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CHAPTER

3

HYDRAULIC DESIGN OF STREETS AND CLOSED CONDUITS

Section 3.1 Street and Closed Conduit Design Overview

3.1.1 Storm Water System Design

3.1.1.1 Introduction

Storm water system design is an integral component of both site and overall storm water management design. Good drainage design must strive to maintain compatibility and minimize interference with existing drainage patterns; control flooding of property, structures, and roadways for design flood events; and minimize potential environmental impacts on storm water runoff.

Storm water collection systems must be designed to provide adequate surface drainage while at the same time meeting other storm water management goals such as water quality, streambank protection, habitat protection, and groundwater recharge.

3.1.1.2 System Components

The storm water system components consist of all the *integrated* site design practices and storm water controls utilized on the site. Three considerations largely shape the design of the storm water systems: water quality, streambank protection, and flood control.

The on-site flood control systems are designed to remove storm water from areas such as streets and sidewalks for public safety reasons. The drainage system can consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, storm water ponds and wetlands, and small underground pipe systems which collect storm water runoff from mid-frequency storms and transport it to structural control facilities, pervious areas, and/or the larger storm water systems (i.e., natural waterways, large man-made conduits, and large water impoundments).

The storm water (major) system consists of natural waterways, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainageways such as overload relief swales and infrequent temporary ponding areas. The storm water system includes not only the trunk line system that receives the water, but also the natural overland relief which functions in case of overflow from or failure of the on-site flood control system. Overland relief must not flood or damage houses, buildings or other property.

This chapter is intended to provide design criteria and guidance on several on-site flood control system components, including street and roadway gutters, inlets, and storm drain pipe systems (Section 3.2). Chapter 4 covers the design of culverts (Section 4.2); vegetated and lined open channels (Section 4.4); storage and design (Section 4.5); ; outlet structures (Section 4.6); and energy dissipation devices for outlet protection (Section 4.7). The rest of this section covers important considerations to keep in mind in the planning and design of storm water drainage facilities.

3.1.1.3 Checklist for Planning and Design

The following is a general procedure for drainage system design on a development site.

A. Analyze topography, including:
   1. Check off-site drainage pattern. Where is water coming onto the site? Where is water leaving the site?
   2. Check on-site topography for surface runoff and storage, and infiltration
      a. Determine runoff pattern: high points, ridges, valleys, streams, and swales. Where is the water going?
      b. Overlay the grading plan and indicate watershed areas: calculate square footage (acreage), points of concentration, low points, etc.

B. Analyze other site conditions, including:
   1. Land use and physical obstructions such as walks, drives, parking, patios, landscape edging, fencing, grassed area, landscaped area, tree roots, etc.
   2. Soil type (infiltration rates).
   3. Vegetative cover (slope protection).

C. Check potential drainage outlets and methods, including:
   1. On-site (structural control, receiving water)
   2. Off-site (highway, storm drain, receiving water, regional control)
   3. Natural drainage system (swales)
   4. Existing drainage system (drain pipe)

D. Analyze areas for probable location of drainage structures and facilities.

E. Identify the type and size of drainage system components required. Design the drainage system and integrate with the overall storm water management system and plan.

3.1.2 Key Issues in Storm Water System Design

3.1.2.1 Introduction

The traditional design of storm water systems has been to collect and convey storm water runoff as rapidly as possible to a suitable location where it can be discharged. This manual takes a different approach wherein the design methodologies and concepts of drainage design are to be integrated with the objectives for water quantity and quality control. This means:

- Storm water systems are to remove water efficiently enough to meet flood protection criteria and level of service requirements, and
- These systems are to complement the ability of the site design and structural storm water controls to mitigate the major storm water impacts of urban development.

The following are some of the key issues in integrating water quantity and quality control consideration in storm water system design.

3.1.2.2 General Design Considerations

- Storm water systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage pathways within a drainage basin. These natural drainage pathways should only be modified as a last resort to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or storm water systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or storm water (minor) systems.
- It is important to ensure that the combined on-site flood control system and major storm water system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor storm water systems and/or major storm water structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of storm water systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major storm water system design capacity.

3.1.2.3 Street and Roadway Gutters

- Gutters are efficient flow conveyance structures. This is not always an advantage if removal of pollutants and reduction of runoff is an objective. Therefore, impervious surfaces should be disconnected hydrologically where possible, and runoff should be allowed to flow across pervious surfaces or through vegetated channels. Gutters should be used only after other options have been investigated and only after runoff has had as much chance as possible to infiltrate and filter through vegetated areas.
- It may be possible not to use gutters at all, or to modify them to channel runoff to off-road pervious areas or open channels. For example, curb opening type designs take roadway runoff to smaller feeder grass channels. Care should be taken not to create erosion problems in off-road areas. Protection during construction, establishment of strong stands of vegetation, and active maintenance may be necessary in some areas.
- Use typical road sections that use grass channels or swales instead of gutters to provide for pollution reduction and reduce the impervious area required. Figure 3.1-1 illustrates a roadway cross section that eliminates gutters for residential neighborhoods. Flow can also be directed to center median strips in divided roadway designs. To protect the edge of pavement, ribbons of concrete can be used along the outer edges of asphalt roads.

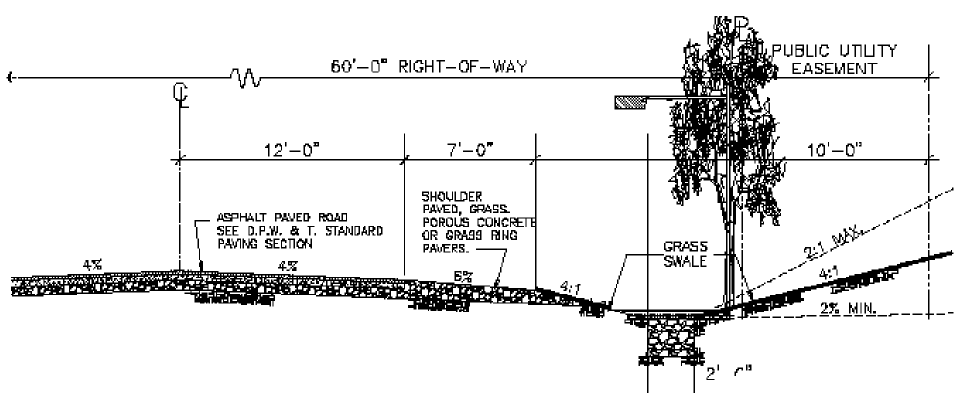


Figure 3.1-1 Alternate Roadway Section without Gutters
(Source: Prince George's County, MD, 1999)

3.1.2.4 Inlets and Drains

- Inlets should be located to maximize the overland flow path, take advantage of pervious areas, and seek to maximize vegetative filtering and infiltration. For example, it might be possible to design a parking lot so water flows into vegetated areas prior to entering the nearest inlet.
- Inlet location should not compromise safety or aesthetics. It should not allow for standing water in areas of vehicular or pedestrian traffic, but should take advantage of natural depression storage where possible.
- Inlets should be located to serve as overflows for structural storm water controls. For example, a bioretention device in a commercial area could be designed to overflow to a catch basin for larger storm events.
- The choice of inlet type should match its intended use. A sumped inlet may be more effective supporting water quality objectives.
- Use several smaller inlets instead of one large inlet in order to:
  1. Prevent erosion on steep landscapes by intercepting water before it accumulates too much volume and velocity.
  2. Provide a safety factor. If a drain inlet clogs, the other surface drains may pick up the water.
  3. Improve aesthetics. Several smaller drains will be less obvious than one large drain.
  4. Spacing smaller drain inlets will give surface runoff a better chance of reaching the drain. Water will have to travel farther to reach one large drain inlet.

3.1.2.5 Closed Conduit Systems (Storm Drains/Sewers)

- The use of *integrated* site design practices (and corresponding site design credits) should be considered to reduce the overall length of a closed conduit storm water system.
- Shorter and smaller conveyances can be designed to carry runoff to nearby holding areas, natural conservation areas, or filter strips (with spreaders at the end of the pipe).
- Ensure that storms in excess of closed conduit design flows can be safely conveyed through a development without damaging structures or flooding major roadways. This is often done through design of both a major and minor drainage system. The on-site flood control system carries the mid-frequency design flows while larger runoff events may flow across lots and along streets as long as it will not cause property damage or impact public safety.

3.1.3 Design Storm Recommendations

Listed below are the design storm recommendations for various storm water drainage system components to be designed and constructed in accordance with the minimum storm water management standards. Some jurisdictions may require the design of storm water conveyance systems, sized for various storm frequencies. Please consult your local review authority to determine the local requirements. It is recommended that the full build-out conditions be used to calculate flows for the design storm frequencies below.

Storm Drainage Systems

Including storm drainage systems and pipes that do not convey runoff under public roadways, sometimes called lateral closed systems.

- 5- to 25-year design storm (for pipe and culvert design)
- 5- to 25-year design storm (for inlet design)
- 100-year design storm (for sumped inlets, unless overflow facilities are provided)

Check Storm

Used to estimate the runoff that is routed through the drainage system and storm water management facilities to determine the effects on the facilities, adjacent property, floodplain encroachment, and downstream areas. The check storm should be contained within public right-of-way and/or easements but should not be conveyed underground in the drainage system.

- 100-year design storm

References

Department of Irrigation and Drainage Malaysia, River Engineering Division, 2000. Urban Stormwater Management Manual for Malaysia (Draft).

Prince George's County, MD, 1999. Low-Impact Development Design Strategies, An Integrated Design Approach.

The Dewberry Companies, 2002, 2nd Edition, Land Development Handbook, McGraw-Hill Companies, Inc., New York, NY.

American Society of Civil Engineers, 1993, Design and Construction of Urban Stormwater Management Systems, Manual and Report No. 77.

Larry W. Mays, Editor, 2001, Stormwater Collection Systems Design Handbook, McGraw-Hill Companies, Inc., New York, NY.

Section 3.2
On-Site Flood Control System Design

3.2.1 Overview

3.2.1.1 Introduction

On-Site Flood Control Systems, also known as minor drainage systems, quickly remove runoff from areas such as streets and sidewalks for public safety purposes. The on-site flood control system consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, and small underground pipe systems which collect storm water runoff and transport it to structural control facilities, pervious areas, and/or the larger storm water system (i.e., natural waterways, large man-made conduits, and large water impoundments).

This section is intended to provide criteria and guidance for the design of on-site flood control system components including:

- Street and roadway gutters
- Storm water inlets
- Storm drain pipe systems

Ditch, channel and swale design criteria and guidance are covered in Section 4.4, *Open Channel Design*.

Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grate, curb, and combination inlets are based on information contained in HEC-12 (USDOT, FHWA, 1984). Storm drain system design is based on the use of the Rational Method Formula.

3.2.1.2 General Criteria

Design Frequency

See Section 3.1 or the local review authority for design storm requirements for the sizing of storm system components. Fully developed conditions should be used for design.

Flow Spread Limits

Inlets shall be spaced so that the spread in the street for the design flow shall not exceed the local criteria or as a guideline the following, as measured from the face of the curb:

- 8 feet if the street is classified as a Collector or Arterial street (for 4-lane streets or greater, spread may extend across one travel lane)
- 16 feet at any given section, but in no case greater than 10 feet on one side of the street, if the street is classified as a Local street

3.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 3.2-1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3.2-1 Symbols and Definitions

Symbol	Definition	Units
a	Gutter depression	in
A	Area of cross section	ft^2
d or D	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
E_o	Ratio of frontal flow to total gutter flow Q_w/Q	-
g	Acceleration due to gravity (32.2 ft/s^2)	ft/s^2
h	Height of curb opening inlet	ft
H	Head loss	ft
K	Loss coefficient	-
L or L_T	Length of curb opening inlet	ft
L	Pipe length	ft
n	Roughness coefficient in the modified Manning's formula for triangular gutter flow	-
P	Perimeter of grate opening, neglecting bars and side against curb	ft
Q	Rate of discharge in gutter	cfs
Q_i	Intercepted flow	cfs
Q_S	Gutter capacity above the depressed section	cfs
S or S_x	Cross Slope - Traverse slope	ft/ft
S or S_L	Longitudinal slope	ft/ft
S_f	Friction slope	ft/ft
S'_w	Depression section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
T_s	Spread above depressed section	ft
V	Velocity of flow	ft/s
W	Width of depression for curb opening inlets	ft
Z	T/d, reciprocal of the cross slope	-

3.2.3 Street and Roadway Gutters

Effective drainage of street and roadway pavements is essential to the maintenance of the roadway service level and to traffic safety. Water on the pavement can interrupt traffic flow, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of storm water on the pavement surface.

This section presents design guidance for gutter flow hydraulics originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual.

3.2.3.1 Formula

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = [0.56/n]\ S_x^{5/3}\ S^{1/2}\ T^{8/3} \qquad (3.2.1)$$

where:

- Q = gutter flow rate, cfs
- S_x = pavement cross slope, ft/ft
- n = Manning's roughness coefficient
- S = longitudinal slope, ft/ft
- T = width of flow or spread, ft

Equation 3.2.1 was first presented by C.F. Izzard in 1946.

3.2.3.2 Nomograph

Figure 3.2-1 is a nomograph for solving Equation 3.2.1. Manning's n values for various pavement surfaces are presented in Table 3.2-2 below. Note: the nomograph will not work on slopes steeper than 3% - 4% for 2 and 3 lane thoroughfares. Also, the “Q” in the nomograph is only for n = 0.016.

3.2.3.3 Manning's n Table

Table 3.2-2 Manning’s n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.014
Asphalt pavement:	
Smooth texture	0.015
Rough texture	0.019
Concrete gutter with asphalt pavement:	
Smooth	0.015
Rough	0.018
Concrete pavement:	
Float finish	0.017
Broom finish	0.019
For gutters with small slopes, where sediment may accumulate, increase above values of n by	0.002

Note: Based on the statement of Izzard (1946) and confirmation by model studies (Ickert and Crosby, 2003), the n-values given in Table 4-3 of HEC No. 22, 2001, were increased by 18% to derive the n-values in this table.

3.2.3.4 Uniform Cross Slope

The nomograph in Figure 3.2-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not; use the product of Q and n (Qn).

Step 4 Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

Condition 2: Find gutter flow, given spread.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n.

Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

Step 4 For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

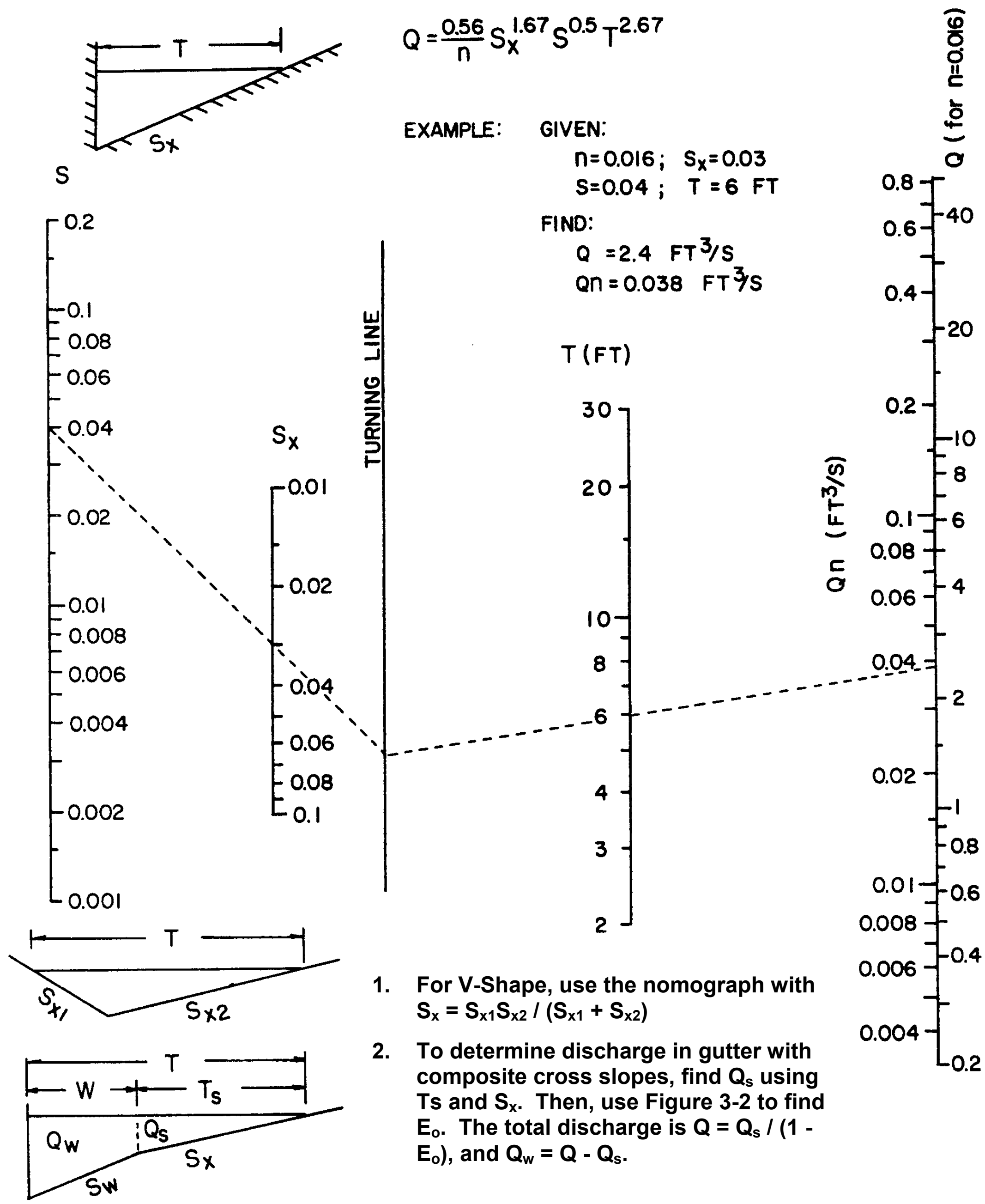


Figure 3.2-1 Flow in Triangular Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

***See Section 3.2.3.2 for applicability**

3.2.3.5 Composite Gutter Sections

Figure 3.2-2 in combination with Figure 3.2-1 can be used to find the flow in a gutter with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

Figure 3.2-3 provides a direct solution of gutter flow in a composite gutter section. The flow rate at a given spread or the spread at a known flow rate can be found from this figure. Figure 3.2-3 involves a complex graphical solution of the equation for flow in a composite gutter section. Typical of graphical solutions, extreme care in using the figure is necessary to obtain accurate results.

Condition 1: Find spread, given gutter flow.

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), depressed section slope (S_w), depressed section width (W), Manning's n, gutter flow (Q), and a trial value of gutter capacity above the depressed section (Q_s).

Step 2 Calculate the gutter flow in W (Q_w), using the equation:

$$\mathbf{Q_w = Q - Q_s} \qquad \textbf{(3.2.2)}$$

Step 3 Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 3.2-2 to find an appropriate value of W/T.

Step 4 Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.

Step 5 Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4.

Step 6 Use the value of T_s from Step 5 along with Manning's n, S, and S_x to find the actual value of Q_s from Figure 3.2-1.

Step 7 Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

Condition 2: Find gutter flow, given spread.

Step 1 Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's n, and depth of gutter flow (d).

Step 2 Use Figure 3.2-1 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes, substituting T_s for T.

Step 3 Calculate the ratios W/T and S_w/S_x, and, from Figure 3.2-2, find the appropriate value of E_o (the ratio of Q_w/Q).

Step 4 Calculate the total gutter flow using the equation:

$$\mathbf{Q = Q_s / (1 - E_o)} \qquad \textbf{(3.2.3)}$$

where:

Q = gutter flow rate, cfs

Q_s = flow capacity of the gutter section above the depressed section, cfs

E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

Step 5 Calculate the gutter flow in width (W), using Equation 3.2.2.

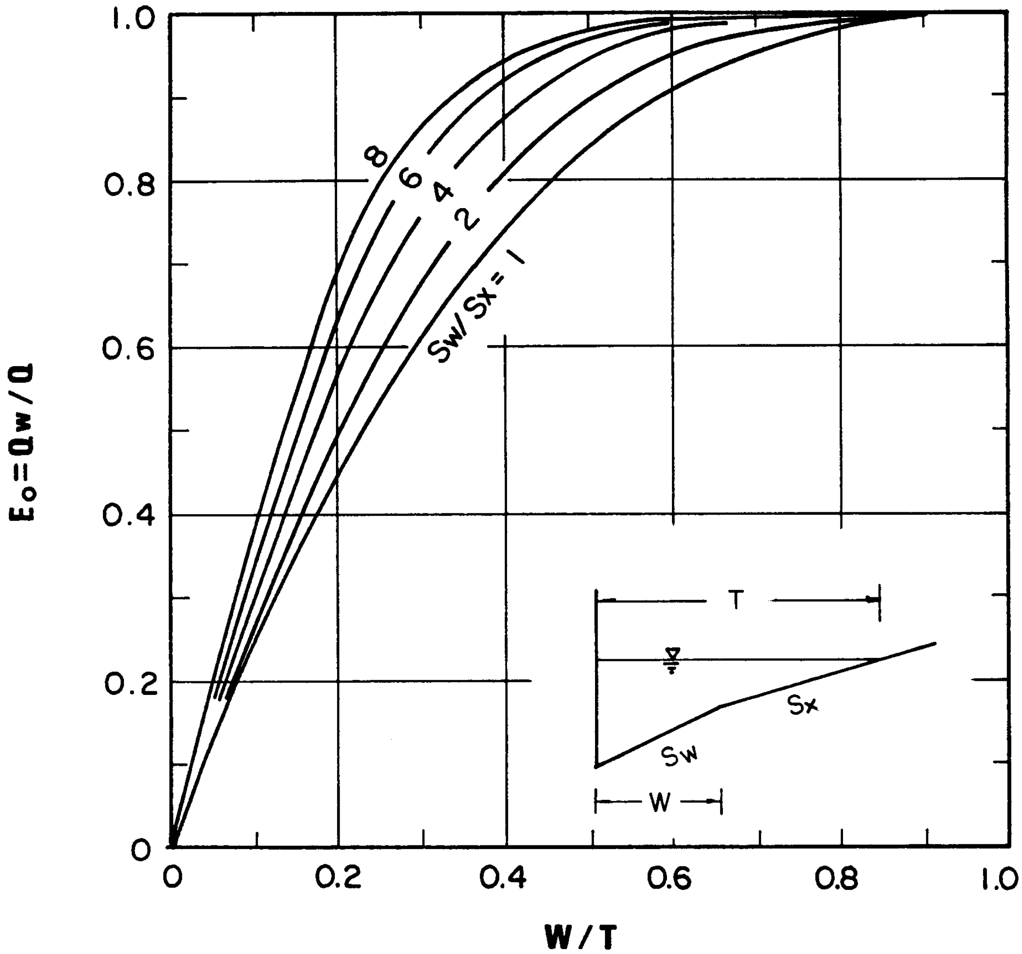


Figure 3.2-2 Ratio of Frontal Flow to Total Gutter Flow
(Source: AASHTO Model Drainage Manual, 1991)

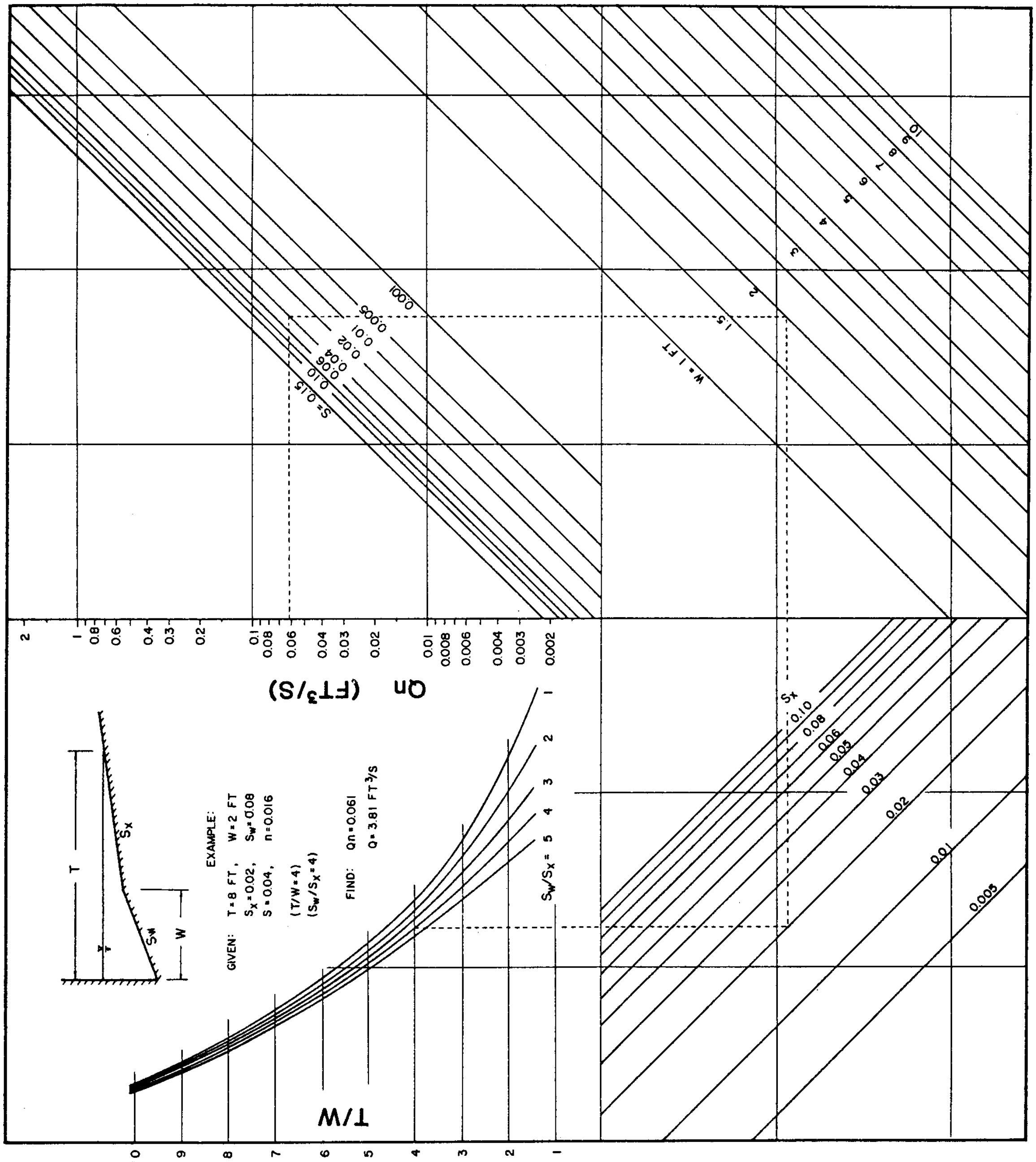


Figure 3.2-3 Flow in Composite Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

3.2.3.6 Examples

Example 1

Given:

T = 8 ft
S_x = 0.025 ft/ft
n = 0.015
S = 0.01 ft/ft

Find:

1. Flow in gutter at design spread
2. Flow in width (W = 2 ft) adjacent to the curb

Solution:

a. From Figure 3.2-1, Qn = 0.03
   Q = Qn/n = 0.03/0.015 = 2.0 cfs

b. T = 8 - 2 = 6 ft
   $(Qn)_2$ = 0.014 (Figure 3.2-1) (flow in 6-foot width outside of width (W))
   Q = 0.014/0.015 = 0.9 cfs
   Q_w = 2.0 - 0.9 = 1.1 cfs

Flow in the first 2 ft adjacent to the curb is 1.1 cfs and 0.9 cfs in the remainder of the gutter.

Example 2

Given:

T = 6 ft
S_w = 0.0833 ft/ft
T_s = 6 - 1.5 = 4.5 ft
W = 1.5 ft
S_x = 0.03 ft/ft
n = 0.014
S = 0.04 ft/ft

Find:

Flow in the composite gutter

Solution:

1. Use Figure 3.2-1 to find the gutter section capacity above the depressed section.
   $Q_s n$ = 0.038
   Q_s = 0.038/0.014 = 2.7 cfs
2. Calculate W/T = 1.5/6 = 0.25 and
   S_w/S_x = 0.0833/0.03 = 2.78
   Use Figure 3.2-2 to find E_o = 0.64
3. Calculate the gutter flow using Equation 3.2.3
   Q = 2.7/(1 - 0.64) = 7.5 cfs
4. Calculate the gutter flow in width, W, using Equation 3.2.2
   Q_W = 7.5 - 2.7 = 4.8 cfs

3.2.3.7 Parabolic Cross Slope

The following methodology regarding Parabolic cross-slopes was excerpted from the City of Austin Drainage Criteria Manual dated July 2003.

Flows in the gutter of a parabolically crowned pavement are calculated from a variation of Manning's Equation, which assumes steady flow in a prismatic open channel. However, this equation is complicated and difficult to solve for each design case.

To provide a means of determining the flow in the gutter, generalized gutter flow equations for combinations of parabolic crown heights, curb splits and street grades of different street widths have been prepared. All of these equations have a logarithmic form.

Note: The street width used in this section is measured from face of curb to face of curb.

Streets Without Curb Split

Curb split is the vertical difference in elevation between curbs at a given street cross section. The gutter flow equation for parabolic crown streets without any curb split is:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 \qquad (3.2.4)$$

where:

- Q = Gutter flow rate, cfs
- S_0 = Street grade, ft/ft
- y_0 = Water depth in the gutter, feet
- K_0, K_1, K_2 = Constant coefficients shown in Table 3.2-3 for different street widths

Table 3.2-3 Coefficients for Equation 3.2.4, Streets Without Curb Split

Street Width* (ft)	Coefficients		
	K_0	K_1	K_2
30	2.85	0.50	3.03
36	2.89	0.50	2.99
40	2.85	0.50	2.89
44	2.84	0.50	2.83
48	2.83	0.50	2.78
60	2.85	0.50	2.74

***Note:** Based on the Transportation Criteria Manual the street width is measured from face of curb to face of curb (FOC-FOC).

Source: City of Austin, Watershed Engineering Division

Streets With Curb Split

The gutter flow equation for parabolic crown streets with curb split is:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K_3 (CS) \qquad (3.2.5)$$

where:

- Q = Gutter flow rate, cfs
- S_0 = Street grade, ft/ft
- y_0 = Water depth in the gutter, feet
- CS = Curb split, feet
- K_0, K_1, K_2, K_3 = Constant coefficients shown in Tables 3.2-4 and 3.2-5

The K values in Equation 3.2.5 are found in Tables 3.2-4 and 3.2-5 for different street widths. Table 3.2-4 is used when calculating the higher gutter flows, and Table 3.2-5 is used when calculating the lower gutter flows.

Table 3.2-4 Coefficients for Equation 3.2.5, Streets With Curb Split – Higher Gutter

Street Width* (ft)	Coefficients				Curb Split Range (ft)
	K_0	K_1	K_2	K_3	
30	2.85	0.50	3.03	-0.131	0.0-0.6
36	2.89	0.50	2.99	-0.140	0.0-0.8
40	2.85	0.50	2.89	-0.084	0.0-0.8
44	2.84	0.50	2.83	-0.091	0.0-0.9
48	2.83	0.50	2.78	-0.095	0.0-1.0
60	2.85	0.50	2.74	-0.043	0.0-1.2

Source: City of Austin, Watershed Engineering Division

Table 3.2-5 Coefficients for Equation 3.2.5, Streets With Curb Split – Lower Gutter

Street Width* (ft)	Coefficients				Curb Split Range (ft)
	K_0	K_1	K_2	K_3	
30	2.70	0.50	2.74	-0.215	0.0-0.6
36	2.74	0.50	2.73	-0.214	0.0-0.8
40	2.75	0.50	2.73	-0.198	0.0-0.8
44	2.76	0.50	2.73	-0.186	0.0-0.9
48	2.77	0.50	2.72	-0.175	0.0-1.0
60	2.80	0.50	2.71	-0.159	0.0-1.2

Source: City of Austin, Watershed Engineering Division

All the crown heights for different street widths are calculated by the following equation:

Crown Height (feet) = 0.5 + [(W 30)/120]

where,

W = street width, feet

Parabolic Crown Location

The gutter flow equation presented for parabolic crowns with split curb heights is based on a procedure for locating the street crown. The procedure allows the street crown to shift from the street center line toward the high ¼ point of the street in direct proportion to the amount of curb split. The maximum curb split occurs with the crown at the ¼ point of the street. The maximum allowable curb split for a street with parabolic crowns is 0.02 feet per foot of street width, except for 30-foot side streets where the maximum curb split is considered to be 5 feet..

Example: Determination of Crown Location

Given: 0.4 feet Design split on 30-foot wide street.

Maximum Curb Split = 0.02 x street width
= 0.2 x 30 feet = 0.6 feet
Maximum Movement = ¼ street width for 30-foot street
= ¼ x 30 feet = 7.5 feet

Split Movement = (Design Split x W/Maximum Split x 4)
= (0.4 x 30/0.6 x 4) = 5 feet

Curb splits that are determined by field survey, whether built intentionally or not, should be considered when determining the capacity of the curb flow.

Special consideration should be given when working with cross sections which have the pavement crown above the top of curb. When the crown exceeds the height of the curb the maximum depth of water is equal to the height of the curb, not the crown height. It should be noted that a parabolic section where the crown equals the top of curb will carry more water than a section which has the crown one (1) inch above the top of curb. For other parabolic roadway sections not included in Tables 3.2-3, 3.2-4, and 3.2-5, see HEC-22 Urban Design Manual by the Federal Highway Administration, dated August 2001.

3.2.4 Storm Water Inlets

Inlets are drainage structures used to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts. Grate inlets subject to traffic should be bicycle safe and be load-bearing adequate. Appropriate frames should be provided.

Inlets used for the drainage of highway surfaces can be divided into three major classes:

- Grate Inlets – These inlets include grate inlets consisting of an opening in the gutter covered by one or more grates, and slotted inlets consisting of a pipe cut along the longitudinal axis with a grate or spacer bars to form slot openings.
- Curb-Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.
- Combination Inlets – These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

Inlets may be classified as being on a continuous grade or in a sump. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sump" condition exists when the inlet is located at a low point and water enters from both directions. Sump areas should have an overflow route or channel.

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

The design of grate inlets will be discussed in subsection 3.2.5, curb inlet design in Section 3.2.6, and combination inlets in Section 3.2.7.

3.2.5 Grate Inlet Design

3.2.5.1 Grate Inlets on Grade

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small.

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. They also handle debris better than other grate inlets but the vanes of the grate must be turned in the proper direction. Where debris is a problem, consideration should be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was made to qualitatively simulate field conditions. Table 3.2-3 presents the results of debris handling efficiencies of several grates. Debris handling efficiencies were based on the total number of simulated leaves arriving at the grate and the number passed.

The ratio of frontal flow to total gutter flow, E_O, for straight cross slope is expressed by the following equation:

$$\mathbf{E_o = Q_w/Q = 1 - (1 - W/T)^{2.67}} \qquad \textbf{(3.2.6)}$$

where:

Q = total gutter flow, cfs

Q_W = flow in width W, cfs

W = width of depressed gutter or grate, ft

T = total spread of water in the gutter, ft

Table 3.2-6 Grate Debris Handling Efficiencies			
Rank	Grate	Longitudinal Slope	
		(0.005)	(0.04)
1	CV - 3-1/4 - 4-1/4	46	61
2	30 - 3-1/4 - 4	44	55
3	45 - 3-1/4 - 4	43	48
4	P - 1-7/8	32	32
5	P - 1-7/8 - 4	18	28
6	45 - 2-1/4 - 4	16	23
7	Reticuline	12	16
8	P - 1-1/8	9	20

Source: "Drainage of Highway Pavements" (HEC-12), Federal Highway Administration, 1984.

Figure 3.2-2 provides a graphical solution of E_o for either depressed gutter sections or straight cross slopes. The ratio of side flow, Q_s, to total gutter flow is:

$$Q_s/Q = 1 - Q_w/Q = 1 - E_o \qquad (3.2.7)$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.09\ (V - V_0) \qquad (3.2.8)$$

where:

V = velocity of flow in the gutter, ft/s (using Q from Figure 3.2-1)

V_o = gutter velocity where splash-over first occurs, ft/s (from Figure 3.2-4)

This ratio is equivalent to frontal flow interception efficiency. Figure 3.2-4 provides a solution of equation 3.2.8, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 3.2-4 is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by:

$$R_s = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})] \qquad (3.2.9)$$

where:

L = length of the grate, ft

Figure 3.2-5 provides a solution to equation 3.2.9.

The efficiency, E, of a grate is expressed as:

$$E = R_fE_o + R_s(1 - E_o) \qquad (3.2.10)$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q[R_fE_o + R_s(1 - E_o)] \qquad (3.2.11)$$

The following example illustrates the use of this procedure.

Given:

$W = 2$ ft

$T = 8$ ft

$S_x = 0.025$ ft/ft

$S = 0.01$ ft/ft
$E_o = 0.69$
$Q = 3.0$ cfs
$V = 3.1$ ft/s
Gutter depression = 2 in

Find:

Interception capacity of:

1. a curved vane grate, and
2. a reticuline grate 2-ft long and 2-ft wide

Solution:

From Figure 3.2-4 for Curved Vane Grate, $R_f = 1.0$
From Figure 3.2-4 for Reticuline Grate, $R_f = 1.0$
From Figure 3.2-5 $R_s = 0.1$ for both grates
From Equation 3.2.11:

$Q_i = 3.0[1.0 \times 0.69 + 0.1(1 - 0.69)] = 2.2$ cfs

For this example, the interception capacity of a curved vane grate is the same as that for a reticuline grate for the sited conditions.

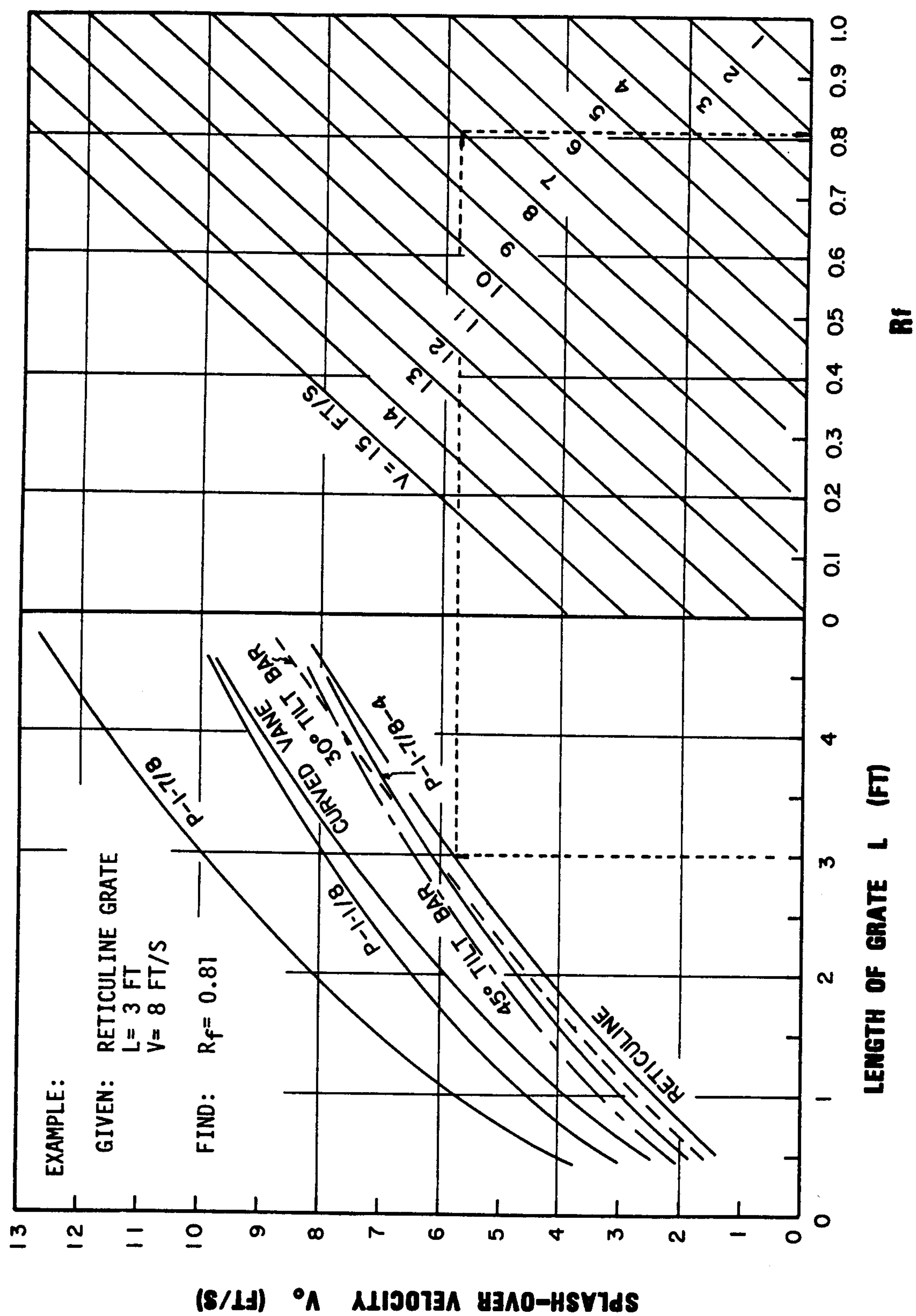


Figure 3.2-4 Grate Inlet Frontal Flow Interception Efficiency
(Source: HEC-12, 1984)

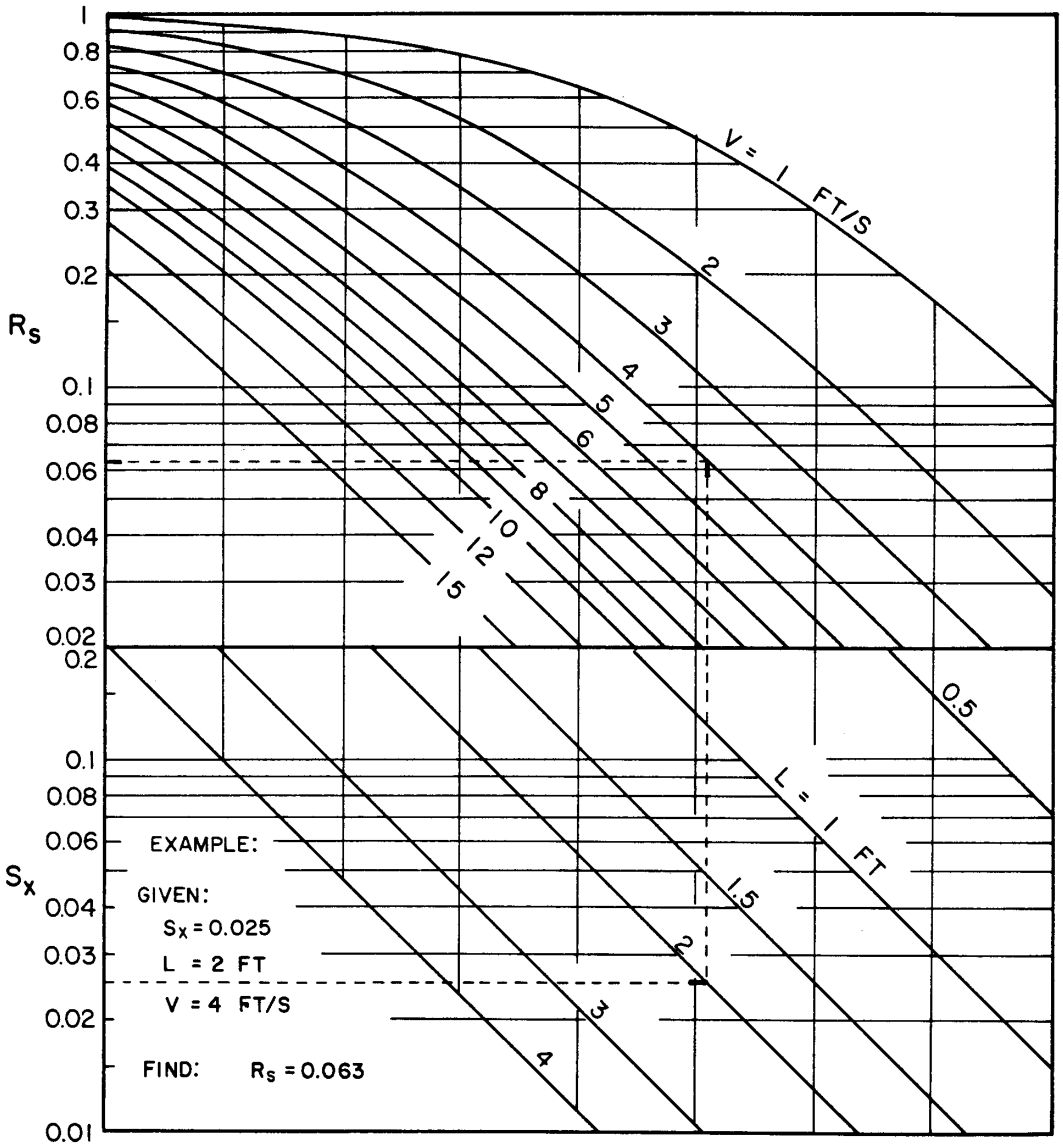


Figure 3.2-5 Grate Inlet Side Flow Interception Efficiency
(Source: HEC-12, 1984)

3.2.5.2 Grate Inlets in Sag

A grate inlet in a sag operates as a weir up to a certain depth, depending on the bar configuration and size of the grate, and as an orifice at greater depths. For a standard gutter inlet grate, weir operation continues to a depth of about 0.4 feet above the top of grate and when depth of water exceeds about 1.4 feet, the grate begins to operate as an orifice. Between depths of about 0.4 feet and about 1.4 feet, a transition from weir to orifice flow occurs.

The capacity of grate inlets operating as a weir is:

$$\mathbf{Q_i = CPd^{1.5}} \qquad \textbf{(3.2.12)}$$

where:

P = perimeter of grate excluding bar widths and the side against the curb, ft
C = 3.0
d = depth of water above grate, ft

and as an orifice is:

$$\mathbf{Q_i = CA(2gd)^{0.5}} \qquad \textbf{(3.2.13)}$$

where:

C = 0.67 orifice coefficient
A = clear opening area of the grate, ft^2
g = 32.2 ft/s^2

Figure 3.2-6 is a plot of equations 3.2.12 and 3.2.13 for various grate sizes. The effect of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used. The following example illustrates the use of this figure.

Given:

A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate.

Q_b = 3.6 cfs
Q = 8 cfs, 25-year storm
T = 10 ft, design
S_x = 0.05 ft/ft
d = TS_x = 0.5 ft

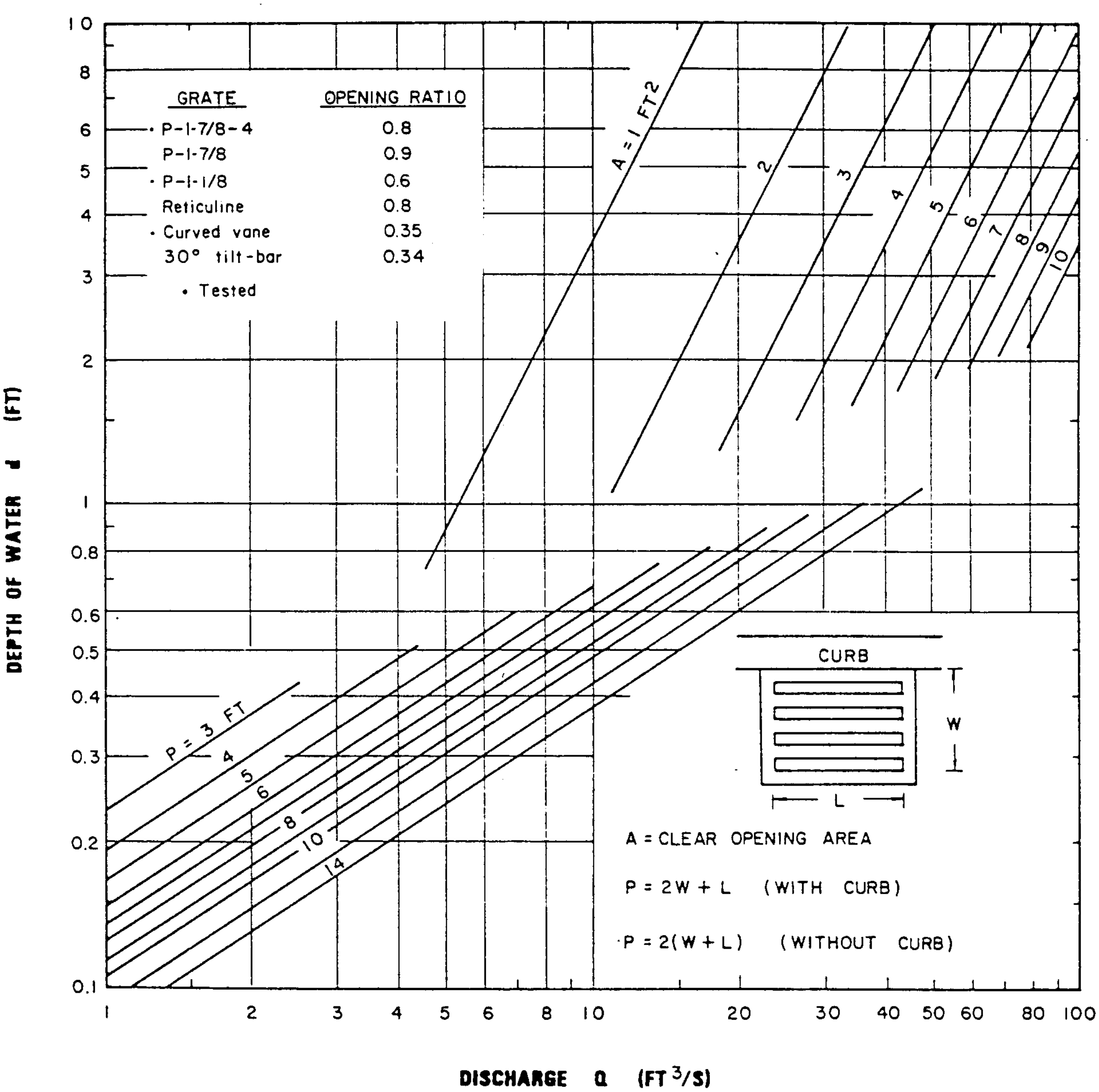
Find:

Grate size for design Q. Check spread at S = 0.003 on approaches to the low point.

Solution:

From Figure 3.2-6, a grate must have a perimeter of 8 ft to intercept 8 cfs at a depth of 0.5 ft.

Some assumptions must be made regarding the nature of the clogging in order to compute the capacity of a partially clogged grate. If the area of a grate is 50% covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50%. For example if a 2-ft x 4-ft grate is clogged so that the effective width is 1 ft, then the perimeter, P = 1 + 4 + 1 = 6 ft, rather than 8 ft, the total perimeter, or 4 ft, half of the total perimeter. The area of the opening would be reduced by 50% and the perimeter by 25%.



Reference: USDOT, FHWA, HEC-12 (1984).

Figure 3.2-6 Grate Inlet Capacity in Sag Conditions
(Source: HEC-12, 1984)

Therefore, assuming 50% clogging along the length of the grate, a 4 x 4, a 2 x 6, or a 3 x 5 grate would meet requirements of an 8-ft perimeter 50% clogged.

Assuming that the installation chosen to meet design conditions is a double 2 x 3 ft grate, for 50% clogged conditions: P = 1 + 6 + 1 = 8 ft

For 25-year flow: d = 0.5 ft (from Figure 3.2-6)

The American Society of State Highway and Transportation Officials (AASHTO) geometric policy recommends a gradient of 0.3% within 50 ft of the level point in a sag vertical curve.

Check T at S = 0.003 for the design and check flow:

Q = 3.6 cfs, T = 8.2 ft (25-year storm) (from Figure 3.2-1)

Thus a double 2 x 3-ft grate inlet with 50% clogging is adequate to intercept the design flow at a spread that does not exceed design spread, and to ensure the spread on the approaches to the low point will not exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet or curb-opening inlet in sag where ponding can occur, as well as flanking inlets on the low gradient approaches.

3.2.6 Curb Inlet Design

3.2.6.1 Curb Inlets on Grade

Following is a discussion of the procedures for the design of curb inlets on grade. Curb-opening inlets are effective in the drainage of pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is determined using Figure 3.2-7. The efficiency of curb-opening inlets shorter than the length required for total interception is determined using Figure 3.2-8.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in the following equation:

$$\mathbf{S_e = S_x + S'_w E_o} \qquad \textbf{(3.2.14)}$$

where:

E_o = ratio of flow in the depressed section to total gutter flow

S'_w = cross slope of gutter measured from the cross slope of the pavement, S_x

S'_w = (a/12W)

where:

a = gutter depression, in

W = width of depressed gutter, ft

It is apparent from examination of Figure 3.2-7 that the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

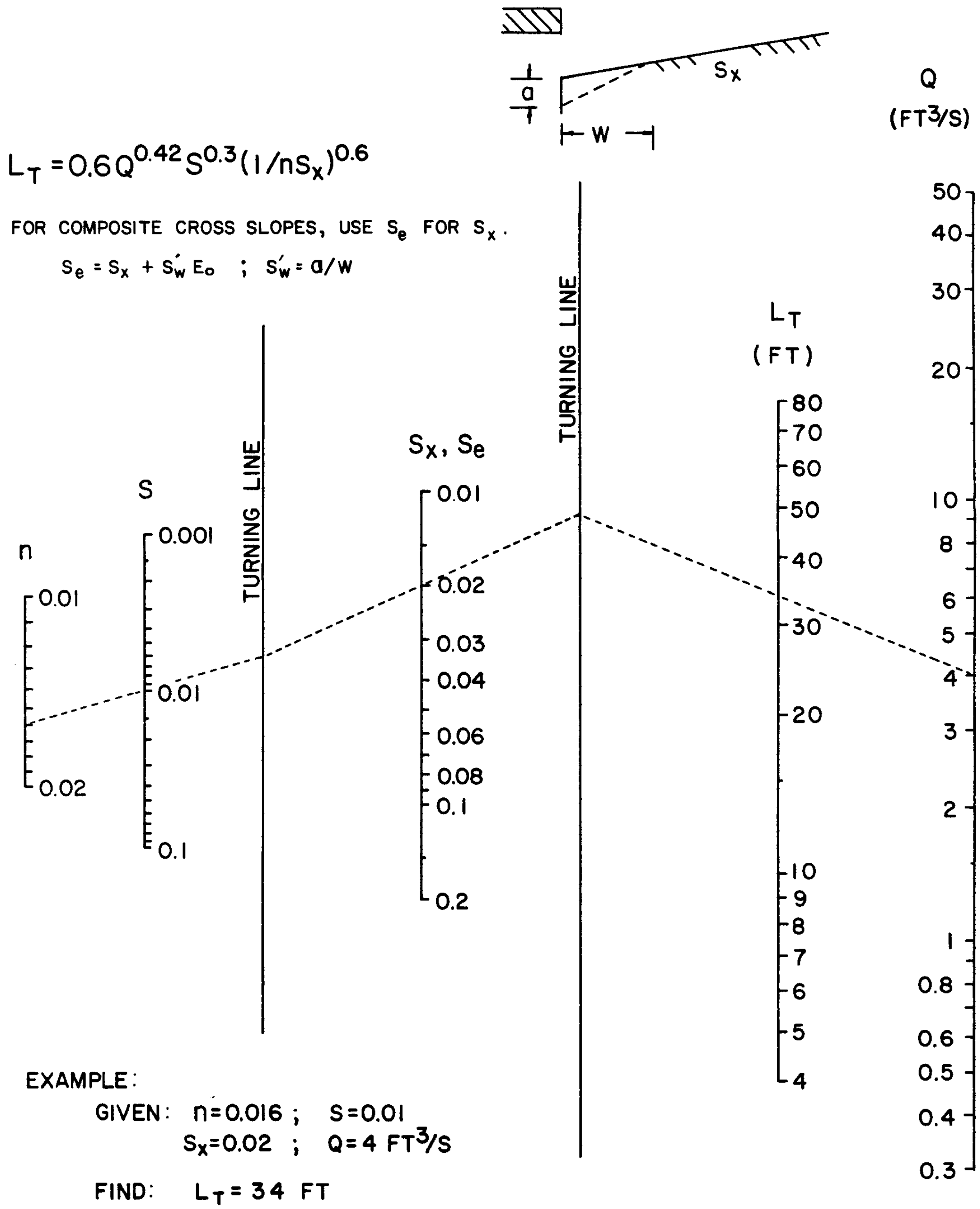


Figure 3.2-7 Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: HEC-12, 1984)

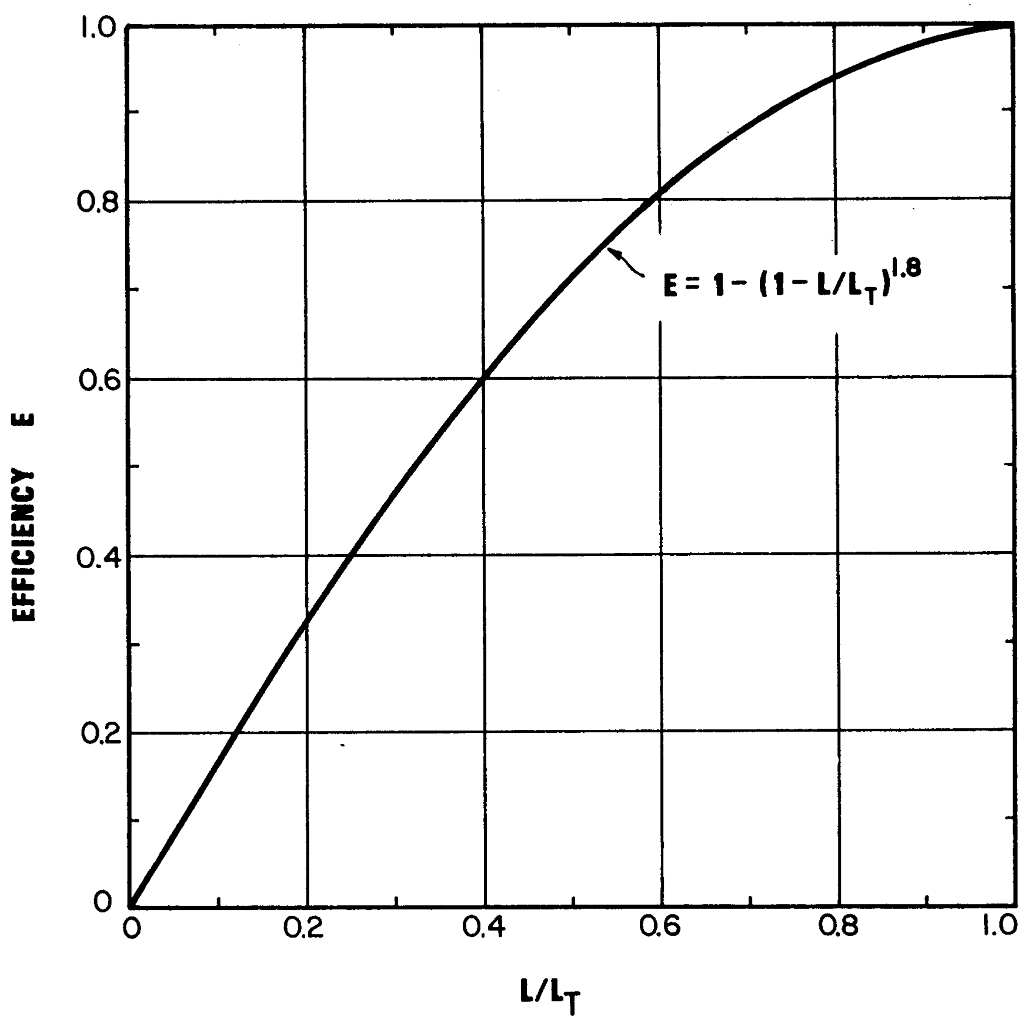


Figure 3.2-8 Curb-Opening and Slotted Drain Inlet Interception Efficiency
(Source: HEC-12, 1984)

Design Steps

Steps for using Figures 3.2-7 and 3.2-8 in the design of curb inlets on grade are given below.

Step 1 Determine the following input parameters:

Cross slope = S_x (ft/ft)

Longitudinal slope = S (ft/ft)

Gutter flow rate = Q (cfs)

Manning's n = n

Spread of water on pavement = T (ft) from Figure 3.2-1

Step 2 Enter Figure 3.2-7 using the two vertical lines on the left side labeled n and S. Locate the value for Manning's n and longitudinal slope and draw a line connecting these points and extend this line to the first turning line.

Step 3 Locate the value for the cross slope (or equivalent cross slope) and draw a line from the point on the first turning line through the cross slope value and extend this line to the second turning line.

Step 4 Using the far right vertical line labeled Q locate the gutter flow rate. Draw a line from this value to the point on the second turning line. Read the length required from the vertical line labeled L_T.

Step 5 If the curb-opening inlet is shorter than the value obtained in Step 4, Figure 3.2-8 can be used to calculate the efficiency. Enter the x-axis with the L/L_T ratio and draw a vertical line upward to the E curve. From the point of intersection, draw a line horizontally to the intersection with the y-axis and read the efficiency value.

Example

Given:

S_x = 0.03 ft/ft
n = 0.016
S = 0.035 ft/ft
Q = 5 cfs
S'_w = 0.083 (a = 2 in, W = 2 ft)

Find:

1. Q_i for a 10-ft curb-opening inlet
2. Q_i for a depressed 10-ft curb-opening inlet with a = 2 in, W = 2 ft, T = 8 ft (Figure 3.2-1)

Solution:

1. From Figure 3.2-7, L_T = 41 ft, L/L_T = 10/41 = 0.24
   From Figure 3.2-8, E = 0.39, Q_i = EQ = 0.39 x 5 = 2 cfs
2. Qn = 5.0 x 0.016 = 0.08 cfs
   S_w/S_x = (0.03 + 0.083)/0.03 = 3.77
   T/W = 3.5 (from Figure 3.2-3)
   T = 3.5 x 2 = 7 ft
   W/T = 2/7 = 0.29 ft
   E_o = 0.72 (from Figure 3.2-2)
   Therefore, $S_e = S_x + S'_w E_o$ = 0.03 + 0.083(0.72) = 0.09
   From Figure 3.2-7, L_T = 23 ft, L/L_T = 10/23 = 0.43
   From Figure 3.2-8, E = 0.64, Q_i = 0.64 x 5 = 3.2 cfs

The depressed curb-opening inlet will intercept 1.6 times the flow intercepted by the undepressed curb opening and over 60% of the total flow.

3.2.6.2 Curb Inlets in Sump

For the design of a curb-opening inlet in a sump location, the inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The capacity of curb-opening inlets in a sump location can be determined from Figure 3.2-9, which accounts for the operation of the inlet as a weir and as an orifice at depths greater than 1.4h. This figure is applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter depression. The height (h) in the figure assumes a vertical orifice opening (see sketch on Figure 3.2-9). The weir portion of Figure 3.2-9 is valid for a depressed curb-opening inlet when $d \leq (h + a/12)$.

The capacity of curb-opening inlets in a sump location with a vertical orifice opening but without any depression can be determined from Figure 3.2-10. The capacity of curb-opening inlets in a sump location with other than vertical orifice openings can be determined by using Figure 3.2-11.

Design Steps

Steps for using Figures 3.2-9, 3.2-10, and 3.2-11 in the design of curb-opening inlets in sump locations are given below.

Step 1 Determine the following input parameters:
Cross slope = S_x (ft/ft)
Spread of water on pavement = T (ft) from Figure 3.2-1
Gutter flow rate = Q (cfs) or dimensions of curb-opening inlet [L (ft) and H (in)]
Dimensions of depression if any [a (in) and W (ft)]

Step 2 To determine discharge given the other input parameters, select the appropriate figure (3.2-9, 3.2-10, or 3.2-11 depending on whether the inlet is in a depression and if the orifice opening is vertical).

Step 3 To determine the discharge (Q), given the water depth (d), locate the water depth value on the y-axis and draw a horizontal line to the appropriate perimeter (p), height (h), length (L), or width x length (hL) line. At this intersection draw a vertical line down to the x-axis and read the discharge value.

Step 4 To determine the water depth given the discharge, use the procedure described in Step 3 except enter the figure at the value for the discharge on the x-axis.

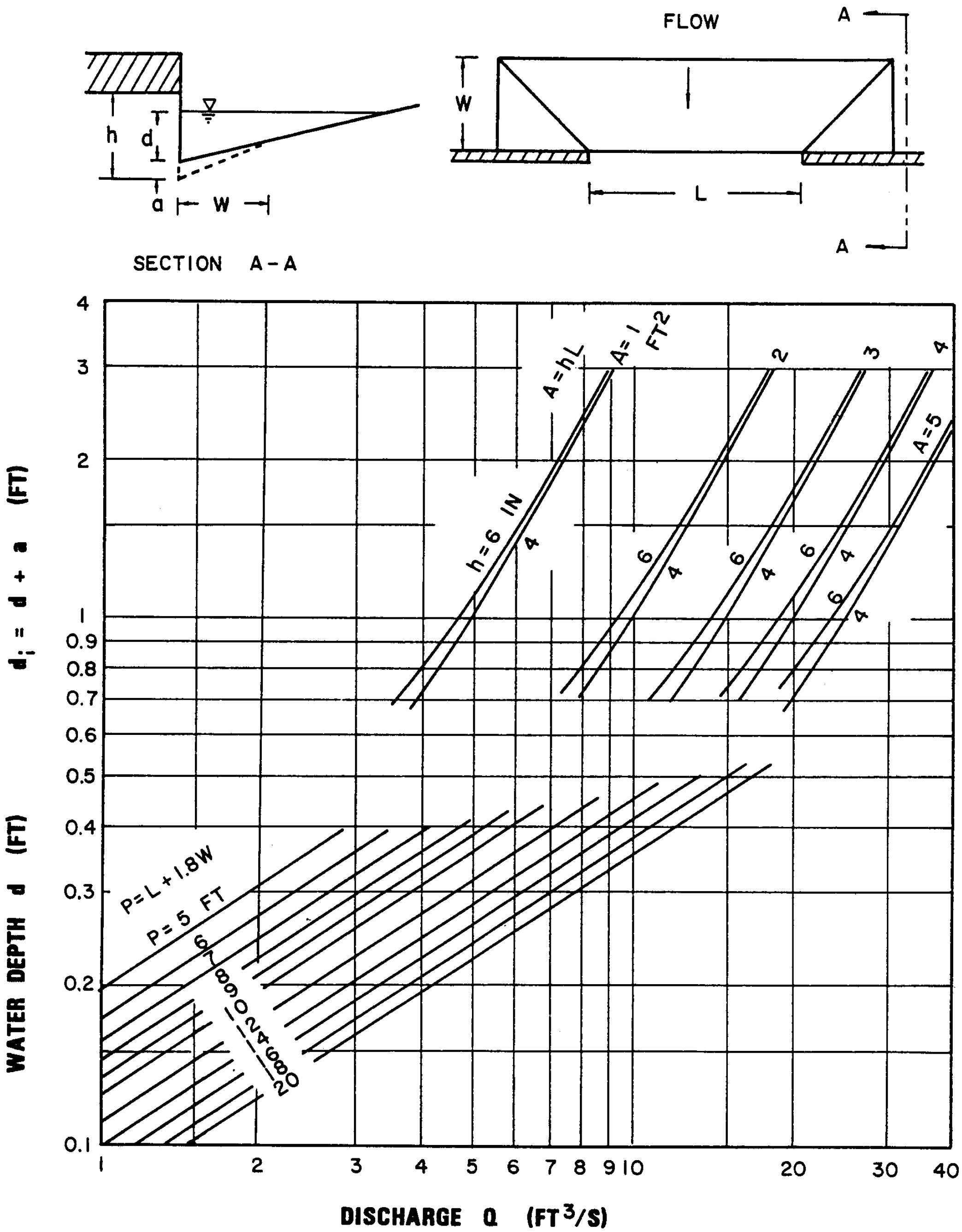


Figure 3.2-9 Depressed Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

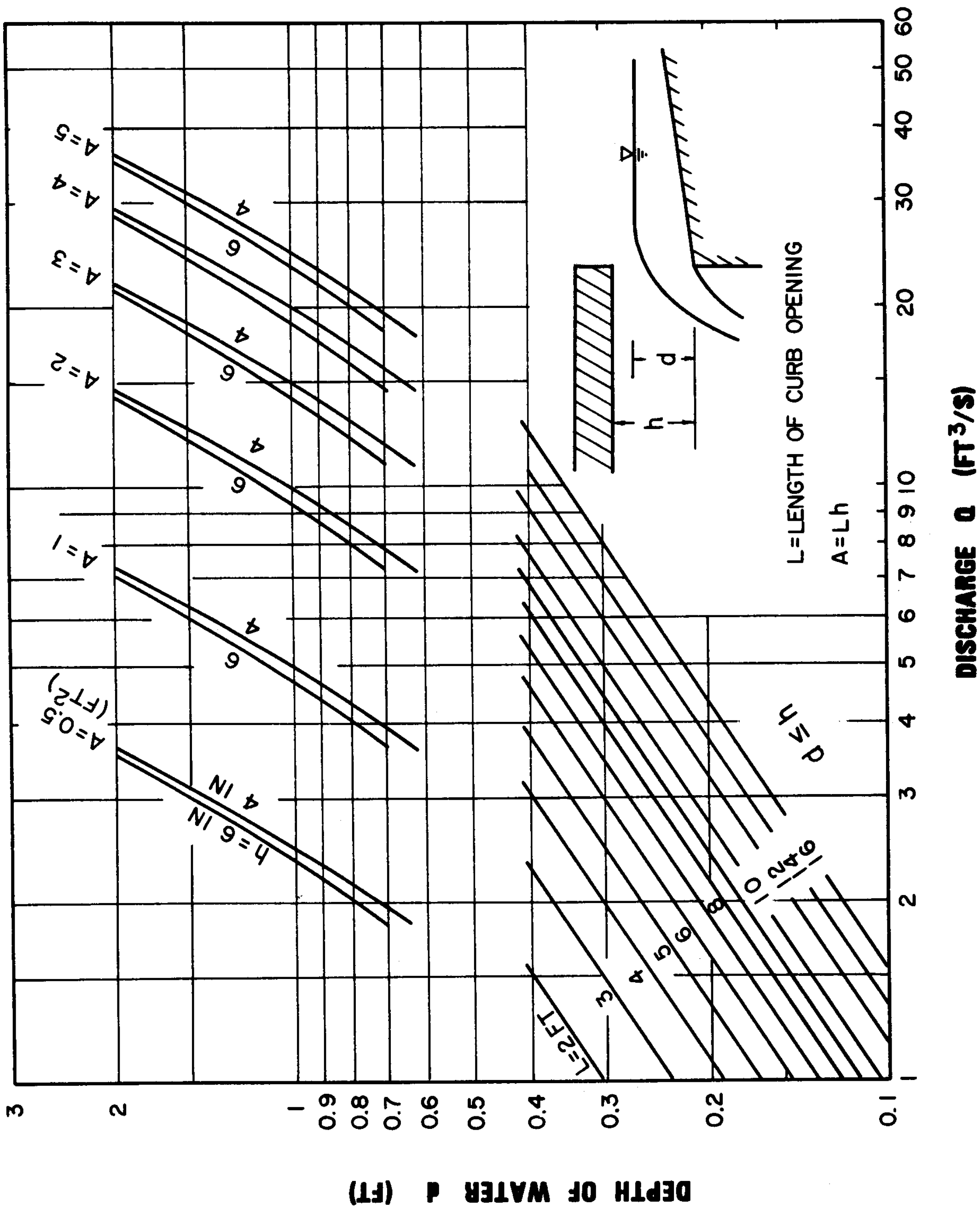


Figure 3.2-10 Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

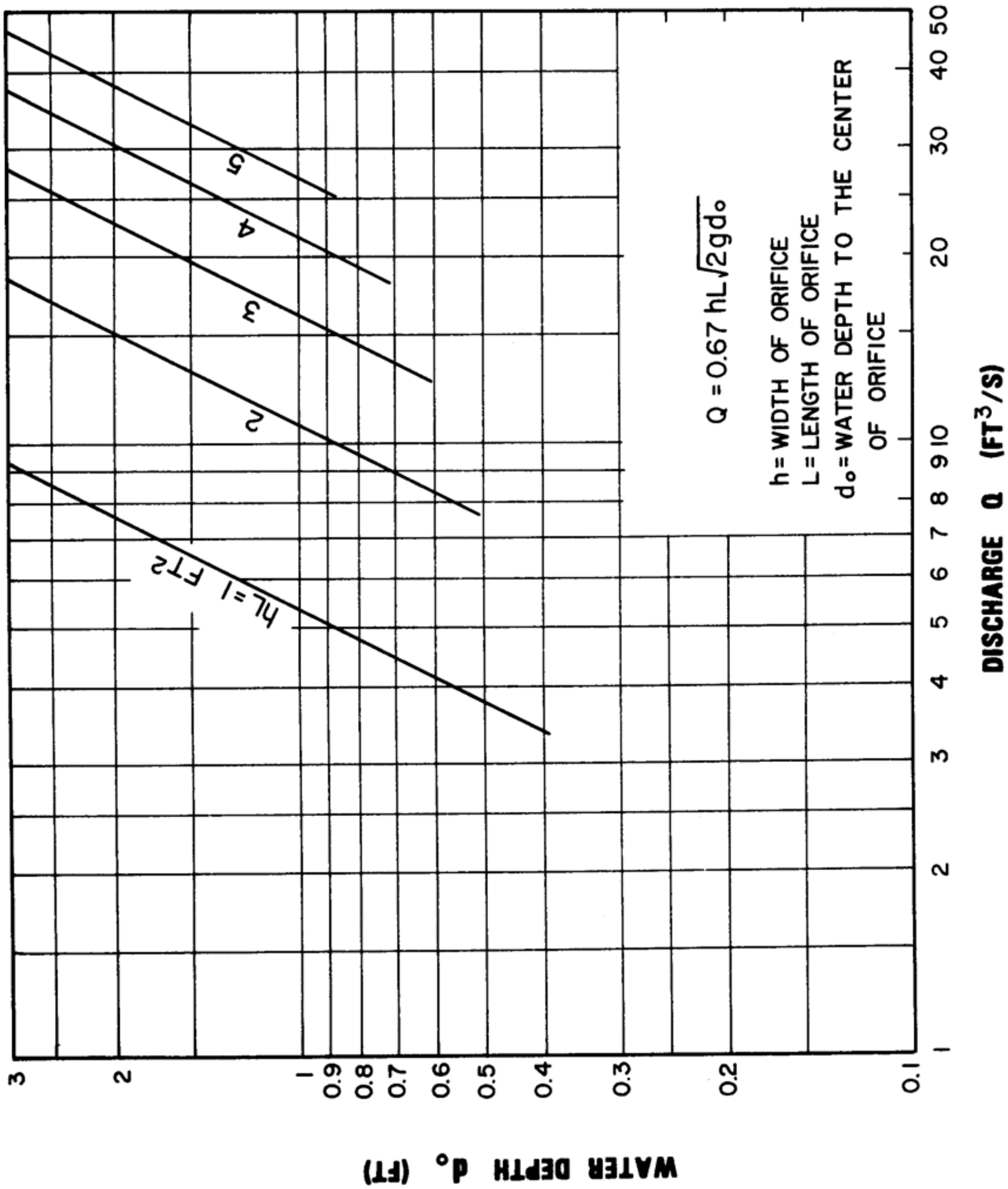


Figure 3.2-11 Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats
(Source: AASHTO Model Drainage Manual, 1991)

Example:"

Given:

Curb-opening inlet in a sump location

L = 5 ft

h = 5 in

1. Undepressed curb opening
   S_x = 0.05 ft/ft
   T = 8 ft
2. Depressed curb opening
   S_x = 0.05 ft/ft
   a = 2 in
   W = 2 ft
   T = 8 ft

Find:

Discharge Q_i

Solution:

1. d = TS_x = 8 x 0.05 = 0.4 ft
   d < h
   From Figure 3.2-10, Q_i = 3.8 cfs
2. d = 0.4 ft
   h + a/12 = (5 + 2/12)/12 = 0.43 ft

   since d < 0.43 the weir portion of Figure 3.2-9 is applicable (lower portion of the figure).

   P = L + 1.8W = 5 + 3.6 = 8.6 ft
   From Figure 3.2-9, Q_i = 5 cfs

At d = 0.4 ft, the depressed curb-opening inlet has about 30% more capacity than an inlet without depression.

3.2.7 Combination Inlets

3.2.7.1 Combination Inlets on Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of Figures 3.2-4, 3.2-5, and 3.2-6.

3.2.7.2 Combination Inlets in Sump

All debris carried by storm water runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity. Assuming complete clogging of the grate, Figures 3.2-9, 3.2-10, and 3.2-11 for curb-opening inlets should be used for design.

3.2.8 Closed Conduit Systems (Storm Drains/Sewers)

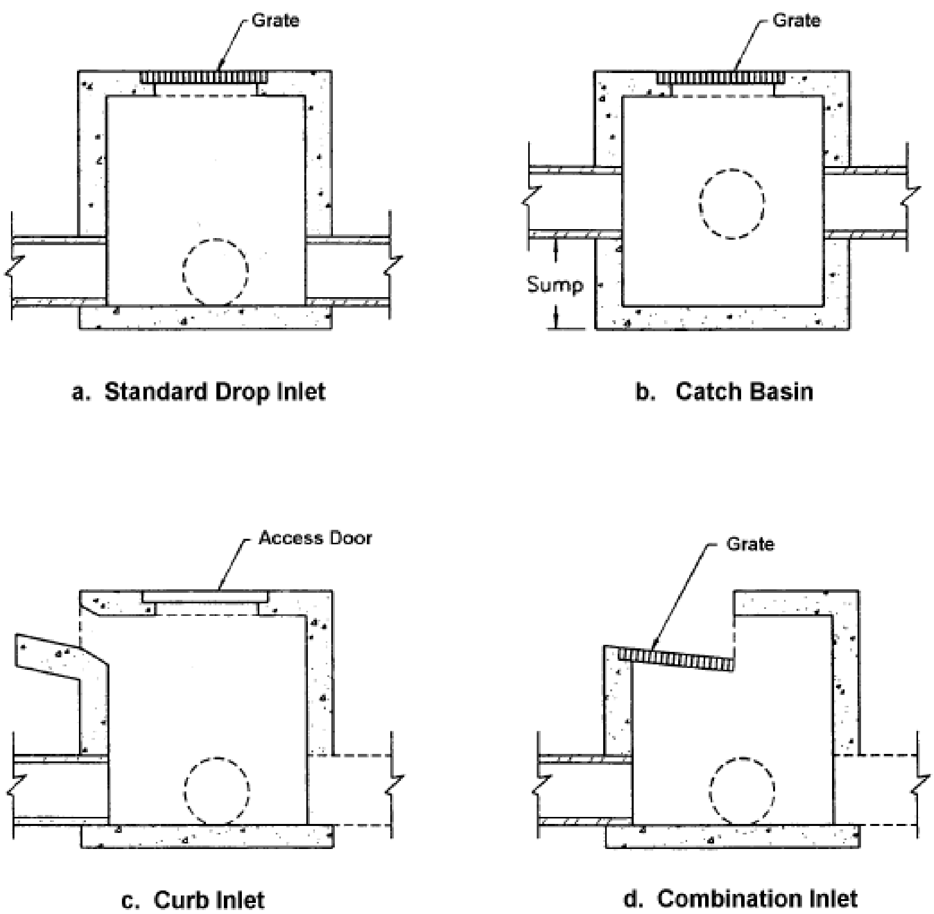
Storm drain pipe systems, also known as *storm sewers*, are pipe conveyances used for transporting runoff from roadway and other inlets to outfalls at other structural storm water controls and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

Closed conduit system are composed of different lengths and sizes of conduits (system segments) connected by appointment structures (system nodes). Segments are most often circular pipe, but can be a box or other enclosed conduit. Materials used are usually corrugated metal, plastic, and concrete but may be of other materials.

Appurtenant structures serve many functions. Inlets, access holes, and junction chambers are presented in sections 3.2.8.1 through 3.2.8.3.

3.2.8.1 Inlets

The primary function is to allow surface water to enter the closed conduit system. Inlet structures may also serve as access points for cleaning and inspection. Typical inlets structures are a standard drop inlet, catch basin, curb inlet, combination inlet, and Y inlet. (See Figures 3.2-12 and 3.2-13).



a. Standard Drop Inlet

b. Catch Basin

c. Curb Inlet

d. Combination Inlet

Figure 3.2-12 Inlet Structures
(HEC 22, 2001)

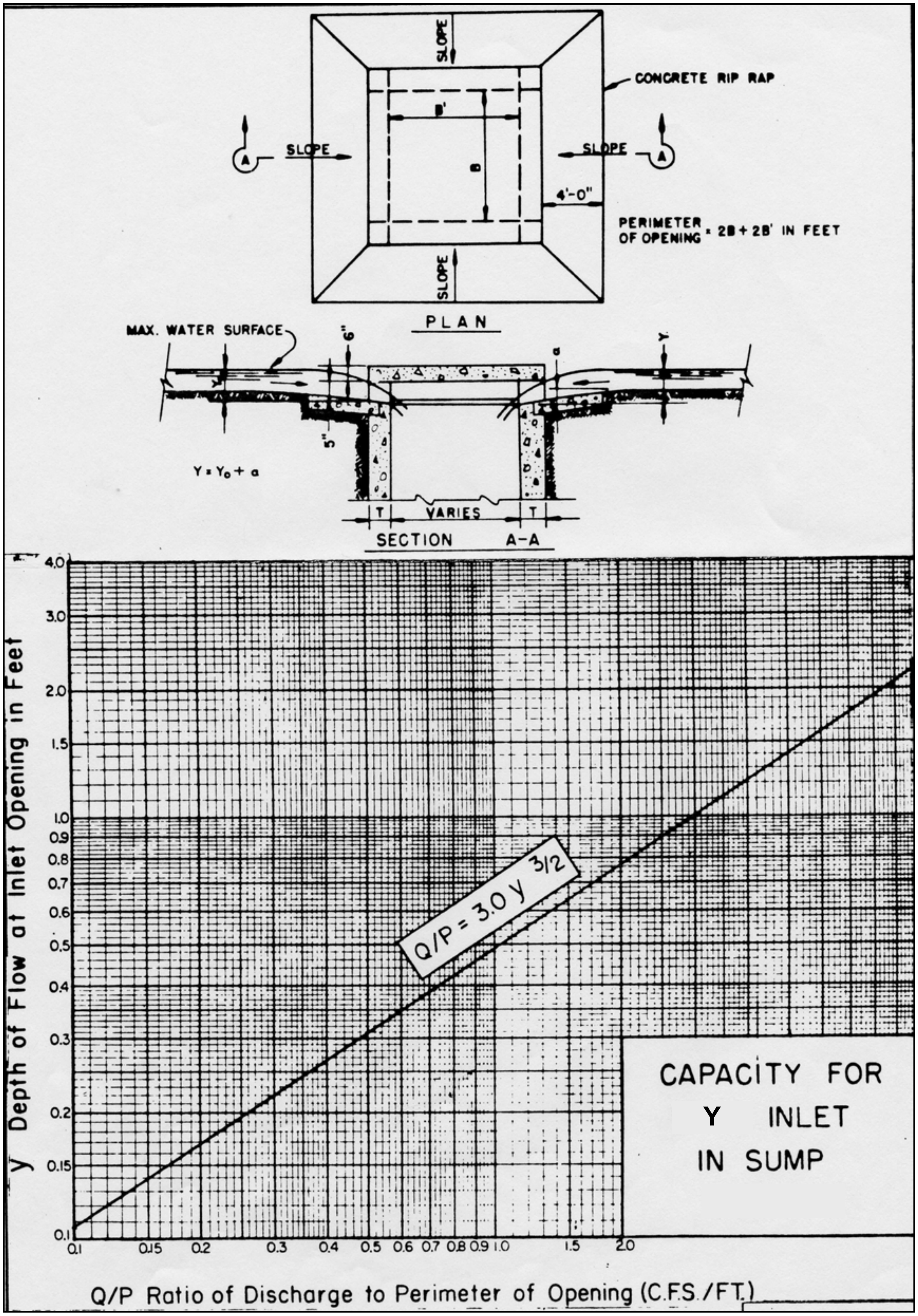


Figure 3.2-13 Capacity for Y Inlet in Sump
(Fort Worth, 1967)

Inlet structures are located at the upstream end and at intermediate points within the closed conduit system. Inlet placement is generally a trial and error procedure that attempts to produce the most economical and hydraulically effective system (HEC 22, 2001).

3.2.8.2 Access Holes (Manholes)

The primary function of an access hole is to provide access to the closed conduit system. An access hole can also serve as a flow junction and can provide ventilation and pressure relief. Typical access holes are shown in Figures 3.2-14 and 3.2-15 (HEC 22, 2001). The materials commonly used for access hole construction are precast concrete and cast-in-place concrete.

Spacing criteria are typically established by local agencies. At a minimum, access holes should be located at the following points:

- Where two or more storm drains converge
- Where pipe sizes change
- Where a change in alignment occurs
- Where a change in grade occurs

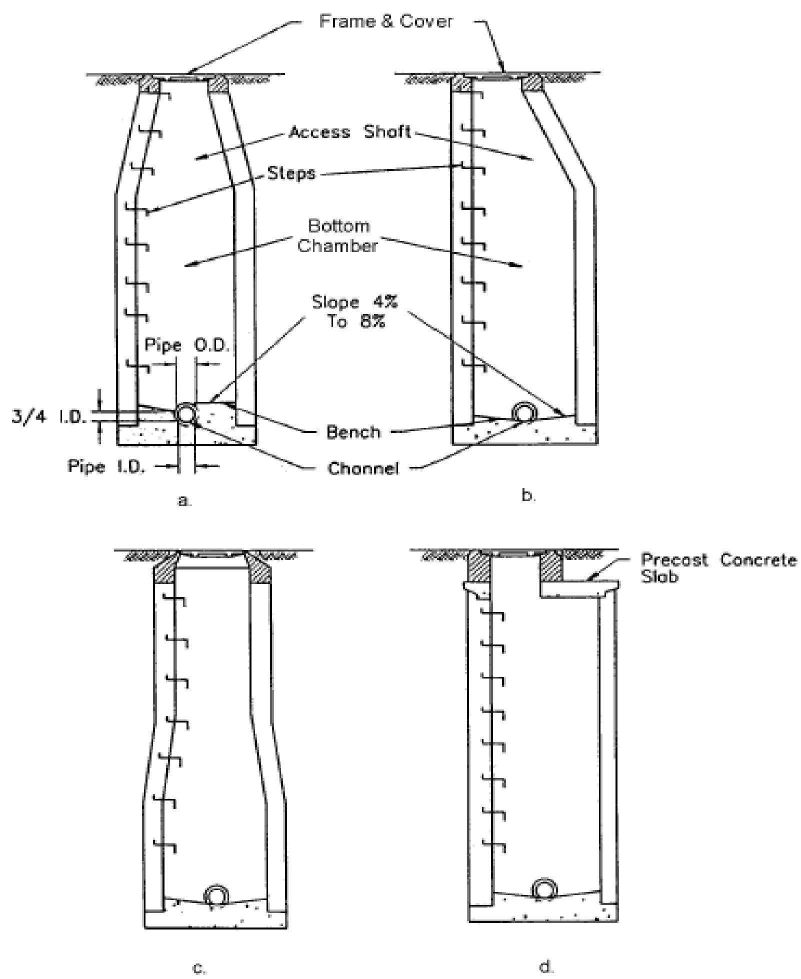


Figure 3.2-14 Typical Access Hole Configurations.
(HEC22, 2001)

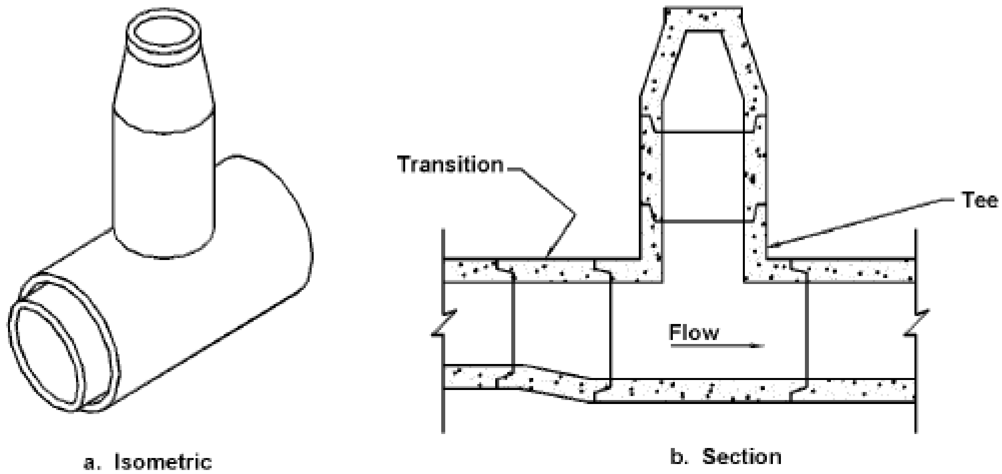


Figure 3.2-15 "Tee" Access Hole for Large Storm Drains
(HEC 22, 2001)

Access holes may be needed at intermediate points along straight runs of closed conduits. Table 3.2-7 gives suggested maximum spacing criteria.

Table 3.2-7 Access Hole Spacing Criteria
(HEC 22, 2001)

Pipe Size (inches)	Suggested Maximum Spacing (feet)
12-24	300
27-36	400
42-54	500
60 and up	1000

3.2.8.3 Junction Chambers

A junction chamber, or junction box, is a special design underground chamber used to join two or more large storm drain conduits. This type of structure is usually required where storm drains are larger than the size that can be accommodated by standard access holes. For smaller diameter storm drains, access holes are typically used instead of junction chambers. Junction chambers by definition do not need to extend to the ground surface and can be completely buried. However, it is recommended that riser structures be used to provide surface access and/or to intercept surface runoff.

Materials commonly used for junction chamber construction include pre-cast concrete and cast-in-place concrete. On storm drains constructed of corrugated steel, the junction chambers are sometimes made of the same material.

To minimize flow turbulence in junction boxes, flow channels and benches are typically built into the bottom of the chambers. Where junction chambers are used as access points for the storm drain system, their location should adhere to the spacing criteria outlined in Table 3.2-7.

3.2.8.4 Design Criteria

Specific design criteria will likely vary from community to community. In the design of closed conduit systems, the following are offered for consideration in setting local criteria:

- For ordinary conditions, storm drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning Formula is recommended for capacity calculations.
- The maximum hydraulic gradient should not produce a velocity that exceeds 15 ft/s.
- The minimum desirable physical slope should be 0.5% or the slope that will produce a velocity of 2.5 feet per second when the storm sewer is flowing full, whichever is greater.
- If the potential water surface elevation exceeds 1 foot below ground elevation for the design flow, the top of the pipe, or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the hydraulic grade line.

3.2.8.5 General Design Procedure

The design of storm drain systems generally follows these steps:

Step 1 Determine inlet location and spacing as outlined earlier in this section.

Step 2 Prepare a tentative plan layout of the storm sewer drainage system including:

a. Location of storm drains
b. Direction of flow
c. Location of manholes
d. Location of existing facilities such as water, gas, or underground cables

Step 3 Determine drainage areas and compute runoff using the Rational Method

Step 4 After the tentative locations of inlets, drain pipes, and outfalls (including tailwaters) have been determined and the inlets sized, compute the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from the upstream end of a line downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream. The storm drain system design computation form (Figure 3.2-25) can be used to summarize hydrologic, hydraulic and design computations.

Step 5 Examine assumptions to determine if any adjustments are needed to the final design.

The rate of discharge at any point in the storm drainage system is not the sum of the inlet flow rates of all inlets above the section of interest. It is generally less that this total. The Rational Method is the most common means of determining design discharges for storm drain design. The time of concentration is very influential in the determination of the design discharge using the Rational Method. The time of concentration is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of interest. The designer is usually concerned with two different times of concentration: one for inlet spacing and the other for pipe sizing. The time of concentration for inlet spacing is the time required for water to flow from the hydraulically most distant point of the unique drainage area contributing only to that inlet. Typically, this is the sum of the times required for water to travel overland to the pavement gutter and along the length of the gutter between inlets. If the total time of concentration to the upstream inlet is less than five minutes, a minimum time of concentration of five minutes is used as the duration of rainfall. The time of concentration for each successive inlet should be determined independently in the same manner as was used for the first inlet.

The time of concentration for pipe sizing is defined as the time required for water to travel for the most hydraulically distant point in the total contributing watershed to the design point. Typically, this time consists of two components: (1) the time for overland and gutter flow to reach the first inlet, and (2) the time to flow through the storm drainage system to the point of interest.

The flow path having the longest time of concentration to the point of interest in the storm drainage system will usually define the duration used in selecting the intensity value in the Rational Method. Exceptions to the general application of the Rational Equation exist. For example, a small relatively impervious area within a larger drainage area may have an independent discharge higher than that of the total area. This anomaly may occur because of the higher runoff coefficient (C value) and higher intensity resulting from a short time of concentration. If an exception does exist, it can generally be classified as one of two exception scenarios.

The first exception occurs when a highly impervious section exists at the most downstream area of a watershed and the total upstream area flows through the lower impervious area. When this situation occurs, two separate calculations should be made.

- First, calculate the runoff from the total drainage area with its weighed C value and the intensity associated with the longest time of concentration.
- Secondly, calculate the runoff using only the smaller less pervious area. The typical procedure would be followed using the C value for the small less pervious area and the intensity associated with the shorter time of concentration.

The results of these two calculations should be compared and the largest value of discharge should be used for design.

The second exception exists when a smaller less pervious area is tributary to the larger primary watershed. When this scenario occurs, two sets of calculations should also be made.

- First, calculate the runoff form the total drainage area with its weighted C value and the intensity associated with the longest time of concentration.
- Secondly, calculate the runoff to consider how much discharge from the larger primary area is contributing at the same time the peak from the smaller less pervious tributary area is occurring. When the small area is discharging, some discharge from the larger primary area is also contributing to the total discharge. In this calculation, the intensity associated with the time of concentration from the small less pervious area is used. The portion of the larger primary area to be considered is determined by the following: A_c=A (t_{c1}/t_{c2}).

A_c is the most downstream part of the larger primary area that will contribute to the discharge during the time of concentration associated with the smaller, less pervious area. A is the area of the larger primary area, t_{c1} is the time of concentration of the smaller, less pervious, tributary area, and t_{c2} is the time of concentration associated with the larger primary area as is used in the first calculation. The C value to be used in this computation should be the weighted C value of the smaller less pervious tributary area and the area A_c. The area to be used in the Rational Method would be the area of the less pervious area plus A_c. The second calculation should only be considered when the less pervious area is tributary to the area with the longer time of concentration and is at or near the downstream end of the total drainage area.

Finally, the results of these calculations should be compared and the largest value of discharge should be used for design.

3.2.8.6 Capacity Calculations

The design procedures presented here assume flow within each storm drain segment is steady and uniform. This means the discharge and flow depth in each segment are assumed to be constant with respect to time and distance. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered to be constant.

In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption.

Although at times flow in a closed conduit may be under pressure or at other times the conduit may flow partially full, the usual design assumption is that the conduit is flowing fill but not under pressure. Under this assumption the rate of head loss is the same as the slope of the pipe (S_f=S, ft/ft). Designing for full flow is a conservative assumption since the peak flow actually occurs at 93 percent of full flow.

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning's Formula, expressed by the following equation:

$$\mathbf{V = (1.486/n)\ R^{2/3}\ S^{1/2}} \qquad \mathbf{(3.2.15)}$$

where:

V = mean velocity of flow, ft/s

R = the hydraulic radius, ft - defined as the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)

S = the slope of hydraulic grade line, ft/ft

n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$\mathbf{Q = (1.486/n)\ A\ R^{2/3}\ S^{1/2}} \qquad \mathbf{(3.2.16)}$$

where:

Q = rate of flow, cfs

A = cross sectional area of flow, ft^2

For pipes flowing full, the area is $(\pi/4)D^2$ and the hydraulic radius is D/4, so, the above equations become:

$$\mathbf{V = [0.590\ D^{2/3}S^{1/2}]/n} \qquad \mathbf{(3.2.17)}$$

$$\mathbf{Q = [0.463\ D^{8/3}S^{1/2}]/n} \qquad \mathbf{(3.2.18)}$$

where:

D = diameter of pipe, ft

S = slope of the pipe = S_f hydraulic grade line, ft/ft

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$\mathbf{H_f = [0.453\ n^2V^2L]/[R^{4/3}]} \qquad \mathbf{(3.2.19)}$$

$$\mathbf{H_f = [(2.87\ n^2V^2L]/[D^{4/3}]} \qquad \mathbf{(3.2.20)}$$

$$\mathbf{H_f = [(185n^2(V^2/2g)L]/[D^{4/3}]} \qquad \mathbf{(3.2.21)}$$

where:

H_f = total head loss due to friction, ft (S_f x L)
n = Manning's roughness coefficient
D = diameter of pipe, ft
L = length of pipe, ft
V = mean velocity, ft/s
R = hydraulic radius, ft
g = acceleration of gravity = 32.2 ft/sec^2

A nomograph solution of Manning's Equation for full flow in circular conduits is presented in Figure 3.2-16. Representative values of the Manning's coefficient for various storm drain materials are provided in Table 3.2-9. It should be remembered that the values in the table are for new pipe tested in a laboratory. Actual field values for conduits may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

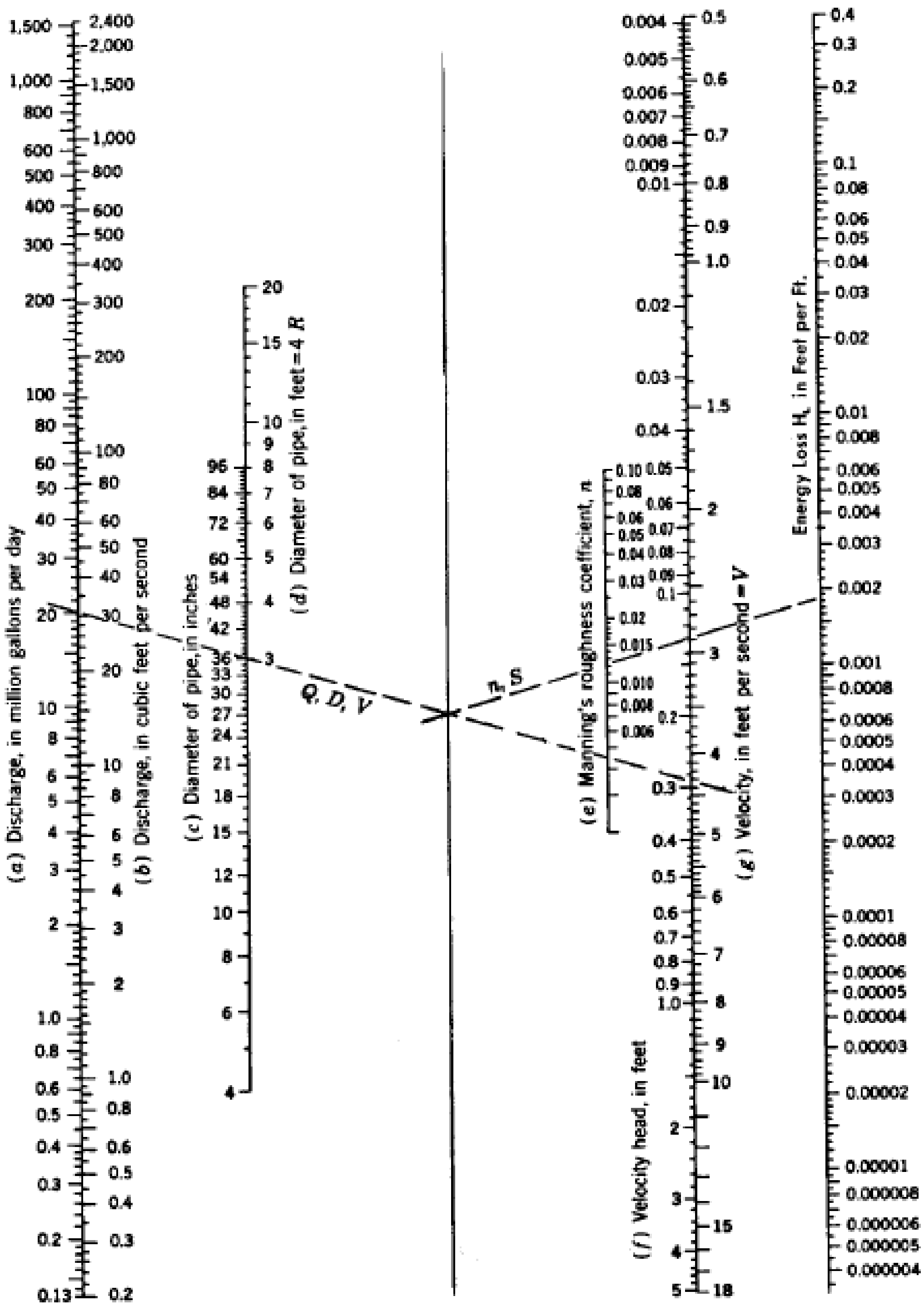
Figure 3.2-17 illustrates storm drain capacity sensitivity to the parameters in the Manning's equation. This figure can be used to study the effect changes in individual parameters will have on storm drain capacity. For example, if the diameter of a storm drain is doubled, its capacity will be increased by a factor of 6.0; if the slope is doubled, the capacity is increased by a factor of 1.4; however, if the roughness is doubled, the pipe capacity will be reduced by 50 percent.

The hydraulic elements graph in Figures 3.2-18a and 3.2-18b is provided to assist in the solution of the Manning's equation for part full flow in storm drains. The hydraulic elements chart shows the relative flow conditions at different depths in a circular pipe and makes the following important points:

1. Peak flow occurs at 93 percent of the height of the pipe. This means that if the pipe is designed for full flow, the design will be slightly conservative.
2. The velocity in a pipe flowing half-full is the same as the velocity for full flow.
3. Flow velocities for flow depths greater than half-full are greater than velocities at full flow.
4. As the depth of flow drops below half-full, the flow velocity drops off rapidly. The shape of a storm drain conduit also influences its capacity. Although most storm drain conduits are circular, a significant increase in capacity can be realized by using an alternate shape. Table 3.2-8 provides a tabular listing of the increase in capacity which can be achieved using alternate conduit shapes that have the same height as the original circular shape, but have a different cross sectional area. Although these alternate shapes are generally more expensive than circular shapes, their use can be justified in some instances based on their increased capacity.

Table 3.2-8 Increase in Capacity of Alternate Conduit Shapes Based on a Circular Pipe with the Same Height
(HEC-22, 2001)

	Area (Percent Increase)	Conveyance (Percent Increase)
Circular	**--**	**--**
Oval	**63**	**87**
Arch	**57**	**78**
Box (B = D)	**27**	**27**



Alignment chart for energy loss in pipes, for Manning's formula.
Note: Use chart for flow computations, $H_L = S$

Figure 3.2-16 Solution of Manning's Equation for Flow in Storm Drains-English Units
(Taken from "Modern Sewer Design" by American Iron and Steel Institute)

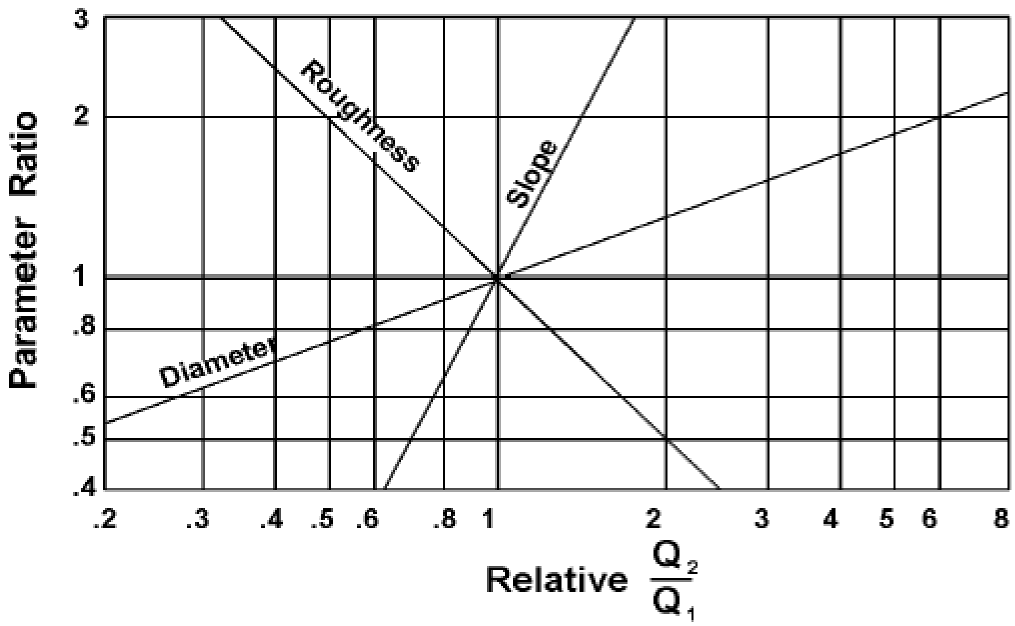


Figure 3.2-17 Storm Drain Capacity Sensitivity
(HEC 22, 2001)

Table 3.2-9 Manning's Coefficients for Storm Drain Conduits
(HEC 22, 2001)

Type of Culvert	Roughness or Corrugation	Manning's n
Concrete Pipe	**Smooth**	**0.010-0.011**
Concrete Boxes	**Smooth**	**0.012-0.015**
Spiral Rib Metal Pipe	**Smooth**	**0.012-0.013**
Corrugated Metal Pipe, Pipe-Arch and Box (Annular or Helical Corrugations -- see Figure B-3 in Reference 2, Manning's n varies with barrel size)	**68 by 13 mm 2-2/3 by 1/2 in Annular**	**0.022-0.027**
	68 by 13 mm 2-2/3 by 1/2 in Helical	**0.011-0.023**
	150 by 25 mm 6 by 1 in Helical	**0.022-0.025**
	125 by 25 mm 5 by 1 in	**0.025-0.026**
	75 by 25 mm 3 by 1 in	**0.027-0.028**
	150 by 50 mm 6 by 2 in Structural Plate	**0.033-0.035**
	230 by 64 mm 9 by 2-1/2 in Structural Plate	**0.033-0.037**
Corrugated Polyethylene	**Smooth**	**0.009-0.015**
Corrugated Polyethylene	**Corrugated**	**0.018-0.025**
Polyvinyl chloride (PVC)	**Smooth**	**0.009-0.011**

***NOTE: The Manning's n values indicated in this table were obtained in the laboratory and are supported by the provided reference. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.**

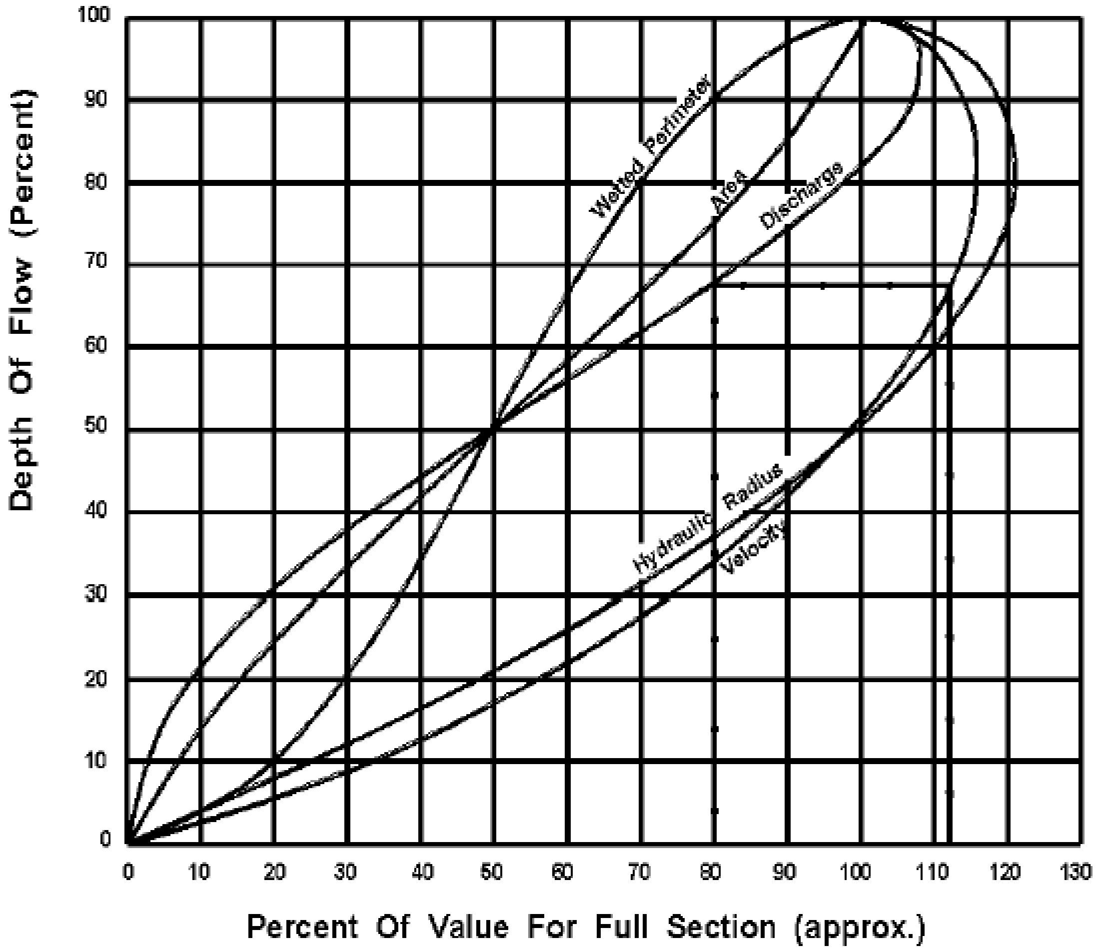


Figure 3.2-18a Hydraulic Elements of Circular Section
(HEC 22, 2001)

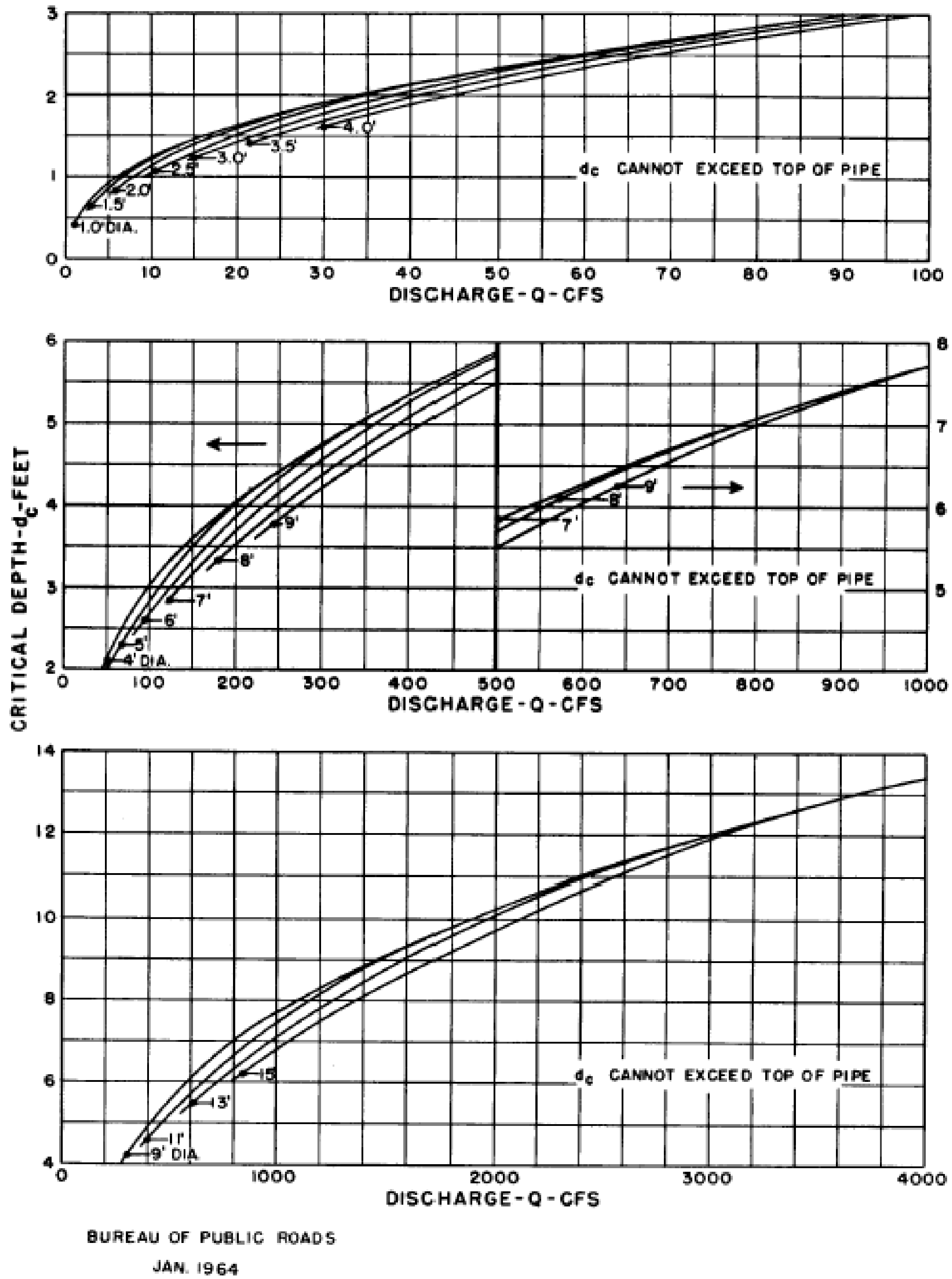


Figure 3.2-18b Critical Depth in Circular Pipe-English Units
(HEC 22, 2001)

3.2.8.7 Minimum Grades and Desirable Velocities

The minimum slopes are calculated by the modified Manning's formula:

$$S = [(nV)^2]/[2.208R^{4/3}] \qquad (3.2.22)$$

where:

- S = the slope of the hydraulic grade line, ft/ft
- n = Manning's roughness coefficient
- V = mean velocity of flow, ft/s
- R = hydraulic radius, ft (area dived by wetted perimeter)

For circular conduits flowing full but not under pressure, R=D/4, and the hydraulic grade line is equal to the slope of the pipe. For these conditions equation 3.2.22 may be expressed as:

$$S = 2.87(nV)^2/D^{4/3} \qquad (3.2.23)$$

For a minimum velocity of 2.5 fps, the minimum slope equation becomes:

$$S = 17.938(n^2/D^{4/3}) \qquad (3.2.24)$$

where:

- D = diameter, ft

Table 3.2-10 gives minimum slopes for two commonly used materials: concrete pipe with an n-value of 0.013 and corrugated metal pipe with an n-value of 0.024.

<u>Minimum Grades</u>

Storm drains should operate with velocities of flow sufficient to prevent excessive deposits of solid materials; otherwise objectionable clogging may result. The controlling velocity is near the bottom of the conduit and considerably less than the mean velocity of the sewer. Storm drains shall be designed to have a minimum mean velocity flowing full of 2.5 fps. Table 3.2-10 gives minimum slopes for two commonly used materials: concrete pipe (n = 0.013) and corrugated metal pipe (n = 0.024), flowing at 2.5 fps.

<u>Desirable Velocities</u>

Velocities in sewers are important mainly because of the possibilities of excessive erosion on the storm drain inverts. Table 3.2-11 shows the desirable velocities for most storm drainage design.

Table 3.2-10 Minimum Grades for Storm Drains for 2.5 fps

Pipe Size (inches)	Concrete Pipe (n = 0.013) Slope ft/ft	Corrugated Metal Pipe (n = 0.024) Slope ft/ft
15	0.0023	0.0077
18	0.0018	0.0060
21	0.0014	0.0049
24	0.0012	0.0041
27	0.0010	0.0035
30	0.0009	0.0030
33	0.0008	0.0027
36	0.0007	0.0024
39	0.0006	0.0021
42	0.0006	0.0020
45	0.0005	0.0018
48	0.0005	0.0016
54	0.0004	0.0014
60	0.0004	0.0012
66	0.0003	0.0011
72	0.0003	0.0010
78	0.0003	0.0009
84	0.0002	0.0008
96	0.0002	0.0006

Table 3.2-11 Desirable Velocity in Storm Drains

Description	Maximum Desirable Velocity
Culverts (All types)	15 fps.
Storm Drains (Inlet laterals)	No Limit
Storm Drains (Collectors)	15 fps.
Storm Drains (Mains)	12 fps.

3.2.8.8 Storm Drain Storage

If downstream drainage facilities are undersized for the design flow, a structural storm water control may be needed to reduce the possibility of flooding. The required storage volume can also be provided by using larger than needed storm drain pipe sizes and restrictors to control the release rates at manholes and/or junction boxes in the storm drain system. The same design criteria for sizing structural control storage facilities are used to determine the storage volume required in the system (see Section 2.2 for more information).

3.2.8.9 Energy Grade Line/Hydraulic Grade Line

The energy grade line (EGL) is an imaginary line that represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head and pressure head.

$$E = V^2/2g + p/\gamma + z \quad (3.2.25)$$

where:

E	=	Total energy, ft
$V^2/2g$	=	Velocity head, ft (kinetic energy)
p	=	Pressure, lbs/ft^2
γ	=	Unit weight of water, 62.4 lbs/ft^3
p/γ	=	Pressure head, ft (potential energy)
z	=	Elevation head, ft (potential energy)

Bernoulli's Law expressed between points one (1) and two (2) in a closed conduit accounts for all energy forms and energy losses. The general form of the law may be written as:

$$V_1^2/2g + p_1/\gamma + z_1 = V_2^2/2g + p_2/\gamma + z_2 - H_f - \Sigma H_m \quad (3.2.26)$$

where:

H_f	=	Pipe friction loss, ft
ΣH_m	=	Sum of minor or form losses, ft

The calculation of the EGL for the full length of the system is critical to the evaluation of a storm drain. In order to develop the EGL it is necessary to calculate all of the losses through the system. The energy equation states that the energy head at any cross section must equal that in any other downstream section plus the intervening losses. The intervening losses are typically classified as either friction losses or form losses. The friction losses can be calculated using the Manning's Equation. Form losses are typically calculated by multiplying the velocity head by a loss coefficient, K. Various tables and calculations exist for developing the value of K depending on the structure being evaluated for loss. Knowledge of the location of the EGL is critical to the understanding and estimating the location of the hydraulic grade line (HGL).

The hydraulic grade line (HGL) is a line coinciding with the level of flowing water at any point along an open channel. In closed conduits flowing under pressure, the hydraulic grade line is the level to which water would rise in a vertical tube at any point along the pipe. The hydraulic grade line is used to aid the designer in determining the acceptability of a proposed storm drainage system by establishing the elevation to which water will rise when the system is operating under design conditions.

HGL, a measure of flow energy, is determined by subtracting the velocity head ($V^2/2g$) from the EGL. Energy concepts can be applied to pipe flow as well as open channel flow. Figure 3.2-19 illustrates the energy and hydraulic grade lines for open channel and pressure flow in pipes.

When water is flowing through the pipe and there is a space of air between the top of the water and the inside of the pipe, the flow is considered as open channel flow and the HGL is at the water surface. When the pipe is flowing full under pressure flow, the HGL will be above the crown of the pipe. When the flow in the pipe just reaches the point where the pipe is flowing full, this condition lies in between open channel flow and pressure flow. At this condition the pipe is under gravity full flow and the flow is influenced by the resistance of the total circumference of the pipe. Under gravity full flow, the HGL coincides with the crown of the pipe.

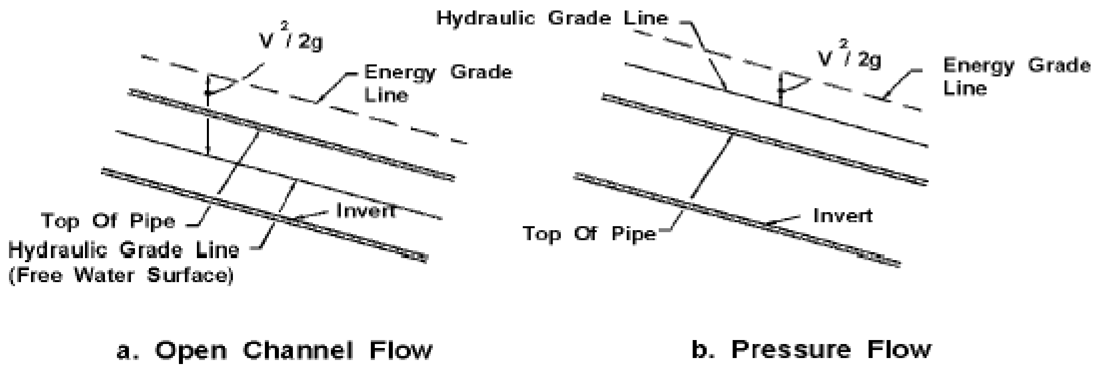


Figure 3.2-19 Hydraulic and Energy Grade Lines in Pipe Flow
(HEC 22, 2001)

Inlet surcharging and possible access hole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made. Storm drainage systems can often alternate between pressure and open channel flow conditions from one section to another.

A detailed procedure for evaluating the energy grade line and the hydraulic grade line for storm drainage systems is presented in section 3.2.8.12.

3.2.8.10 Storm Drain Outfalls

All storm drains have an outlet where flow from the storm drainage system is discharged. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the storm water. The procedure for calculating the energy grade line through a storm drainage system begins at the outfall. Therefore, consideration of outfall conditions is an important part of storm drain design.

Several aspects of outfall design must be given serious consideration. These include the flowline or invert (inside bottom) elevation of the proposed storm drain outlet, tailwater elevations, the need for energy dissipation, and the orientation of the outlet structure.

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

The tailwater depth or elevation in the storm drain outfall must be considered carefully. Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation. For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth of the outlet. The tailwater may also occur between the critical depth and the invert of the outlet. However, the starting point for the hydraulic grade line determination should be either the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, $(d_c + D)/2$, whichever is greater.

An exception to the above rule would be for a very large outfall with low tailwater where a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the design tailwater elevation, whichever was highest.

If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by a comparison of the drainage area of the receiving stream to the area of the storm drainage system. For example, if the storm drainage system has a drainage area much smaller than that of the receiving stream, the peak discharge from the storm drainage system may be out of phase with the peak discharge from the receiving watershed. Table 3.2-8 provides a comparison of discharge frequencies for coincidental occurrence for the 2-, 5-, 10-, 25-, 50-, and 100-year design storms. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. For example, if the receiving stream has a drainage area of 200 acres and the storm drainage system has a drainage area of 2 acres, the ratio of receiving area to storm drainage area is 200 to 2 which equals 100 to 1. From Table 3.2-8 and considering a 10-year design storm occurring over both areas, the flow rate in the main stream will be equal to that of a five year storm when the drainage system flow rate reaches its 10-year peak flow at the outfall. Conversely, when the flow rate in the main channel reaches its 10-year peak flow rate, the flow rate from the storm drainage system will have fallen to the 5- year peak flow rate discharge. This is because the drainage areas are different sizes, and the time to peak for each drainage area is different.

Table 3.2-12 Frequencies for Coincidental Occurrences
(TxDOT, 2002)

Area ratio	2-year design		5-year design	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	2	1	5
	2	1	5	1
1,000:1	1	2	2	5
	2	1	5	2
100:1	2	2	2	5
	2	2	5	5
10:1	2	2	5	5
	2	2	5	5
1:1	2	2	5	5
	2	2	5	5
Area ratio	**10-year design**		**25-year design**	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	10	2	25
	10	1	25	2
1,000:1	2	10	5	25
	10	2	25	5
100:1	5	10	10	25
	10	5	25	10
10:1	10	10	10	25
	10	10	25	10
1:1	10	10	25	25
	10	10	25	25

Table 3.2-12 Frequencies for Coincidental Occurrences
(TxDOT, 2002)

Area ratio	50-year design		100-year design	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	2	50	2	100
	50	2	100	2
1,000:1	5	50	10	100
	50	5	100	10
100:1	10	50	25	100
	50	10	100	25
10:1	25	50	50	100
	50	25	100	50
1:1	50	50	100	100
	50	50	100	100

There may be instances in which an excessive tailwater causes flow to back up the storm drainage system and out of inlets and access holes, creating unexpected and perhaps hazardous flooding conditions. The potential for this should be considered. Flap gates placed at the outlet can sometimes alleviate this condition; otherwise, it may be necessary to isolate the storm drain from the outfall by use of a pump station.

Energy dissipation may be required to protect the storm drain outlet. Protection is usually required at the outlet to prevent erosion of the outfall bed and banks. Riprap aprons or energy dissipators should be provided if high velocities are expected. See Section 4.7 for guidance on design of Energy Dissipation Structures.

The **orientation of the outfall** is another important design consideration. Where practical, the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for excessive erosion. If the outfall structure can not be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of the receiving channel, care must be taken to avoid erosion on the opposite channel bank. If erosion potential exists, a channel bank lining of riprap or other suitable material should be installed on the bank. Alternatively, an energy dissipator structure could be used at the storm drain outlet.

3.2.8.11 Energy Losses

Prior to computing the hydraulic grade line, all energy losses in pipe runs and junctions must be estimated. In addition to the principal energy involved in overcoming the friction in each conduit run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access holes. The following sections present relationships for estimating typical energy losses in storm drainage systems. The application of some of these relationships is included in the design example in section 8.2.8.13.

3.2.8.11.1 Pipe Friction Losses

The major loss in a storm drainage system is the friction or boundary shear loss. The head loss due to friction in a pipe is computed as follows:

$$H_f = S_f L \qquad (3.2.27)$$

where:

H_f = friction loss, ft

S_f = friction slope, ft/ft
L = length of pipe, ft

Section 3.2.8.6 gives the equation for computing the friction loss in pipes flowing full.

The friction slope in equation 3.2.27 is also the slope of the hydraulic gradient for a particular pipe run. As indicated by equation 3.2.27, the friction loss is simply the hydraulic gradient multiplied by the length of the run. Since this design procedure assumes steady uniform flow in open channel flow, the friction slope will match the pipe slope for part full flow. Pipe friction losses for full flow can be determined by the use of Equation 3.2.20.

3.2.8.11.2 Exit Losses

The exit loss from a storm drain outlet is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as at an endwall, the exit loss is:

$$\mathbf{H_o = 1.0\ [(V_o^2/2g) - (V_d^2/2g)]} \qquad (3.2.28)$$

where:
V_o = average outlet velocity
V_d = channel velocity downstream of outlet

Note that when V_d = 0, as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with water moving in the same direction as the outlet water, the exit loss may be reduced to virtually zero.

3.2.8.11.3 Bend Losses

The bend loss coefficient for storm drain design is minor but can be estimated using the following formula (AASHTO, 1991):

$$\mathbf{h_b = 0.0033\ (\Delta)\ (V^2/2g)} \qquad (3.2.29)$$

where:
Δ = angle of curvature in degrees

3.2.8.11.4 Transition Losses

A transition is a location where a conduit or channel changes size. Typically, transitions should be avoided and access holes should be used when pipe size increases. However, sometimes transitions are unavoidable. Transitions include expansions, contractions, or both. In small storm drains, transitions may be confined within access holes. However, in larger storm drains or when a specific need arises, transitions may occur within pipe runs as illustrated in Figures 3.2-14 and 3.2-20.

Energy losses in expansions or contractions in non-pressure flow can be expressed in terms of the kinetic energy at the two ends. Contraction and expansion losses can be evaluated with equations 3.2-30 and 3.2-31 respectively.

$$\mathbf{H_c=K_c\ [V_1^2/(2g)- V_2^2/(2g)]} \qquad (3.2.30)$$

$$\mathbf{H_e=K_e\ [V_1^2/(2g)- V_2^2/(2g)]} \qquad (3.2.31)$$

where:
K_e = expansion coefficient
K_c = contraction coefficient (0.5 K_e)
V_1 = velocity upstream of transition
V_2 = velocity downstream of transition
g = acceleration due to gravity (32.2 ft/s_2)

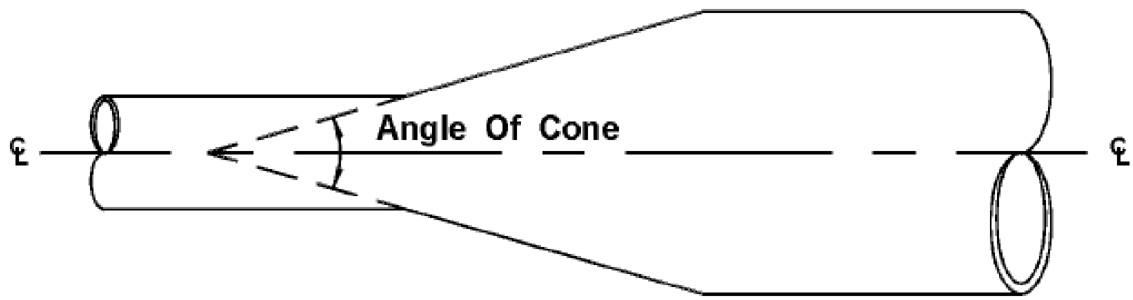


Figure 3.2-20 Angle of Cone for Pipe Diameter Changes

For gradual contractions, it has been observed that K_c = 0.5 K_e. Typical values of K_e for gradual enlargements are tabulated in Table 3.2-10a. Typical values of K_c for sudden contractions are tabulated in Table 3.2-10b. The angle of the cone that forms the transition is defined in Figure 3.2-20.

Table 3.2-13a Typical Values for K_e for Gradual Enlargement of Pipes in Non-Pressure Flow

D_2/D_1	Angle of Cone						
	10°	20°	45°	60°	90°	120°	180°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00

D_2/D_1= Ratio of Diameter of larger pipe to smaller pipe (ASCE, 1992)

Table 3.2-13b Typical Values of K_c for Sudden Pipe Contractions

D_2/D_1	K_c
0.2	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1	0

D_2/D_1= Ratio of Diameter of smaller pipe to larger pipe (ASCE, 1992)

For storm drain pipes functioning under pressure flow, the loss coefficients listed in Tables 3.2-14 and 3.2-15 can be used with Equation 3.2.32 for sudden and gradual expansions respectively. For sudden contractions in pipes with pressure flow, the loss coefficients listed in Table 3.2-16 can be used in conjunction with Equation 3.2.33 (ASCE, 1992).

$$H_e=K_e\,(V_1^2/2g) \quad (3.2.32)$$

$$H_c=K_c\,(V_2^2/2g) \quad (3.2.33)$$

where:

K_e = expansion coefficient (Tables 3.2-14 and 3.2-15)

K_c = contraction coefficient (Table 3.2.16)

V_1 = velocity upstream of transition

V_2 = velocity downstream of transition

g = acceleration due to gravity 32.2 ft/s^2

Table 3.2-14 Values of K_e for Determining Loss of Head due to Sudden Enlargement in Pipes

D_2/D_1	Velocity, V_1, in feet Per Second											
	2.0	3.0	4.0	5.0		7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09	0.08
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22	0.21	0.20
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33	0.32	0.32
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42	0.41	0.40
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50	0.48	0.47
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69	0.67	0.65
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76	0.74	0.72
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80	0.77	0.75
10.0	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.80
∞	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86	0.83	0.81

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (upstream of transition)
(ASCE, 1992)

Table 3.2-15 Values of K_e for Determining Loss of Head due to Gradual Enlargement in Pipes

D_2/D_1	Angle of Cone										
	2°	6°	10°	15°	20°	25°	30°	35°	40°	50°	60°
1.1	0.01	0.01	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.21	0.23
1.2	0.02	0.02	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.35	0.37
1.4	0.02	0.03	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.50	0.53
1.6	0.03	0.04	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.57	0.61
1.8	0.03	0.04	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.61	0.65
2.0	0.03	0.04	0.07	0.16	0.29	0.38	0.46	0.52	0.56	0.63	0.68
2.5	0.03	0.04	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.65	0.70
3.0	0.03	0.04	0.08	0.16	0.31	0.40	0.48	0.55	0.59	0.66	0.71
∞	0.03	0.05	0.08	0.16	0.31	0.40	0.49	0.56	0.60	0.67	0.72

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
Angle of cone is the angle in degrees between the sides of the tapering section
(ASCE, 1992)

Table 3.2-16 Values of K_e for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity, V_1, in feet Per Second											
	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.06
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.11	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.20
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.32	0.29	0.27
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	0.31

Table 3.2-16 Values of K_e for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity, V_1, in feet Per Second												
	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
∞	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (upstream of transition)
(ASCE, 1992)

3.2.8.11.5 Junction Losses

A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of an access hole structure. The minor loss equation for a pipe junction is a form of the momentum equation as follows:

$$H_j=[((Q_o V_o) - (Q_i V_i) - (Q_l V_l \cos\theta)) / (0.5g(A_o-A_i))] + h_i - h_o \qquad (3.2.34)$$

where:

H_j = junction loss (ft)

Q_o, Q_i, Q_l = outlet, inlet, and lateral flows respectively (ft^3/s)

V_o, V_i, V_l = outlet, inlet, and lateral velocities, respectively (ft/s)

h_o, h_i = outlet and inlet velocity heads (ft)

A_o, A_i = outlet and inlet cross-sectional areas (ft^2)

θ = angle between the inflow and outflow pipes (Figure 3.2-21)

3.2.8.11.6 Inlet and Access Hole Losses - Preliminary Estimate

The initial layout of a storm drain system begins at the upstream end of the system. The designer must estimate sizes and establish preliminary elevations as the design progresses downstream. An approximate method for estimating losses across an access hole is provided in this section. This is a preliminary estimate only and will not be used when the energy grade line calculations are made. Methods defined in Section 3.2.8.11.7 will be used to calculate the losses across an access hole when the energy grade line is being established.

The approximate method for computing losses at access holes or inlet structures involves multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 3.2.35. Applicable coefficients (K_{ah}) are tabulated in Table 3.2-17. This method can be used to estimate the initial pipe crown drop across an access hole or inlet structure to offset energy losses at the structure. The crown drop is then used to establish the appropriate pipe invert elevations. However, this method is used only in the preliminary design process and should not be used in the EGL calculations.

$$H_{ah}=K_{ah} (V_o^2/2g) \qquad (3.2.35)$$

Table 3.2-17 Head Loss Coefficients (FHA, Revised 1993)	
Structure Configuration	K_{ah}
Inlet-straight run	0.5
Inlet-angled through	
90°	1.5
60°	1.25
45°	1.1
22.5°	0.7
Manhole-straight run	0.15
Manhole-angled through	
90°	1
60°	0.85
45°	0.75
22.5°	0.45

3.2.8.11.7 Inlet and Access Hole Losses for EGL Calculations - Energy-Loss Methodology

Various methodologies have been advanced for evaluating losses at access holes and other flow junctions. The energy loss method presented in this section is based on laboratory research and does not apply when the inflow pipe invert is above the water level in the access hole.

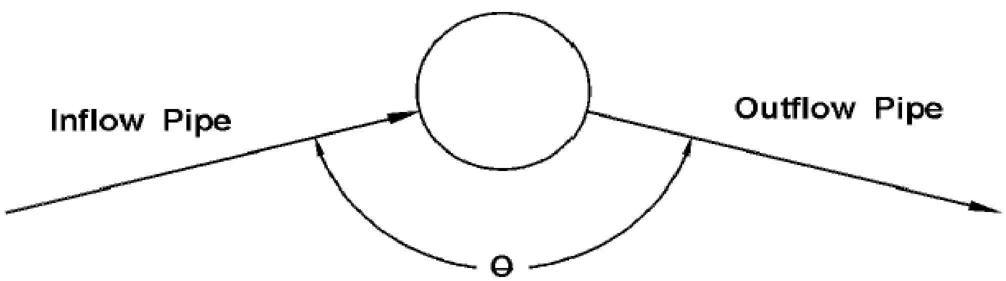


Figure 3.2-21. Head Loss Coefficients

The energy loss encountered going from one pipe to another through an access hole is commonly represented as being proportional to the velocity head of the outlet pipe. Using K to represent the constant of proportionality, the energy loss, H_{ah}, is approximated by Equation 3.2.36. Experimental studies have determined that the K value can be approximated by the relationship in Equation 3.2.37 when the inflow pipe invert is below the water level in the access hole.

$$H_{ah} = K (V_o^2/2g) \qquad (3.2.36)$$

$$K = K_O C_D C_d C_Q C_p C_B \qquad (3.2.37)$$

where:

K = adjusted loss coefficient
K_o = initial head loss coefficient based on relative access hole size
C_D = correction factor for pipe diameter (pressure flow only)
C_d = correction factor for flow depth

C_Q = correction factor for relative flow
C_p = correction factor for plunging flow
C_B = correction factor for benching
V_o = velocity of outlet pipe

For cases where the inflow pipe invert is above the access hole water level, the outflow pipe will function as a culvert, and the access hole loss and the access hole HGL can be computed using procedures found in *Hydraulic Design of Highway Culverts* (HDS-5, 1985). If the outflow pipe is flowing full or partially full under outlet control, the access hole loss (due to flow contraction into the outflow pipe) can be computed by setting K in Equation 3.2.36 to K_e as reported in Table 3.2-18. If the outflow pipe is flowing under inlet control, the water depth in the access hole should be computed using the inlet control nomographs in HDS- 5 (for example see Figure 4.3-2a in Section 4.2).

The initial head loss coefficient, K_o in Equation 3.2.37, is estimated as a function of the **relative access hole** size and the angle of deflection between the inflow and outflow pipes as represented in Equation 3.2.6. This deflection angle is represented in Figure 3.2-21.

$$K_o = 0.1\ (b/D_o)(1-\sin\theta) + 1.4(b/D_o)^{0.15}\sin\theta \qquad (3.2.38)$$

where:

θ = angle between the inflow and outflow pipes (Figure 3.2-21)
b = access hole or junction diameter
D_o = outlet pipe diameter

A change in head loss due to differences in **pipe diameter** is only significant in pressure flow situations when the depth in the access hole to outlet pipe diameter ratio, d_{aho}/D_o, is greater than 3.2. In these cases a correction factor for pipe diameter, C_D, is computed using Equation 3.2.39. Otherwise C_D is set equal to 1.

$$C_D=(D_o/D_i)^3 \qquad (3.2.39)$$

where:

D_o = outgoing pipe diameter
D_i = inflowing pipe diameter

Table 3.2-18 Coefficients for Culverts; Outlet Control, Full, or Partly Full

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12 D)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7^o or 45^o levels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Project from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7^o or 45^o bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30^o to 75^o to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/2 barrel dimension, or beveled top edge	0.2
Wingwall at 10^o to 25^o to barrel	
Square-edged at crown	0.05
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End-section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance.
(Source: Reference HDS No.5, 1985)

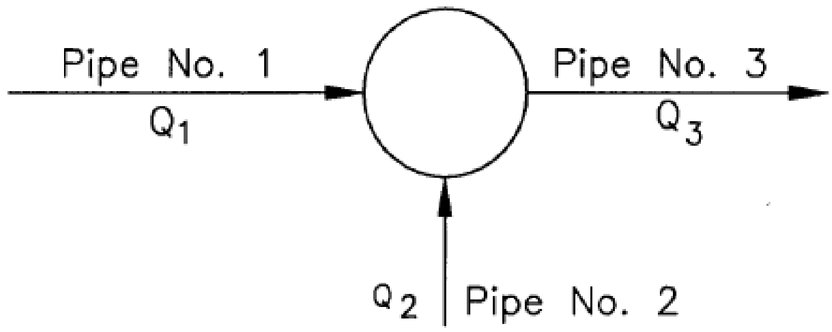


Figure 3.2-22 Relative flow effect

The correction factor for **flow depth**, C_d, is significant only in cases of free surface flow or low pressures, when the d_{aho}/D_o ratio is less than 3.2. In cases where this ratio is greater than 3.2, C_d is set equal to 1. To determine the applicability of this factor, the water depth in the access hole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor is calculated using Equation 3.2.38.

$$C_D = 0.5(d_{aho}/D_o)^{0.6} \tag{3.2.40}$$

where:

d_{aho} = water depth in access hole above the outlet pipe invert

D_o = outlet pipe diameter

The correction factor for **relative flow**, C_Q, is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed using Equation 3.2.39. The correction factor is only applied to situations where there are 3 or more pipes entering the structure at approximately the same elevation. Otherwise, the value of C_Q is equal to 1.0.

$$C_Q = (1- 2\sin\theta)\,[1-(Q_i / Q_o)]^{0.75} + 1 \tag{3.2.41}$$

where:

C_Q = correction factor for relative flow

θ = the angle between the inflow and outflow pipes (Figure 3.2-22)

Q_i = flow in the inflow pipe

Q_o = flow in the outflow pipe

As can be seen from Equation 3.2.39, C_Q is a function of the angle of the incoming flow as well as the ratio of inflow coming through the pipe of interest and the total flow out of the structure. To illustrate this effect, consider the access hole shown in Figure 3.2-23 and assume the following two cases to determine the correction factor of pipe number 2 entering the access hole. For each of the two cases, the angle between the inflow pipe number 1 and the outflow pipe, θ, is 180°.

Case 1:
$Q_1 = 3$ ft³/s
$Q_2 = 1$ ft³/s
$Q_3 = 4$ ft³/s

Using Equation 3.2.39,
$C_Q = (1 - 2\sin\theta)(1 - Q_i/Q_o)^{0.75} + 1$
$C_Q = (1 - 2\sin 180^o)(1 - 3/4)^{0.75} + 1$
$C_Q = 1.35$

Case 2:
$Q_1 = 1.0$ ft³/s
$Q_2 = 3.0$ ft³/s
$Q_3 = 4.0$ ft³/s

Using Equation 3.2.39,
$C_Q = (1 - 2\sin\theta)(1 - Q_i/Q_o)^{0.75} + 1$
$C_Q = (1 - 2\sin 180^o)(1 - 1/4)^{0.75} + 1$
$C_Q = 1.81$

The correction factor for **plunging flow**, C_p, is calculated using Equation 3.2.40. This correction factor corresponds to the effect another inflow pipe, plunging into the access hole, has on the inflow pipe for which the head loss is being calculated. Using the notations in Figure 3.2-23, C_p is calculated for pipe #1 when pipe #2 discharges plunging flow. The correction factor is only applied when $h > d_{aho}$. Additionally, the correction factor is only applied when a higher elevation flow plunges into an access hole that has both an inflow line and an outflow in the bottom of the access hole. Otherwise, the value of C_p is equal to 1.0. Flows from a grate inlet or a curb opening inlet are considered to be plunging flow and the losses would be computed using Equation 3.2.40.

$$C_p = 1 + 0.2(h/D_o)\,[(h - d_{aho})/D_o] \qquad (3.2.42)$$

where:

C_p = correction for plunging flow

h = vertical distance of plunging flow from the flow line of the higher elevation inlet pipe to the center of the outflow pipe

D_o = outlet pipe diameter

d_{aho} = water depth in access hole relative to the outlet pipe invert

The correction for **benching** in the access hole, C_B, is obtained from Table 3.2-15. Figure 3.2-24 illustrates benching methods listed in Table 3.2-15. Benching tends to direct flow through the access hole, resulting in a reduction in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Table 3.2-19 Correction for Benching
(HEC 22, 2001)

Bench Type	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat or Depressed Floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07

*pressure flow, $d_{aho}/D_o \geq 3.2$

**free surface flow, $d_{aho}/D_o \leq 3.2$

In summary, to estimate the head loss through an access hole from the outflow pipe to a particular inflow pipe using the energy-loss method, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

3.2.8.11.8 Composite Energy Loss Method

The Energy Loss Method described in Section 3.2.8.11.7 resulted from preliminary experimental and analytical techniques that focused on relatively simple access hole layout and a small number of inflow pipes. A more suitable method is available to analyze complex access holes that have, for example, many inflow pipes. This complex method, referred to as the Composite Energy Loss Method, is implemented in the FHWA storm drain analysis and design package HYDRA (GKY, 1994). Details on the method are described in the HYDRA program technical documentation and the associated research report (Chang, et. al., 1994).

This complex minor loss computation approach focuses on the calculation of the energy loss from the inflow pipes to the outflow pipe (Chang, et. al., 1994). The methodology can be applied by determining the estimated energy loss through an access hole given a set of physical and hydraulic parameters. Computation of the energy loss allows determination and analysis of the energy gradeline and hydraulic gradeline in pipes upstream of the access hole. This methodology only applies to subcritical flow in pipes.

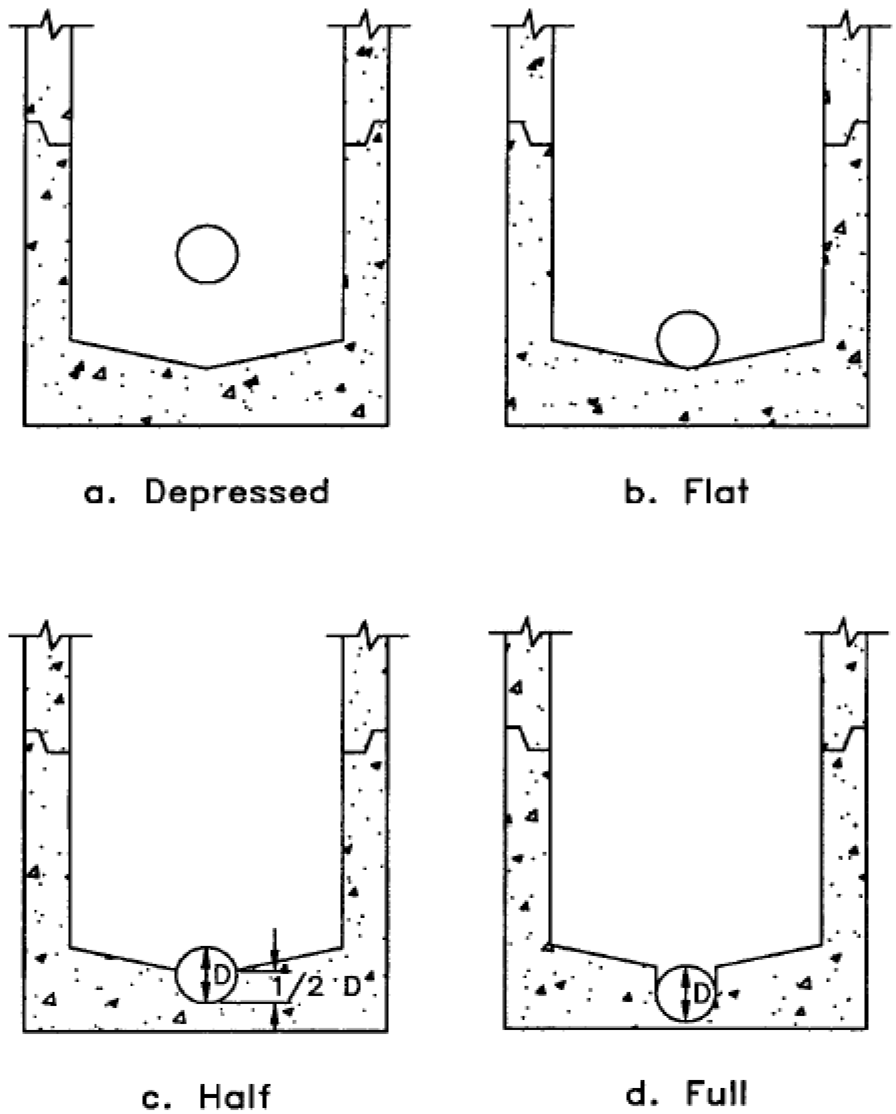


Figure 3.2-23 Access to Benching Methods

3.2.8.12 Preliminary Design Procedure

The preliminary design of storm drains can be accomplished by using the following steps and the storm drain computation sheet provided in Figure 3.2-25. This procedure assumes that each storm drain will be initially designed to flow full under gravity conditions. The designer must recognize that when the steps in this section are complete, the design is only preliminary. Final design is accomplished after the energy grade line and hydraulic grade line computations have been completed (See Section 3.2.8.9).

Step 1 Prepare a working plan layout and profile of the storm drainage system establishing the following design information:

a. Location of storm drains.

b. Direction of flow.

c. Location of access holes and other structures.
d. Number or label assigned to each structure.
e. Location of all existing utilities (water, sewer, gas, underground cables, etc.).

Step 2 Determine the following hydrologic parameters for the drainage areas tributary to each inlet to the storm drainage system:

a. Drainage areas.
b. Runoff coefficients.
c. Travel time

Step 3 Using the information generated in Steps 1 and 2, complete the following information on the design form for each run of pipe starting with the upstream most storm drain run:

a. "From" and "To" stations, Columns 1 and 2b, "Length" of run, Column 3
b. "Length" of run, Column 3
c. "Inc." drainage area, Column 4

   The incremental drainage area tributary to the inlet at the upstream end of the storm drain run under consideration.

d. "C," Column 6

   The runoff coefficient for the drainage area tributary to the inlet at the upstream end of the storm drain run under consideration. In some cases a composite runoff coefficient will need to be computed.

e. "Inlet" time of concentration, Column 9

   The time required for water to travel from the hydraulically most distant point of the drainage area to the inlet at the upstream end of the storm drain run under consideration.

f. "System" time of concentration, Column 10

   The time for water to travel from the most remote point in the storm drainage system to the upstream end of the storm drain run under consideration. For the upstream most storm drain run this value will be the same as the value in Column 9. For all other pipe runs this value is computed by adding the "System" time of concentration (Column 10) and the "Section" time of concentration (Column 17) from the previous run together to get the system time of concentration at the upstream end of the section under consideration (See Section 3.2.8.3 for a general discussion of times of concentration).

Step 4 Using the information from Step 3, compute the following:

a. "TOTAL" area, Column 5

   Add the incremental area in Column 4 to the previous sections total area and place this value in Column 5.

b. "INC." area x "C," Column 7

   Multiply the drainage area in Column 4 by the runoff coefficient in Column 6. Put the product, CA, in Column 7.

c. "TOTAL" area x "C," Column 8

   Add the value in Column 7 to the value in Column 8 for the previous storm drain run and put this value in Column 8.

d. "I," Column 11

   Using the larger of the two times of concentration in Columns 9 and 10, and an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity, I, and place this value in Column 11.

e. "TOTAL Q," Column 12

Calculate the discharge as the product of Columns 8 and 11. Place this value in Column 12.

f. "SLOPE," Column 21

Place the pipe slope value in Column 21. The pipe slope will be approximately the slope of the finished roadway. The slope can be modified as needed.

g. "PIPE DIA.," Column 13

Size the pipe using relationships and charts presented in Section 3.2.8.6 to convey the discharge by varying the slope and pipe size as necessary. The storm drain should be sized as close as possible to a full gravity flow. Since most calculated sizes will not be available, a nominal size will be used. The designer will decide whether to go to the next larger size and have part full flow or whether to go to the next smaller size and have pressure flow.

h. "CAPACITY FULL," Column 14

Compute the full flow capacity of the selected pipe using Equation 3.2.18 and put this information in Column 14.

i. "VELOCITIES," Columns 15 and 16

Compute the full flow and design flow velocities (if different) in the conduit and place these values in Columns 15 and 16. If the pipe is flowing full, the velocities can be determined from V = Q/A, Equations 3.2.17 and 3.2.18. If the pipe is not flowing full, the velocity can be determined from Figure 3.2-18a.

j. "SECTION TIME," Column 17

Calculate the travel time in the pipe section by dividing the pipe length (Column 3) by the design flow velocity (Column 16). Place this value in Column 17.

k. "CROWN DROP," Column 20

Calculate an approximate crown drop at the structure to off-set potential structure energy losses using Equation 3.2.33 introduced in Section 3.2.8.11.6. Place this value in Column 20.

l. "INVERT ELEV.," Columns 18 and 19

Compute the pipe inverts at the upper (U/S) and lower (D/S) ends of this section of pipe, including any pipe size changes that occurred along the section.

Step 5 Repeat steps 3 and 4 for all pipe runs to the storm drain outlet. Use equations and nomographs to accomplish the design effort.

Step 6 Check the design by calculating the energy grade line and hydraulic grade line as described in Section 3.2.8.9.

PRELIMINARY STORM DRAIN COMPUTATION SHEET

COMPUTED BY ______ DATE ______ ROUTE ______
CHECKED BY ______ DATE ______ SECTION ______
PAGE ______ OF ______ COUNTY ______

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO	( )	INC ( )	TOTAL ( )		INC. ( )	TOTAL ( )	INLET (min)	SYSTEM (min)	(/hr)	(³/s)	( )	(³/s)	FULL (/s)	DESIGN (/s)	(min)	U/S ( )	D/S ( )	( )	(/)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

Figure 3.2-24 Preliminary Storm Drain Computation Sheet

3.2.8.13 Energy Grade Line Evaluation Procedure

This section presents a step-by-step procedure for manual calculation of the energy grade line (EGL) and the hydraulic grade line (HGL) using the energy loss method. For most storm drainage systems, computer methods such as HYDRA (GKY, 1994) are the most efficient means of evaluating the EGL and the HGL. However, it is important that the designer understand the analysis process so that he can better interpret the output from computer generated storm drain designs.

Figure 3.2-25 provides a sketch illustrating use of the two grade lines in developing a storm drainage system. The following step-by-step procedure can be used to manually compute the EGL and HGL. The computation tables in Figure 3.2-26 and Figure 3.2-27 can be used to document the procedure outlined below.

Before outlining the computational steps in the procedure, a comment relative to the organization of data on the form is appropriate. In general, a line will contain the information on a specific structure and the line downstream from the structure. As the table is started, the first two lines may be unique. The first line will contain information about the outlet conditions. This may be a pool elevation or information on a known downstream system. The second line will be used to define the conditions right at the end of the last conduit. Following these first two lines the procedure becomes more general. A single line on the computation sheet is used for each junction or structure and its associated outlet pipe. For example, data for the first structure immediately upstream of the outflow pipe and the outflow pipe would be tabulated in the third full line of the computation sheet (lines may be skipped on the form for clarity). Table A (Figure 3.2-26) is used to calculate the HGL and EGL elevations while table B (Figure 3.2-27) is used to calculate the pipe losses and structure losses. Values obtained in table B are transferred to table A for use during the design procedure. In the description of the computation procedures, a column number will be followed by a letter A or B to indicate the appropriate table to be used.

EGL computations begin at the outfall and are worked upstream taking each junction into consideration. Many storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and access hole losses are summed to determine the upstream EGL levels. If supercritical flow occurs, pipe and access losses are not carried upstream. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream pipe section to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime.

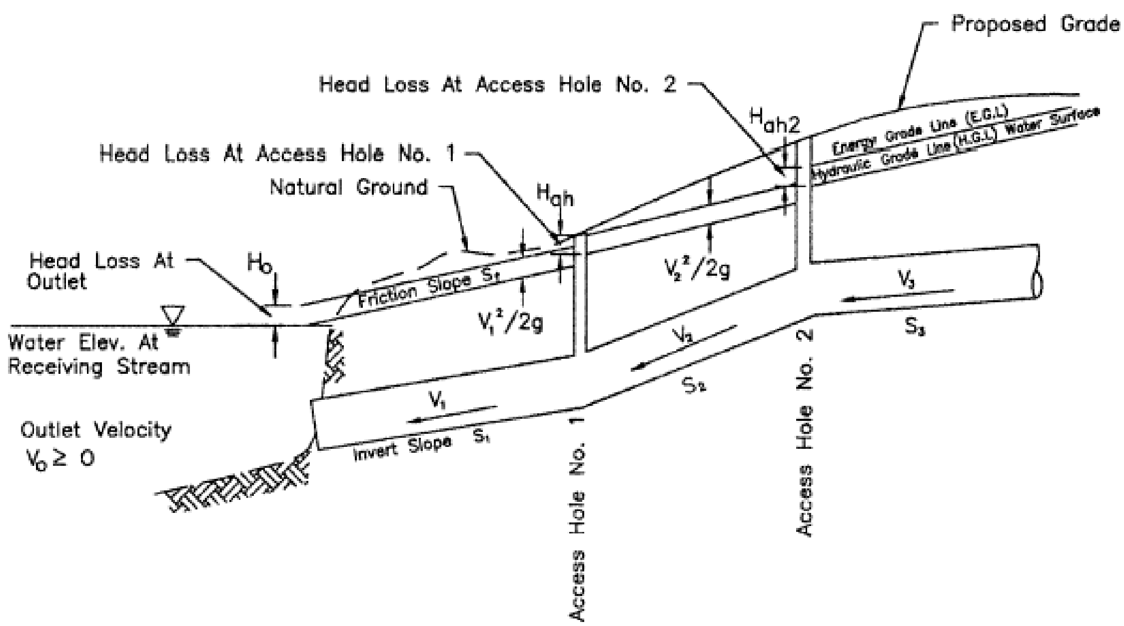


Figure 3.2-25 Energy and Hydraulic Grade Line Illustration

The EGL computational procedure follows:

Step 1 The first line of Table A includes information on the system beyond the end of the conduit system. Define this as the stream, pool, existing system, etc. in column 1A. Determine the EGL and HGL for the downstream receiving system. If this is a natural body of water, the HGL will be at the water surface. The EGL will also be at the water surface if no velocity is assumed or will be a velocity head above the HGL if there is a velocity in the water body. If the new system is being connected to an existing storm drain system, the EGL and the HGL will be that of the receiving system. Enter the HGL in Column 14A and the EGL in Column 10A of the first line on the computation sheet.

Step 2 Identify the structure number at the outfall (this may be just the end of the conduit, but it needs a structure number), the top of conduit (TOC) elevation at the outfall end, and the surface elevation at the outfall end of the conduit. Place these values in Columns 1A, 15A, and 16A respectively. Also add the structure number in Col.1B.

Step 3 Determine the EGL just upstream of the structure identified in Step 2. Several different cases exist as defined below when the conduit is flowing full:

Case 1: If the TW at the conduit outlet is greater than $(d_c + D)/2$, the EGL will be the TW elevation plus the velocity head for the conduit flow conditions.

Case 2: If the TW at the conduit outlet is less than $(d_c + D)/2$, the EGL will be the HGL plus the velocity head for the conduit flow conditions. The equivalent hydraulic grade line, EHGL, will be the invert plus $(d_c + D)/2$.

ENERGY GRADE LINE COMPUTATION SHEET - TABLE A

COMPUTED BY ________ DATE ________ ROUTE ________
CHECKED BY ________ DATE ________ SECTION ________
PAGE ________ OF ________ COUNTY ________
INITIAL TAILWATER ELEV. ________

Str. ID	D	Q	L	V	d	d_c	$V^2/2g$	S_f	Total Pipe Loss (table B)	EGL_o	K (table B)	$K(V^2/2g)$	EGL_i	HGL	U/S TOC	Surf. Elev.
	( )	(³/s)	( )	(/s)	( )	( )	( )	(/)	( )	( )		( )	( )	( )	( )	( )
(1)	(2)	(3)	(4)	(5)	(6a)	(6b)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)

Figure 3.2-26 Energy Grade Line Computation Sheet - Table A

ENERGY GRADE LINE COMPUTATION SHEET - TABLE B

COMPUTED BY ________ DATE ________ ROUTE ____________
CHECKED BY ________ DATE ________ SECTION ____________
PAGE ________ OF ________ COUNTY ____________

	Pipe Losses ( )						Structure Losses ( )							
Str. ID (1)	H_f (2)	h_b (3)	H_c (4)	H_e (5)	H_j (6)	Total (7)	d_{aho} (8)	K_o (9)	C_D (10)	C_d (11)	C_Q (12)	C_p (13)	C_B (14)	K (15)

Figure 3.2-27 Energy Grade Line Computation Sheet - Table B

The velocity head needed in either Case 1 or 2 will be calculated in the next steps, so it may be helpful to complete Step 4 and work Step 5 to the point where velocity head (Col. 7A) is determined and then come back and finish this step. Put the EGL in Column 13A.

Note: The values for d_c for circular pipes can be determined from Figure 3.2-18b. Charts for other conduits or other geometric shapes can be found in *Hydraulic Design of Highway Culverts*, HDS-5, and cannot be greater than the height of the conduit.

Step 4 Identify the structure ID for the junction immediately upstream of the outflow conduit (for the first conduit) or immediately upstream of the last structure (if working with subsequent lines) and enter this value in Columns 1A and 1B of the next line on the computation sheets. Enter the conduit diameter (D) in column 2A, the design discharge (Q) in Column 3A, and the conduit length (L) in Column 4A.

Step 5 If the barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in column 7A. Put “full” in Column 6a and not applicable (n/a) in Column 6b of Table A. Continue with Step 6. If the barrel flows only partially full, continue with Step 5A.

Note: If the pipe is flowing full because of high tailwater or because the pipe has reached its capacity for the existing conditions, the velocity will be computed based on continuity using the design flow and the full cross sectional area. Do not use the full flow velocity determined in Column 15 of the Preliminary Storm Drain Computation Form for part-full flow conditions. For part-full conditions discussed in Step 5, the calculations in the preliminary form may be helpful. Actual flow velocities need to be used in the EGL/HGL calculations.

Step 5A Part full flow: Using the hydraulic elements graph in Figure 3.2-18a with the ratio of part full to full flow (values from the preliminary storm drain computation form), compute the depth and velocity of flow in the conduit. Enter these values in Column 6a and 5 respectively of Table A. Compute the velocity head ($V_2/2g$) and place in Column 7A.

Step 5B Compute critical depth for the conduit using Figure 3.2-18b. If the conduit is not circular, see HDS-5 for additional charts. Enter this value in Column 6b of Table A.

Step 5C Compare the flow depth in Column 6a (Table A) with the critical depth in Column 6b (Table A) to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical, continue with Step 6. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical, continue with Step 5D. In either case, remember that the EGL must be higher upstream for flow to occur. If after checking for super critical flow in the upstream section of pipe, assure that the EGL is higher in the pipe than in the structure.

Step 5D Pipe losses in a supercritical pipe section are not carried upstream. Therefore, enter a zero (0) in Column 7B for this structure.

Step 5E Enter the structure ID for the next upstream structure on the next line in Columns 1A and 1B. Enter the pipe diameter (D), discharge (Q), and conduit length (L) in Columns 2A, 3A, and 4A respectively of the same line.

Note: After a downstream pipe has been determined to flow in supercritical flow, it is necessary to check each succeeding upstream pipe for the type of flow that exists. This is done by calculating normal depth and critical depth for each pipe. If normal depth is less than the diameter of the pipe, the flow will be open channel flow and the critical depth calculation can be used to determine whether the flow is sub or supercritical. If the flow line elevation through an access hole drops enough that the invert of the upstream pipe is not inundated by the flow in the downstream pipe, the designer goes back to Step1A and begins a new design as if the downstream section did not exist.

Step 5F Compute normal depth for the conduit using Figure 3.2-18a and critical depth using Figure 3.2-18b. If the conduit is not circular see HDS-5 for additional charts. Enter these values in Columns 6A and 6b of Table A.

Step 5G If the pipe barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in Column 7A. Go to Step 3, Case 2 to determine the EGL at the outlet end of the pipe. Put this value in Column 10A and go to Step 6. For part full flow, continue with Step 5H.

Step 5H Part full flow: Compute the velocity of flow in the conduit and enter this value in Column 5A. Compute the velocity head ($V^2/2g$) and place in Column 7A.

Step 5I Compare the flow depth in Column 6a with the critical depth in Column 6b to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical, continue with Step 5J. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical, continue with Step 5K.

Step 5J Subcritical flow upstream: Compute EGL_o at the outlet of the previous structure as the outlet invert plus the sum of the outlet pipe flow depth and the velocity head. Place this value in Column 10A of the appropriate structure and go to Step 9.

Step 5K Supercritical flow upstream: Access hole losses do not apply when the flow in two (2) successive pipes is supercritical. Place zeros (0) in Columns 11A, 12A, and 15B of the intermediate structure (previous line). The HGL at the structure is equal to the pipe invert elevation plus the flow depth. Check the invert elevations and the flow depths both upstream and downstream of the structure to determine where the highest HGL exists. The highest value should be placed in Column 14A of the previous structure line. Perform Steps 20 and 21 and then repeat Steps 5E through 5K until the flow regime returns to subcritical. If the next upstream structure is end-of-line, skip to step 10b then perform Steps 20, 21, and 24.

Step 6 Compute the friction slope (S_f) for the pipe using Equation 3.2.19 divided by L[$S_f = H_f/L = [185\ n^2\ (V^2/2g)]/D^{4/3}$] for a pipe flowing full. Enter this value in Column 8A of the current line. If full flow does not exist, set the friction slope equal to the pipe slope.

Step 7 Compute the friction loss (H_f) by multiplying the length (L) in Column 4A by the friction slope (S_f) in Column 8A and enter this value in Column 2B. Compute other losses along the pipe run such as bend losses (h_b), transition contraction (H_c) and expansion (H_e) losses, and junction losses (H_j) using Equations 3.2.27 through 3.2.32 and place the values in Columns 3B, 4B, 5B, and 6B, respectively. Add the values in 2B, 3B, 4B, 5B, and 6B and place the total in Column 7B and 9A.

Step 8 Compute the energy grade line value at the outlet of the structure (EGL_o) as the EGL_i elevation from the previous structure (Column 13A) plus the total pipe losses (Column 9A). Enter the EGL_o in Column 10A.

Step 9 Estimate the depth of water in the access hole (estimated as the depth from the outlet pipe invert to the hydraulic grade line in the pipe at the outlet). Computed as EGL_o (Column 10A) minus the pipe velocity head in Column 7A minus the pipe invert elevation (from the preliminary storm drain computation form). Enter this value in Column 8B. If supercritical flow exists in this structure, leave this value blank and skip to Step 5E.

Step 10 If the inflow storm drain invert is submerged by the water level in the access hole, compute access hole losses using Equations 3.2.36 and 3.2.37. Start by computing the initial structure head loss coefficient, K_o, based on relative access hole size. Enter this value in Column 9B. Continue with Step 11. If the inflow storm drain invert is not submerged by the water level in the access hole, compute the head in the access hole using culvert techniques from HDS-5 as follows:

a. If the structure outflow pipe is flowing full or partially full under outlet control, compute the access hole loss by setting K in Equation 3.2.35 to K_e as reported in Table 3.2-16. Enter this value in Column 15B and 11A, continue with Step 17. Add a note on Table A indicating that this is a drop structure.

b. If the outflow pipe functions under inlet control, compute the depth in the access hole (HGL) using Chart 28 or 29. If the storm conduit shape is other than circular, select the appropriate inlet control nomograph from HDS-5. Add these values to the access hole invert to determine the HGL. Since the velocity in the access hole is negligible, the EGL and HGL are the same. Enter HGL in Col.14A and EGL in Col.13A. Add a note on Table A indicating that this is a drop structure. Go to Step 20.

Step 11 Using Equation 3.2.39 compute the correction factor for pipe diameter, C_D, and enter this value in Column 10B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is greater than 3.2.

Step 12 Using Equation 3.2.40 compute the correction factor for flow depth, C_D, and enter this value in Column 11B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is less than 3.2.

Step 13 Using Equation 3.2.41, compute the correction factor for relative flow, C_Q, and enter this value in Column 12B. This factor = 1.0 if there are less than 3 pipes at the structure.

Step 14 Using Equation 3.2.42, compute the correction factor for plunging flow, C_p, and enter this value in Column 13B. This factor = 1.0 if there is no plunging flow. This correction factor is only applied when $h>d_{aho}$.

Step 15 Enter in Column 14B the correction factor for benching, C_B, as determined from Table 3.2-18. Linear interpolation between the two columns of values will most likely be necessary.

Step 16 Using Equation 3.2.37, compute the value of K and enter this value in Column 15B and 11A.

Step 17 Compute the total access hole loss, H_{ah}, by multiplying the K value in Column 11A by the velocity head in Column 7A. Enter this value in Column 12A.

Step 18 Compute EGL_i at the structure by adding the structure losses in Column 12A to the EGL_o value in Column 10A. Enter this value in Column 13A.

Step 19 Compute the hydraulic grade line (HGL) at the structure by subtracting the velocity head in Column 7A from the EGL_i value in Column 13A. Enter this value in Column 14A.

Step 20 Determine the top of conduit (TOC) value for the inflow pipe (using information from the storm drain computation sheet) and enter this value in Column 15A.

Step 21 Enter the ground surface, top of grate elevation or other high water limits at the structure in Column 16A. If the HGL value in Column 14A exceeds the limiting elevation, design modifications will be required.

Step 22 Enter the structure ID for the next upstream structure in Column 1A and 1B of the next line. When starting a new branch line, skip to Step 24.

Step 23 Continue to determine the EGL through the system by repeating Steps 4 through 23. (Begin with Step 2 if working with a drop structure. This begins the design process again as if there were no system down stream from the drop structure).

Step 24 When starting a new branch line, enter the structure ID for the branch structure in Column 1A and 1B of a new line. Transfer the values from Columns 2A through 10A and 2B to 7B associated with this structure on the main branch run to the corresponding columns for the branch line. If flow in the main storm drain at the branch point is subcritical, continue with Step 9; if supercritical, continue with Step 5E.

3.2.8.14 Storm Drain Design Example

The following storm drain design example illustrates the application of the design procedures outlined in Sections 3.2.8.11 and 3.2.8.12.

Example of Preliminary Storm Drain Design

Given: The roadway plan and section illustrated in Figure 3.2-29, duration intensity information in Table 3.2-21, and inlet drainage area information in Table 3.2-20. All grates are type P 50 x 100, all piping is reinforced concrete pipe (RCP) with a Manning's n value of 0.013, and the minimum design pipe diameter = 18 in for maintenance purposes.

Find:

(1) Using the procedures outlined in Section 3.2.8.11 determine appropriate pipe sizes and inverts for the system illustrated in Figure 3.2-29.

(2) Evaluate the HGL for the system configuration determined in part (1) using the procedure outlined in Section 3.2.8.12.

Solution:

(1) Preliminary Storm Drain Design

Step 1.Figure 3.2-29 illustrates the proposed system layout including location of storm drains, access holes, and other structures. All structures have been numbered for reference. Figure 3.2-30 (a) and (b) illustrate the corresponding storm drain profiles.

Step 2.Drainage areas, runoff coefficients, and times of concentration are tabulated in Figure 3.2-31. Example problems documenting the computation of these values are included in this chapter.

Starting at the upstream end of a conduit run, Steps 3 and 4 from Section 3.2.8.11 are completed for each storm drain pipe. A summary tabulation of the computational process is provided in Figure 3.2-31. The column by column computations for each section of conduit follow:

Table 3.2-20 Drainage Area Information for Design Example

Inlet No.	Drainage Area (ac)	"C"	Time of Concentration (min)
40	0.64	0.73	3
41	0.35	0.73	2
42	0.32	0.73	2
43	--	--	--
44			

Table 3.2-21 Intensity/Duration Data Design Example									
Time (min)	5	10	15	20	30	40	50	60	120
Intensity (in/hr)	7.1	5.9	5.1	4.5	3.5	3	2.6	2.4	1.4

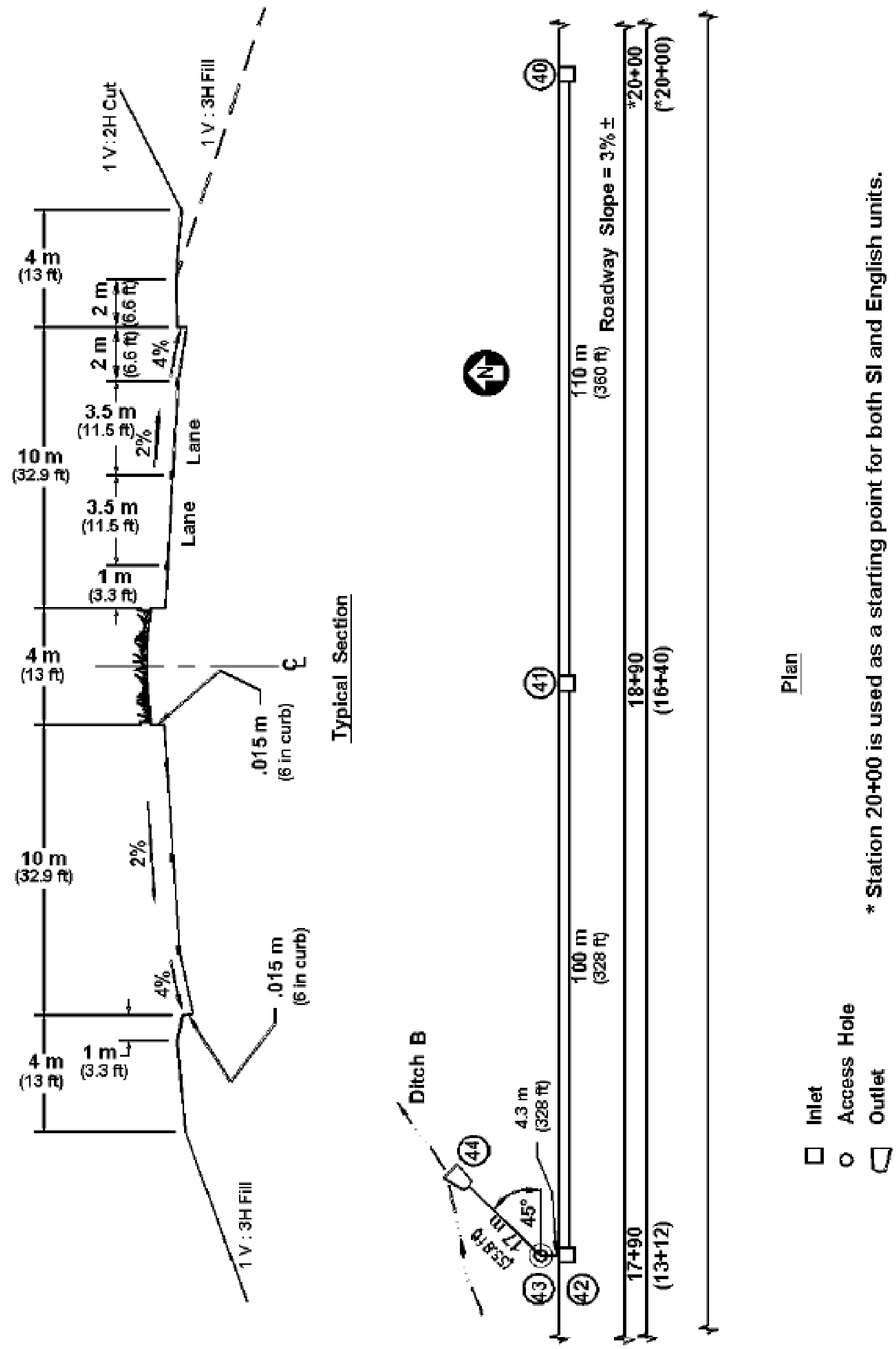


Figure 3.2-28 Roadway Plan and Sections for Example

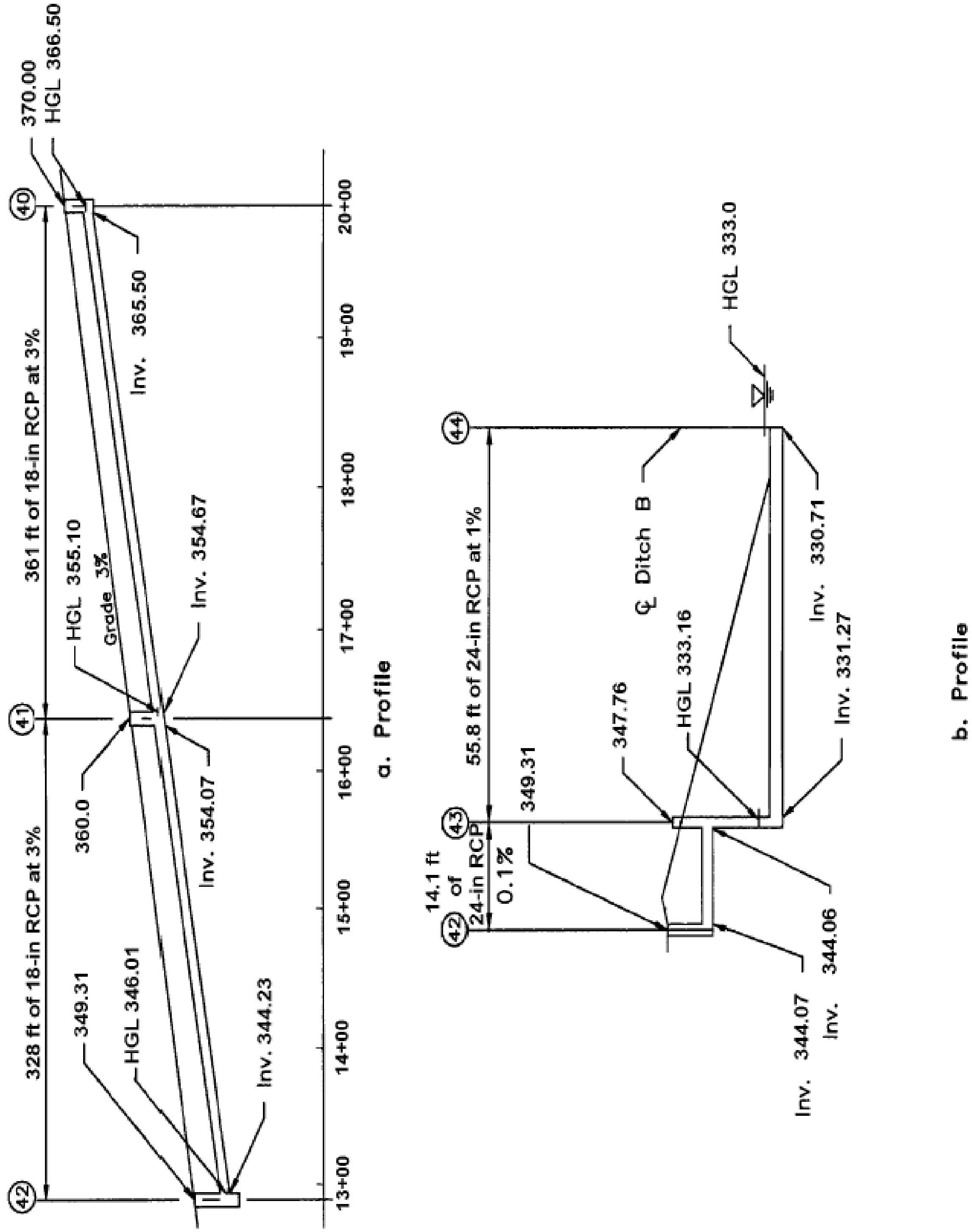


Figure 3.2-29 Storm Drain Profiles for Example

PRELIMINARY STORM DRAIN COMPUTATION SHEET
(English Solution)

COMPUTED BY ______ DATE ______
CHECKED BY ______ DATE ______
PAGE ______ OF ______

ROUTE ______
SECTION ______
COUNTY ______

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO		INC (ac)	TOTAL (ac)		INC. (ac)	TOTAL (ac)	INLET (min)	SYSTEM (min)					FULL (ft/s)	DESIGN (ft/s)		U/S (ft)	D/S (ft)		
		(Ft)								(in/hr)	(ft^3/s)	(ft)	(ft^3/s)			(min)			(ft)	(ft/ft)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
40	41	361.0	0.64	0.64	0.73	0.47	0.47	3	5	7.1	3.3	1.50	18.1	10.3	7.52	1	365.50	354.67	----	0.03
41	42	328.0	0.35	0.99	0.73	0.25	0.72	2	5	7.1	5.1	1.5	18.1	10.3	8.7	1	354.07	344.23	0.6	0.03
42	43	14.1	0.32	1.31	0.73	0.23	0.95	2	5	7.1	6.75	2.0	7.12	2.28	2.6	0	344.07	344.06	0.16	0.001
43	44	55.8	0.00	1.31	-	-	0.95	-	5	7.1	6.75	2.0	22.52	7.22	6.10	0	331.27	330.71	12.79	0.01

Figure 3.2-30 Storm Drain Computation Sheet for Example

Structure 40 to 41

Col. 1 From structure 40

Col. 2 To structure 41

Col. 3 Run Length	L = 2000 ft - 1639 ft L = 361 ft	Figure 3.2-30
Col. 4 Inlet Area	A_i = 0.64 ac	Figure 3.2-31
Col. 5 Total Area	A_t = 0.64 ac	Total area up to inlet 40
Col. 6 "C"	C = 0.73	Figure 3.2-31
Col. 7 Inlet CA	CA = (0.64)(0.73) CA = 0.47 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.47 + 0 ΣCA= 0.47 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	t_i = 3 min	Figure 3.2-31
Col. 10 Sys. Time	t_c = 3 min (use 5 min)	same as Col. 9 for upstream most section
Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21; System time less than 5 minutes therefore, use 5 minutes
Col. 12 Runoff	$Q = C_f (CA) (I)$ $Q = (0.47) (7.1) / 1.0$ $Q = 3.3\ ft^3/sec$	Equation 2.1.3; C_f = 1.0 Col. 8 times Col. 11 multiplied by 1.0
Col. 21 Slope	S = 0.03	select desired pipe slope
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(3.3)(0.013)/(0.46)(0.03)^{0.5})]^{0.375}$ D = 0.8 ft D_{min} = 1.5 ft	Equation 3.2.18 or Figure 3.2-16 use D_{min}
Col. 14 Full Cap	$Q_f = (K_Q/n) D^{2.67} S_o^{0.5}$ $Q_f = (0.46/0.013) (1.5)^{2.67} (0.03)^{0.5}$ $Q_f = 18.1 ft^3/s$	Equation 3.2.18 or Figure 3.2-16
Col. 15 Vel. Full	$V_f = (K_V/n) D^{0.67} S_o^{0.5}$ $V_f = (0.59/0.013) (1.5)^{0.67} (0.03)^{0.5}$ V_f = 10.3 ft/s	Equation 3.2.17 or Figure 3.2-16
Col. 16 Vel. Design	Q/Q_f = 3.3/18.1 = 0.18 V/V_f = 0.73 V = (0.73) (10.3) V = 7.52 ft/s	Figure 3.2-18a
Col. 17 Sect. Time	t_s = L/V = 361 / 7.52 / 60 t_s = 0.8 min; use 1 min	Col. 3 divided by Col. 16

Col. 20 Crown Drop	= 0	Upstream most invert
Col. 18 U/S Invert	= Grnd - 3.0 ft - dia = 370.0 - 3.0 - 1.5 = 365.5 ft	3 ft = min cover Ground elevation from Figure 3.2-30
Col. 19 D/S Invert	= (365.5) - (361.0)(0.03) = 354.67 ft	Col. 18 - (Col. 3)(Col. 21)

At this point, the pipe should be checked to determine if it still has adequate cover.

354.67 + 1.5 + 3.0 = 359.17 — Invert elev. + Diam + min cover

Ground elevation of 360.0 ft is greater than 359.17 ft so OK

Structure 41 to 42

Col. 1 From	= 41	
Col. 2 To	= 42	
Col. 3 Run Length	L = 1639 - 1311 \\ L = 328 ft	Figure 3.2-30
Col. 4 Inlet Area	A_i = 0.35 ac	Figure 3.2-31
Col. 5 Total Area	A_t = 0.35 + 0.64 A_t = 0.99 ac	
Col. 6 "C"	C = 0.73	Figure 3.2-31
Col. 7 Inlet CA	CA = (0.35)(0.73) CA = 0.25 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.25 + 0.47 ΣCA = 0.72 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	t_i = 2 min	Table 3.2-20
Col. 10 Sys. Time	t_c = 4 min (use 5 min)	Col. 9 + Col. 17 for line 40-41
Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21; system time equals 5 min
Col. 12 Runoff	$Q = (CA)(I)/(K_u)$ Q = (0.72) (7.1) / 1.0 Q = 5.1 ft^3/sec	Equation 2.1.3 Col. 8 times Col. 11 divided by 1.0
Col. 21 Slope	S = 0.03	select desired pipe slope
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(5.1)(0.013)/(0.46)(0.03)^{0.5}]^{0.375}$ D = 0.93 ft D_{min} = 1.5 ft use D_{min}	Equation 3.2.18 or Figure 3.2-16 use D_{min}

Col. 14 Full Cap.	$Q_f = (K_Q/n)\ D^{2.67}\ S_o^{0.5}$ $Q_f = (0.46/0.013)(1.5)^{2.67}(0.03)^{0.5}$ $Q_f = 18.1\ ft^3/s$	Equation 3.2.18 or Figure 3.2-16
Col. 15 Vel. Full	$V_f = (K_V/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59/0.013)(1.5)^{0.67}\ (0.03)^{0.5}$ $V_f = 10.3$ ft/s	Equation 3.2.18 or Figure 3.2-16
Col. 16 Vel. Design	$Q/Q_f = 5.1/18.1 = 0.28$ $V/V_f = 0.84$ V = (0.84) (10.3) V = 8.7 ft/s	 Figure 3.2-18a
Col. 17 Sect. Time	$T_s = L/V = 328 / 8.75 / 60$ $T_s = 0.6$ min; use 1 min	Col. 3 divided by Col. 16
Col. 20 Crown Drop	$= H_{ah} = K_{ah}\ (V^2 / 2g)$ $= (0.5)(8.7)^2 / [(2)(32.2)]$ = 0.6 ft	Equation 3.2.36 with Table 3.2-16 $K_{ah} = 0.5$ for inlet - straight run
Col. 18 U/S Invert	= 354.67 - 0.6 = 354.07 ft	Downstream invert of upstream conduit minus estimated structure loss (drop)
Col. 19 D/S Invert	= (354.07) - (328)(0.03) = 344.23 ft	Col. 18 - (Col. 3)(Col. 21)

Structure 42 to 43

Col. 1 From structure	= 42	
Col. 2 To structure	= 43	
Col. 3 Run Length	L = 14.1 ft	Figure 3.2-30
Col. 4 Inlet Area	$A_i = 0.32$ ac	Figure 3.2-31
Col. 5 Total Area	$A_t = 0.32 + 0.99$ $A_t = 1.31$ ac	Col. 4 plus structure 41 total area
Col. 6 "C"	C = 0.73	Figure 3.2-31
Col. 7 Inlet CA	CA = (0.32)(0.73) CA = 0.23 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.23 + 0.72 ΣCA = 0.95 ac	Col. 7 plus structure 41 total CA values
Col. 9 Inlet Time	$t_i = 2$ min	Table 3.2-20
Col. 10 Sys. Time	$t_c = 5$ min	Col. 9 + Col. 17 for line 40-41 plus Col.17 for line 41-42

Col. 11 Intensity	$I = 7.1$ in/hr	Table 3.2-21
Col. 12 Runoff	$Q = (CA)(I)$ $Q = (0.95)(7.1)$ $Q = 6.75\ ft^3/sec$	Col. 8 times Col. 11
Col. 21 Slope	$S = 0.001$	Select desired pipe slope
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(6.75)(0.013)/(0.46)(0.001)^{0.5}]^{0.375}$ $D = 1.96$ ft $D = 2.0$ ft	Equation 3.2.18 or Figure 3.2-16 Use nominal size
Col. 14 Full Cap .	$Q_f = (K_Q/n)(D^{2.67})(S_o^{0.5})$ $Q_f = (0.46/(0.013)(2.0)^{2.67}(0.001)^{0.5}$ $Q_f = 7.12\ ft^3/s$	Equation 3.2.18 or Figure 3.2-16
Col. 15 Vel. Full	$V_f = (K_v/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59)/(0.013)(2.0)^{0.67}(0.001)^{0.5}$ $V_f = 2.28$ ft/s	Equation 3.2.18 or Figure 3.2-16
Col. 16 Vel. Design	$Q/Q_f = 6.75/7.12 = 0.95$ $V/V_f = 1.15$ $V = (1.15)(2.28)$ $V = 2.6$ ft/s	Figure 3.2-18a
Col. 17 Sect. Time	$t_s = L/V = 14.1 / 2.6 / 60$ $t_s = 0.09$ min, use 0.0 min	Col. 3 divided by Col. 16
Col. 20 Crown Drop	$= H_{ah} = K_{ah}(V^2 / 2g)$ $= (1.5)(2.6)^2 / [(2)(32.2)]$ $= 0.16$ ft	Equation 3.2.36 and Table 3.2-13; $K_{ah} = 1.5$ for inlet - angled through 90 degrees
Col. 18 U/S Invert	= 344.23 - 0.16 = 344.07 ft	Downstream invert of upstream conduit minus estimated structure loss (drop)
Col. 19 D/S Invert	= 344.07 - (14.1)(0.001) = 344.06 ft	Col. 18 - (Col. 3)(Col. 21)

Structure 43 to 44

Col. 1 From	= 43	
Col. 2 To	= 44	
Col. 3 Run Length	$L = 55.8$ ft	Figure 3.2-30
Col. 4 Inlet Area	$A_i = 0.0$ ac	Figure 3.2-31
Col. 5 Total Area	$A_t = 1.31$ ac	Col. 4 plus structure 42 total area

Col. 6 "C"	C = n/a	Figure 3.2-31
Col. 7 Inlet CA	CA = 0.0	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.00 + 0.95 ΣCA = 0.95 ac	Col. 7 plus structure 42 total CA value
Col. 9 Inlet Time	n/a	No inlet
Col. 10 Sys. Time	t_c = 5 min	Col. 10 + Col. 17 for line 42-43
Col. 11 Intensity	I = 7.1 in/hr	Table 3.2-21
Col. 12 Runoff	Q = (CA) I Q = (0.95) (7.1) Q = 6.75 ft^3/sec	Col. 8 times Col. 11
Col. 21 Slope	S = 0.01	Select desired pipe slope
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q\,S_o^{0.5})]^{0.375}$ $D = [(6.75)(0.013)/(0.46)(0.01)^{0.5}]^{0.375}$ D = 1.27 ft D = 2.0 ft	Equation 3.2.18 or Figure 3.2-16 U/S conduit was 2.0 ft. - Do not reduce size inside the system
Col. 14 Full Cap.	$Q_f = (K_Q/n)(D^{2.67})(S_o^{0.5})$ $Q_f = (0.46)/(0.013)(2.0)^{2.67}\,(0.01)^{0.5}$ Q_f = 22.52 ft^3/s	Equation 3.2.18 or Figure 3.2-16
Col. 15 Vel. Full	$V_f = (K_v/n)\,D^{0.67}\,S_o^{0.5}$ $V_f = (0.59)/(0.013)(2.0)^{0.67}\,(0.01)^{0.5}$ V_f = 7.22 ft/s	Equation 3.2.17 or Figure 3.2-16
Col. 16 Vel. Design	Q/Q_f = 6.75/22.52 = 0.30 V/V_f = 0.84 V = (0.84) (7.22) V = 6.1 ft/s	Figure 3.2-18a
Col. 17 Sect. Time	t_s = 55.8 /6.1 / 60 t_s = 0.15 min, use 0.0 min	Col. 3 divided by Col. 16
Col. 19 D/S Invert	= 330.71 ft	Invert at discharge point in ditch
Col. 18 U/S Invert	= 330.71 + (55.8)(0.01) = 331.27 ft	Col. 19 + (Col. 3)(Col. 21)
Col. 20 Crown Drop	= 344.06 - 331.27 = 12.79 ft straight run	Col. 19 previous run - Col. 18

(2) Energy Grade Line Evaluation Computations - English Units

The following computational procedure follows the steps outlined in Section 3.2.8.12 above. Starting at structure 44, computations proceed in the upstream direction. A summary tabulation of the computational process is provided in Figure 3.2-32 English and Figure 3.2-33 English. The column by column computations for each section of storm drain follow:

RUN FROM STRUCTURE 44 TO 43

Outlet

Step 1	Col. 1A	Outlet	
	Col. 14A	HGL = 333.0	Downstream pool elevation
	Col. 10A	EGL = 333.0	Assume no velocity in pool

Structure 44

Step 2	Col. 1A, 1B	Str. ID = 44	Outlet
	Col. 15A	Invert = 330.71 ft	Outfall invert
		TOC = 330.71 + 2.0	Top of storm drain at outfall
		TOC = 332.71	
		Surface Elev = 332.71	Match TOC
Step 3		HGL = TW = 333.0	From Step 1
		EGL_i = HGL + $V^2/2g$	Use Case 1 since TW is above the top of conduit
		EGL_i = 333.0 +0.07	
	Col. 13A	EGL_i = 333.07	EGL_i for str. 44

Structure 43

Step 4	Col. 1A, 1B	Str. ID = 43	Next Structure
	Col. 2A	D = 2.0 ft	Pipe Diameter
	Col. 3A	Q = 6.75 cfs	Conduit discharge (design value)
	Col. 4A	L = 55.8 ft	Conduit length
Step 5	Col. 5A	V = Q/A	Velocity; use full barrel velocity since outlet is submerged.
		$V = 6.75/[(\pi /4)(2.0)^2]$	
		V = 2.15 ft/s	
	Col. 7A	$V^2/2g = (2.15)^2/(2)(32.2)$	Velocity head in conduit
		= 0.07 ft	
Step 6	Col. 8A	$S_f = [(Qn)/(K_Q D^{2.67})]^2$	Equation 3.2.18
		$S_f = [(6.75)(0.013)/(0.46)(2.0)^{2.67}]^2$	
		S_f = 0.00090 ft/ft	
Step 7	Col. 2B	$H_f = S_f L$	Equation 3.2.27
		H_f = (0.0009) (55.8)	Col. 8A x Col. 4A
		H_f = 0.05	
	Col. 7B &	$h_b, H_c, H_e, H_j = 0$	
	Col. 9A	Total = 0.05 ft	

ENERGY GRADE LINE COMPUTATION SHEET - TABLE A
(English Solution)

COMPUTED BY ______ DATE ______ ROUTE ______
CHECKED BY ______ DATE ______ SECTION ______
PAGE ______ OF ______ COUNTY ______
INITIAL TAILWATER ELEV. ______

Str. ID	D	Q	L	V	d	d_c	$V^2/2g$	S_f	Total Pipe Loss (table B)	EGL_o	K table B	$K(V^2/2g)$	EGL_i	HGL	U/S TOC	Surf. Elev.
	(ft)	(ft^3/s)	(ft)	(fps)	(ft)		(ft)	(ft / ft)	(ft)	(ft)		(ft)	(ft)	(ft)	(ft)	(ft)
(1)	(2)	(3)	(4)	(5)	(6a)	(6b)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16
OUTLET										333.00				333.00		
44													333.07		332.71	332.71
43	2.0	6.75	55.8	2.15	FULL	n/a	0.07	0.0009	0.05	333.12	0.5	0.04	333.16	*333.16	346.06	347.76
											*(Drop Structure)					
43	(New Outlet)			2.6		0.8	0.10						345.56	345.46	346.06	347.76
		(Above calculations are for inlet end of STR 43 for conduit 42-43)														
42	2.0	6.75	14.1	2.6	1.56	0.80	0.10	0.001	0.014	345.57	0.62	0.06	346.11	346.01	345.73	349.31
41	1.5	5.10	328.0	8.65	0.56	0.85	1.16	-	0	355.79	-	-	355.98	355.10	356.17	360.0
40	1.5	3.35	361.0	7.52	0.43	0.70	0.88		0	-	0	0	366.50	366.50	367.0	370.0

Figure 3.2-31 Energy Grade Line Computation Sheet, Table A, for English Example

ENERGY GRADE LINE COMPUTATION SHEET - TABLE B
(English Solution)

COMPUTED BY ______ DATE ______ ROUTE ______
CHECKED BY ______ DATE ______ SECTION ______
PAGE ______ OF ______ COUNTY ______

	Pipe Losses (ft)						Structure Losses (ft)							
Str. ID (1)	H_f (2)	h_b (3)	H_c (4)	H_e (5)	H_j (6)	Total (7)	d_{aho} (8)	K_o (9)	C_D (10)	C_d (11)	C_Q (12)	C_p (13)	C_B (14)	K (15)
44														
	0.05					0.05								
43							1.89							0.5
	0.014					0.014								
42							1.40	1.55	1.0	0.40	1.0	1.0	1.0	0.62
						0.0								
41														
														0.0
40							1.0							

Figure 3.2-32 Energy Grade Line Computation Sheet, Table B, for English Example

Step 8	Col. 10A	$EGL_o = EGL_i$ + pipe loss EGL_o = 333.07 + 0.05 EGL_o = 333.12 ft	
		HGL = 333.12 - 0.07 = 333.05 TOC = 331.27 + 2.0 = 333.27	Check for full flow - close Assumption OK
Step 9	Col. 8B	Not applicable due to drop structure	
Step 10	Col. 9B and 11A	K_e = 0.5	Inflow pipe invert much higher than d_{aho}. Assume square edge entrance
Step 17	Col. 12A	$K(V^2/2g) = (0.50)(0.07)$ $K(V^2/2g)$ = 0.04 ft	Col. 11A times Col. 7A
Step 18	Col. 13A	$EGL_i = EGL_o$ EGL_i = 333.12 + 0.04 EGL_i = 333.16 ft	Col 10A plus 12A
Step 19	Col. 14A	HGL = EGL_i = 333.16 ft d_{aho} = HGL- invert = 333.16 - 331.27 = 1.89 ft	For drop structures, the HGL is the same as the EGL Col. 8B
Step 20	Col. 15A	U/S TOC = Inv. + Dia. U/S TOC = 344.06 + 2.0 U/S TOC = 346.06 ft	From storm drain comp. sheet (Figure 3.2-31)
Step 21	Col. 16A	Surf. Elev. = 347.76 ft 347.76 > 333.09	From Figure 3.2-30. Surface elev. exceeds HGL, OK
Step 2	Col. 1A, 1B Col. 15A Col. 16A	Str. ID = 43 U/S TOC = 344.06 + 2.0 = 346.06 Surface Elev = 347.76	Drop Structure - new start
Step 3	 Col. 14A	HGL' = inv. + $(d_c+D)/2$ HGL' = 344.06 + (0.80 + 2.0)/2 HGL = 345.46 ft	Calculate new HGL - Use Case 2 d_c from Figure 3.2-18b
		EGL = HGL + $V^2/2g$	V = 2.6 fps from Prelim. Comp. Sht.
	 Col. 13A	EGL = 345.46 + 0.10 EGL = 345.56 ft	

Structure 42

Step 4	Col.1A	Str. ID = 42	
	Col. 2A	D = 2.0 ft	Pipe Diameter
	Col. 3A	Q = 6.75 cfs	Conduit discharge (design value)
	Col. 4A	L = 14.1 ft	Conduit length
Step 5A	Col. 5A	V = 2.6 ft/s Q/Qf = 6.75 / 7.12 = 0.95	For flow: Actual velocity from storm drain computation sheet.
	Col. 6A	d_n = 1.56 ft Chart 26	Figure 3.2-31
	Col. 7A	$V^2/2g = (2.6)^2/(2)(32.2)$ $V^2/2g = 0.10$ ft	Velocity head in conduit
Step 5B	Col.6bA	d_c = 0.80 ft	From HDS-5
Step 5C		$d_n < d_c$	Flow is subcritical
Step 6	Col. 8A	S_f = 0.001	Conduit not full so S_f = pipe slope d_n = 1.56 (Figure 3.2.18a) d_c = 0.80 (HDS-5) Flow is subcritical
Step 7		$H_f = S_f L$ H_f = (0.001) (14.1)	Equation 3.2.27 Col. 8A x Col. 5A
	Col. 2B	H_f = 0.014 ft h_b, H_c, H_e, H_j = 0	
	Col. 7B and 9A	Total = 0.014 ft	
Step 8		$EGL_o = EGL_i$ + total pipe loss EGL_o = 345.56 + 0.014	Col. 14A plus Col. 9A
	Col. 10A	EGL_o = 345.57 ft	
Step 9	Col. 8B	$d_{aho} = EGL_o$ - velocity head - pipe invert d_{aho} = 345.57 - 0.10 - 344.07 d_{aho} = 1.40 ft	Col. 10A - Column 7A - pipe invert
Step 10	Col. 9B	$K_o = 0.1(b/D_o)(1-\sin\theta) + 1.4(b/D_o)^{0.15}\sin(\theta)$ b = 4.0 ft D_o = 2.0 ft $\theta = 90^o$ $K_o = 0.1(4.0/2.0)(1 - \sin 90) + 1.4(4.0/2.0)^{0.15}\sin 90$ K_o = 1.55	Equation 3.2.38 Access hole diameter. Col. 2A - outlet pipe diam Flow deflection angle
Step 11	Col. 10B	$C_D = (D_o/D_i)^3$ d_{aho} = 1.40 d_{aho}/D_o = (1.40/2.0) $d_{aho}/D_o = 0.70 < 3.2$ C_D = 1.0	Equation 3.2.39; pipe diameter Column 8B therefore

Step 12	Col. 11B	$C_d = 0.5\ (d_{aho}/ D_o)^{0.6}$ $d_{aho}/D_o = 0.70 < 3.2$ $C_d = 0.5\ (1.4/2.0)^{0.6}$ $C_d = 0.40$	Equation 3.2.40; Flow depth correction.
Step 13	Col. 12B	$C_Q = (1\text{-}2 \sin \theta)(1\text{-}Q_i/Q_o)^{0.75}+1$ $C_Q = 1.0$	Equation 3.2.41; relative flow No additional pipes entering
Step 14	Col. 13B	$C_p = 1 + 0.2(h/D_o)[(h\text{-}d)/D_o]$ $C_p = 1.0$	Equation 3.2.42; plunging flow No plunging flow
Step 15	Col. 14B	$C_B = 1.0$	Benching Correction, flat floor (Table 3.2-15)
Step 16	Col. 15B and 11A	$K = K_o\, C_D\, C_d\, C_Q\, C_p\, C_B$ $K = (1.55)(1.0)(0.40)(1.0)(1.0)(1.0)$ $K = 0.62$	Equation 3.2.37
Step 17	Col. 12A	$K(V^2/2g) = (0.62)(0.10)$ $K(V^2/2g) = 0.06$ ft	Col. 11A times Col. 7A
Step 18	Col. 13A	$EGL_i = EGL_o + K(V_2/2g)$ $EGL_i = 346.05 + 0.06$ $EGL_i = 346.11$	Col. 10A plus 12A
Step 19	Col. 14A	$HGL = EGL_i - V^2/2g$ HGL = 346.11 - 0.10 HGL = 346.01 ft	Col. 13A minus Col. 7A
Step 20	Col 15A	U/S TOC = Inv. + Dia. U/S TOC = 344.23 + 1.5 U/S TOC = 345.73 ft	Information from storm drain comp. sheet (Figure 3.2-31)
Step 21	Col 16A	Surf. Elev. = 349.31 ft 349.31 > 345.96	From Figure 3.2-30 Surface elev. exceeds HGL, OK

Structure 41

Step 4	Col. 1A, 1B Col. 2A Col. 3A Col. 4A	Str. ID = 41 D = 1.50 ft Q = 5.10 cfs L = 328 ft	Next Structure Pipe Diameter Conduit discharge (design value) Conduit length
Step 5	Part full flow from column's 12 and 15 of storm drain computation sheet.		Continue with Step 5A
Step 5A	 Col. 6aA	$Q/Q_f = 5.1/18.1 = 0.28$ $d/d_f = 0.37$ d = (0.37) (1.5) d = 0.56 ft	 Figure 3.2-18a
	  Col. 5A	$V/V_f = 0.84$ V = (0.84)(10.3) V = 8.65 fps	Figure 3.2-18a

		$V^2/2g = (8.65)_2/(2)(32.2)$	Velocity head
	Col. 7A	$V^2/2g = 1.16$ ft	
Step 5B	Col. 6bA	$d_c = 0.85$ ft	Figure 3.2-18b
Step 5C		$0.56 < 0.85$	Supercritical flow since $d_n < d_c$
Step 5D	Col. 7B	Total pipe loss = 0	

Structure 40

Step 5E	Col. 1A,1B	Str. Id. = 40	Next structure
	Col. 2A	D = 1.5 ft	Pipe diameter
	Col. 3A	Q = 3.35 cfs	Conduit discharge (design)
	Col. 4A	L = 361.0 ft	Conduit length
Step 5F		$Q/Q_f = 3.3/18.1 = 0.18$	
		$d/d_c = 0.29$	Figure 3.2-18a
		d = (0.29)(1.5)	
	Col. 6aA	d = 0.43 ft	
	Col. 6bA	$d_c = 0.7$ ft	Figure 3.2-18b
Step 5H		$V/V_f = 0.73$	Figure 3.2-18a
		V = (0.73)(10.3)	
	Col. 5A	V = 7.52 fps	
	Col. 7A	$V^2/2g = (7.52)^2/(2)(32.2)$	Velocity head
		$V^2/2g = 0.88$ ft	
Step 5I		$d_n = 0.43$ ft < 0.70 ft$= d_c$	Supercritical flow since $d_n < d_c$
Step 5K	Col. 11A, and 15B	K = 0.0	Str. 41 line; supercritical flow;
	Col. 12A	$K(V^2/2g) = 0$	no structure losses

Since both conduits 42-41 and 41-40 are supercritical - establish HGL and EGL at each side of access hole 41.

		HGL = Inv. + d	
		HGL = 354.07 + 0.56	D/S Invert + Flow depth
		HGL = 354.63 ft	
		EGL = 354.63 + 1.16	
		HGL + velocity head	
	Col. 10A	EGL = 355.79 ft	EGL_o of Str.41
		HGL = 354.67 + 0.43	U/S invert + Flow depth
	Col. 14A	HGL = 355.10 ft	Highest HGL
		EGL = 355.10 + 0.88	HGL + velocity head
	Col. 13A	EGL = 355.98 ft	EGL_i of Str. 41
Step 20	Col. 15A	U/S TOC = Inv. + Dia.	Information from storm drain comp
		U/S TOC = 354.67 + 1.5	Sheet (Figure 3.2-31) for Str. 41
		U/S TOC = 356.17 ft	
Step 21	Col. 16A	Surf. Elev. = 360.0 ft	From Figure 3.2-30.
		360.0 > 355.10	Surface elev. > HGL, OK

Step 10b	Col. 8B	d_{aho} = 0.67 (1.5) = 1.0 ft	Figure 3.3-2a, HW/D = 0.67
		HGL = Str. 40 Inv. + d_{aho}	Structure Inv. from storm
		HGL = 365.50 + 1.0.	drain comp. sheet
	Col. 14A	HGL = 366.50 ft	
	Col.13A	EGL = 366.50 ft	Assume no velocity in str.
Step 20	Col. 15A	U/S TOC = Inv. + Dia.	Information from storm drain
		U/S TOC = 365.5 + 1.5	comp. sheet (Figure 3.2-31) for
		U/S TOC = 367.0 ft	Str. 40
Step 21	Col. 16A	Surf. Elev. = 370.0 ft	From Figure 3.2-30
		370.0 ft > 366.50 ft	Surface Elev. > HGL, OK

See Figures 3.2-32 and 3.2-33 for the tabulation of results. The final HGL values are indicated in Figure 3.2-30.

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