameters. | The current standard requires that the Hydraulic Gradient be at least one foot below the inlet elevations.<br><br>For systems where the distance between the Hydraulic Gradient and the gutter elevation is greater than the standard, the diameter of one or more pipe segments may be reduced to raise the Hydraulic Gradient. Here, try to reduce the larger-diameter pipe segments first, since this will provide a greater cost reduction than reducing the size of the smaller-diameter segments.<br><br>For systems where the distance between the Hydraulic Gradient and the gutter elevation is less than the standard, you will need to increase the diameter of one or more pipe segments to lower the Hydraulic Gradient. Here, increase the smaller-diameter pipe segments first, since this will provide less of a cost increase than increasing the size of the larger-diameter pipe segments. Look for “flow-pipe size” combinations that have substantial friction losses. For example, there is very little reduction in the losses by increasing the diameter of a 24-inch pipe that is carrying only 3 cfs. Alternatively, if another 24-inch pipe were carrying 15 cfs, increasing the pipe diameter could achieve a significant reduction in friction losses. |
| **RECALCULATE THE RUNOFF AND HYDRAULIC GRADE LINE ELEVATION** | |
| 23. Return to Step 14, working the changes through the system. | When you have made enough iterations through the system that any changes in diameters of pipe segments would cause the distance between the Hydraulic Gradient and the gutter elevation to be less than the standard, your design is essentially complete. |

Note:
Examples 6.6-1 and 6.6-2 were created before the *Plan Preparation Manual* (Volume 2, Chapter 1.3) required that flow lines be shown to two decimal places and before the *Drainage Manual* required that HGL Clearance be provided in the storm tab. The examples have not been revised to reflect these changes. Although the examples have not been revised, they still represent a valid design procedure.

**Example 6.6-1 Flat System—Determining Appropriate Pipe Sizes**
Given:

- Inlets, Pipes, Runoff Coefficients & Details shown in Figure 6.6-1 and Table 6.6-1
- System discharges to a pond that stages to elevation 8.3 during a five-year design storm

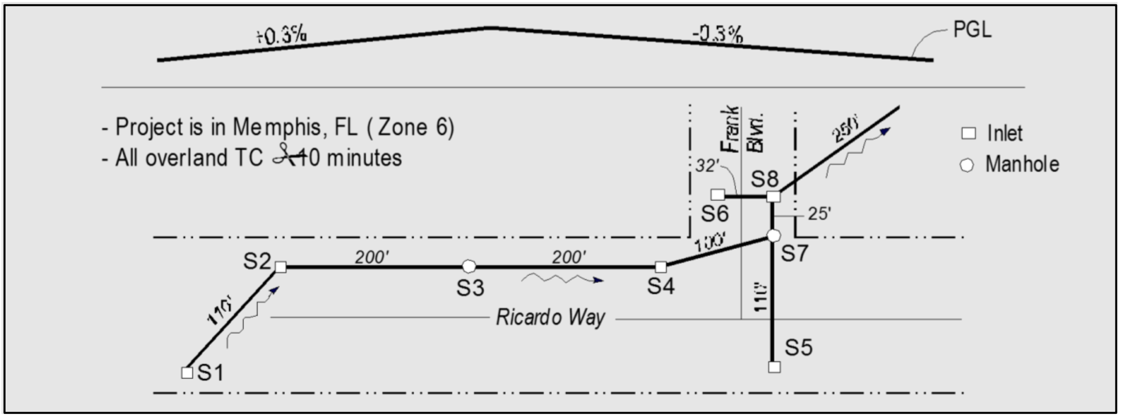



**Figure 6.6-1: Example 6.6-1 – Given Information**

**Table 6.6-1: Data for Example 6.6-1**

| LOCATION OF UPPER END | | | STRUCTURE NO. | TYPE OF STRUCTURE | LENGTH (ft) | DRAINAGE AREA (ac) | | INLET ELEV (ft) | NOTES AND REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| ALIGNMENT NAME | | | | | | C = 0.95 | | | FREQUENCY (yrs): 5 |
| | | | | | | C = | | | MANNING'S "n": 0.012 |
| | | | | | | C = 0.20 | | | TAILWATER EL. (ft) 8.3 |
| STATION | DISTANCE (ft ) | SIDE | UPPER | | | INCREMENT | TOTAL | | All overland $t_c$ < 10 min. |
| | | | LOWER | | | | | | |
| Ricardo Way | | | 1 | P5 | 110 | 0.3 | | 10.90 | |
| | | | | | | | | | |
| 40 + 80 | 46.5 | R | 2 | | | 0.05 | | | |
| Ricardo Way | | | 2 | P5 | 200 | 0.2 | | 11.10 | |
| | | | | | | | | | |
| 41 + 25 | 46.5 | L | 3 | | | 0.03 | | | |
| Ricardo Way | | | 3 | MH | 200 | --- | | 11.40 | |
| | | | | | | | | | |
| 43 + 25 | 44 | L | 4 | | | --- | | | |
| Ricardo Way | | | 4 | P5 | 100 | 0.4 | | 11.10 | |
| | | | | | | | | | |
| 45 + 25 | 46.5 | L | 7 | | | 0.1 | | | |
| Ricardo Way | | | 5 | P5 | 110 | 0.4 | | 10.90 | |
| | | | | | | | | | |
| 46 + 00 | 46.5 | R | 7 | | | 0.5 | | | |
| Frank Blvd | | | 7 | MH | 25 | --- | | 10.50 | |
| | | | | | | | | | |
| 30 + 50 | 10 | R | 8 | | | --- | | | |
| Frank Blvd | | | 6 | P5 | 32 | 0.15 | | 9.60 | |
| | | | | | | | | | |
| 30 + 75 | 16 | L | 8 | | | 0.5 | | | |
| Frank Blvd | | | 8 | P5 | 250 | 0.25 | | 9.60 | |
| | | | | | | | | | |
| 30 + 75 | 16 | R | outlet | | | 0.5 | | | |

Find:

- The pipe sizes to meet the standard hydraulic gradient clearance of 1.13 feet to the inlet (edge of pavement) elevation

1. Add the areas that contribute flow to each pipe segment. For each of the pipe segments, the total impervious area (C = 0.95) is obtained as follows.

Total Area $P_{1\text{-}2}$ = Inc. $Areas_{S\text{-}1}$; no upstream pipes
= 0.3 ac.
Total Area $P_{2\text{-}3}$ = Inc. $Areas_{S\text{-}2}$ + Total Area $P_{1\text{-}2}$
= 0.2 + 0.3 = 0.5 ac
Total Area $P_{3\text{-}4}$ = Inc. $Areas_{S\text{-}3}$ + Total Area $P_{2\text{-}3}$
= 0.0 + 0.5 = 0.5 ac
Total Area $P_{4\text{-}7}$ = Inc. $Areas_{S\text{-}4}$ + Total Area $P_{3\text{-}4}$
= 0.4 + 0.5 = 0.9 ac

Total Area $P_{5\text{-}7}$ = Inc. $\text{Area}_{S\text{-}5}$; no upstream pipes
= 0.4 ac.
Total Area $P_{7\text{-}8}$ = Inc. $\text{Area}_{S\text{-}7}$ + Tot. Area $P_{4\text{-}7}$ + Tot. Area $P_{5\text{-}7}$
= 0.0 + 0.9 + 0.4 = 1.3 ac
Total Area $P_{6\text{-}8}$ = Inc. $\text{Area}_{S\text{-}6}$; no upstream pipes
= 0.15 ac.
Total Area $P_{8\text{-out}}$= Inc. $\text{Area}_{S\text{-}8}$ + Tot. Area $P_{7\text{-}8}$ + Tot. Area $P_{6\text{-}8}$
= 0.25 + 1.3 + 0.15 = 1.7 ac

The same approach is applied to drainage areas associated with the pervious runoff coefficient. Table 6.6-2 is a partial tabulation form with the above information.

**Table 6.6-2: Partially Completed Tabulation Form with Data from Example**

| STRUCTURE NO. | DRAINAGE AREA (ac.) | |
|---|---|---|
| | C= 0.95 | |
| | C= | |
| | C= 0.20 | |
| UPPER / LOWER | INCREMENT | TOTAL |
| 1 | 0.3 | 0.3 |
| | | |
| 2 | 0.05 | 0.05 |
| 2 | 0.2 | 0.5 |
| | | |
| 3 | 0.03 | 0.08 |
| 3 | --- | 0.5 |
| | | |
| 4 | --- | 0.08 |
| 4 | 0.4 | 0.9 |
| | | |
| 7 | 0.1 | 0.18 |
| 5 | 0.4 | 0.4 |
| | | |
| 7 | 0.5 | 0.5 |
| 7 | --- | 1.3 |
| | | |
| 8 | --- | 0.68 |
| 6 | 0.15 | 0.15 |
| | | |
| 8 | 0.5 | 0.5 |
| 8 | 0.25 | 1.7 |
| | | |
| outlet | 0.5 | 1.68 |

2. For each pipe section, multiply the total area associated with each runoff coefficient by the corresponding runoff coefficient to obtain the subtotal CA values.

   $P_{1\text{-}2}$: 0.3 x 0.95 = 0.29
   0.05 x 0.20 = 0.01
   $P_{2\text{-}3}$: 0.5 x 0.95 = 0.48
   0.08 x 0.20 = 0.02
   Etc.

3. For each pipe section, add the subtotal CA values to obtain the Total CA.

   $P_{1\text{-}2}$: 0.29 +0.01 = 0.3
   $P_{2\text{-}3}$: 0.48 + 0.02 = 0.5
   Etc.

Table 6.6-3 is a partial tabulation form with the above information.

ESTIMATE A PRELIMINARY HYDRAULIC GRADE LINE (HGL) SLOPE

4. Determine the design TW.
   The crown of the outlet pipe is not known at this time, so we will use the given information about the stage (8.3 feet) of the stormwater management facility.
5. Assume which inlet will be critical.
   For this example, we will assume that S-1 is critical. Elevation S-1 = 10.9 feet
6. Calculate a preliminary HGL slope.
   For this example, we will base the preliminary HGL slope on the following formula.

**Table 6.6-3: Partially Completed Tabulation Form with Data from Example**

| STRUCTURE NO. | DRAINAGE AREA (ac.) | | SUB-TOTAL C •A | TOTAL C •A |
|---|---|---|---|---|
| UPPER | C= 0.95 | | | |
| | C= | | | |
| | C= 0.20 | | | |
| LOWER | INCREMENT | TOTAL | | |
| 1 | 0.3 | 0.3 | 0.29 | **0.30** |
| | | | | |
| 2 | 0.05 | 0.05 | 0.01 | |
| 2 | 0.2 | 0.5 | 0.48 | **0.50** |
| | | | | |
| 3 | 0.03 | 0.08 | 0.02 | |
| 3 | --- | 0.5 | 0.48 | **0.50** |
| | | | | |
| 4 | --- | 0.08 | 0.02 | |
| 4 | 0.4 | 0.9 | 0.86 | **0.9** |
| | | | | |
| 7 | 0.1 | 0.18 | 0.04 | |
| 5 | 0.4 | 0.4 | 0.38 | **0.48** |
| | | | | |
| 7 | 0.5 | 0.5 | 0.1 | |
| 7 | --- | 1.3 | 1.24 | **1.38** |
| | | | | |
| 8 | --- | 0.68 | 0.14 | |
| 6 | 0.15 | 0.15 | 0.14 | **0.24** |
| | | | | |
| 8 | 0.5 | 0.5 | 0.1 | |
| 8 | 0.25 | 1.7 | 1.62 | **1.96** |
| | | | | |
| outlet | 0.5 | 1.68 | 0.34 | |

$$\text{Slope} = \frac{\text{Critical Inlet Elev - DTW - 1 foot}}{\text{System Length between Outlet \& Critical Inlet}}$$

The total pipe length is best seen in Figure 6.6-1.
110 + 200 + 200 + 100 + 25 + 250 = 885 feet
Prelim HGL Slope = 10.9 – 8.3 – 1/885
= 0.0018 ft/ft
= 0.18%

CALCULATE RUNOFF FLOW RATES (FIRST PASS DOWN THE SYSTEM)

Starting with pipe section $P_{1-2}$,

7. Determine the time of concentration. [$P_{1-2}$]
   Since this is the first inlet in the system and it has an overland $t_c$ of 10 minutes or less, use 10-minute minimum.

8. Determine the intensity. [$P_{1-2}$]
   From the NOAA Atlas 14 data, the 10-minute intensity is 6.5 in/hr.

9. Calculate the total runoff for the pipe section. [$P_{1-2}$]
   Q = Total CA (Step 3) times the intensity (previous step)
   Q = 0.3 x 6.5 = 1.95 cfs

10. Determine pipe size. [$P_{1-2}$]
    For the first pass, we assume full flow.
    Using the hydraulic calculator, an 18-inch pipe is acceptable because the friction slope (0.03 percent) is flatter than the preliminary HGL slope (0.18 percent). The minimum physical slope is 0.15 percent (see discussion in Chapter 6.4.2.1). Record the pipe size, and the minimum physical slope. Also, record the full-flow friction slope as the HGL slope. Although it is not necessary to record the HGL slope at this step, it will be used later when moving up the system and calculating the hydraulic gradient. It may save time to record this while the hydraulic calculator is set for the flow and pipe size.

11. Determine the pipe flow lines, physical slope, and fall. [$P_{1-2}$]
    For this example, we will use 4.5 feet clearance between the inlet (edge of pavement) elevation and the flow line of an 18-inch pipe. (The minimum clearance for an 18-inch pipe in a precast Type P-5 structure is 4.2 feet. See Table 6.4-5.) Then:
    Upper-End Flow Line = 10.9 – 4.5 = 6.4 feet

    For this example, we will assume there are no constraints such as utilities that would prevent the pipe from being set at the minimum physical pipe slope (0.15

percent). Then:
Minimum pipe fall = 110 ft x 0.15% = 0.17 feet
Pipe fall = 0.2 ft (minimum fall rounded up to nearest 0.1 foot)
Physical Slope = 0.2 ft/110 ft = 0.18 percent
Lower End Flow Line = 6.4 – 0.2 = 6.2 feet

| |
|---|
| If this were an actual project, you also should check that the minimum cover heights in Appendix C of the *Drainage Manual* are satisfied. To simplify this example, we will assume that adequate cover is provided. |

12. Calculate the actual flow velocity. [$P_{1-2}$]
    Vel = Q/A = 1.95 cfs/πD$^2$/4)
    The full-flow cross sectional area is used for the first pass down the system. This is reasonable for a flat system like this example. If you know the pipe is flowing partially full, use the average cross sectional flow area. See the discussion on page 56 and the next example.

    Using the hydraulic calculator, the velocity of 1.95 cfs flowing full through an 18-inch pipe is 1.1 fps.

    Calculate the physical velocity. [$P_{1-2}$]
    Using the hydraulic calculator, the full-flow velocity for an 18-inch pipe sloped at 0.18 percent = 2.7 fps

13. Calculate the time of flow in pipe section. [$P_{1-2}$]
    Time = Length/Actual Velocity
    = 110 ft/1.1 fps = 100 seconds = 1.7 minutes

A partially completed tabulation form is shown in Table 6.6-4.

**Table 6.6-4: Partially Completed Tabulation Form with Data from Example**

| STRUCTURE NUMBER (UPPER / LOWER) | TYPE OF STRUCTURE | LENGTH (ft) | TIME OF CONCEN-TRATION (min) | TIME OF FLOW IN SECTION (min) | INTENSITY (in/hr) | TOTAL C • A | TOTAL FLOW (cfs) | INLET ELEV. (ft) | HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV. (ft) | LOWER END ELEV. (ft) | FALL (ft) | PIPE SIZE (IN.) RISE / SPAN | SLOPE (%) HYD. GRAD. / PHYSICAL / MIN. PHYS. | ACTUAL VELOCITY (FPS) / PHYSICAL VELOCITY (FPS) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | P5 | 110 | 10 | 1.7 | 6.5 | 0.30 | 1.95 | 10.90 | | | | 18 | .03 | 1.1 |
| | | | | | | | | | 7.9 | 7.7 | 0.2 | | .18 | |
| 2 | | | | | | | | | 6.4 | 6.2 | | 18 | .15 | 2.7 |

For pipe section $P_{2-3}$,

14. Determine the Time of Concentration. [$P_{2-3}$]
    $t_c$ overland ≤ 10 min.
    $t_c$ system = 10 + 1.7 = 11.7 min. therefore
    $t_c$ = 11.7 min.

15. Determine the intensity. [$P_{2-3}$]
    From the  from NOAA Atlas 14 data, the 11.7-minute intensity is 6.1 in/hr.

16. Calculate the total runoff for the pipe section. [$P_{2-3}$]
    Flow rate = Total CA x Intensity
    = 0.5 x 6.1 = 3.1

17. Determine pipe size. [$P_{2-3}$]
    Using the hydraulic calculator, an 18-inch pipe is acceptable because the friction slope (0.07 percent) is less than the preliminary HGL slope (0.18 percent). As done for the previous pipe section, record the pipe size, and the minimum physical slope. Also, record the friction slope as the HGL slope.

18. Determine the pipe flow lines, physical slope, and fall. [$P_{2-3}$]
    Since this inlet is higher than S-1, the potential conflict with the inlet top will not control the flow lines. For this example, we will attempt to match flow line elevations across structures. Therefore:

| | |
|---|---|
| Upper-end flow line | = 6.2 (previous pipe section downstream flow line) |
| Minimum pipe fall | = length x min. phys. slope |
| | = 200 ft x 0.15% = 0.3 ft |
| Pipe fall | = 0.3 ft |
| Physical slope | = 0.3 ft/200 ft = 0.15% |
| Lower-end flow line | = 6.2 – 0.3 = 5.9 feet |

19. Calculate the actual flow velocity. [$P_{2\text{-}3}$]
    Vel = Q/A = 1.95 cfs/(π $D^2$/4).
    The full-flow cross sectional area is used for the first pass through the system. Using the hydraulic calculator, the velocity of 3.1 cfs flowing full through an 18-inch pipe is approximately 1.8 fps.

    Calculate the physical velocity. [$P_{2\text{-}3}$]
    Using the hydraulic calculator, the full-flow velocity for an 18-inch pipe sloped at 0.15 percent = 2.5 fps

20. Calculate the time of flow in pipe section. [$P_{2\text{-}3}$]
    Time = Length/Actual Velocity
    = 200 ft/1.8 fps = 111 seconds = 1.9 minutes

A partially completed tabulation form is shown in Table 6.6-5.

**Table 6.6-5: Partially Completed Tabulation Form with Data from Example**

| STRUCTURE NO. UPPER / LOWER | TYPE OF STRUCTURE | LENGTH (ft) | TIME OF CONCEN-TRATION (min) | TIME OF FLOW IN SECTION (min) | INTENSITY (in/hr) | TOTAL C • A | TOTAL FLOW (cfs) | INLET ELEV. (ft) | HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV. (ft) | HYD. GRADIENT / CROWN / FLOW LINE: LOWER END ELEV. (ft) | FALL (ft) | PIPE SIZE (IN) / RISE / SPAN | SLOPE (%): HYD. GRAD. / PHYSICAL / MIN. PHYS. | ACTUAL VELOCITY (FPS) / PHYSICAL VELOCITY (FPS) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | P5 | 110 | 10 | 1.7 | 6.5 | 0.30 | 1.95 | 10.90 | | | | 18 | 0.03 | 1.1 |
| | | | | | | | | | 7.9 | 7.7 | 0.2 | | 0.18 | |
| 2 | | | | | | | | | 6.4 | 6.2 | | 18 | 0.15 | 2.7 |
| 2 | P5 | 200 | 11.7 | 1.9 | 6.1 | 0.50 | 3.1 | 11.10 | | | | 18 | 0.07 | 1.8 |
| | | | | | | | | | 7.7 | 7.4 | 0.3 | | 0.15 | |
| 3 | | | | | | | | | 6.2 | 5.9 | | 18 | 0.15 | 2.5 |

Step 14 through Step 20 are repeated for the remaining pipe sections. Situations that are different from the above pipe sections are discussed below. Table 6.6-6 shows the results of doing these steps for the entire system.

Pipe Section $P_{3\text{-}4}$

The manhole contributes no additional flow to the system, nor does it combine flow from several pipes. The time of concentration and the intensity are not applicable. The flow through the pipe section is the same as the upstream pipe section.

Pipe Section $P_{4\text{-}7}$,

The time of concentration is 11.7 + 1.9 +1.9 = 15.5 minutes.

Pipe Sections $P_{5-7}$ and $P_{6-8}$

They receive only overland flow like pipe section $P_{1-2}$, thus their time of concentration is based on overland flow time. Their flow lines are determined by matching the flow lines of the downstream structure, using the minimum physical slope to the upstream structure, and rounding to the nearest 0.1' such that the minimum slope is maintained.

Pipe Section $P_{7-8}$

This is similar to similar to pipe section $P_{3-4}$ in that the manhole contributes no flow. It is different from pipe section $P_{3-4}$ in that two pipes drain to the manhole. Because of this difference, the pipe section is treated like the other inlets along the main line. The $t_c$, intensity, and flow are calculated for the section. The time of concentration is 15.5 + 0.6 = 16.1 minutes.

As stated in Step 18, we will attempt to match flow lines across structures for this example. The upper-end flow line is set to match the lower-end flow line of pipe section $P_{4-7}$. The lower-end flow line is set to match the flow line of S-8.

Pipe Section $P_{8\text{-out}}$

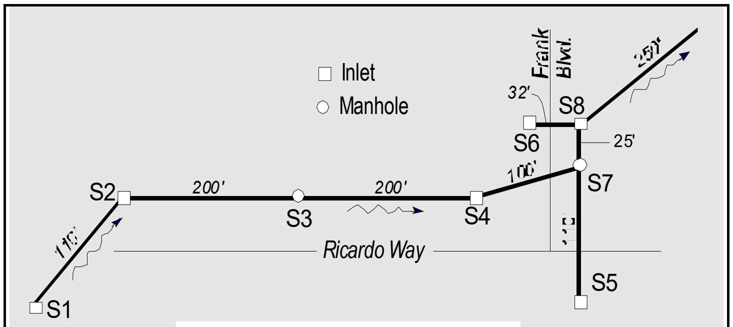
For this example, we will use a 5.1-foot clearance between the inlet (edge of pavement) elevation and the flow line of a 24-inch pipe. (The minimum clearance for a 24-inch pipe in a precast Type P-5 structure is 4.7 feet. See Table 6.4-5.) Then, upper-end FL = 9.6 – 5.1 = 4.5 feet. The lower-end FL is set to match the minimum physical slope with the FL rounded to the closest 0.1 foot such that the minimum slope is maintained.

## Table 6.6-6: Results of First Pass Down the System

| STRUCTURE NO. (UPPER / LOWER) | TYPE OF STRUCTURE | LENGTH (ft) | TIME OF CONCENTRATION (min) | TIME OF FLOW IN SECTION (min) | INTENSITY (in/hr) | TOTAL C • A | TOTAL FLOW (cfs) | INLETELEV. (ft) | HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV. (ft) | HYD. GRADIENT / CROWN / FLOW LINE: LOWER END ELEV. (ft) | FALL (ft) | PIPE SIZE (IN): RISE / SPAN | SLOPE (%): HYD. GRAD. / PHYSICAL / MIN. PHYS. | ACTUAL VELOCITY (FPS) / PHYSICAL VELOCITY (FPS) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | P5 | 110 | 10 | 1.7 | 6.5 | 0.30 | 1.95 | 10.90 | | | | 18 | .03 | **1.1** |
| | | | | | | | | | 7.9 | 7.7 | 0.2 | | .18 | |
| 2 | | | | | | | | | 6.4 | 6.2 | | 18 | .15 | **2.7** |
| 2 | P5 | 200 | 11.7 | 1.9 | 6.1 | 0.50 | 3.1 | 11.10 | | | | 18 | 0.07 | **1.8** |
| | | | | | | | | | 7.7 | 7.4 | 0.3 | | 0.15 | |
| 3 | | | | | | | | | 6.2 | 5.9 | | 18 | 0.15 | **2.5** |
| 3 | MH | 200 | N/A | 1.9 | N/A | 0.50 | 3.1 | 11.40 | | | | 18 | 0.07 | **1.8** |
| | | | | | | | | | 7.4 | 7.1 | 0.3 | | 0.15 | |
| 4 | | | | | | | | | 5.9 | 5.6 | | 18 | 0.15 | **2.5** |
| 4 | P5 | 100 | 15.5 | 0.6 | 5.4 | 0.9 | 4.9 | 11.10 | | | | 18 | 0.18 | **2.8** |
| | | | | | | | | | 7.1 | 6.9 | 0.2 | | 0.2 | |
| 7 | | | | | | | | | 5.6 | 5.4 | | 18 | 0.15 | **2.9** |
| 5 | P5 | 110 | 10 | 1.7 | 6.5 | 0.48 | 3.1 | 10.90 | | | | 18 | 0.07 | **1.8** |
| | | | | | | | | | 7.1 | 6.9 | 0.2 | | 0.18 | |
| 7 | | | | | | | | | 5.6 | 5.4 | | 18 | 0.15 | **2.7** |
| 7 | MH | 25 | 16.1 | 0.2 | 5.3 | 1.38 | 7.3 | 10.50 | | | | 24 | 0.09 | **2.4** |
| | | | | | | | | | 7.4 | 6.5 | 0.9 | | 3.6 | |
| 8 | | | | | | | | | 5.4 | 4.5 | | 24 | 0.1 | **14.8** |
| 6 | P5 | 32 | 10 | 0.4 | 6.5 | 0.24 | 1.56 | 9.60 | | | | 18 | 0.02 | **1.3** |
| | | | | | | | | | 6.1 | 6.0 | 0.1 | | 0.3 | |
| 8 | | | | | | | | | 4.6 | 4.5 | | 18 | 0.15 | **3.5** |
| 8 | P5 | 250 | 16.3 | 1.9 | 5.3 | 1.96 | 10.4 | 9.60 | | | | 24 | 0.18 | **3.4** |
| | | | | | | | | | 6.5 | 6.2 | 0.3 | | 0.12 | |
| outlet | | | | | | | | | 4.5 | 4.2 | | 24 | 0.1 | **2.7** |

21. Check for peak flow from reduced area.
Reviewing the size and runoff coefficient for each drainage area, it does not appear that most of the area or most of the imperviousness is concentrated near the lower end of the system. Since we would not expect to have peak flow from reduced area, detailed calculations would not be necessary. For this example, we will check it just to demonstrate an approach.

From the schematic, it appears that a logical reduced area would be area flowing overland to S4, S5, S6, and S8. The overland $t_c$ to S4 was given as 10 minutes. So let's apply a 10-minute storm to pipe section $P_{4-7}$. Doing so reduces the contributing area from S3.



Schematic

An approach to finding the reduced contributing area is to multiply the area (or the CA product) from S3 by the ratio of the times of concentration. From Table 6.6-6, the $t_c$ for the flow from S3 is 15.5 minutes. So, reduce the Total CA from S3 by the ratio 10/15.5, or 0.65.

From Table 6.6-6, the Total CA from S3 = 0.5.

So the Total CA from S3 <u>is reduced by</u>: 0.5 (1 - 0.65) = 0.18 This value is not the reduced CA; it is the amount the Total CA is reduced by.

Reducing the Total CA for pipe section $P_{4-7}$ by this amount yields: 0.9 – 0.18 = 0.72

The five-year intensity for a 10-minute storm = 6.5 in/hour.

The flow in the pipe downstream of S4 = CAi = 0.72 x 6.5 = 4.7 cfs.

This is less than the 4.9 cfs calculated for the entire contributing area for $P_{4-7}$, as shown in Table 6.6-6. So, peak flow in pipe section $P_{4-7}$ does not result from reduced area. Although other pipe sections could be checked for peak flow from reduced area, this effort shown above is acceptable for this system.

CALCULATE THE HYDRAULIC GRADE LINE ELEVATION (first pass up the system)

For pipe section $P_{8\text{-}out}$:

22. Determine the lower-end HG elevation. [$P_{8\text{-}out}$]
For the outlet pipe, the lower-end HG is the design tailwater. For this example, the design tailwater is the higher of (1) the crown of the pipe (elev. 6.2 feet), or (2) the peak stage of the stormwater facility during the storm drain design event (elev. 8.3 feet). Thus, the lower-end HG = 8.3 feet

23. Determine the upper-end HG elevation. [$P_{8\text{-}out}$]
For this example, the lower-end HG submerges the entire pipe section; therefore,

Upper-end HG elev. = Lower-end HG elev. + Full-flow friction loss
& Full-Flow Friction Loss = Full-flow friction slope x Pipe length
= 0.18% x 250 ft = 0.45 feet

The full-flow friction slope was previously recorded as the hydraulic gradient slope in Table 6.6-6 when we moved down the system calculating flow rates.

Then upper-end HG elev. = Lower-end HG elev. + Full-flow friction loss
= 8.3 + 0.45 = 8.75 feet (see table below)

**Table 6.6-7**

<table>
<tr><th>STRUCTURE NO.</th><th rowspan="3">LENGTH (ft)</th><th colspan="3">HYD. GRADIENT<br>CROWN<br>FLOW LINE</th><th>SLOPE (%)</th></tr>
<tr><th>UPPER</th><th rowspan="2">UPPER ELEV (ft)</th><th rowspan="2">LOWER ELEV (ft)</th><th rowspan="2">FALL (ft)</th><th>HYD. GRAD.<br>PHYSICAL</th></tr>
<tr><th>LOWER</th><th>MIN. PHYS.</th></tr>
<tr><td rowspan="2">8</td><td rowspan="3">250</td><td>8.75</td><td>8.3</td><td>0.45</td><td>0.18</td></tr>
<tr><td>6.5</td><td>6.2</td><td rowspan="2">0.3</td><td>0.12</td></tr>
<tr><td>outlet</td><td>4.5</td><td>4.2</td><td>0.1</td></tr>
</table>

Pipe sections $P_{6\text{-}8}$ and $P_{5\text{-}7}$ are stubs and their hydraulic gradient will not be calculated during the first pass up the system.

For pipe section $P_{7\text{-}8}$:

24. Determine the lower-end HG elevation.
The downstream pipe upper-end HG elevation (8.75 feet) is higher than the lower-end Crown Elevation (6.5 feet); therefore, the lower-end HG elevation = downstream pipe upper-end HG = 8.75 feet

25. Determine the upper-end HG elevation. [$P_{7\text{-}8}$]
For this example, the lower-end HG submerges the entire pipe section; therefore:
Upper-end HG elev. = Lower-end HG elev. + Full-flow friction loss
Full-flow friction loss = Full-flow friction slope x Pipe length
= 0.09% x 25 ft = 0.02 feet
The full-flow friction slope was recorded previously as the hydraulic gradient slope in Table 6-6.
Then, upper-end HG elev. = Lower-end HG elev. + Full-flow friction loss
= 8.75 + 0.02 = 8.77 feet

The same steps are repeated for the remaining mainline pipe sections. Table 6.6-8 shows the results of doing these steps for the entire system.

## Table 6.6-8: Results of First Pass Up the System

| STRUCTURE NO. (UPPER / LOWER) | LENGTH (ft) | TOTAL FLOW (cfs) | INLET ELEV. (ft) | HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV (ft) | LOWER END ELEV (ft) | FALL (ft) | PIPE SIZE (IN): RISE / SPAN | SLOPE (%): HYD. GRAD. / PHYSICAL / MIN. PHYS. |
|---|---|---|---|---|---|---|---|---|
| 1 | 110 | 1.95 | 10.90 | 9.26 | 9.23 | 0.03 | 18 | 0.03 |
| | | | | 7.9 | 7.7 | 0.2 | | 0.18 |
| 2 | | | | 6.4 | 6.2 | | 18 | 0.15 |
| 2 | 200 | 3.1 | 11.10 | 9.23 | 9.09 | 0.14 | 18 | 0.07 |
| | | | | 7.7 | 7.4 | 0.3 | | 0.15 |
| 3 | | | | 6.2 | 5.9 | | 18 | 0.15 |
| 3 | 200 | 3.1 | 11.40 | 9.09 | 8.95 | 0.14 | 18 | 0.07 |
| | | | | 7.4 | 7.1 | 0.3 | | 0.15 |
| 4 | | | | 5.9 | 5.6 | | 18 | 0.15 |
| 4 | 100 | 4.9 | 11.10 | 8.95 | 8.77 | 0.18 | 18 | 0.18 |
| | | | | 7.1 | 6.9 | 0.2 | | 0.20 |
| 7 | | | | 5.6 | 5.4 | | 18 | 0.15 |
| 5 | 110 | 3.1 | 10.90 | | | | 18 | 0.07 |
| | | | | 7.1 | 6.9 | 0.2 | | 0.18 |
| 7 | | | | 5.6 | 5.4 | | 18 | 0.15 |
| 7 | 25 | 7.3 | 10.50 | 8.77 | 8.75 | 0.02 | 24 | 0.09 |
| | | | | 7.4 | 6.5 | 0.9 | | 3.60 |
| 8 | | | | 5.4 | 4.5 | | 24 | 0.10 |
| 6 | 32 | 1.56 | 9.60 | | | | 18 | 0.02 |
| | | | | 6.1 | 6.0 | 0.1 | | 0.30 |
| 8 | | | | 4.6 | 4.5 | | 18 | 0.15 |
| 8 | 250 | 10.4 | 9.60 | 8.75 | 8.3 | .45 | 24 | 0.18 |
| | | | | 6.5 | 6.2 | 0.3 | | 0.12 |
| outlet | | | | 4.5 | 4.2 | | 24 | 0.10 |

26. Compare the hydraulic gradient to the standard and adjust pipe sizes.
    The standard clearance of 1.13 feet between the hydraulic gradient and the inlet elevation (edge of pavement) is not met for S-8 and probably not met for S-6. The remaining inlets have adequate clearance. Increasing the size of the outlet pipe $P_{8\text{-}out}$ to 30 inches will reduce the hydraulic gradient at S-6 and S-8, so we will try that. To reduce costs, we also will try reducing the pipe size of section $P_{7\text{-}8}$ to 18 inches.

27. Calculate the hydraulic gradient using the changed pipe sizes.
    Table 6.6-9 shows the new slopes and the recalculated hydraulic gradient for the entire system.

**Table 6.6-9**: **Results of Second Pass Up the System**

| STRUCTURE NO. | LENGTH (ft) | TOTAL FLOW (cfs) | INLET ELEV. (ft) | HYD. GRADIENT / CROWN / FLOW LINE | | | PIPE SIZE (IN) | SLOPE (%) |
|---|---|---|---|---|---|---|---|---|
| UPPER / LOWER | | | | UPPER END ELEV (ft) | LOWER END ELEV (ft) | FALL (ft) | RISE / SPAN | HYD. GRAD. / PHYSICAL / MIN. PHYS. |
| 1 | 110 | 1.95 | 10.90 | 9.04 | 9.01 | 0.03 | 18 | 0.03 |
| | | | | 7.9 | 7.7 | 0.20 | | 0.18 |
| 2 | | | | 6.4 | 6.2 | | 18 | 0.15 |
| 2 | 200 | 3.1 | 11.10 | 9.01 | 8.87 | 0.14 | 18 | 0.07 |
| | | | | 7.7 | 7.4 | 0.30 | | 0.15 |
| 3 | | | | 6.2 | 5.9 | | 18 | 0.15 |
| 3 | 200 | 3.1 | 11.40 | 8.87 | 8.73 | 0.14 | 18 | 0.07 |
| | | | | 7.4 | 7.1 | 0.30 | | 0.15 |
| 4 | | | | 5.9 | 5.6 | | 18 | 0.15 |
| 4 | 100 | 4.9 | 11.10 | 8.73 | 8.55 | 0.18 | 18 | 0.18 |
| | | | | 7.1 | 6.9 | 0.20 | | 0.20 |
| 7 | | | | 5.6 | 5.4 | | 18 | 0.15 |
| 5 | 110 | 3.1 | 10.90 | 8.63 | 8.55 | 0.08 | 18 | 0.07 |
| | | | | 7.1 | 6.9 | 0.20 | | 0.18 |
| 7 | | | | 5.6 | 5.4 | | 18 | 0.15 |
| 7 | 25 | 7.3 | 10.50 | 8.55 | 8.45 | 0.10 | 18 | 0.40 |
| | | | | 6.9 | 6.0 | 0.90 | | 3.60 |
| 8 | | | | 5.4 | 4.5 | | 18 | 0.15 |
| 6 | 32 | 1.56 | 9.60 | 8.46 | 8.45 | 0.01 | 18 | 0.02 |
| | | | | 6.1 | 6.0 | 0.10 | | 0.30 |
| 8 | | | | 4.6 | 4.5 | | 18 | 0.15 |
| 8 | 250 | 10.4 | 9.60 | 8.45 | 8.3 | 0.15 | 30 | 0.06 |
| | | | | 7.0 | 6.8 | 0.20 | | 0.08 |
| outlet | | | | 4.5 | 4.3 | | 30 | 0.08 |

28. Compare the HG to the standard and adjust pipe sizes.
    From Table 6.6-9, the standard 1.13 feet of clearance between the hydraulic gradient and the inlet elevation (edge of pavement) exists throughout the system.

29. Recalculate the flow.
    Changing pipe sizes changes the velocity, thus changing the time of flow in the section and the time of concentration. These changes affect only the changed pipes and the pipes downstream of the changed pipes. For this example, only pipe sections $P_{7\text{-}8}$ and $P_{8\text{-out}}$ are affected.

    The increased velocity in pipe section $P_{7\text{-}8}$ reduced the time of flow in the pipe by only 0.1 minute because the pipe is so short. As a result, the time of concentration of the outlet pipe was reduced by only 0.1 minute from 16.3 to 16.2 minutes. The change in the intensity for a change in $t_c$ of 0.1 minute is negligible. Although we changed the size of the two pipes, there was no noticeable change to the flow rate in the system.

A completed tabulation form is shown in Table 6.6-10.

## Example 6.6-1

### Table 6.6-10: Completed Tabulation Form For Example 6.6-1

| LOCATION OF UPPER END<br>ALIGNMENT NAME | | | STRUCTURE NO. | TYPE OF STRUCTURE | LENGTH (ft) | DRAINAGE AREA (ac) | | C • A SUBTOTAL | TIME OF CONCENTRATION (MIN) | TIME OF FLOW IN SECTION (MIN) | INTENSITY (in/hr) | C • A TOTAL | TOTAL FLOW (cfs) | INLET ELEV. (ft) | HYD. GRADIENT<br>CROWN<br>FLOW LINE | | | PIPE SIZE (IN)<br>RISE<br>SPAN | SLOPE (%)<br>HYD. GRAD<br>PHYSICAL<br>MIN. PHYS. | ACTUAL VELOCITY(FPS)<br>PHYSICAL VELOCITY (FPS) | NOTES AND REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| STATION | DISTANCE (ft) | SIDE | UPPER<br>LOWER | | | C = **0.95**<br>C =<br>C = **0.20**<br>INCRE-MENT | TOTAL | | | | | | | | UPPER END ELEV (ft) | LOWER END ELEV (ft) | FALL (ft) | | | | FREQUENCY (Yrs) **: 5**<br>MANNING'S "n" : 0.012<br>TAILWATER EL. (ft): 8.3<br>All overland $t_c$ < 10 Min. |
| Ricardo Way | | | 1 | P5 | 110 | 0.3 | 0.3 | 0.29 | 10 | 1.7 | 6.5 | 0.3 | 1.95 | 10.90 | 9.04 | 9.01 | 0.03 | 18 | 0.03 | 1.1 | |
| | | | | | | | | | | | | | | | 7.9 | 7.7 | 0.2 | | 0.18 | | |
| 40 + 80 | 46.5 | R | 2 | | | 0.05 | 0.05 | 0.01 | | | | | | | 6.4 | 6.2 | | | 0.15 | 2.7 | |
| Ricardo Way | | | 2 | P5 | 200 | 0.2 | 0.5 | 0.48 | 11.7 | 1.9 | 6.1 | 0.5 | 3.1 | 11.10 | 9.01 | 8.87 | 0.14 | 18 | 0.07 | 1.8 | |
| | | | | | | | | | | | | | | | 7.7 | 7.4 | 0.3 | | 0.15 | | |
| 41 + 25 | 46.5 | L | 3 | | | 0.03 | 0.08 | 0.02 | | | | | | | 6.2 | 5.9 | | | 0.15 | 2.5 | |
| Ricardo Way | | | 3 | MH | 200 | --- | 0.5 | 0.48 | N/A | 1.9 | N/A | 0.5 | 3.1 | 11.40 | 8.87 | 8.73 | 0.14 | 18 | 0.07 | 1.8 | |
| | | | | | | | | | | | | | | | 7.4 | 7.1 | 0.3 | | 0.15 | | |
| 43 + 25 | 44 | L | 4 | | | --- | 0.08 | 0.02 | | | | | | | 5.9 | 5.6 | | | 0.15 | 2.5 | |
| Ricardo Way | | | 4 | P5 | 100 | 0.4 | 0.9 | 0.86 | 15.5 | 0.6 | 5.4 | 0.9 | 4.9 | 11.10 | 8.73 | 8.55 | 0.18 | 18 | 0.18 | 2.8 | |
| | | | | | | | | | | | | | | | 7.1 | 6.9 | 0.2 | | 0.2 | | |
| 45 + 25 | 46.5 | L | 7 | | | 0.1 | 0.18 | 0.04 | | | | | | | 5.6 | 5.4 | | | 0.15 | 2.9 | |
| Ricardo Way | | | 5 | P5 | 110 | 0.4 | 0.4 | 0.38 | 10 | 1.0 | 6.5 | 0.48 | 3.1 | 10.90 | 8.63 | 8.55 | 0.08 | 18 | 0.07 | 1.8 | |
| | | | | | | | | | | | | | | | 7.1 | 6.9 | 0.2 | | 0.18 | | |
| 46 + 00 | 46.5 | R | 7 | | | 0.5 | 0.5 | 0.1 | | | | | | | 5.6 | 5.4 | | | 0.15 | 2.7 | |
| Frank Blvd. | | | 7 | MH | 25 | --- | 1.3 | 1.24 | 16.1 | 0.1 | 5.3 | 1.38 | 7.3 | 10.50 | 8.55 | 8.45 | 0.1 | 18 | 0.40 | 4.1 | |
| | | | | | | | | | | | | | | | 6.9 | 6.0 | 0.9 | | 3.6 | | |
| 30 + 50 | 10 | R | 8 | | | --- | 0.68 | 0.14 | | | | | | | 5.4 | 4.5 | | | 0.15 | 12.3 | |
| Frank Blvd. | | | 6 | P5 | 32 | 0.15 | 0.15 | 0.14 | 10 | 0.4 | 6.5 | 0.24 | 1.56 | 9.60 | 8.46 | 8.45 | 0.01 | 18 | 0.02 | 1.3 | |
| | | | | | | | | | | | | | | | 6.1 | 6.0 | 0.1 | | 0.3 | | |
| 30 + 75 | 16 | L | 8 | | | 0.5 | 0.5 | 0.1 | | | | | | | 4.6 | 4.5 | | | 0.15 | 3.5 | |
| Frank Blvd | | | 8 | P5 | 250 | 0.25 | 1.7 | 1.62 | 16.2 | 1.9 | 5.3 | 1.96 | 10.4 | 9.60 | 8.45 | 8.3 | 0.15 | 30 | 0.06 | 2.2 | |
| | | | | | | | | | | | | | | | 7.0 | 6.8 | 0.2 | | 0.08 | | |
| 30 + 75 | 16 | R | Outlet | | | 0.5 | 1.68 | 0.34 | | | | | | | 4.5 | 4.3 | | | 0.08 | 2.5 | |

**Example 6.6-2 Steep System—Determining Appropriate Pipe Sizes**

Given:

- Inlets, pipes, runoff coefficients, and details in Figure 6.6-6 and Table 6.6-11
- Designer chooses to match crown elevations across structures

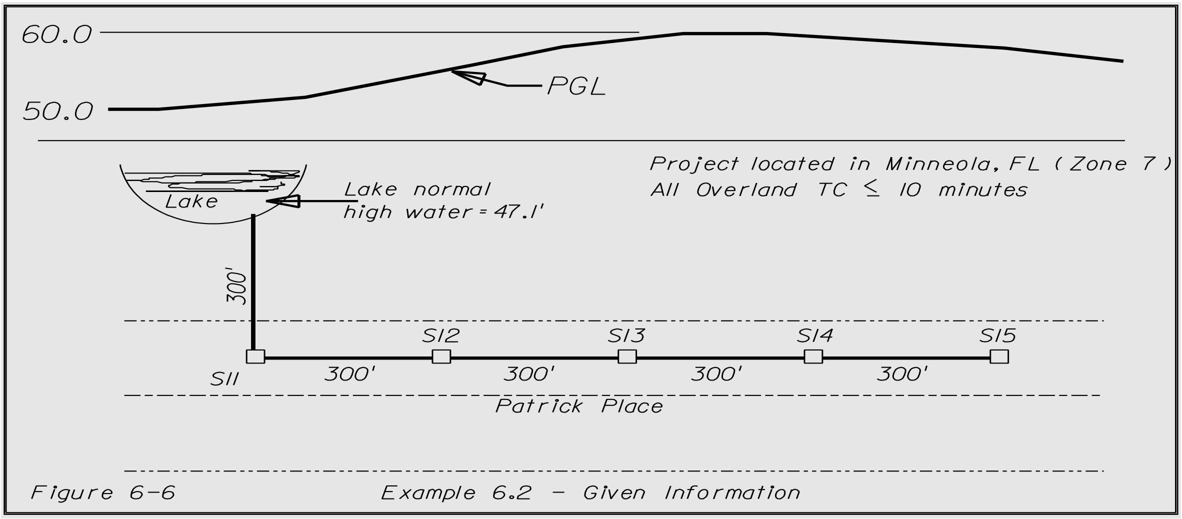



**Figure 6.6-6. Details for Example 6.6-2**

**Table 6.6-11: Data for Example 6.6-2**

| LOCATION OF UPPER END | | | STRUCTURE NO. | TYPE OF STRUCTURE | LENGTH (ft) | DRAINAGE AREA (ac) | | INLET ELEV. (ft) | NOTES AND REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| ALIGNMENT NAME | | | | | | C = | | | |
| | | | | | | C = **0.80** | | | |
| | | | | | | C = | | | FREQUENCY (Yrs): 5 |
| STATION | DISTANCE (ft) | SIDE | | | | INCRE-MENT | TOTAL | | MANNING'S "n": 0.012 |
| | | | | | | | | | TAILWATER EL. (ft): 47.1 |
| | | | UPPER | | | | | | All overland $t_c \le 10$ min. |
| | | | LOWER | | | | | | |
| Patrick Place | | | **15** | **P1** | **300** | **0.2** | | **59.70** | |
| | | | **14** | | | | | | |
| Patrick Place | | | **14** | **P1** | **300** | **1.0** | | **59.80** | |
| | | | **13** | | | | | | |
| Patrick Place | | | **13** | **P1** | **300** | **0.6** | | **59.00** | |
| | | | **12** | | | | | | |
| Patrick Place | | | **12** | **P1** | **300** | **0.5** | | **54.50** | |
| | | | **11** | | | | | | |
| Patrick Place | | | **11** | **P1** | **300** | **1.1** | | **50.50** | |
| | | | **outlet** | | | | | | |

## Example 6.6-2—Given Information

Find:

- The pipe sizes to meet the standard hydraulic gradient clearance of 1.13 feet to the inlet (edge of pavement) elevation.

1. Add the areas that contribute flow to each pipe segment. For each of the pipe segments, the total area is obtained as in Example 6.6-1.

2. For each pipe section, multiply the total area associated with each runoff coefficient by the corresponding runoff coefficient to obtain the subtotal CA values.

3. For each pipe section, add the subtotal CA values to obtain the total CA value.

Table 6.6-12 is a partial tabulation form complete with the information from the first three steps.

**Table 6.6-12: Partially Completed Tabulation Form with Data from Example**

| STRUCTURE NO. | | DRAINAGE AREA (ac) | | SUB-TOTAL C • A | TOTAL C • A |
|---|---|---|---|---|---|
| | | C = | | | |
| | | C = 0.80 | | | |
| | | C = | | | |
| UPPER | LOWER | INCRE-MENT | TOTAL | | |
| 15 | 14 | 0.2 | 0.2 | 0.16 | **0.16** |
| 14 | 13 | 1 | 1.2 | 0.96 | **0.96** |
| 13 | 12 | 0.6 | 1.8 | 1.44 | **1.44** |
| 12 | 11 | 0.5 | 2.3 | 1.84 | **1.84** |
| 11 | outlet | 1.1 | 3.4 | 2.72 | **2.72** |

ESTIMATE A PRELIMINARY HGL SLOPE

4. Determine the design TW.
   The crown of the outlet pipe is not known at this time, so we will use the given information about the lake stage. DTW = 47.1 feet.

5. Assume which inlet will be critical. For this example, we will assume that S-15 is critical.

6. For this example, we will base the preliminary HGL slope on the following formula.

$$\text{Slope} = \frac{\text{Critical Inlet Elev - DTW - 1 foot}}{\text{System Length between Outlet \& Critical Inlet}}$$

Slope = (59.7 – 47.1 – 1)/1,500 = 0.8%

CALCULATE RUNOFF FLOW RATES
FIRST PASS DOWN THE SYSTEM

Starting with pipe section $P_{15\text{-}14}$:

7. Determine the time of concentration. [$P_{15\text{-}14}$]
Since this is the first inlet in the system and it has an overland $t_c$ of 10 minutes or less, use the 10-minute minimum.

8. Determine the intensity. [$P_{15\text{-}14}$]
From the NOAA Atlas 14 data, the 10-minute intensity is 6.5 in/hr.

9. Calculate the flow rate for the pipe section. [$P_{15\text{-}14}$]
Q = Total CA (Step 3) times the intensity (previous step)
Q = 0.16 x 6.5 = 1.04 cfs

10. Determine pipe size. [$P_{15\text{-}14}$]
*For the first pass, we are assuming full flow.*
Using the hydraulic calculator, an 18-inch pipe is acceptable because the friction slope (<0.04 percent) is flatter than the preliminary HGL slope. The minimum physical slope is 0.15 percent (see discussion in Section 6.4.2.1). Record the pipe size and the minimum physical slope. Also, record the full-flow friction slope as the HGL slope. For this flow rate through an 18-inch pipe, the friction loss is so small that the Department's hydraulic calculator does not show the slope. The loss could be calculated from the equation in Section 6.5.1, but for now we will record the HGL slope as zero.

11. Determine the pipe flow lines, physical slope, and fall. [$P_{15\text{-}14}$]
For this example, we will use a 4.5-foot clearance between the inlet (edge of pavement) elevation and the flow line of an 18-inch pipe. (The minimum clearance for standard precast structures is 4.2 feet. See Table 6.4-5.) Then:
Upper-end flow line = 59.7 – 4.5 = 55.2 feet

For this example, we will assume there are no constraints such as utilities that would prevent the pipe from being set at the minimum physical pipe slope (0.15 percent). Then:

| | |
|---|---|
| Minimum pipe fall | = 300 ft x 0.15% = 0.45 ft |
| Pipe fall | = 0.5 ft (minimum fall rounded up to nearest 0.1 foot) |
| Physical Slope | = 0.5 ft/300 ft = 0.167% |
| Lower-end flow line | = 55.2 – 0.5 = 54.7 feet |

If this were an actual project, you also should check that the minimum cover heights in Appendix C of the *Drainage Manual* are satisfied. To simplify this example, we will assume that adequate cover is provided.

12. Calculate the actual flow velocity. [$P_{15\text{-}14}$]
    $V = Q/A = 1.04$ cfs/($\pi D^2/4$)

    The full-flow cross sectional area used for the first pass through the system. Using the hydraulic calculator, the velocity of 1.04 cfs flowing full through an 18-inch pipe is 0.59 fps.

    Calculate the physical velocity. [$P_{15\text{-}14}$]

    Using the hydraulic calculator, the full-flow velocity for an 18-inch pipe sloped at 0.17 percent = 2.6 fps

13. Calculate the time of flow in the pipe section. [$P_{15\text{-}14}$]
    Time = Length/Actual Velocity
    = 300 ft/0.59 fps = 508 seconds = 8.5 minutes

A partially completed tabulation form with the information from this pipe is shown below.

**Table 6.6-13: Partially Completed Tabulation Form with Data from Example**

<table>
<tr><th>STRUCTURE NO.</th><th rowspan="3">LENGTH (ft)</th><th rowspan="3">TIME OF CONCEN-TRATION (MIN)</th><th rowspan="3">TIME OF FLOW IN SECTION (MIN)</th><th rowspan="3">INTENSITY (in/hr)</th><th rowspan="3">TOTAL C • A</th><th rowspan="3">TOTAL FLOW (cfs)</th><th rowspan="3">INLET ELEV. (ft)</th><th colspan="3">HYD. GRADIENT<br>CROWN<br>FLOW LINE</th><th>PIPE SIZE (IN)<br>RISE</th><th>SLOPE (%)</th><th>ACTUAL VELOCTIY (FPS)</th></tr>
<tr><th>UPPER</th><th rowspan="2">UPPER END ELEV (ft)</th><th rowspan="2">LOWER END ELEV (ft)</th><th rowspan="2">FALL (ft)</th><th rowspan="2">SPAN</th><th>HYD. GRAD.<br>PHYSICAL</th><th rowspan="2">PHYSICAL CELOCITY (FPS)</th></tr>
<tr><th>LOWER</th><th>MIN. PHYS.</th></tr>
<tr><td rowspan="2">15</td><td rowspan="3">300</td><td rowspan="3">10</td><td rowspan="3">8.5</td><td rowspan="3">6.5</td><td rowspan="3">0.16</td><td rowspan="3">1.04</td><td rowspan="3">59.70</td><td></td><td></td><td></td><td rowspan="2">18</td><td>0</td><td rowspan="2">.59</td></tr>
<tr><td>56.7</td><td>56.2</td><td rowspan="2">0.5</td><td>.167</td></tr>
<tr><td>14</td><td>55.2</td><td>54.7</td><td>18</td><td>.15</td><td>2.6</td></tr>
</table>

Step 7 through Step 13 are repeated for the remaining pipe sections. We have assumed that all the pipes are flowing full for this pass down the system. Situations that are different from the above pipe section are discussed below.

For Pipe Section $P_{14-13}$:

- The time of concentration is 10 + 8.5 = 18.5 minutes
- The upper-end flow line = 54.7 (set by matching the crowns across the structure)
- The lower-end flow line = 54.7 – 0.5 = 54.2 (set by minimum pipe slope as was done for $P_{15-14}$)

For Pipe Section $P_{13-12}$:

- The time of concentration is 18.5 + 1.8 = 20.3 minutes
- The upper-end flow line = 54.2 (set by matching the crowns across the structure)
- The lower-end flow line = S12 gutter elev. – inlet clearance = 54.5 – 4.5 = 50.0
- The physical slope = (54.2 – 50.0)/300 = 1.4 percent

For Pipe Section $P_{12-11}$:

- The time of concentration is 20.3 + 1.3 = 21.6 minutes.
- The upper-end flow line = 50.0 (set by matching the crowns across the structure)
- The lower-end flow line = S11 gutter elev. – inlet clearance = 50.5 – 4.5 = 46.0
- The physical slope = (50.0 – 46.0)/300 = 1.33 percent

For Pipe Section $P_{11\text{-out}}$:

- Size could be 18-inch pipe or 24-inch pipe based on comparing the full-flow friction loss slope to the preliminary HGL slope. Try 18-inch pipe, since the other pipes seem oversized.
- The time of concentration is 21.6 + 1.0 = 22.6 minutes
- The upper-end flow line = 46.0 (set by matching the crowns across the structure).

- Several factors may control the lower-end flow line, such as but not limited to cover requirements under roads around the lake, the lake bottom elevation, purposely submerging the outlet to minimize potential erosion at the outlet. For this example, we arbitrarily chose 44.5 feet.
- The physical slope = (46.0 – 44.5)/300 = 0.5 percent

The full-flow friction slope was recorded as the hydraulic gradient slope for all the pipes. Table 6.6-14 shows the results of doing Step 7 through Step 13 for the entire system.

**Table 6.6-14: Results of First Pass Down the System**

| STRUCTURE NO. (UPPER / LOWER) | LENGTH (ft) | TIME OF CONCEN-TRATION ( MIN ) | TIME OF FLOW IN SECTION ( MIN ) | INTENSITY ( in/hr ) | TOTAL C•A | TOTAL FLOW ( cfs ) | INLET ELEV (ft ) | HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV ( ft ) | LOWER END ELEV ( ft ) | FALL ( ft ) | PIPE SIZE (IN): RISE / SPAN | SLOPE (%): HYD GRAD / PHYSICAL / MIN PHYS | ACTUAL VELOCITY (FPS) / PHYSICAL VELOCITY (FPS) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 15 | 300 | 10 | 8.5 | 6.5 | 0.16 | 1.04 | 59.70 | | | | 18 | ≈0 | .59 |
| | | | | | | | | 56.7 | 56.2 | 0.5 | | 0.167 | |
| 14 | | | | | | | | 55.2 | 54.7 | | 18 | .15 | 2.6 |
| 14 | 300 | 18.5 | 1.8 | 5.1 | 0.96 | 4.9 | 59.80 | | | | 18 | 0.18 | 2.8 |
| | | | | | | | | 56.2 | 55.7 | 0.5 | | 0.167 | |
| 13 | | | | | | | | 54.7 | 54.2 | | 18 | 0.15 | 2.6 |
| 13 | 300 | 20.3 | 1.3 | 4.9 | 1.44 | 7.05 | 59.00 | | | | 18 | 0.38 | 4.0 |
| | | | | | | | | 55.7 | 51.5 | 4.2 | | 1.4 | |
| 12 | | | | | | | | 54.2 | 50.0 | | 18 | 0.15 | 7.6 |
| 12 | 300 | 21.6 | 1.0 | 4.7 | 1.84 | 8.65 | 54.50 | | | | 18 | 0.58 | 4.9 |
| | | | | | | | | 51.5 | 47.5 | 4.0 | | 1.33 | |
| 11 | | | | | | | | 50.0 | 46.0 | | 18 | 0.15 | 7.5 |
| 11 | 300 | 22.6 | - | 4.6 | 2.72 | 12.5 | 50.50 | | | | 18 | 1.2 | 7.0 |
| | | | | | | | | 47.5 | 46.0 | 1.5 | | 0.50 | |
| | | | | | | | | 46.0 | 44.5 | | 18 | 0.15 | 4.5 |

CALCULATE THE HYDRAULIC GRADE LINE ELEVATION

For pipe section $P_{11\text{-out}}$

14. Determine the Lower-end HG elevation. [$P_{11\text{-out}}$]
    For the outlet pipe, the lower-end HG is the design tailwater (DTW). For this example, the design tailwater is the higher of: (1) the crown of the pipe (elev. 46.0 feet), or (2) the normal high water stage (47.1) of the lake. Thus:
    Lower-end HG = 47.1 feet

15. Determine the upper-end HG elevation. [$P_{11\text{-}out}$]
The upper-end HG is the higher of:
1) Lower-end HG + full-flow friction loss = 47.1 + 1.2% x 300 feet
= 47.1 + 3.6 = 50.7 feet

OR

2) The elevation of normal depth upstream. The above elevation is higher than the upper-end crown, so normal depth cannot control.

Then: Upper-end HG = 50.7 feet

The standard HG clearance is not met (S-11 inlet elev. = 50.5). We will increase this pipe size to 24 inches before continuing upstream. To match the crowns at the upper end, the flow line of the 24-inch pipe will be set 0.5 foot lower than for the 18-inch pipe. [$P_{11\text{-}out}$]

| | |
|---|---|
| Upper-end flow line | = 45.5 ft |
| Pipe fall | = 45.5 – 44.5 = 1 ft (holding lower-end flow line) |
| Physical slope | = 1/300 = 0.33 percent |

Starting at the outlet pipe again:

16. Determine the lower-end HG elevation. [$P_{11\text{-}out}$]
Using the same approach as in Step 14, the lower-end HG elevation = 47.1 feet

17. Determine the upper-end HG elevation. [$P_{11\text{-}out}$]

The Upper End HG is higher of:

1) Lower-end HG + full-flow friction loss = 47.1 + 0.26% x 300 ft
= 47.1 + 0.78 = 47.9 ft

OR

2) The elevation of normal depth upstream. The above elevation (47.9 feet) is higher than the crown (47.5 feet), so normal depth does not apply.

Then, upper-end HG = 47.9 feet

Repeat Step 16 and Step 17 for the other pipe sections.

Table 6.6-16 shows the results of completing these steps for the entire system.
For Pipe Section $P_{12\text{-}11}$:

The lower-end HG is the higher of:

1) Downstream pipe upper-end HG: = 47.9 ft

OR

2) Controlling pipe condition at the lower end. The above elevation (47.9 feet) is higher than the crown (47.5 feet), so normal depth does not apply.

Then, lower-end HG = 47.9 feet

The upper-end HG is the higher of:

1) Lower-end HG + full-flow friction loss = 47.9 + 0.58% x 300 ft
= 47.9 + 1.74 = 49.64 ft

OR

2) The elevation of normal depth upstream. Using the hydraulic calculator (Q = 8.65 cfs, 18-inch pipe @ 1.33 percent slope), the normal depth = 0.6 x Diameter

   Upper-end normal depth elev. = 50.0 + 0.6 x 1.5 = 50.9 ft ($d_{NORM}$ = 0.6 x D)

Then, upper-end HG = 50.9 feet

For Pipe Section $P_{13\text{-}12}$:

The lower-end HG is the higher of:

1) Downstream pipe upper-end HG = 50.9 ft

OR

2) Controlling pipe condition at the lower end. The pipe slope (1.4 percent) is steeper than the full-flow friction slope (0.38 percent) so, if the downstream HG is low enough, the flow depth at the lower end is normal depth. Thus, the controlling pipe condition is normal depth (Figure 5-1C). Using the hydraulic calculator (Q = 7.05 cfs, 18-inch pipe @ 1.4 percent slope) the normal depth = 0.52 x Diameter

Lower-end normal depth elev. = 50.0 + 0.52 x 1.5 = 50.78 ft

Then, lower-end HG = 50.9 feet

The upper-end HG is the higher of:

1) Lower-end HG + full-flow friction loss = 50.9 + .38% x 300 ft
= 50.9 + 1.14 = 52.04 ft

OR

2) The elevation of normal depth upstream. = 54.2 + 0.52 x 1.5 = 54.98 ft ($d_{NORM}$ = 0.52 x D)

Then, upper-end HG = 54.98 feet

For Pipe Section $P_{14\text{-}13}$:

The lower-end HG is the higher of:

1) Downstream pipe upper end HG = 54.98 ft

OR

2) Controlling pipe condition at the lower end. The pipe slope (0.167 percent) is flatter than the full-flow friction slope (0.18 percent), so use the crown of the pipe (Figure 6.5-1D) as the controlling pipe condition at the lower end. Lower-end crown elev. = 55.7 ft

Then, lower-end HG = 55.7 feet

The upper-end HG is the higher of:

1) Lower-end HG + full-flow friction loss = 55.7 + 0.18% x 300 ft
= 55.7 + 0.54 = 56.24 ft

OR

2) The elevation of normal depth upstream. The above elevation (56.24 feet) is higher than the crown (56.2 feet), so normal depth does not apply.

Then, upper-end HG = 56.24 feet

For Pipe Section $P_{15\text{-}14}$:

The Lower End HG is the higher of:

1) Downstream pipe upper-end HG = 56.24 ft

OR

2) Controlling pipe condition at the lower end. The above elevation (56.24 feet) is higher than the crown (56.2 feet), so normal depth does not apply.

Then, lower-end HG = 56.24 feet

The Upper End HG is the higher of:

1) Lower-end HG + full-flow friction loss = 56.24 + 0.0 ft = 56.24 ft

OR

2) The elevation of normal depth upstream. Using the hydraulic calculator (Q = 1.0 cfs, 18-inch pipe @ 0.17 percent slope), the normal depth = 0.32 x Diameter

   Normal depth elevation= 55.2 + 0.32 x 1.5 = 55.68 ft

Then, upper-end HG = 56.24 feet

Table 6.6-16 shows the results of doing Step 16 and Step 17 for the entire system.

## Table 6.6-15: Results of First Pass up the System

| STRUCTURE NO. (UPPER / LOWER) | LENGTH (ft) | TOTAL FLOW (cfs) | INLET ELEV (ft) | HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV (ft) | HYD. GRADIENT / CROWN / FLOW LINE: LOWER END ELEV (ft) | FALL (ft) | PIPE SIZE (IN): RISE / SPAN | SLOPE (%): HYD. GRAD. / PHYSICAL / MIN. PHYS. |
|---|---|---|---|---|---|---|---|---|
| 15 | 300 | 1.04 | 59.70 | 56.24 | 56.24 | | 18 | ≈0 |
| | | | | 56.7 | 56.2 | 0.5 | | 0.167 |
| 14 | | | | 55.2 | 54.7 | | 18 | 0.15 |
| 14 | 300 | 4.9 | 59.80 | 56.24 | 55.7 | 0.54 | 18 | 0.18 |
| | | | | 56.2 | 55.7 | 0.5 | | 0.167 |
| 13 | | | | 54.7 | 54.2 | | 18 | 0.15 |
| 13 | 300 | 7.05 | 59.00 | 54.98 | 50.9 | | 18 | 0.38 |
| | | | | 55.7 | 51.5 | 4.2 | | 1.4 |
| 12 | | | | 54.2 | 50.0 | | 18 | 0.15 |
| 12 | 300 | 8.65 | 54.50 | 50.9 | 47.9 | | 18 | 0.58 |
| | | | | 51.5 | 47.5 | 4.0 | | 1.33 |
| 11 | | | | 50.0 | 46.0 | | 18 | 0.15 |
| 11 | 300 | 12.5 | 50.50 | 47.9 | 47.1 | 0.78 | 24 | 0.26 |
| | | | | 47.5 | 46.5 | 1.0 | | 0.33 |
| outlet | | | | 45.5 | 44.5 | | 24 | 0.1 |

The HG slopes shown for pipe sections $P_{15-14}$, $P_{13-12}$, $P_{12-11}$ are the full-flow friction slopes. The values are not the true HG slopes because these pipes are flowing part full. The values will be revised in subsequent iterations through the system. The full-flow friction slopes have been shown in Table 6.6-16 to help follow the discussion of Step 16 and Step 17 for the entire system.

18. Compare the hydraulic gradient elevation to the standard.
    Throughout the system, the hydraulic gradient elevation is more than 1.13 feet below the inlet elevation (edge of pavement), so it meets the current standard. We will recalculate the flow rates and check again.

19. Recalculate the flow rates.
    Several pipes are flowing partly full, so we need to recalculate the velocities and times of flow in those sections. This will change the times of concentration and the flow rates. Pipe sections $P_{15-14}$, $P_{13-12}$, and $P_{12-11}$ are flowing part full and the others are flowing full based on the calculations up to this point. We will assume these modes of flow as we work downstream recalculating flow. The velocity in the three pipes flowing partly full will be based on normal depth velocity (see Figure 6.5-3). Table 6.6-17 shows the results of recalculating the flow rates.

### Table 6.6-16: Results of Second Pass down the System

| STRUCTURE NO.<br>UPPER<br>LOWER | LENGTH (ft) | TIME OF CONCEN-TRATION (min) | TIME OF FLOW IN SECTION (min) | INTENSITY (in/hr) | TOTAL C•A | TOTAL FLOW (cfs) | PIPE SIZE (IN)<br>RISE<br>SPAN | SLOPE (%)<br>HYD. GRAD.<br>PHYSICAL<br>MIN PHYS | ACTUAL VELOCITY (FPS)<br>PHYSICAL VELOCITY (FPS) | NOTES AND REMARKS<br>FREQUENCY (Yrs): 5<br>MANNINGS “n”: 0.012<br>TAILWATER EL. (ft): 47.1<br>All overland $t_c \leq$ 10 min. |
|---|---|---|---|---|---|---|---|---|---|---|
| 15 | 300 | 10.0 | 2.4 | 6.5 | 0.16 | 1.04 | 18 | ≈ 0<br>0.167 | 2.1 | Act Vel based on normal Depth. d/D = 0.32 |
| 14 | | | | | | | 18 | .15 | 2.6 | |
| 14 | 300 | 12.4 | 1.5 | 6.0 | 0.96 | 5.76 | 18 | 0.25<br>0.167 | 3.25 | |
| 13 | | | | | | | 18 | 0.15 | 2.6 | |
| 13 | 300 | 13.9 | 0.6 | 5.7 | 1.44 | 8.2 | 18 | 0.5<br>1.4 | 8.0 | Act Vel based on normal Depth. d/D = 0.55 |
| 12 | | | | | | | 18 | 0.15 | 7.6 | |
| 12 | 300 | 14.5 | 0.6 | 5.6 | 1.84 | 10.3 | 18 | 0.8<br>1.33 | 8.3 | Act Vel based on normal Depth. d/D = 0.67 |
| 11 | | | | | | | 18 | 0.15 | 7.5 | |
| 11 | 300 | 15.1 | - | 5.6 | 2.72 | 15.2 | 24 | 0.38<br>0.33 | 4.8 | |
| Outlet | | | | | | | 24 | 0.10 | 4.5 | |

20. Recalculate the hydraulic gradient elevation.
    Work up the system, as was done previously in Step 16 and Step 17. Table 6.6-18 shows the results.

## Table 6.6-17: Results of Second Pass up the System

| STRUCTURE NO. | LENGTH ( ft ) | TOTAL FLOW (cfs) | INLET ELEV. (ft) | HYD. GRADIENT | | | PIPE SIZE (IN) | SLOPE (%) | NOTES AND REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| | | | | CROWN | | | | | |
| | | | | FLOW LINE | | | | | FREQUENCY (Yrs): 5 |
| | | | | UPPER END ELEV. (ft) | LOWER END ELEV. (ft) | FALL (ft) | RISE | | MANNING'S "n": 0.012 |
| | | | | | | | | HYD GRAD | TAILWATER EL. (ft): 47.1 |
| UPPER | | | | | | | SPAN | PHYSICAL | |
| LOWER | | | | | | | | MON PHYS | All overland $t_c \leq 10$ min. |
| 15 | 300 | 1.04 | 59.70 | 56.45 | 56.45 | | 18 | ≈ 0 | Act.Vel based on Norm Depth. d/D = 0.32 |
| | | | | 56.7 | 56.2 | 0.5 | | 0.167 | |
| 14 | | | | 55.2 | 54.7 | | 18 | 0.15 | |
| 14 | 300 | 5.76 | 59.80 | 56.45 | 55.7 | 0.75 | 18 | 0.25 | |
| | | | | 56.2 | 55.7 | 0.5 | | 0.167 | |
| 13 | | | | 54.7 | 54.2 | | 18 | 0.15 | |
| 13 | 300 | 8.02 | 59.00 | 55.03 | 51.0 | | 18 | 0.5 | Act Vel & Upper End HG Based on Norm Depth D/D = 0.55 |
| | | | | 55.7 | 51.5 | 4.2 | | 1.4 | |
| 12 | | | | 54.2 | 50.0 | | 18 | 0.15 | |
| 12 | 300 | 10.3 | 54.50 | 51.0 | 48.24 | | 18 | 0.8 | Act Vel & Upper End HG Based on Norm Depth D/D = 0.67 |
| | | | | 51.5 | 47.5 | 4.0 | | 1.33 | |
| 11 | | | | 50.0 | 46.0 | | 18 | 0.15 | |
| 11 | 300 | 15.2 | 50.50 | 48.24 | 47.1 | 1.14 | 24 | 0.38 | |
| | | | | 47.5 | 46.5 | 1.0 | | 0.33 | |
| OUT | | | | 45.5 | 44.5 | | 24 | 0.10 | |

The HG slopes shown for pipe sections $P_{15\text{-}14}$, $P_{13\text{-}12}$, $P_{12\text{-}11}$ are the full-flow friction slopes. The values are not the true HG slopes because these pipes are flowing part full. The full-flow friction slopes have been shown in Table 6.6-18 to help you compare HG elevations as you work through the system. The values are changed in Table 6.6-19, which reflects the completed design.

21. Compare the hydraulic gradient to the standard.

    Throughout the system, the hydraulic gradient elevation is more than 1.13 feet below the inlet elevation (edge of pavement), so it meets the current standard. Pipe section $P_{11\text{-}out}$ cannot be reduced in diameter without violating the standard HG clearance at S-11 (see Step 15). The other pipes are the minimum standard diameter, so their diameter cannot be reduced.

    Pipe section $P_{15\text{-}14}$ is flowing full for about half of its length. Consequently, the flow velocity is less than the 2.1 fps we estimated in Table 6.6-17. We could make another iteration through the system recalculating flows based on the reduced velocity, but there is nothing to be gained from doing that here. None of the pipe diameters can be reduced. A completed tabulation form is shown in Table 6.6-19.

## Example 6.6-2

### Table 6.6-18: Completed Tabulation Form For Example 6.6-2

| LOCATION OF UPPER END / ALIGNMENT NAME / STATION | DISTANCE (ft) | SIDE | STRUCTURE NO. (UPPER / LOWER) | TYPE OF STRUCTURE | LENGTH (ft) | DRAINAGE AREA (ac) C = / C = 0.80 / C = — INCREMENT | TOTAL | SUBTOTAL C•A | TIME OF CONCENTRATION (min) | TIME OF FLOW IN SECTION (min) | INTENSITY (in/hr) | TOTAL C•A | TOTAL FLOW (cfs) | INLET ELEV (ft) | HYD. GRADIENT / CROWN / FLOW LINE — UPPER END ELEV (ft) | LOWER END ELEV (ft) | FALL (ft) | PIPE SIZE (IN) RISE / SPAN | SLOPE (%) HYD GRAD / PHYSICAL / MIN PHYS | ACTUAL VELOCITY (FPS) / PHYSICAL VELOCITY (FPS) | NOTES AND REMARKS — FREQUENCY (Yrs): 5; MANNING'S "n": 0.012; TAILWATER EL. (ft): 47.1; All overland $t_c \le 10$ min |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Patrick Place | | | 15 | P1 | 300 | 0.2 | 0.2 | 0.16 | 10 | 2.4 | 6.5 | 0.16 | 1.04 | 59.70 | 56.45 | 56.45 | ≈ 0 | 18 | ≈ 0 | 2.1 | Act Vel based on Norm Depth. d/D = 0.32 |
| | | | | | | | | | | | | | | | 56.7 | 56.2 | 0.5 | | 0.167 | | |
| | | | 14 | | | | | | | | | | | | 55.2 | 54.7 | | 18 | 0.15 | 2.6 | |
| Patrick Place | | | 14 | P1 | 300 | 1.0 | 1.2 | 0.96 | 12.4 | 1.5 | 6.0 | 0.96 | 5.76 | 59.80 | 56.45 | 55.7 | 0.75 | 18 | 0.25 | 3.25 | |
| | | | | | | | | | | | | | | | 56.2 | 55.7 | 0.5 | | 0.167 | | |
| | | | 13 | | | | | | | | | | | | 54.7 | 54.2 | | 18 | 0.15 | 2.6 | |
| Patrick Place | | | 13 | P1 | 300 | 0.6 | 1.8 | 1.44 | 13.9 | 0.6 | 5.7 | 1.44 | 8.2 | 59.00 | 55.03 | 51.0 | 4.03 | 18 | 1.3 | 8.0 | Act Vel & Upper End HG based on Norm Depth. d/D = 0.55 |
| | | | | | | | | | | | | | | | 55.7 | 51.5 | 4.2 | | 1.4 | | |
| | | | 12 | | | | | | | | | | | | 54.2 | 50.0 | | 18 | 0.15 | 7.6 | |
| Patrick Place | | | 12 | P1 | 300 | 0.5 | 2.3 | 1.84 | 14.5 | 0.6 | 5.6 | 1.84 | 10.3 | 54.50 | 51.0 | 48.24 | 2.79 | 18 | 0.9 | 8.3 | Act Vel & Upper End HG based on Norm Depth. d/D = 0.67 |
| | | | | | | | | | | | | | | | 51.5 | 47.5 | 4.0 | | 1.33 | | |
| | | | 11 | | | | | | | | | | | | 50.0 | 46.0 | | 18 | 0.15 | 7.5 | |
| Patrick Place | | | 11 | P1 | 300 | 1.1 | 3.4 | 2.72 | 15.1 | 1.0 | 5.6 | 2.72 | 15.2 | 50.50 | 48.24 | 47.1 | 1.14 | 24 | 0.38 | 4.8 | |
| | | | | | | | | | | | | | | | 47.5 | 46.5 | 1.0 | | 0.33 | | |
| | | | OUT | | | | | | | | | | | | 45.5 | 44.5 | | 24 | 0.10 | 4.5 | |