CHAPTER

5

HYDRAULIC DESIGN

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5.1 Hydraulic Design Overview

The stormwater system consists of all the site design practices and stormwater controls utilized on the site. Three considerations largely shape the design of the stormwater system: water quality, channel protection, and flood control. Previous chapters have discussed methods for calculating water quality and channel protection volumes, rainfall-runoff modelling to determine runoff rates for local design storms, design of detention practices to protect against peak flow increases, and flood routing techniques to determine peak flows on site and at key points downstream.

This chapter is provided to detail the required hydraulic engineering practices for design of stormwater facilities in Valley Center and Sedgwick County. Hydraulic design generally uses the flows determined through hydrologic analysis as an input. These flow inputs are used to size facilities to safely handle the peak flows over the range of stormwater flows likely to occur. Protection from nuisance flooding for the design storm must be provided using the minor drainage system. Structures must be protected from flooding during the 100-year storm using the major drainage system.

The minor drainage systems are designed to minimize nuisance flooding and to remove stormwater from areas such as streets and sidewalks for public safety reasons. The minor drainage system typically consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, stormwater ponds and wetlands, and small underground pipe systems which collect stormwater runoff from small to mid-frequency storms and transport it to structural control facilities, pervious areas, and/or the larger major stormwater system (i.e., natural waterways, large man-made conduits, and large water impoundments). The minor drainage system must handle the design storm specified for the land use which it serves.

The major drainage system consists of natural waterways, open channels, 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 major drainage system includes not only the main sewer 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.

The design storm frequencies for the stormwater system depend on the type of land use served by the system, as indicated in the following table:

Table 5-1 Design Storm Frequency

Land Use		Design Storm Return Period
1	Residential and Public Park Areas	2 years
2	General Commercial Areas	5 years
3	Public Building Areas	5 years
4	Industrial Areas	5-10 years
5	High Value Downtown Business Areas	5-10 years

Additionally, all structures shall be designed such that the lowest opening into the structure is elevated a minimum of 2 feet above the base flood elevation when located in an area with a mapped floodplain, or 2 feet minimum above the maximum elevation of localized flooding that would be caused by the 100-year return frequency storm event (as defined in Chapter 4) if the structure is not located in a mapped 100-year floodplain.

It is important that stormwater discharges from a new development or redevelopment onto adjacent property do not occur in a manner that is more severe to downstream property than pre-development conditions. Discharges from the developed or redeveloped site shall occur at the same location and in the same manner (e.g., distributed overland flow) as for the pre-development condition unless the altered manner of discharge can be demonstrated to be beneficial or not more severe than the pre-developed condition.

This chapter is intended to provide design criteria and guidance on minor and major drainage system components, including storm sewer systems (Section 5.62), culverts (Section 5.3), bridges (Section 5.4), vegetated and lined open channels (Section 5.5), outlet structures (Section 5.6), energy dissipation devices for outlet protection (Section 5.7), and level spreaders (Section 5.8). The section on channels is intended to cover development-scale channels only and is not intended to provide guidance on major flood control channels. In addition, the section on bridges is for general guidance only, and is not intended to provide detailed design criteria for bridges.

5.2 Storm Sewer Systems

5.2.1 Introduction

Minor drainage systems are intended to quickly remove runoff from areas such as streets, parking lots, and sidewalks. The minor drainage system consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, and small underground pipe systems which collect stormwater runoff and transport it to structural control facilities, pervious areas, and/or the larger stormwater system (i.e., natural waterways, large man-made conduits, and large water impoundments).

This section provides criteria and guidance for the design of minor drainage system components including:

- Street and roadway gutters;
- Stormwater inlets;
- Storm drain pipe systems.

Ditch, channel, and swale design criteria and guidance are covered in Section 5.5.

5.2.2 General Policies Criteria

The following policies and criteria shall govern the design of storm sewer systems in the City of Valley Center and Sedgwick County.

- The primary goal of storm sewer systems is to drain water from roadways, parking lots and sidewalks so that traffic can proceed safely without hydroplaning or becoming swamped. The minor drainage system must provide for the following water-free road travel widths for the design storm. Table 5-2 provides general guidance for allowable pavement encroachment. Ideally, the City of Valley Center prefers that roadside ditches contain the 100-year design event right-of-way to right-of-way, however it is recognized that this is not practical in all situations. Please consult the local jurisdiction if this, or any of the allowed encroachments cannot be realized in the site design.

Table 5-2 Allowable Pavement Encroachment for Design Storm

Street Classification		Allowed Encroachment
1	Local	No curb overtopping. Flow may spread across crown.
2	Collector	No curb overtopping. One 8’ lane width must be free of water.
3	Arterial	No curb overtopping. One 8’ through lane width in each direction must be free of water.
4	Expressways and Freeways	No curb overtopping. Encroachment on travel lanes not permitted.

- Gutter flow and inlet capacity calculations shall follow the procedures contained in the HEC-22 Urban Drainage Design Manual (USDOT, FHWA, 2001). The detailed calculation methods are presented in this chapter. Basic criteria for the application of those procedures are summarized below.
- A Manning’s roughness value of 0.016 shall be used for asphalt and concrete roads and gutters.
- The standard road and gutter cross slope of 3/8” per foot or 0.031 ft/ft shall be used unless specialty applications are approved by the local jurisdiction.
- The standard inlet geometries presented in Table 5-3 shall be used unless specialty applications are approved by the local jurisdiction.

Table 5-3 Local Inlet Geometries

		Type I			Type IA			Type II		
		Single	Double	Triple	Single	Double	Triple	Single	Double	Triple
h	Curb Orifice Height (in)	6			6			5		
L	Curb Weir Length (ft)	5	10	15	5	10	15	2	4	6
A	Curb Opening Area (ft^2)	2.5	5	7.5	2.5	5	7.5	0.83	1.67	2.50
	Grate Type	N/A			N/A			Curved Vane		
	Grate Width (ft)	N/A			N/A			1.17		
L	Grate Length (ft)	N/A			N/A			2.3	4.6	6.9
A	Grate Area (ft^2)	N/A			N/A			1.4	2.8	4.2
P	Grate Perimeter (ft)	N/A			N/A			4.6	6.9	9.2
a	Local Depression (in)	2			4			2		
S_W	Std. Gutter Cross Slope (ft/ft)	0.031			0.031			0.031		
S_X	Std. Road Cross Slope (ft/ft)	0.031			0.031			0.031		
W	Effective Gutter Width (ft)	1.83			4.17			1.83		

- The maximum junction criteria in Table 5-4 shall be used unless other specifications are approved by the local jurisdiciton.

Table 5-4 Maximum Junction Spacing Criteria for Storm Sewer Access

Pipe Size (inches)	Maximum Spacing (feet)
18 and smaller	300
24 and larger	400

- Manning's formula shall be used to calculate the flow of water in storm sewers. The water surface elevation shall not be higher than 1 foot below ground elevation for the design storm flow.
- Concrete pipe shall be used for all pipes in the right of way, in public drainage easements, and those used as outlets from stormwater facilities. Corrugated metal and other pipe materials may be used in selected applications if they meet local specifications.
- Maximum allowable storm sewer velocities are shown below.

Table 5-5 Maximum Allowable Velocities for Storm Drains

Description	Maximum Allowable Velocity
Culverts (All types)	15 ft/s
Storm Drains (Inlet laterals)	18 ft/s
Storm Drains (Collectors)	15 ft/s
Storm Drains (Mains)	12 ft/s

- Minimum allowable storm sewer slopes for proper self-cleaning are shown below. This table is based on achieving a velocity of 2 ft/s at a pipe flow depth of (0.2 x full depth). This standard can be used for calculating minimum slopes for alternative pipe shapes and materials.

Table 5-6 Minimum Allowable Slopes for Reinforced Concrete Pipe

d_o	S_{min}	d_o	S_{min}
inches	%	inches	%
12	0.51	39	0.11
15	0.38	42	0.1
18	0.3	48	0.08
21	0.24	54	0.07
24	0.2	60	0.06
27	0.17	66	0.05
30	0.15	72	0.05
33	0.13	84	0.04
36	0.12	96	0.03

- Outlets with flap gates shall have the bottom of the flap gate set a minimum of one foot above the anticipated or historical sediment accumulation elevation or baseflow elevation for the receiving channel, whichever is more restrictive.
- Drops through manholes or junction boxes for storm sewer design may be based on one or more of these “rules-of’thumb” criteria:
  - Set flow line elevations of pipes in structures such that the crowns of the pipes are at the same elevation;
  - When main line pipe size changes at a structure, the drop in flow line through the structure is set equal to the difference in the diamenter (rise, for non-circular) of the pipe;
  - Provide a 0.2 foot drop in the main line pipes through structures when the structure includes one side lateral; provide 0.3 foot drop in the main line pipes through the structure when the structure includes two or more laterals;
  - Provide 0.1 foot drop in the main line pipes through structures whenever the horizontal alignment of the main line pipes changes by 45-degrees or more.
- For concrete pipes, a minimum clearance of one foot is required between the top of the outside diameter of the pipe (including pipe bells, where applicable) and the bottom of pavements (including base course) when pipes are installed under or in close proximity to such pavement.

5.2.3 Street Gutters

Effective drainage of street 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 stormwater on the pavement surface. This section presents design guidance for gutter flow hydraulics published in HEC-22.

Manning's Equation (Equation 5-1) shall be used to evaluate gutter flow hydraulics (HEC-22, 2001).

Equation 5-1 $$Q = \left(\frac{0.56}{n}\right) S_x^{5/3} S^{1/2} T^{8/3}$$

where:

Q	=	gutter flow rate, cfs
S_x	=	pavement cross slope, ft/ft (3.1%)
n	=	Manning's roughness coefficient
S	=	longitudinal slope, ft/ft
T	=	width of flow or spread, ft

Figure 5-2 is a nomograph for solving Equation 5-1. A Manning's n of 0.016 shall be used for asphalt and concrete paving and gutters. Manning's n values for various additional surfaces are presented in Appendix A for specialty applications.

The following two problems demonstrate the use of Figure 5-2 to check for allowable road water spread.

Example Problem – Gutter Flow #1

You are designing a standard 66' local residential collector street with a longitudinal slope of 4%. Can it handle 2.4 cfs on one half of the street and meet the road spread standard?

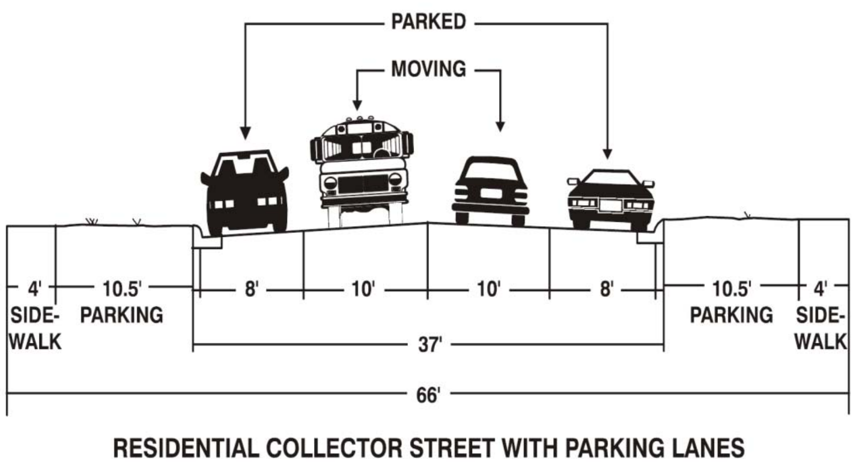


Figure 5-1 Example Street

Enter Figure 5-2 at the longitudinal slope (S) value of 0.04 ft/ft and draw a line through the S_X scale at the standard cross slope (S_X) value of 0.031. Extend the line to the turning line.

Draw a line between the turning line intercept and a Q value of 2.4 cfs. Read the spread (T).

T = 6 feet

Determine the open width:

Open width = ½ road width – Spread
Open width = 18 – 6 = 12 ft

The road spread standard for a collector street is to prevent curb overtopping and leave on 8' lane open for travel. This standard has been met.

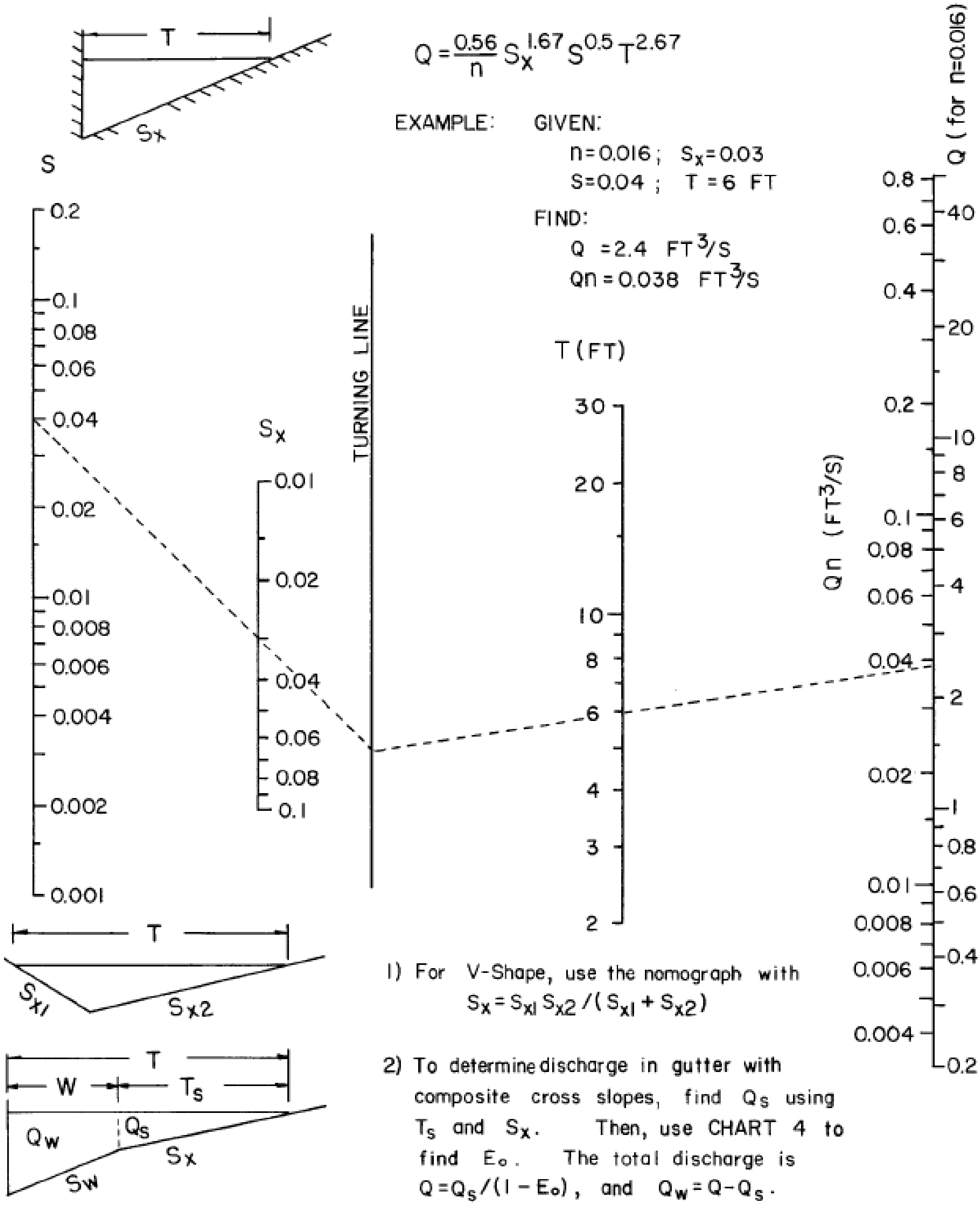


Figure 5-2 Flow in Triangular Gutter Sections - English Units
(HEC-22, 2001)

Example Problem – Gutter Flow #2

For the same residential collector street with a longitudinal slope of 1%, what would be the maximum flow for ½ of the street?

Determine the allowable spread.

Open width = ½ road width – Spread
Spread = ½ road width – Open width
Spread = 18 – 8 = T
Spread = 10 = T

Enter Figure 5-2 at the longitudinal slope (S) value of 0.01 ft/ft and draw a line through the S_X scale at the standard cross slope (S_X) value of 0.031. Extend the line to the turning line.

Draw a line between the turning line intercept and a T value of 10 ft. Read the flow (Q).

Q = 5.0 cfs

5.2.3.1 Gutter Flow vs. Total Flow

Figure 5-3 can be used to find the flow in a gutter with width (W) less than the total spread (T). The figure provides the ratio (E_O) of gutter flow (Q_W) to total flow (Q) in the ½ road section for a range of gutter and road cross slopes. Roads that have a steeper gutter cross slope than road cross slope are called composite gutters, and S_W/S_X will be greater than 1. However, in Wichita and Sedgwick County a S_W/S_X ratio of 1 shall be used for standard designs since the standard road slope is 3/8” per foot and the standard gutter cross slope is 3/8” per foot. This figure can also be used to determine the portion of total flow that will cross the front of a grate inlet by setting W equal to the width of the grate.

For cases where the road cross slope, gutter cross slope or road crown shape are non-standard, Appendix C of HEC-22 provides procedures for determining capacity using nomographs. The nomograph procedure involves a complex graphical solution of the equation for flow in the non-standard road shapes. Typical of graphical solutions, extreme care is necessary to obtain accurate results.

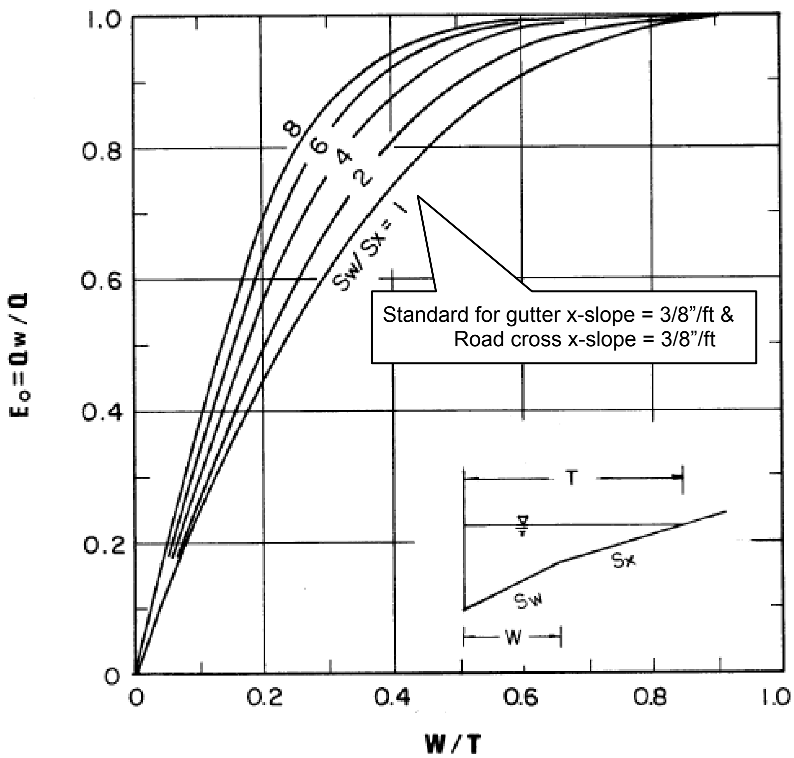


Figure 5-3 Ratio of Gutter Flow to Total Road Section Flow
(HEC-22, 2001)

5.2.4 Stormwater Inlets

Inlets are drainage structures used to collect surface water through grate or curb openings and convey it to storm drains. Inlets used for drainage of highway surfaces can be divided into three major classes:

Grate Inlets: These inlets consist of an opening in the gutter covered by one or more grates. These inlets are typically used in parking lots and yard drainage situations.

Curb-Opening Inlets: These inlets are vertical openings in the curb covered by a top slab. These inlets are particularly suited to sump applications and areas where heavy litter accumulation is expected because of their high capacity and anti-clogging tendencies. Local standard types I and IA fall into this category.

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. These inlets are particularly suited to roads with high longitudinal slopes because of their ability to capture faster moving water. Local standard type II falls into this category.

Figure 5-4 present standard inlets. 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 multiple directions. Sump areas must have an overflow route or channel to protect local structures should the inlet clog. Flanking inlets shall be placed on each side of road sump inlets. The flanking inlets shall be placed so they will limit spread on low gradient approaches and act in relief of the inlet at the low point if it should become clogged.

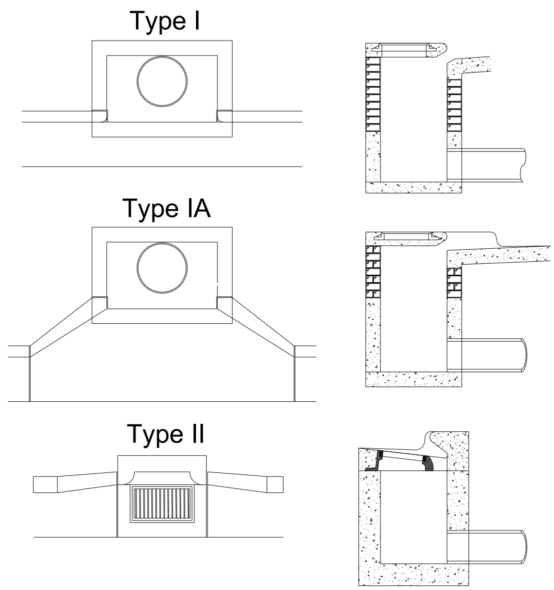


Figure 5-4 Local Standard Inlets

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).

The American Society of State Highway and Transportation Officials (AASHTO) geometric policy recommends a minimum gradient of 0.3% within 50 ft of the level point in a sag vertical curve. This longitudinal slope value shall be used when checking for spread at sag inlets.

The geometric values found in Table 5-3 shall be used for modelling the standard inlets, unless specialty applications are allowed by the Director or his/her designee.

5.2.4.1 Grate Inlets on Grade

The capacity of an inlet depends on 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 flows and 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 side the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the flow and/or velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is usually small.

The ratio of frontal flow to total gutter flow, E_O, for a straight (uniform) cross slope is expressed by the Equation 5-2 (HEC-22, 2001):

Equation 5-2 $$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67}$$

where:

Q	=	total 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

Figure 5-3 provides a graphical solution of E_O for either composite gutters or straight cross slopes. The ratio of side flow, Q_s, to total gutter flow calculated using Equation 5-3 (HEC-22, 2001).

Equation 5-3 $$\frac{Q_s}{Q} = \frac{1 - Q_w}{Q} = 1 - E_o$$

The ratio of frontal flow intercepted by a grate to total frontal flow, R_f, is expressed by the Equation 5-4 (HEC-22, 2001).

Equation 5-4 $$R_f = 1 - 0.09(V - V_o)$$

where:

V	=	velocity of flow in the gutter, ft/s
V_o	=	gutter velocity where splash-over first occurs, ft/s (a function of grate length)

Figure 5-5 provides a solution of Equation 5-4, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 5-5 is total gutter flow divided by the cross-sectional area of flow.

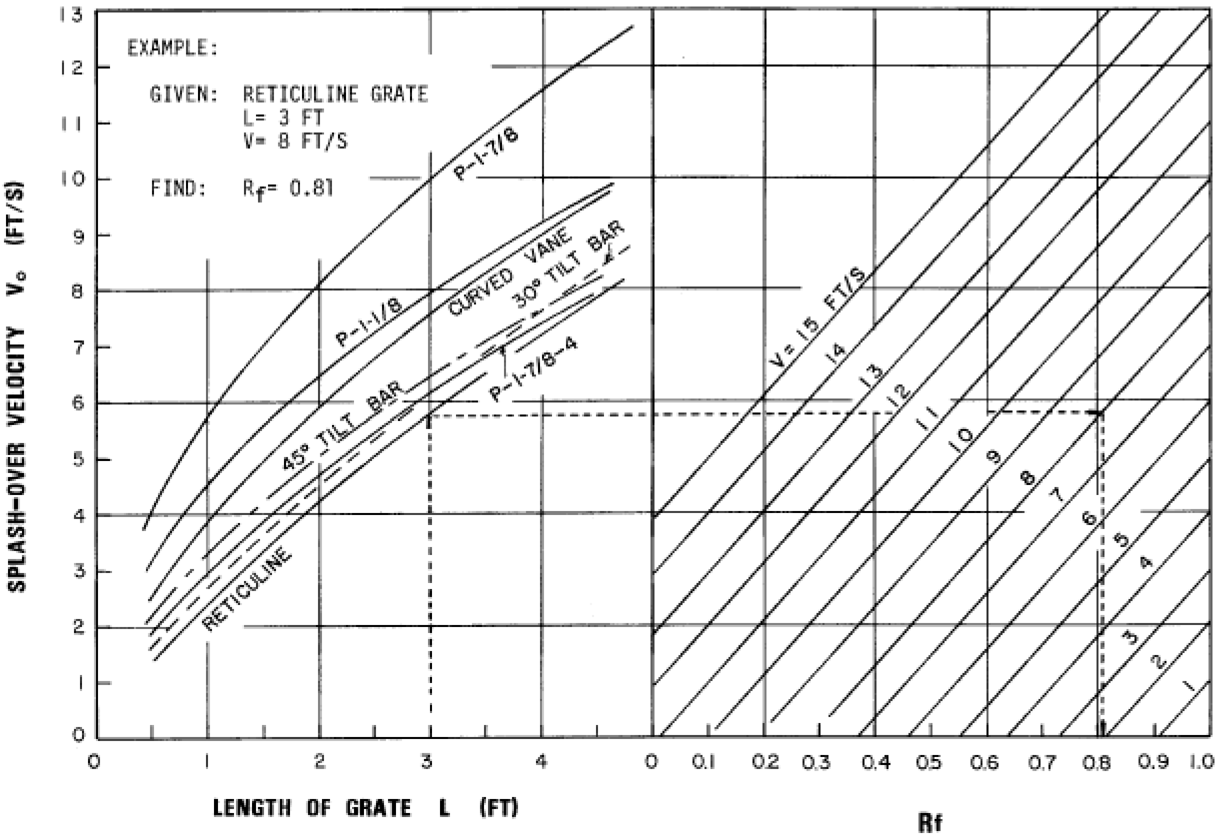


Figure 5-5 Grate Inlet Frontal Flow Interception Efficiency
(HEC-22, 2001)

The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency (HEC-22, 2001), is expressed by Equation 5-5 below. Figure 5-6 provides a solution to Equation 5-5.

Equation 5-5
$$R_s = \frac{1}{\left[1 + \left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)\right]}$$

where:

L = length of the grate, ft

The overall efficiency, E, of a grate (HEC-22, 2001) is expressed in Equation 5-6.

Equation 5-6
$$E = R_f E_O + R_s(1 - E_O)$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow as shown in Equation 5-7 (HEC-22, 2001).

Equation 5-7
$$Q_i = EQ = Q[R_f E_O + R_s(1 - E_O)]$$

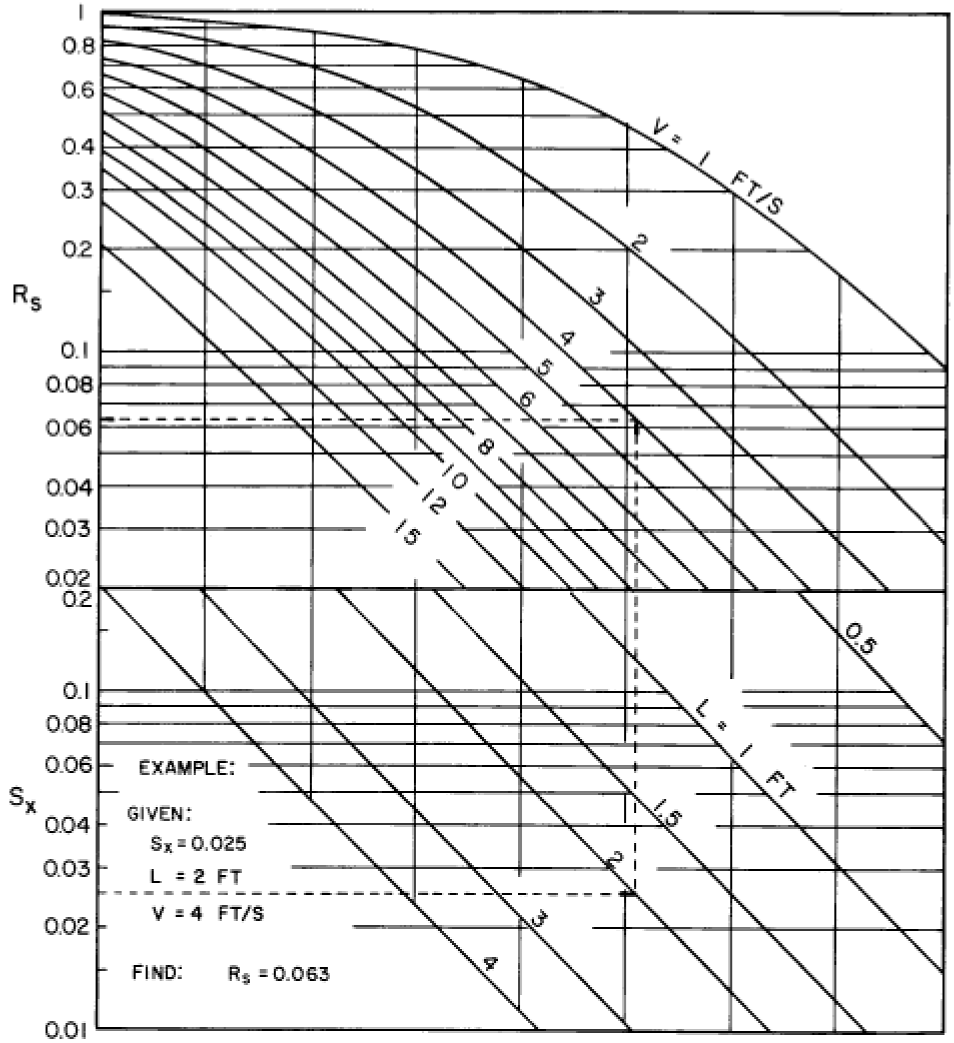


Figure 5-6 Grate Inlet Side Flow Interception Efficiency
(HEC-22, 2001)

The following problem illustrates the design of grate inlets on grade.

Example Problem – Grate Inlets on Grade

Determine the interception capacity of a single Type II inlet for a ½ street with spread of 10 feet and longitudinal slope of 3.1%.

Determine the total flow rate using Equation 5-1.

$$Q = \left(\frac{0.56}{n}\right) S^{5/3} S^{1/2} T^{8/3}$$

$$Q = \left(\frac{0.56}{n}\right)0.031^{5\!/\!3}0.03^{1\!/\!2}10^{8\!/\!3}$$
$$Q = \left(\frac{0.56}{0.016}\right)0.003 * 0.173 * 464 = 8.4\,cfs$$

Determine the flow depth at the curb.

Flow depth = TS_X = (10)(0.031) = 0.31 ft

Determine the flow area

Area of a right triangle = ½*LW* = (½)(10)(0.31) = 1.6 ft^2

Determine the flow velocity.

V = *Q/A*
V = 8.4/1.6 = 5.3 ft/s

Determine E_O for the spread and standard gutter width.

$$E_O = 1 - \left(1 - \frac{W}{T}\right)^{2.67}$$
$$E_O = 1 - \left(1 - \frac{W}{T}\right)^{2.67} = 1 - \left(1 - \frac{1.83}{10}\right)^{2.67} = 0.42$$

Determine R_f from Figure 5-5. Start at bottom left for standard single grate length of 2.3 ft, turn at curved vane grate curve, turn at V = 5.3 ft/s. Since V = 5.3 ft/s is off of the nomograph, R_f = 1.0 and frontal flow capture is complete.

Determine R_s from Figure 5-6. Start at bottom left for S_x = 0.031, turn at the standard grate length of 2.3 ft, turn at V = 5.3 ft/s, arrive at R_S = 0.065.

Determine interception capacity.

$$Q_i = QE = Q(R_f E_O + R_S(1 - E_O))$$
$$Q_i = QE = 8.4(1 * 0.42 + 0.065(1 - 0.42))$$
$$Q_i = QE = 8.4(0.42 + 0.038)$$
$$Q_i = QE = 3.8\,cfs$$

5.2.4.2 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 typical gutter inlet grate, weir operation continues to a depth of about 0.4 to 0.6 feet above the top of grate and when depth of water exceeds about 1.0 to 1.5 feet, the grate begins to operate as an orifice. Between depths of about 0.4 - 0.6 feet and about 1.0 - 1.5 feet, a transition from weir to orifice flow occurs.

The capacity of a grate inlet operating as a weir (HEC-22, 2001) is:

Equation 5-8 $$Q_i = CPd^{1.5}$$

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

The capacity of a grate inlet operating as an orifice (HEC-22, 2001) is:

Equation 5-9 $$Q_i = CA(2gd)^{0.5}$$

where:

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

Figure 5-7 is a plot of Equation 5-8 and Equation 5-9 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.

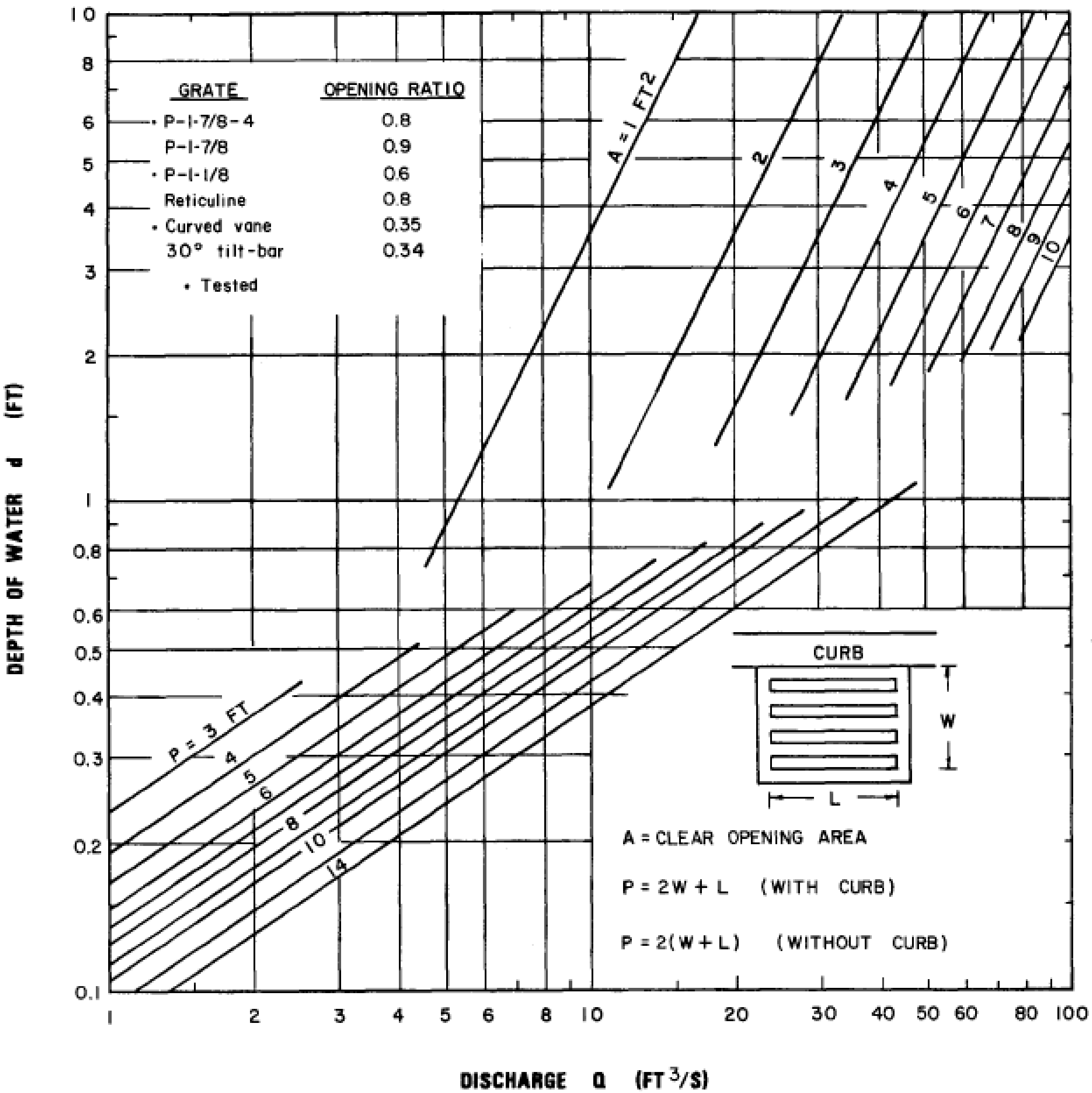


Figure 5-7 Grate Inlet Capacity in Sump Conditions – English Units
(HEC-22, 2001)

Example Problem – Grate Inlets in Sag

How many Type II grates are needed for a design spread of 10 ft and flow of 2 cfs for a sag vertical curve? Allow for 50% clogging of the grate (based on designer's judgement.)

Check spread at S = 0.003 for approaches to the low point per the AASHTO sag guidance.

Determine depth at the curb, given the road cross slope and spread.

$D = TS_X = (10)(0.031) = 0.31$ ft

Determine the effective perimeter and area of various grate sizes. Effective perimeter is equal to the perimeter multiplied by the fraction of the area free of clogging.

Single Unit P = (4.6)(0.5) = 2.3 ft
Double Unit P = (6.9)(0.5) = 3.45 ft
Triple Unit P = (9.2)(0.5) = 4.6 ft

Single Unit A = (1.4)(0.5) = 0.7 ft^2
Double Unit A = (2.8)(0.5) = 1.4 ft^2
Triple Unit A = (4.2)(0.5) = 2.1 ft^2

The depth of 0.3 ft is in the weir flow range. Determine the total effective perimeter necessary to intercept 2 cfs at a depth of 0.31 ft from Figure 5-7.

From the figure, the grate must have an effective perimeter of 4 ft.

Determine the number of grates necessary.

A triple unit will provide the required perimeter of 4 ft, and should be selected.

Check spread (T) at S = 0.003 and Q = 2 cfs using Figure 5-2.

T = 9 ft
Thus, the triple grate can safely limit spread in the sag approach to less than the allowable 10 ft.

5.2.4.3 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 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, computed with the Equation 5-10 (HEC-22, 2001).

Equation 5-10 $$S_e = S_x + S'_w E_o$$

where:

E_O = ratio of flow in the depressed section to total gutter flow
S'_w = cross slope of depressed gutter measured relative to the cross slope of the pavement, ft/ft
S'_w = a / (12 W), (a = Gutter depression, in and w = width of gutter, ft), diagram found in Figure 5-8

a	=	gutter depression, in
W	=	width of depressed gutter, ft
S_e	=	equivalent cross slope, ft/ft
S_x	=	pavement cross slope, ft/ft

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 5-8. It is apparent from examination of Figure 5-8 that the length of curb opening required for total interception can be significantly reduced by increasing the cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

The efficiency of curb-opening inlets shorter than the length required for total interception is determined using Figure 5-9.

The following problem illustrates the design of curb inlets on grade.

Example Problem – Curb Inlets on Grade

For a road with longitudinal slope of 3.5% and ½ street flow of 5 cfs, what is the captured flow for a double Type IA inlet?

Determine the spread (T) for a flow of 5 cfs, standard cross slope of 0.031 and standard roughness of 0.016. Using Figure 5-2, T = 8.5 ft.

Determine W/T for the gutter in front of the Type IA inlet.

W/T = 4.17/8.5 = 0.49 (note: Type I inlets should use W = 1.83)

Determine the frontal flow efficiency (E_O) using W/T and standard S_W/S_X=1 using Figure 5-5.

$E_O = 0.84$

Determine the equivalent cross slope (S_e).

$$S_e = S_x + S'_w E_o$$
$$S_e = 0.031 + \left(4 / 12 * 4.17\right) 0.84 = 0.098$$
(note: Type I inlets should simply use $S_e = S_X$)

Use Figure 5-8 to determine the length for total interception (LT). Using Figure 5-8, LT = 18 ft.

Determine the inlet efficiency using Figure 5-9 for L/LT of 10/18 or 0.56. Using Figure 5-9, E = 0.77 .

Determine the captured flow rate (Qi)

Q_i = EQ = (0.77)(5) = 3.9 cfs

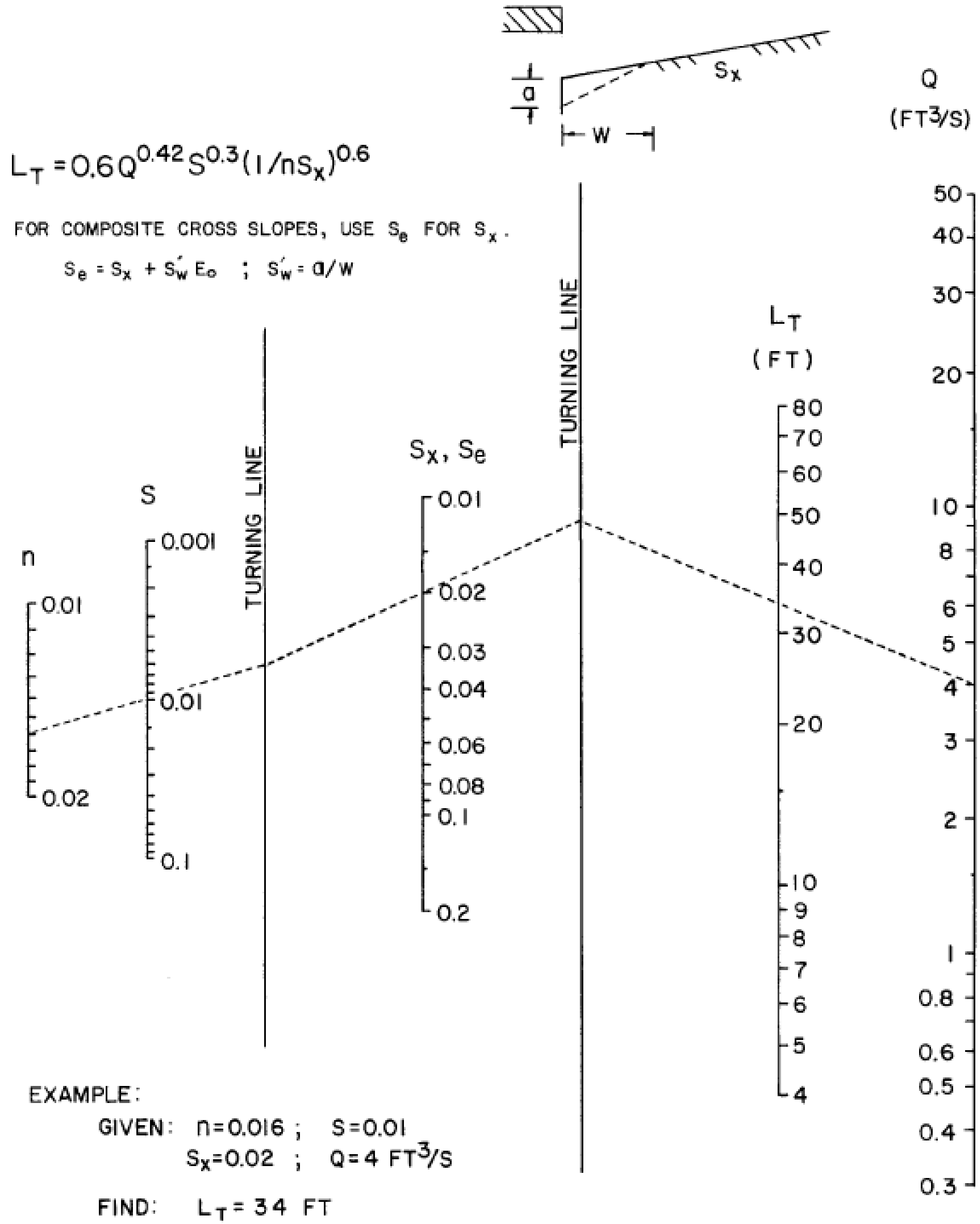


Figure 5-8 Curb-Opening Inlet Length for Total Interception
(HEC-22, 2001)

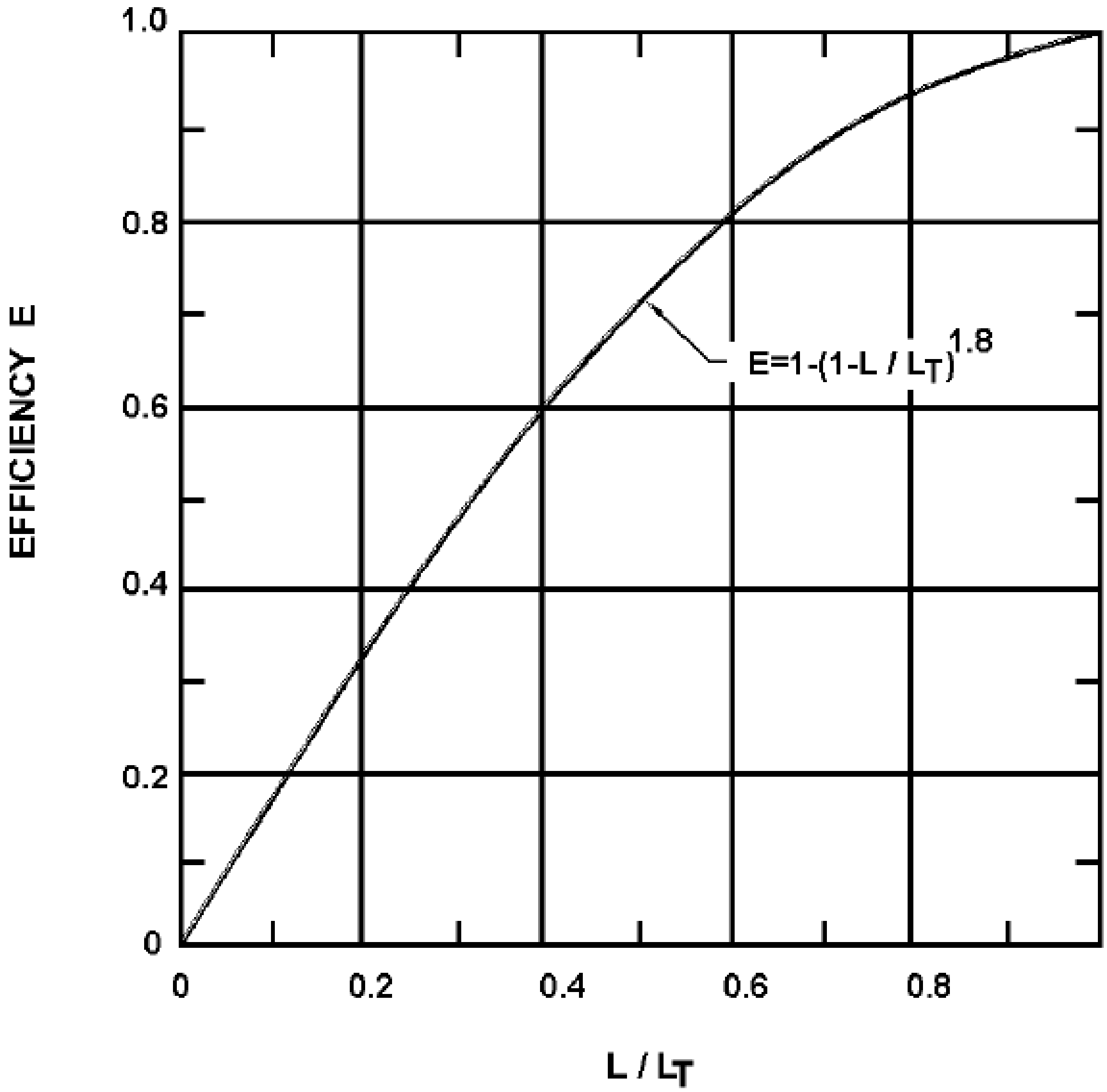


Figure 5-9 Curb-Opening and Slotted Drain inlet Interception Efficiency
(HEC-22, 2001)

5.2.4.4 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 (h) and as an orifice at depths greater than approximately 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 5-10, which accounts for the operation of the inlet as a weir up to the opening height and as an orifice at depths greater than 1.4h.

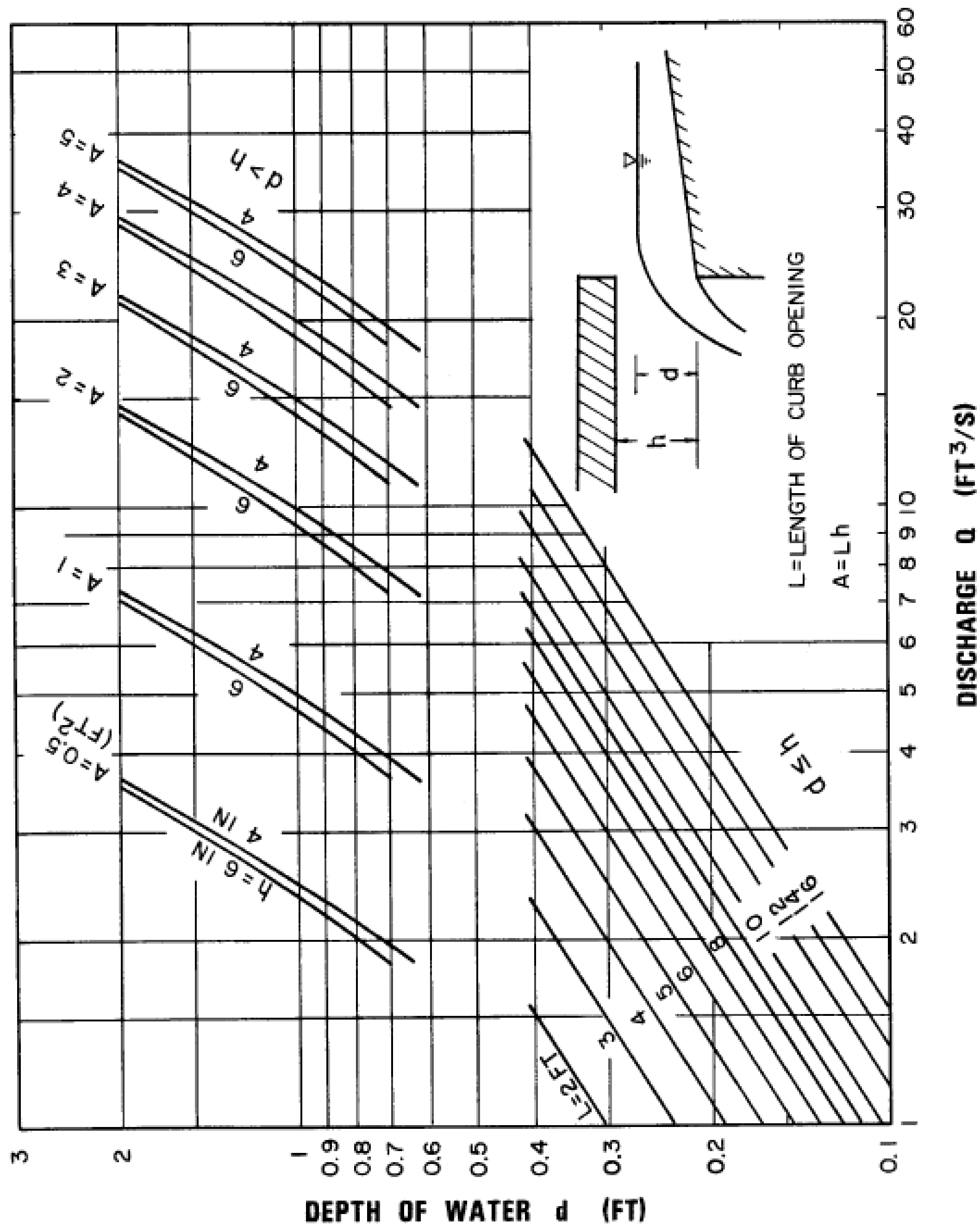


Figure 5-10 Curb-Opening Inlet Capacity in Sump Locations
(HEC-22, 2001)

The following problem illustrates the design of curb inlets in sump locations.

Example Problem – Curb Inlets in Sump

Determine discharge for a single Type I inlet in a sump location with road spread of 12 feet.

Determine depth at curb face

$d = \quad TS_x = (12)(0.031) = 0.37 \text{ ft} = 4.46 \text{ in}$

Determine Q_i from Figure 5-10 for the standard single Type I inlet length of 5 ft, d = 0.37 ft and d < h

$Q_i = 3.4$ cfs (note: the inlet is acting as a weir at this depth)

5.2.4.5 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 (per HEC-22 guidance). Thus, capacity shall be computed by neglecting the curb opening inlet, except for any curb opening length placed upstream of the grate in a “sweeper” configuration. In the latter case, total capacity may be used.

5.2.4.6 Combination Inlets in Sump

All debris carried by stormwater 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, the capacity of a combination inlet at a sump shall be calculated by neglecting the grate inlet capacity.

5.2.4.7 Manholes

The primary functions of a stormwater manhole are to provide access to the closed conduit system where it is not provided by inlets, and to provide junctions for pipes. A stormwater manhole can also provide ventilation and pressure relief. Typical manholes are shown in Figure 5-11. At a minimum, manholes shall 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 pipe grade occurs; and
- According to the maximum spacing criteria found in Table 5-4.

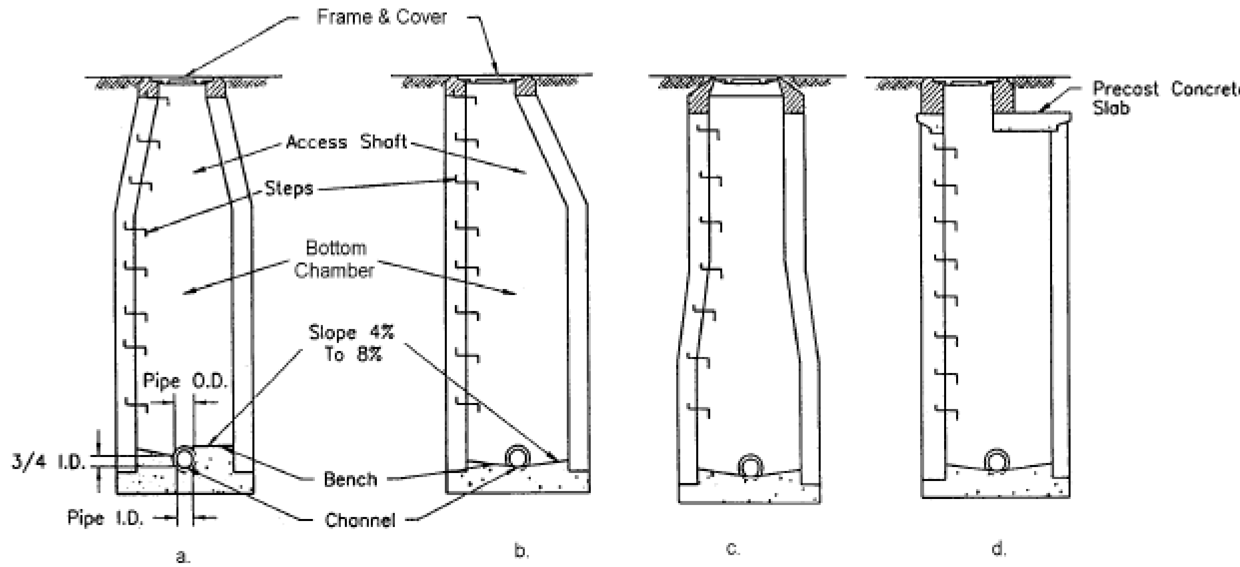


Figure 5-11 Typical Manhole Configurations
(HEC 22, 2001)

5.2.4.8 Junction and Inlet Boxes

A junction box is an underground chamber used to join two or more large storm drain conduits. This type of structure is required where storm drains are larger than the size that can be accommodated by standard manholes. Junction boxes do not need to extend to the ground surface and can be completely buried. However, riser structures must be used to provide access.

Inlet boxes are similar to junction boxes, except they are used in conjunction with inlets. They are often called inlet manholes or inlet boxes.

Where junction boxes or inlet boxes are used as access points for the storm drain system, their location must adhere to the spacing criteria outlined in Table 5-4.

5.2.4.9 Residential Backyard Drainage

Local experience has shown that many residential backyard drainage easements in Wichita and Sedgwick County tend to pond water if not adequately drained by ditches and storm sewers with enough inlets to receive the local runoff. In order to address this problem, backyard drainage ditches or swales must have a minimum slope of 1%. Inlets shall be placed to drain a maximum of 1.5 acres, or a maximum flow path length of 450 feet, whichever is most limiting. The inlets shall be sized for the residential design storm of 2-year recurrence interval.

Additionally, home lots shall be graded to provide a minimum of 1 foot of drop within the first 20 feet horizontally from the home.

5.2.5 Storm Sewers

Storm sewers are pipe conveyances used for transporting runoff from roadway and other inlets to outfalls at structural stormwater controls such as detention basins and to receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of open channels is not feasible.

Closed conduit systems are comprised of different lengths and sizes of conduits connecting structures. Segments are most often circular pipe, but may be a box or other enclosed conduit.

5.2.5.1 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, 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.

At times flow in a closed conduit may be under pressure. At other times the conduit may flow partially full. However, the usual design assumption is that the conduit is flowing full but not under pressure. Under this assumption the rate of friction head loss is assumed to be the same as the slope of the pipe ($S_f = S$). Designing for full flow is a slightly conservative assumption since the peak capacity actually occurs at slightly less than full flow.

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity flow is Manning's formula, expressed by the Equation 5-11 (HEC-22, 2001).

Equation 5-11 $$V = \left(\frac{1.486}{n}\right) R^{2/3} S^{1/2}$$

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 Equation 5-12.

Equation 5-12 $$Q = \left(\frac{1.486}{n}\right) A R^{2/3} S^{1/2}$$

where:

Q	=	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 can be transformed to Equation 5-13 and 5-14 (HEC-22, 2001).

Equation 5-13 $$V = \frac{0.590 D^{2/3} S^{1/2}}{n}$$

Equation 5-14 $$Q = \frac{0.463 D^{8/3} S^{1/2}}{n}$$

where:

D	=	diameter of pipe, ft
S	=	slope of the pipe, ft/ft

A nomograph solution of Manning's equation for full flow in circular conduits is presented in Figure 5-12. Representative values of the Manning's coefficient for various storm drain materials are provided in Appendix A.

Figure 5-13 illustrates storm drain capacity sensitivity to the parameters in 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 Figure 5-14 is provided to assist in the solution of the Manning's equation for partially full flow in storm drains. The hydraulic elements chart shows the relative flow conditions at different depths in a circular pipe and illustrates the following important points:

- Peak capacity occurs at 93 percent of the height of the circular pipe. This means that if the pipe is designed for full flow, the design will be slightly conservative.
- The velocity in a pipe flowing half-full is the same as the velocity for full flow.
- Flow velocities for flow depths greater than half-full are greater than velocities at full flow.
- Velocities in sewers are important mainly because of the possibilities of excessive erosion on the storm drain inverts at high velocity, and clogging at low velocity.

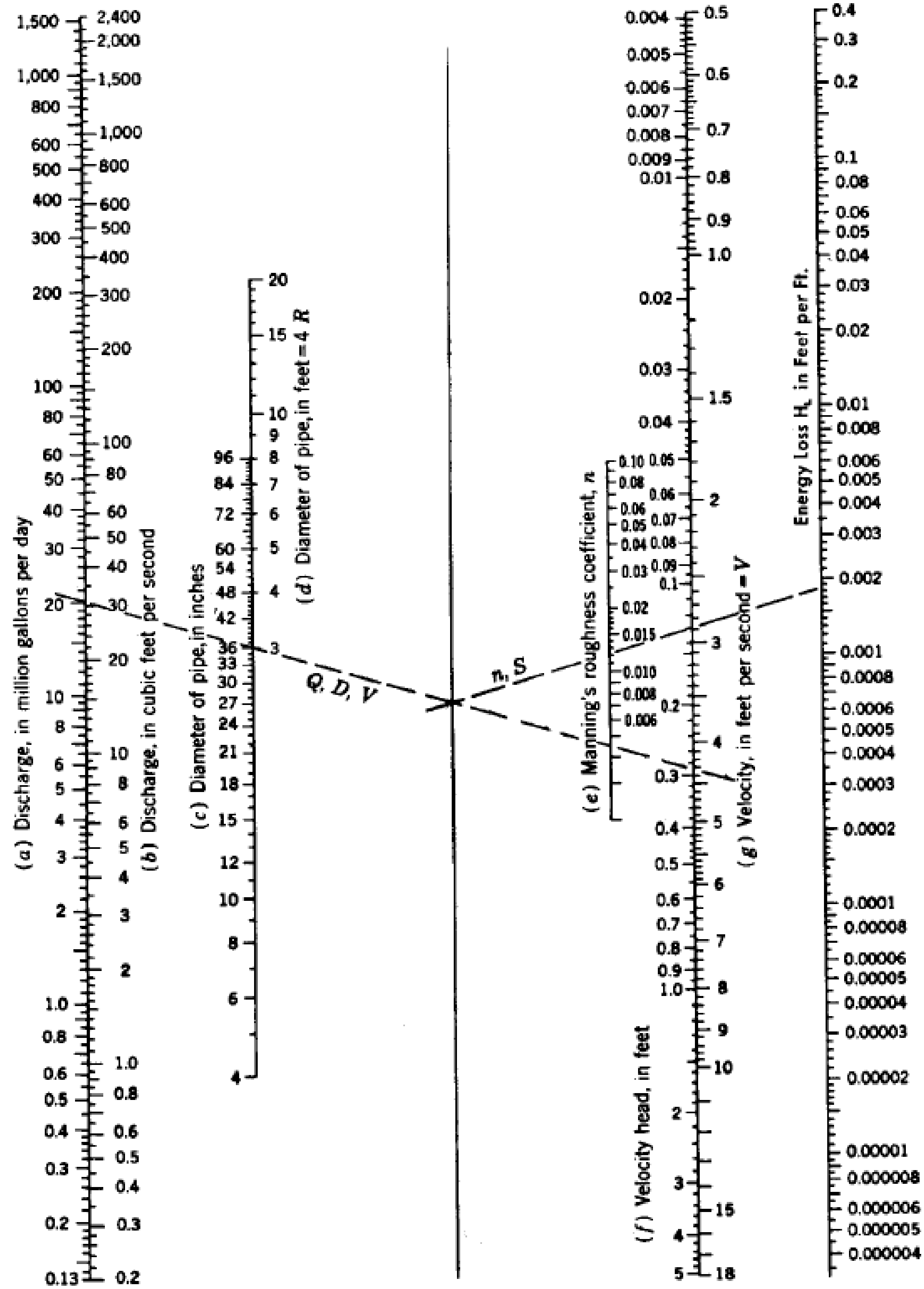


Figure 5-12 Solution of Manning's Equation for Flow in Storm Drains-English Units
(HEC-22, 2001)

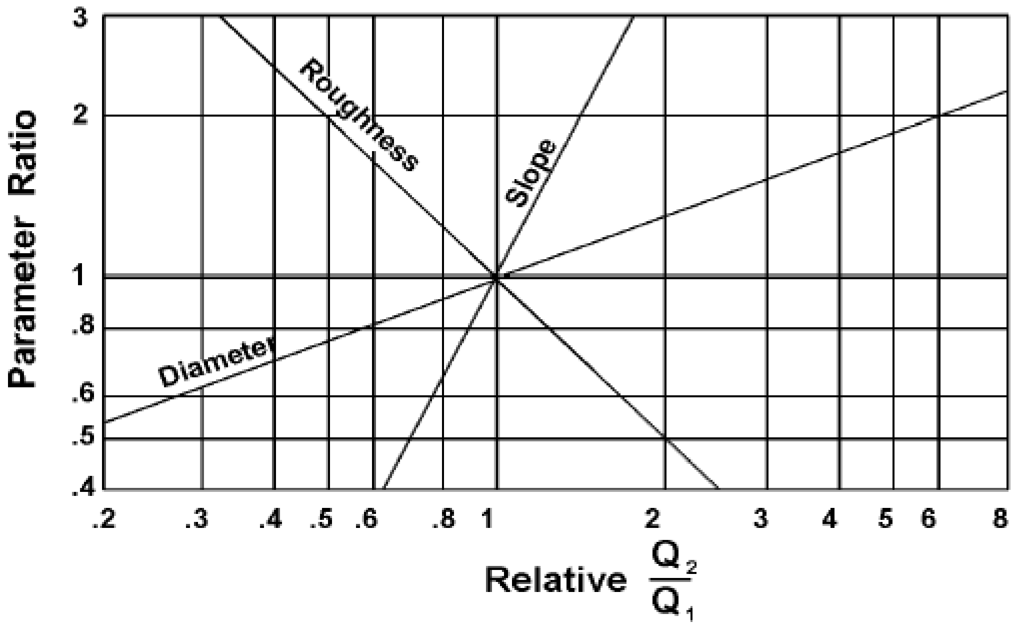


Figure 5-13 Storm Drain Capacity Sensitivity
(HEC 22, 2001)

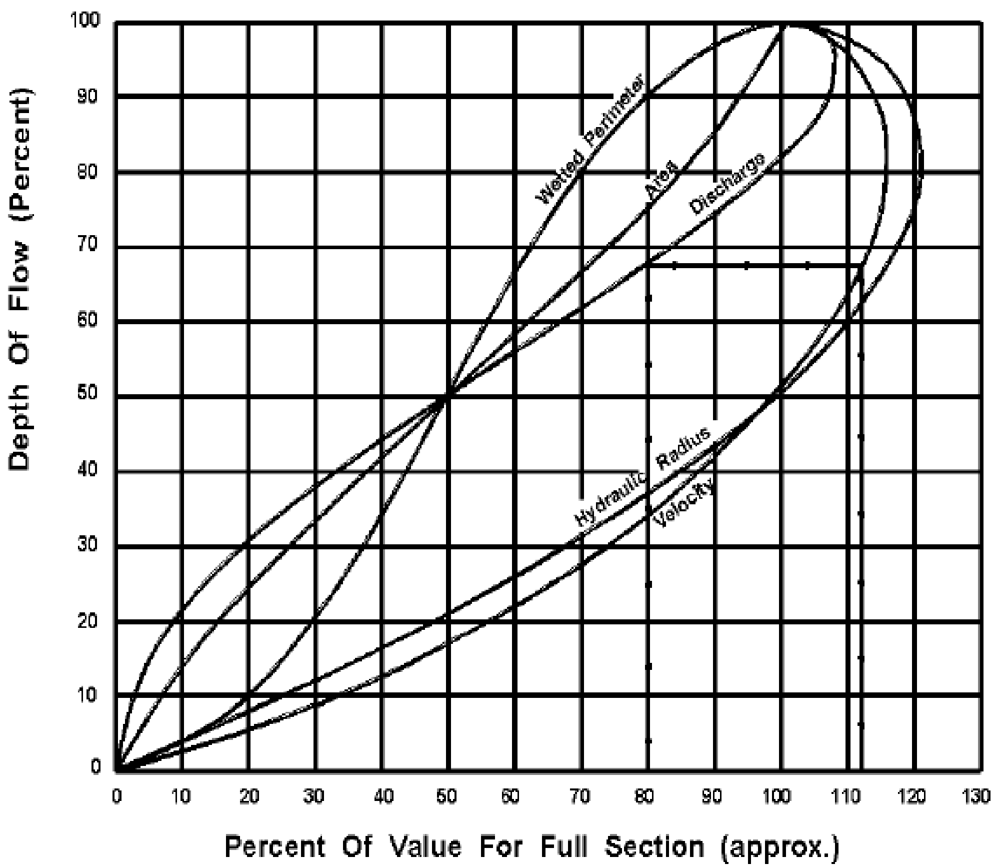


Figure 5-14 Hydraulic Elements of Circular Section (HEC 22, 2001)

5.2.5.2 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 as shown in Equation 5-15 (HEC-22, 2001).

Equation 5-15 $$E = \frac{V^2}{2g} + \frac{p}{\gamma} + z$$

where:

E	=	Total energy, ft
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)
V	=	Velocity, ft/s
g	=	Gravity, 32.2 ft/s^2

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 shown in Equation 5-16.

Equation 5-16 $$\frac{V_1^2}{2g} + \frac{p_1}{\gamma} + z_1 = \frac{V_2^2}{2g} + \frac{p_2}{\gamma} + z_2 - H_f - \sum H_m$$

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 Manning's equation. Form losses are typically calculated by multiplying the velocity head by a loss coefficient, K. The HEC-22 method shall be used for developing the value of K depending on the structure or condition being evaluated for loss. Knowledge of the location of the EGL is critical to understanding and estimating the location of the hydraulic grade line.

The hydraulic grade line (HGL) is a line coinciding with the water level 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.

The HGL is determined by subtracting the velocity head ($V^2/2g$) from the EGL. Energy concepts can be applied to both pipe flow and open channel flow. Figure 5-15 illustrates the energy and hydraulic grade lines for open channel and pressure flow in pipes.

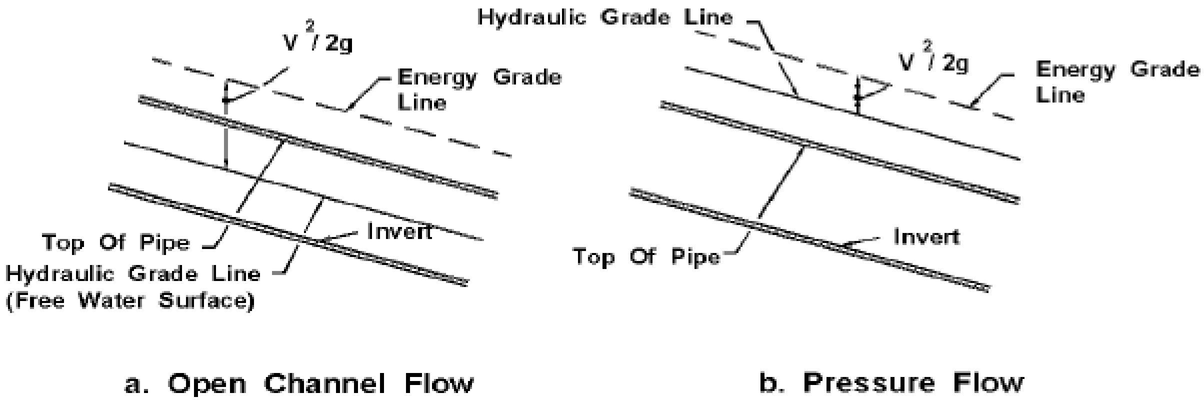


Figure 5-15 Hydraulic and Energy Grade Lines in Pipe Flow
(HEC 22, 2001)

When water is flowing through the pipe and there is a ventilated 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.

Inlet surcharging and possible manhole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must include 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 EGL and the HGL for storm drainage systems is presented in Section 5.2.5.5.

5.2.5.3 Energy Losses

Prior to computing the HGL, 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, and junction structures. The following sections present relationships for estimating typical energy losses in storm drainage systems.

The major loss in a storm drainage system is usually the friction or boundary shear loss. The head loss due to friction in a pipe is computed as shown in Equation 5-17 (HEC-22, 2001).

Equation 5-17 $H_f = S_f L$

where:

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

The friction slope in Equation 5-17 is also the slope of the hydraulic gradient for a particular uniform flow pipe run. As indicated by the equation, 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 partially full flow. Pipe friction losses for full flow can be determined by the use of Equation 5-17.

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 calculated using Equation 5-18 (HEC-22, 2001).

Equation 5-18 $H_o = 1.0\left[\frac{V_o^2}{2g} - \frac{V_d^2}{2g}\right]$

where:

H_o	=	outlet loss, ft
V_o	=	average outlet velocity, ft/s
V_d	=	channel velocity downstream of outlet, ft/s
g	=	gravity, 32.2 ft/s^2

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

An approximate method for estimating losses across manholes, junction boxes and inlet boxes/structures is provided in this section. The method involves multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 5-19. Applicable coefficients (K_{ah}) are provided in Table 5-7. For more than one pipe entering a structure, use the largest applicable value of K_{ah} in Equation 5-19 (HEC-22, 2001).

Equation 5-19 $H_{ah} = K_{ah}\left(\frac{V_o^2}{2g}\right)$

where:

H_{ah}	=	approximate structure loss, ft
K_{ah}	=	structure head loss coefficient (Table 5-7)
V_o	=	velocity at outlet pipe, ft/s
g	=	gravity, 32.2 ft/s^2

Table 5-7 Structure Head Loss Coefficients (HEC-22, 2001)

Structure Configuration	K_{ah}	Structure Configuration	K_{ah}
Inlet-straight run	0.5	Manhole-straight run	0.15
Inlet-angled through		Manhole-angled through	
90°	1.5	90°	1
60°	1.25	60°	0.85
45°	1.1	45°	0.75
22.5°	0.7	22.5°	0.45

Loss coefficients calculated using the approximate method are acceptable for sites where these types of losses are not a critical factor in system design. The more detailed procedures provided in HEC-22 must be used for sites where junction losses are a critical factor. The HEC-22 methodology is generally based on lab-scale and full-scale testing of flow through junction structures.

5.2.5.4 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 5-16. 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 5.2.5.5).

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

- Location of storm drains;
- Direction of flow;
- Location of junction structures;
- Number or label assigned to each structure;
- 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:

- Drainage areas;
- Runoff coefficients;
- 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 most upstream storm drain run:

- "From" and "To" stations, Columns 1 and 2.
- "Length" of run, Column 3.
- "Inc." drainage area, Column 4. The incremental drainage area tributary to the inlet at the upstream end of the storm drain run under consideration.
- "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.
- "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.
- "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 section under consideration. For the most upstream storm drain section this value will be the same as the value in Column 9. For all other pipe sections this value is computed by adding the "System" time of concentration (Column 10) and the "Section" time of concentration (Column 17) from the previous section together to get the system time of concentration at the upstream end of the section under consideration. If Column 10 is less than 15 minutes the designer shall use 15 minutes.

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

- "TOTAL" area, Column 5. Add the incremental area in Column 4 to the previous sections total area and place this value in Column 5.
- "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.
- "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.
- "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.
- "TOTAL Q," Column 12. Calculate the discharge as the product of Columns 8 and 11. Place this value in Column 12.
- "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.
- "PIPE DIA.," Column 13. Size the pipe using relationships and charts presented in Section 5.2.5.1 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 partially full flow or whether to go to the next smaller size and have pressure flow.

- "CAPACITY FULL," Column 14. Compute the full flow capacity of the selected pipe using Equation 5-14 and put this information in Column 14.
- "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. If the pipe is not flowing full, the velocity can be determined from Figure 5-14.
- "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.
- "CROWN DROP," Column 20. Calculate an approximate crown drop at the structure to off-set potential structure energy losses using the “previously described “rules-of-thumb.” Place this value in Column 20.
- "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 5.2.5.5. Adjust as necessary.

Preliminary Storm Drain Computation Sheet

Computed By ____________ Checked By ____________ Page ______

Date ____________ Date ____________ Of ______

Str ID		Length	Drainage Area		Runoff Coefficient	"Area" * "C"		Time of Concentration		Rain	Runoff	Pipe Diameter	Q Full	Velocity		Sec. Time	Invert Elev.		Crown Drop	Slope
From	To		Inc	Total	"C"	Inc	Total	Inlet	System	"I"	"Q"			Full	Design		U/S	D/S		
		(ft)	(Ac)	(Ac)		(Ac)	(Ac)	(min)	(min)	(in/hr)	(ft^3/s)	( )	(ft^3/s)	(ft/s)	(ft/s)	(min)	(ft)	(ft)	(ft)	(ft/ft)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21

Figure 5-16 Preliminary Storm Drain Computation Sheet (HEC-22)

5.2.5.5 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 EPA-SWMM are the most efficient means of evaluating the EGL and the HGL. However, it is important that the designer understand the analysis process to better interpret the output from computer generated storm drain designs.

Figure 5-17 provides a sketch illustrating use of the two grade lines in developing a storm drainage system. (Note that in this figure, extracted from HEC-22, junction structures are referred to as “access holes.”) The following step-by-step procedure can be used to manually compute the EGL and HGL. The computation tables in Figure 5-18 and Figure 5-19 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 about 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 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). Sheet A (Figure 5-18) is used to calculate the HGL and EGL elevations while Sheet B (Figure 5-19) is used to calculate the pipe losses and structure losses. Values obtained in Sheet B are transferred to Sheet 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. Most storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and junction structure 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.

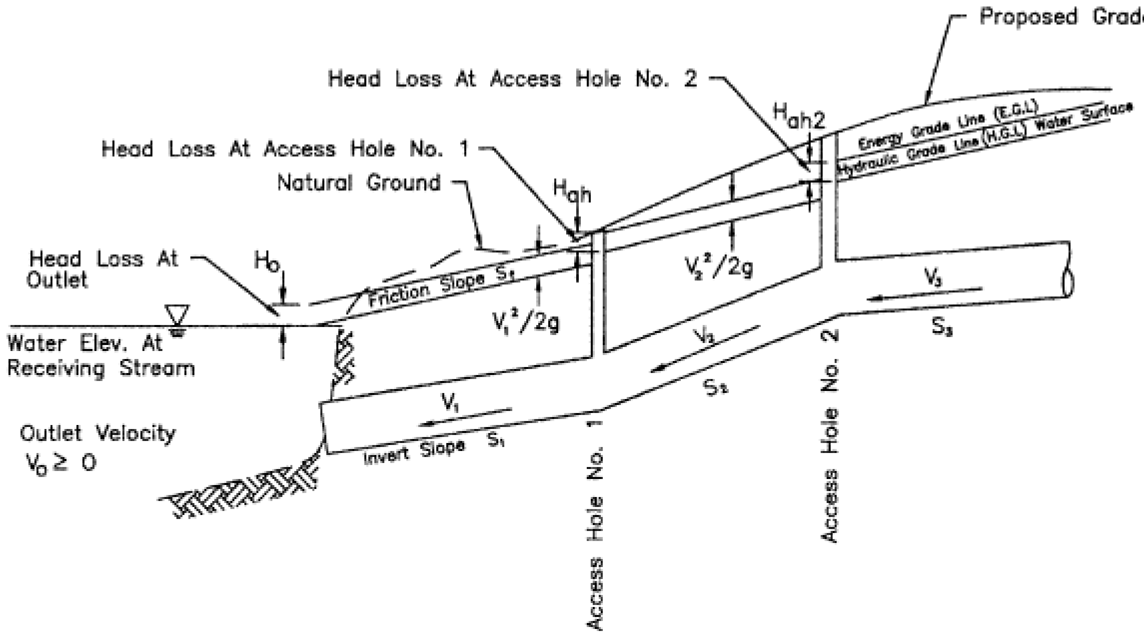


Figure 5-17 Energy and Hydraulic Grade Line Illustration
(HEC 22, 2001)

The EGL computational procedure follows:

Step 1 The first line of Sheet 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 Column 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 (dc + D)/2, the EGL will be the TW elevation plus the velocity head for the conduit flow conditions (where dc is the critical depth and D is the conduit diameter).

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

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 (Column 7A) is determined and then come back and finish this step. Put the EGL in Column 13A.

Note: The values for dc for circular pipes can be determined from Figure 5-25. 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 Sheet 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.

- **5A**: Partially full flow: Using the hydraulic elements graph in Figure 5-14 with the ratio of partially 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 5A respectively of Sheet A. Compute the velocity head ($V^2/2g$) and place in Column 7A.
- **5B**: Compute critical depth for the conduit using Figure 5-25. If the conduit is not circular, see HDS-5 for additional charts. Enter this value in Column 6b of Sheet A.
- **5C**: Compare the flow depth in Column 6a (Sheet A) with the critical depth in Column 6b (Sheet 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.

- **5D**: Pipe losses in a supercritical pipe section are not carried upstream. Therefore, enter a zero (0) in Column 7B for this structure.
- **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 a junction structure drops enough that the invert of the upstream pipe is not inundated by the flow in the downstream pipe, the designer goes back to Step1 and begins a new design as if the downstream section did not exist.
- **5F**: Compute normal depth for the conduit using Figure 5-12 and critical depth using Figure 5-25. If the conduit is not circular see HDS-5 for additional charts. Enter these values in Columns 6A and 6b of Sheet A.
- **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 partially full flow, continue with Step 5H.
- **5H**: Partially 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.
- **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.
- **5J**: Subcritical flow upstream: Compute EGLo 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.
- **5K**: Supercritical flow upstream: manhole losses do not apply when the flow in two (2) successive pipes is supercritical. Place zeros (0) in Columns 11A, 12A, and 7B 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 by dividing friction loss by length for a pipe flowing full, as shown below (n = Manning's n value). 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.

$$S_f = \frac{H_f}{L} = \frac{185n^2\left(\frac{V^2}{2g}\right)}{D^{4/3}}$$

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 expansion (H_e) losses, and junction losses (H_j) and place the values in Columns 3B and 4B, respectively. Add the values in 3B and 4B and place the total in Column 5B and 9A.

Step 8 Compute the energy grade line value at the outlet of the structure (EGL_o) as the EGLi 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 manhole (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 6B. 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 junction structure, compute the structure head loss coefficient, K_{ah}, based on the pipe deflection angle (θ) through the structure. Enter the deflection angle in Column 7B. Look up K_{ah} in Table 5-7 based on the deflection angle. Enter this value in Column 7B and 11A. If the inflow storm drain invert is not submerged by the water level in the junction structure, compute the head in the junction structure using culvert techniques from HDS-5 as follows:

- **10A:** If the structure outflow pipe is flowing full or partially full under outlet control, compute the manhole loss per Equation 5-18. Enter this value in Column 12A and 11A, continue with Step 12. Add a note on Sheet A indicating that this is a drop structure.
- **10B:** If the outflow pipe functions under inlet control, compute the depth in the junction structure (HGL) using Figure 5-22. If the storm conduit shape is other than circular, select the appropriate inlet control nomograph from HDS-5. Add these values to the manhole invert to determine the HGL. Since the velocity in

the junction structure is negligible, the EGL and HGL are the same. Enter HGL in Column 14A and EGL in Column 13A. Add a note on Sheet A indicating that this is a drop structure. Go to Step 14.

Step 11 Compute the total junction structure loss, H_{ah}, by multiplying the K_{ah} value in Column 11A by the velocity head in Column 7A. Enter this value in Column 12A.

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

Step 13 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 14 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 15 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 16 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 18.

Step 17 Continue to determine the EGL through the system by repeating Steps 4 through 18. (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 18 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.

Energy Grade Line Computation Sheet - Table A

Computed By ________ Checked By ________ Initial Tailwater Elevation ________ Page ________
Date ________ Date ________ Of ________

Str ID	D	Q	L	V	d	dc	$V^2/2g$	S_f	Total Pipe Loss (Table B)	EGL_o	K_{ah} (Table B)	$K_{ah}(V^2/2g)$	EGL_i	HGL	U/S TOC	Surf Elev
	(ft)	(cfs)	(ft)	(ft / s)	(ft)	(ft)	(ft)	(ft / ft)	(ft)	(ft)		(ft)	(ft)	(ft)	(min)	(ft)
1A	2A	3A	4A	5A	6a	6b	7A	8A	9A	10A	11A	12A	13A	14A	15A	16A

Figure 5-18 Energy Grade Line Computation Sheet - Sheet A (HEC-22)

Energy Grade Line Computation Sheet - Table B

Page ________ Computed By ________________ Checked By ________________

Of ________ Date ________________ Date ________________

Str ID	Losses (ft)				Depth (ft)	Structure Loss Coefficient (dim)
	H_f	H_e	H_j	Total	d_{aho}	K_{ah}
1B	2B	3B	4B	5B	6B	7B

Figure 5-19 Energy Grade Line Computation Sheet - Sheet B

5.2.6 Software for Storm Sewer Analysis

The calculation of gutter flow, inlet capture and pipe hydraulic grade lines can be extremely time consuming when performed by hand. There are several widely-used programs created by public agencies that can be acquired at no cost to speed these calculations.

Gutter and inlet calculations can be made at minimal effort using the FHWA "HY-22 Urban Drainage Design" program, which runs the procedures found in HEC-22. A screen capture from HY-22 is shown below, displaying the parameters previously discussed for inlet design.

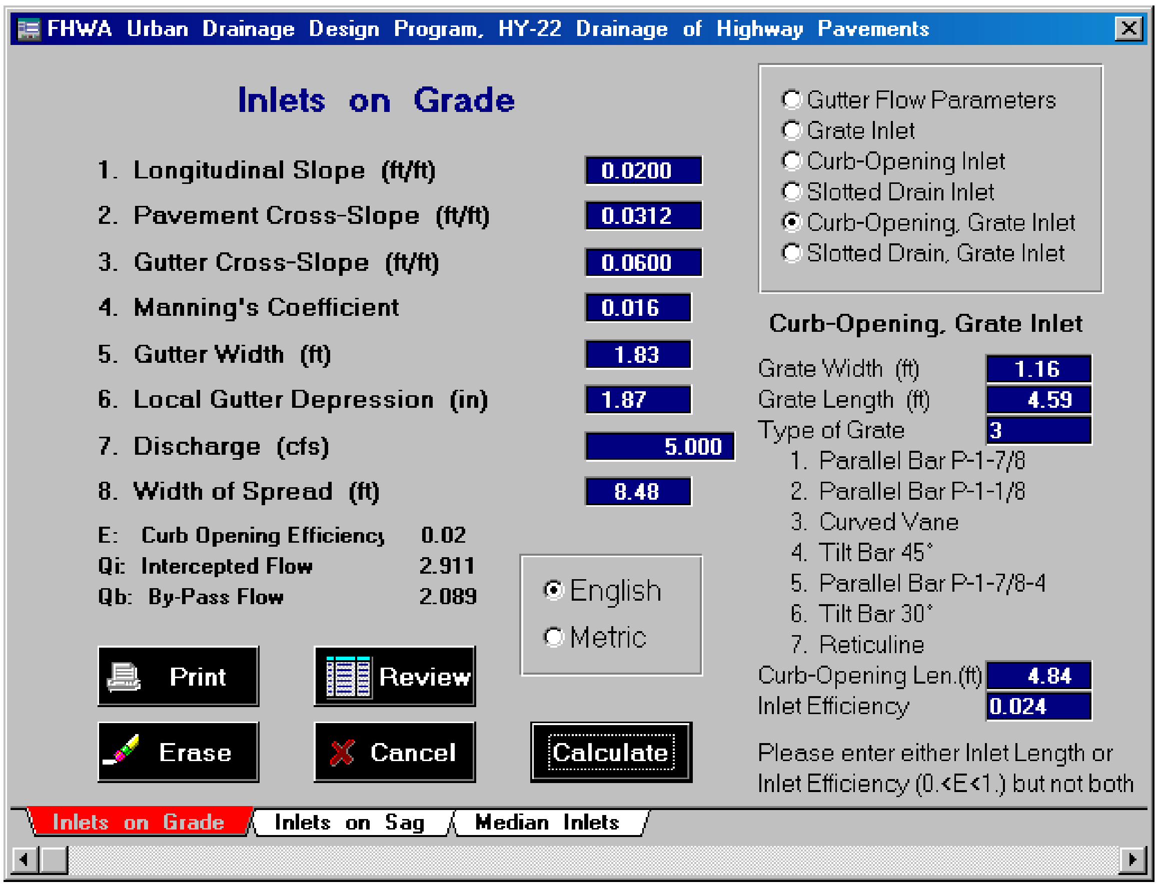


Figure 5-20 Screen Capture from Visual Urban

Visual Urban can be downloaded from:

http://www.fhwa.dot.gov/engineering/hydraulics/software/softwaredetail.cfm

The Urban Drainage and Flood Control District (UDFCD) of Denver has also published an inlet design spreadsheet based on the HEC-22 methodology. This spreadsheet can be used to create head-discharge curves for a wide variety of gutter and inlet conditions. These curves can be used with EPA-SWMM to create a dynamic model of gutter and storm sewer flow. The UDFCD spreadsheet can be downloaded from:

http://www.udfcd.org/downloads/down_software.htm

Storm sewer hydraulic grade line calculations can be performed using the EPA program, “EPA-SWMM”. In addition to benefits described elsewhere, EPA-SWMM can be used when considering complex tailwater effects, storage routing, channel flow, and storm sewer flow. Inlets can be modeled using an outlet with head-discharge relationships based on HEC-22. EPA-SWMM can be downloaded from:

http://www.epa.gov/ednnrmrl/models/swmm/

Proprietary software may also be used for storm sewer analysis. However, commercial third-party models must be pre-approved by the City and County, not only on the basis of technical acceptance but also on the basis of the cost for the City and County to maintain a license of the product for review purposes.

5.3 Culverts

5.3.1 Introduction

A culvert is a short, closed (covered) conduit that conveys stormwater runoff under an embankment or away from the street right-of-way. The primary purpose of a culvert is to convey surface water, but it may also be designed to restrict flow and reduce downstream peak flows.

For economy, engineers may design culverts to operate with the inlet submerged during flood flows, if conditions permit. Design considerations include site and roadway data, design parameters (including shape, material, and orientation), hydrology (flood magnitude versus frequency relation), and channel analysis (stage versus discharge relation).

5.3.2 General Criteria

The design of a culvert for new developments or redevelopments must take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following design criteria shall be used for all culvert designs as applicable.

Culvert design shall follow the analytical procedures outlined in: FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001. The FHWA computer program HY-8 is recommended to implement these procedures.

Culverts that do not cross public roadways shall provide allowable headwater protection for the design storm. Culverts that do cross public roadways shall provide allowable headwater protection for the 100-year storm. The allowable headwater is the depth of water that can be ponded at the upstream end of the culvert, which will be limited by one or more of the following constraints:

- Allowable headwater shall be no greater than one foot below the low point in the road grade unless overflow has been allowed by the roadway design and approved by the Director or his/her designee.
- Allowable headwater shall be no greater than the elevation where flow diverts around the culvert.
- Ponding shall not increase base flood elevations for streams with current FEMA floodplain mapping.
- The 100-year frequency storm shall be analyzed through all culverts to ensure a minimum of 2 feet of freeboard for building structures (e.g., houses and commercial buildings).

Either the headwater shall be controlled to produce acceptable exit velocities per Section 5.7, or energy dissipation shall be provided where these velocities are exceeded.

To ensure self-cleaning velocities during partial depth flow, a minimum velocity of 2.5 feet per second is required for the 1-year flow where practicable.

Culverts shall have a slope no less than 0.3% and no more than 10%.

Reinforced concrete pipe (RCP), pre-cast concrete boxes, or cast-in-place concrete boxes shall be used under all public roadways and for all flowing streams.

In the City the minimum allowable pipe diameter shall be 15 inches for pipes in the right-of-way, pipes conveying offsite drainage, or pipes draining structural stormwater facilities. This minimum pipe diameter shall be 18 inches in the County.

The hydraulic conditions downstream of the culvert site must be evaluated to determine a tailwater depth for a range of discharges. The following conditions must be considered when setting tailwater elevations:

- If the culvert outlet is operating with a free outfall, the critical depth and equivalent hydraulic grade line shall be determined.
- For culverts that discharge to an open channel, the stage-discharge curve for the channel shall be determined.
- If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert shall establish the design tailwater depth for the upstream culvert.
- If the culvert discharges to a lake, pond, or other major water body, the corresponding high water elevation of the particular water body shall establish the culvert tailwater.

The culvert inlet loss coefficients given in Table 5-8 shall be used for all culverts per HDS-5, 2001.

Table 5-8 Inlet Coefficients

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12(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 bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal[1]	
Projecting form fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered 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
Slide- 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 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 [1/12(D)] or beveled top edge	0.2
Wingwalls at 10^o or 25^o to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

[1]Although laboratory tests have not been completed on K_e values for High-Density Polyethylene (HDPE) pipes, the K_e values for corrugated metal pipes are recommended for HDPE pipes.

*Note: "End Section conforming to fill slope" 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. These latter sections can be designed using the information given for the beveled inlet.

Source: HDS-5, 2001

5.3.3 Design Procedures

Culvert Shapes

The cross sectional shape of a culvert can be square, rectangular, circular, elliptical, or arch shaped. Reinforced concrete boxes are square or rectangular. Reinforced concrete pipes are available in circular, arch, and elliptical shapes. Pipe-arch, elliptical, and box shapes are appropriate in situations with limited cover or low allowable headwater. Pipe-arch and box shapes can also be used where less obstruction of the waterway is desirable. Round pipes are generally more economical than noncircular pipes. Therefore, the use of noncircular pipe should be justified by dimensional constraints or unusual allowable headwall elevation constraints.

Box Culverts

Box culverts are available in a wide range of sizes with one or more barrels. Box culverts are either cast in place in the field by forming and pouring or delivered to the site as precast sections.

Multiple Barrel Culverts

Culverts with multiple barrels are appropriate in situations with limited cover, low allowable flow depth or large capacity requirements. Also, a multiple cell box culvert is generally more economical than a box culvert with a single long span. They can be used in wide, shallow channels to limit the constriction of the flow. However, widening a natural channel to accommodate a multiple-barrel culvert often results in sediment and debris accumulation in the widened channel section and in the culvert. Low flow barrels may be required by the Director or his/her designee to ensure self-cleaning velocities for the 1-year storm.

Types of Flow Control

There are two basic types of flow control conditions for culverts, and these conditions are defined by the location of the control section and the critical flow depth. Variations of these basic types are numerous. Please refer to HDS-5 (FHWA Hydraulic Design of Highway Culverts) for more information on various culvert flow regimes.

<u>Inlet Control:</u> The Inlet control condition occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. This typically happens when a culvert is operating on a steep slope. The control section of a culvert is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.

<u>Outlet Control:</u> The outlet control condition occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for the outlet control condition in a culvert is located at the barrel exit or further downstream. Subcritical or pressure flow exists in the culvert barrel under these conditions.

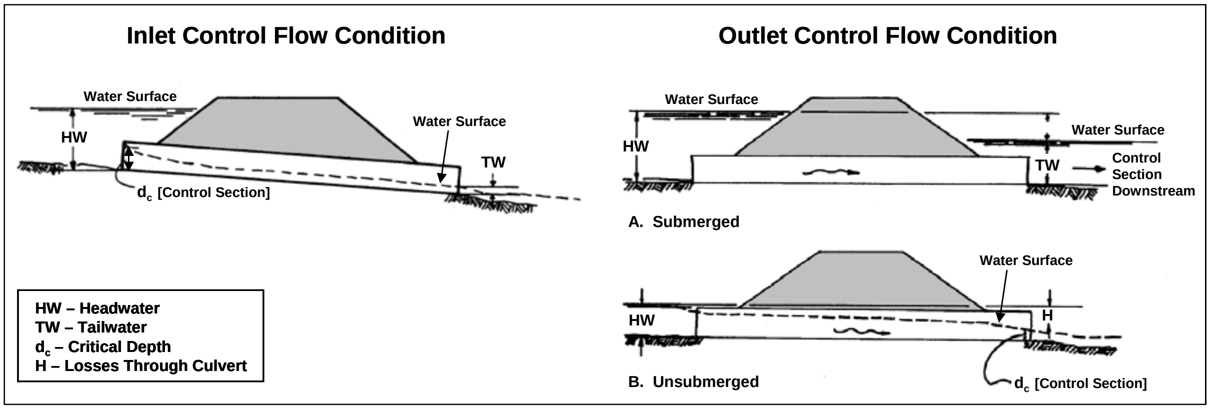


Figure 5-21 Culvert Flow Conditions
(Adapted from: HDS-5, 2001)

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern the particular culvert design.

Procedures

The culvert design process includes the following basic steps:

- Define the location, orientation, shape, length and material (usually concrete) for the culvert to be designed.
- With consideration of the site data, compute design flows and establish maximum barrel height, allowable outlet velocity, tailwater elevations, and maximum allowable headwater.
- Using the culvert sizing methods described below, size the culvert and entrance treatments.
- Optimize the culvert configuration by repeating the procedure as necessary.
- Treat any excessive outlet velocity with an energy dissipator (see Section 5.7).

There are three basic procedures used for designing culverts: (1) inlet control and pipe flow design equations, (2) manual use of inlet and outlet control nomographs, and (3) the use of computer programs such as FHWA HY-8. It is highly recommended that the HY-8 computer model be used for culvert design. However, the nomograph solution method is presented below as an example of the basic design method.

Nomographs

The use of culvert design nomographs requires a trial and error solution. Nomograph solutions provide reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional, separate computations beyond what can be obtained from the nomographs. Figure 5-22 and Figure 5-23 show examples of inlet control and outlet control nomographs for the design of circular concrete pipe culverts. For other culvert designs, refer to the complete set of nomographs in FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001.

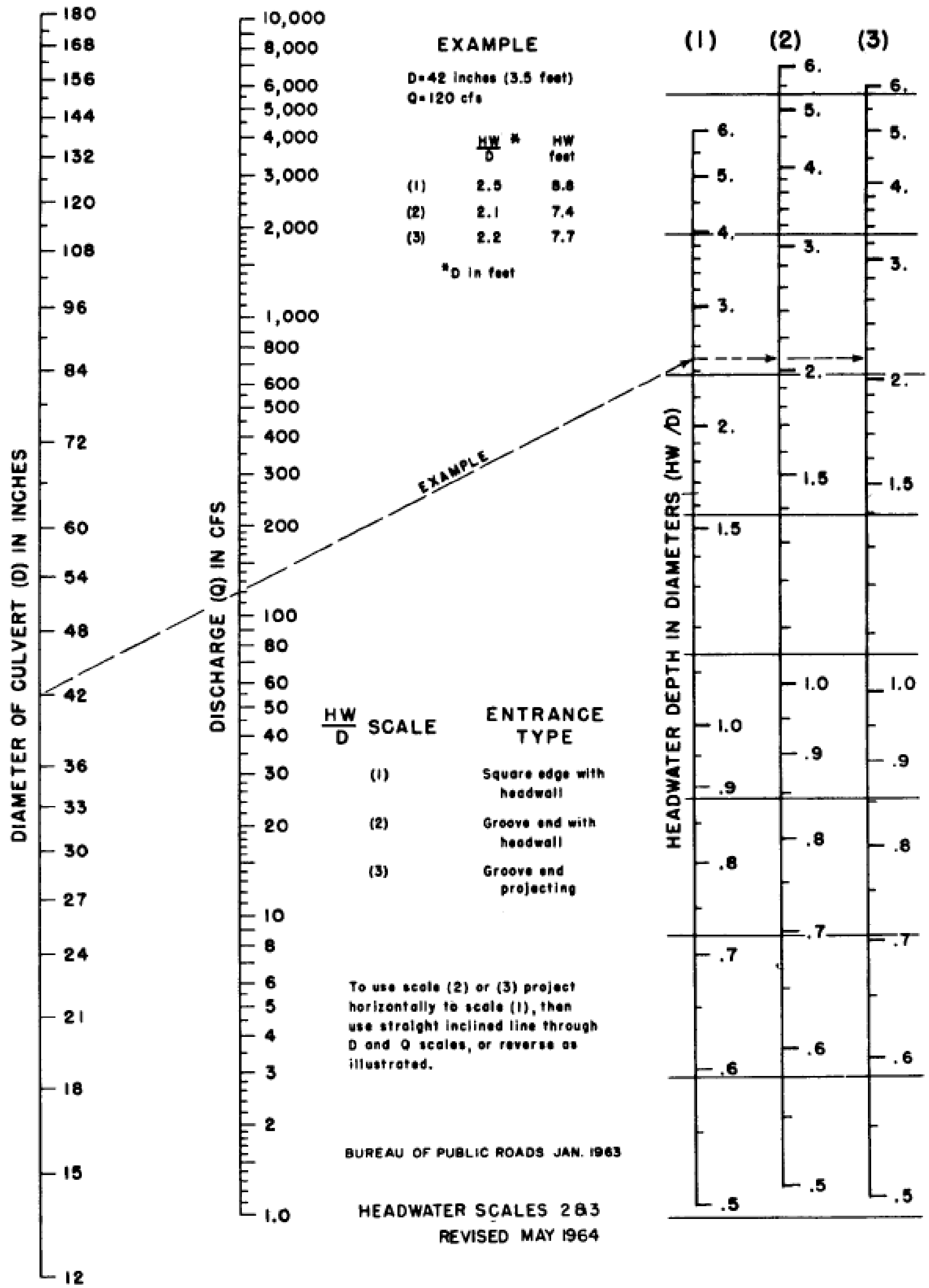


Figure 5-22 Headwater Depth for Circular Concrete Pipe Culvert with Inlet Control
(HEC-22, 2001)

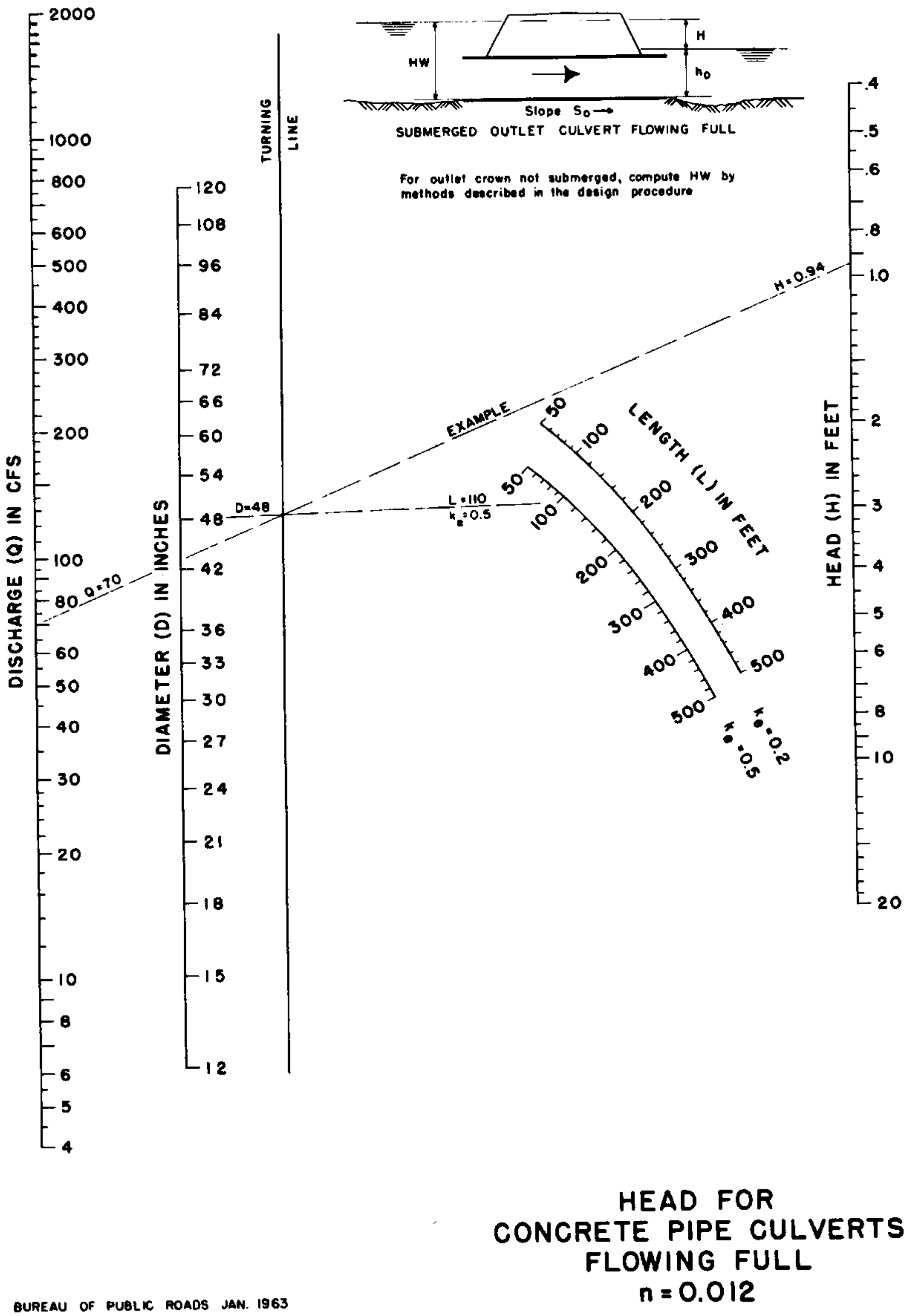


Figure 5-23 Head for Circular Concrete Pipe Culverts Flowing Full

(HDS-5, 2001)

Design Procedure

The following design procedure requires the use of inlet and outlet control nomographs.

Step 1 List design data:

Q	=	discharge (cfs)
L	=	culvert length (ft)
S	=	culvert slope (ft/ft)
TW	=	tailwater depth (ft)
V	=	velocity for trial diameter (ft/s)
K_e	=	inlet loss coefficient
HW	=	allowable headwater depth for the design storm (ft)

Step 2 Determine a trial culvert size by assuming a trial velocity of 3 to 5 ft/s and computing the culvert area, A = Q/V. Determine the culvert diameter.

Step 3 Find the actual HW for the trial size culvert for both inlet and outlet control.

- For inlet control, enter inlet control nomograph with D and Q and find HW/D for the proper entrance type.

  Compute HW and, if too large or too small, try another culvert size before computing HW for outlet control.

- For outlet control enter the outlet control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter. (Outlet loss is assumed to be one velocity head with this nomograph.)

  To compute HW, connect the length scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from the equation per (HDS-5, 2001):

Equation 5-20 $$HW = H + h_o - LS$$

where:

H	=	headloss, ft
h_o	=	½ (critical depth + D), or tailwater depth, whichever is greater
L	=	culvert length, ft
S	=	culvert slope, ft/ft

For critical depth, see Figure 5-25.

Step 4 Compare the computed headwaters and use the higher HW nomograph to determine if the culvert is under inlet or outlet control.

- If inlet control governs, then the design is complete and no further analysis is required.

- If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

Step 5 Calculate exit velocity and if erosion problems might be expected, refer to Section 5.7 for appropriate energy dissipation designs. (An energy dissipation design that significantly changes the tailwater would affect outlet hydraulics of the culvert, and adjustments to the analysis would be required.)

Performance Curves - Roadway Overtopping

A performance curve for any culvert can be obtained from the nomographs by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. A graph is then plotted of headwater versus discharge with sufficient points so that a curve can be drawn through the range of interest. These curves are applicable through a range of headwater versus discharges for culvert. Such computations are made much easier by the use of computer program HY-8.

To complete the culvert design, roadway overtopping shall be analyzed if overtopping is allowed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed.

The overall performance curve can be determined as follows:

Step 1 Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters shall be calculated.

Step 2 Combine the inlet and outlet control performance curves to define a single performance curve for the culvert based on the lower flow.

Step 3 When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation 5-21 to calculate flow rates across the roadway per (HDS-5, 2001).

Equation 5-21 $$Q = C_d L (HW_r)^{1.5}$$

where:
Q = Overtopping flow rate (ft^3/s)
C_d = overtopping discharge coefficient
L = length of roadway (ft)

HW_r = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown (ft)

Note: See Figure 5-24 for guidance on determining a value for C_d. For more information on calculating overtopping flow rates see pages 38 - 44 in HDS-5, 2001.

Step 4 Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

Storage Routing

Storage capacity behind a roadway embankment may significantly attenuate a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced. However, storage should be taken into consideration only if the storage area will remain available for the life of the culvert as a result of ownership, right-of-way or easement. If significant storage is anticipated behind a culvert, the design should be checked by routing the design hydrographs through the culvert to determine the discharge and stage behind the culvert. See Volume 2, Chapter 4 for more information on routing. Additional routing procedures are outlined in HDS-5.

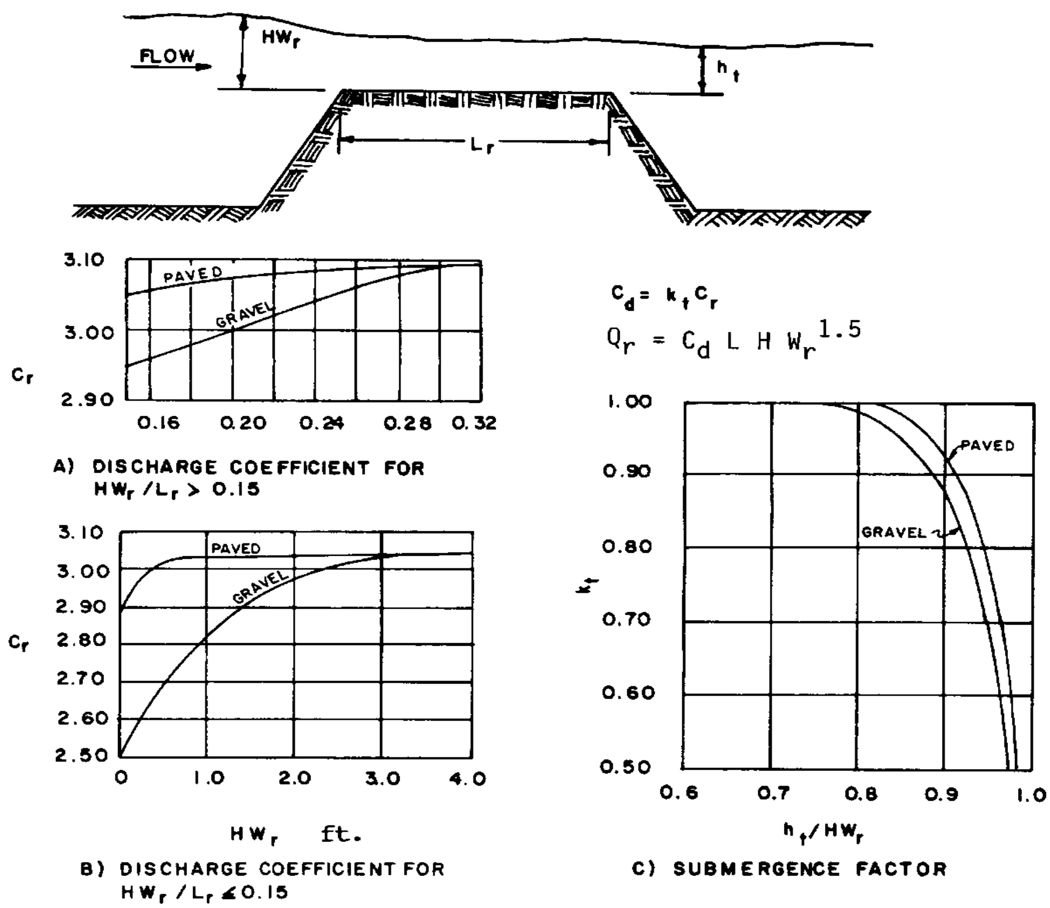


Figure 5-24 Discharge Coefficients for Roadway Overtopping

(HDS-5, 2001)

Example Problem – Culvert Design

Size a circular concrete culvert given the following example data, which were determined by physical limitations at the culvert site.

- Discharge for 1-yr flood = 35 cfs
- Discharge for 25-yr flood = 70 cfs
- Allowable HW for 25-yr discharge = 5.25 ft
- Length of culvert = 100 ft
- Slope of culvert = 0.012
- Natural channel invert elevations: inlet = 15.50 ft, outlet = 14.30 ft
- Tailwater depth for 25-yr discharge = 3.5 ft
- Tailwater depth is assumed to be approximately the normal depth in the downstream channel in this example
- Entrance type = groove end with headwall

Assume a trial culvert velocity of 5 ft/s. Required flow area = (70 cfs)/(5 ft/s) = 14 ft^2 (for the 25-yr recurrence flood).

The corresponding culvert diameter is about 48 in. This can be calculated by using the formula for area of a circle: Area = $(\pi D^2)/4$ or D = $(\text{Area}*4/\pi)^{1/2}$ Therefore: D = $((14 \times 4)/3.14)^{1/2} \times 12$ in/ft) = 50.7 in

A grooved end concrete culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 5-22) with a pipe diameter of 48 inches and a discharge of 70 cfs; read a HW/D value of 0.93.

The depth of headwater (HW) is (0.93) x (4) = 3.72 ft, which is less than the allowable headwater of 5.25 ft. Since 3.72 ft is considerably less than 5.25 try a smaller culvert.

Using the same procedures outlined in steps 3 and 4 the following results were obtained.

- 42-inch culvert – HW = 4.13 ft
- 36-inch culvert – HW = 5.04 ft

Select a 36-inch culvert to check for outlet control.

The culvert is checked for outlet control by using Figure 5-23. With an entrance loss coefficient K_e of 0.20 (Table 5-8) a flow rate of 70 cfs, a culvert length of 100 ft, and a pipe diameter of 36 inches, an H value of 2.8 ft is determined. The headwater for outlet control is computed by the equation: HW = H + h_o - LS

Compute h_o:

h_o = T_w or ½ (critical depth in culvert + D), whichever is greater.

h_o = 3.5 ft or h_o = ½ (2.7 + 3.0) = 2.85 ft

Note: critical depth is obtained from Figure 5-25.

Therefore: h_o = 3.5 ft

The headwater depth for outlet control is:

HW = H + h_o - LS = 2.8 + 3.5 - (100) x (0.012) = 5.10 ft

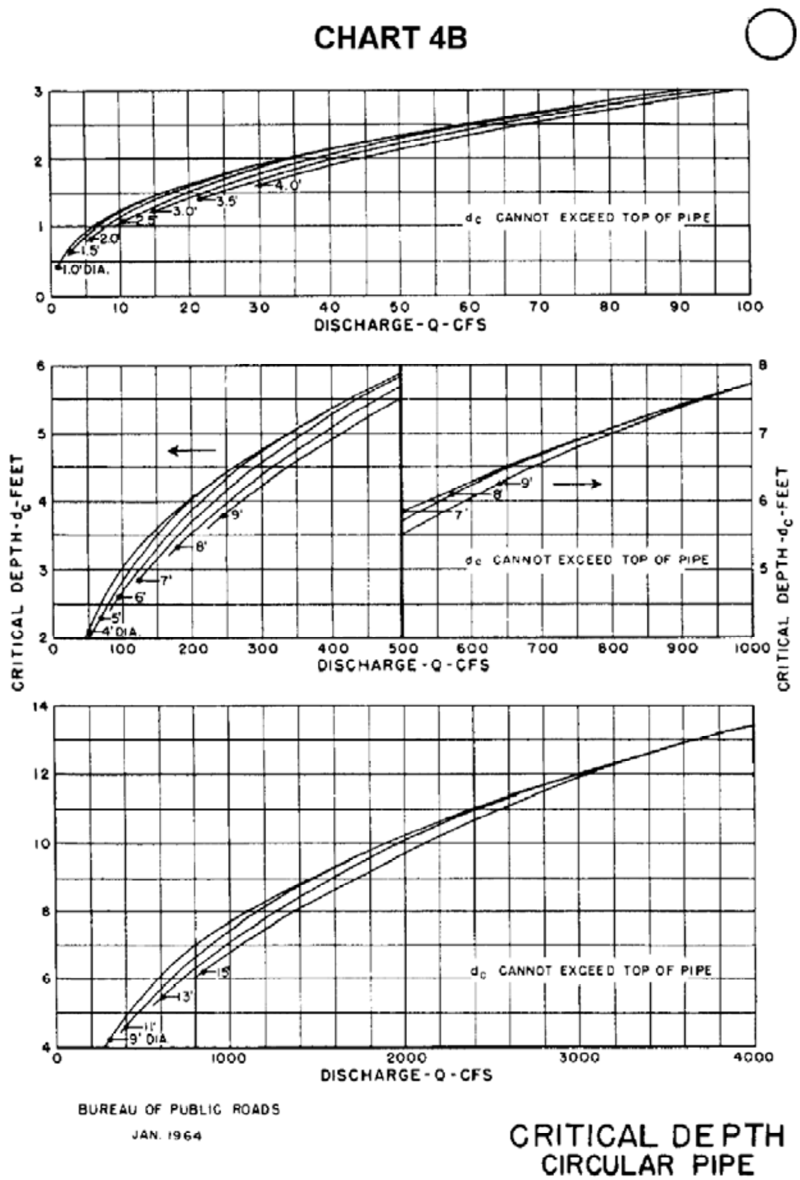


Figure 5-25 Critical Depth for Circular Pipe
(HDS-5, 2001)

Since HW for outlet control (5.10 ft) is greater than the HW for inlet control (5.04 ft), outlet control governs the culvert design. Thus, the maximum headwater expected for a 25-year recurrence flood is 5.10 ft, which is less than the allowable headwater of 5.25 ft.

Estimate outlet exit velocity. Since this culvert is in outlet control and discharges into an open channel downstream with tailwater above the culvert, the culvert will be flowing full at the flow depth in the channel. Using the design peak discharge of 70 cfs and the area of a 36-inch or 3.0-foot diameter culvert the average exit velocity will be:

Q = VA
Therefore: $V = 70 / ((3.14(3.0)^2)/4) = 9.9$ ft/s
With this high velocity, provide an energy dissipator at the culvert outlet. See Section 5.57.

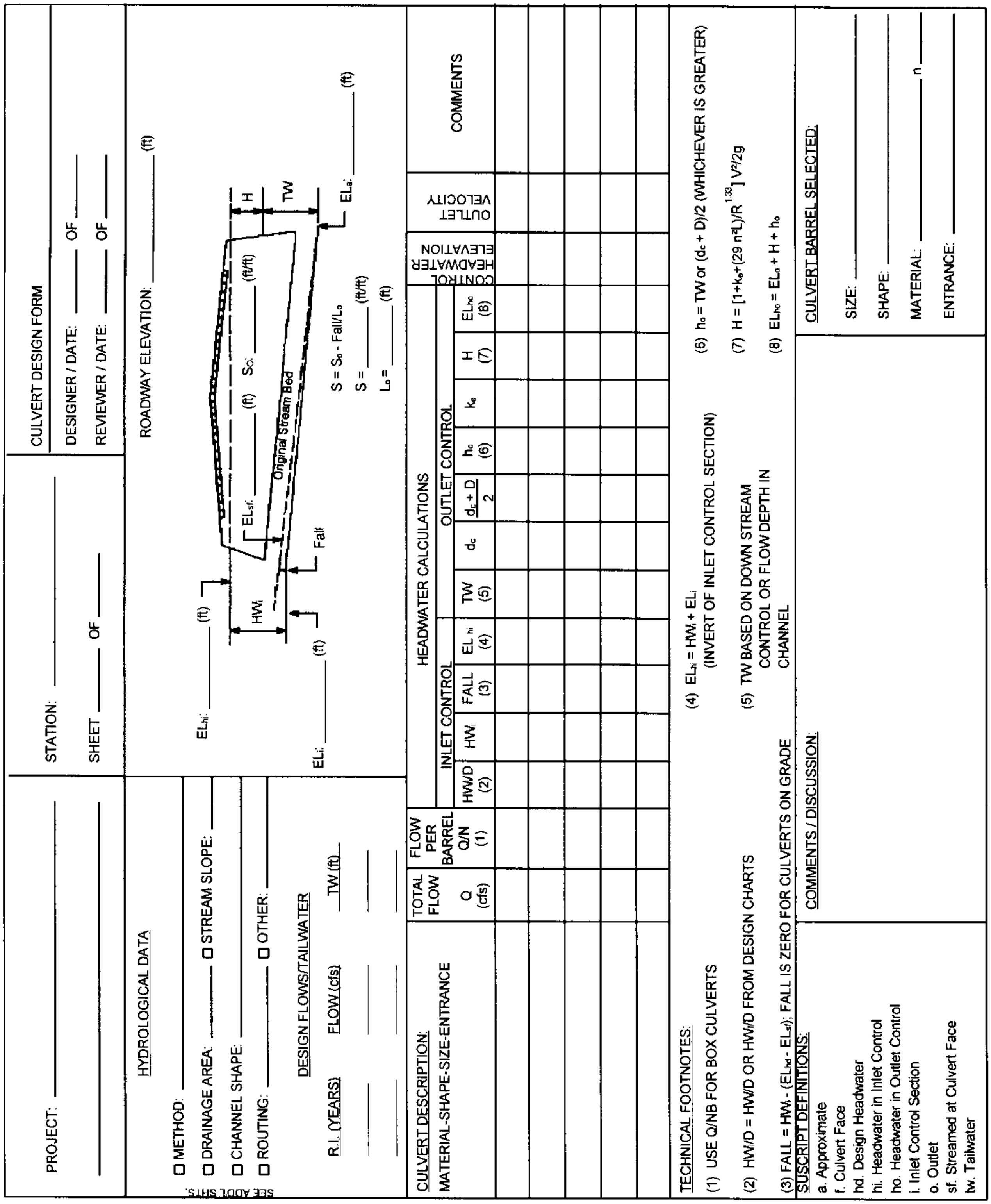
Check for minimum velocity using the 1-year flow of 35 cfs.

Therefore: $V = 35 / ((3.14(3.0)^2/4) = 5.0$ ft/s, therefore V > minimum of 2.5 – OK

(Note that for checking minimum velocity, the assumption of full pipe flow is conservative. If the computed velocity had been too low, then the resulting pipe flow velocity would need to be checked using Manning's equation or HY-8.)

The 100-year flow shall be analyzed through the culvert to determine if any flooding problems will be associated with this culvert.

Figure 5-26 provides a convenient form to organize culvert design calculations.



PROJECT: ______

STATION: ______

SHEET ____ OF ____

CULVERT DESIGN FORM

DESIGNER / DATE: ____ OF ____

REVIEWER / DATE: ____ OF ____

HYDROLOGICAL DATA

SEE ADD'L SHTS.

☐ METHOD: ______

☐ DRAINAGE AREA: ____ ☐ STREAM SLOPE: ____

☐ CHANNEL SHAPE: ______

☐ ROUTING: ____ ☐ OTHER: ____

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION: ______ (ft)

CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL				OUTLET CONTROL							CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
			HW/D (2)	HW_i	FALL (3)	EL_{hi} (4)	TW (5)	d_c	$\frac{d_c + D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)			

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) HW/D = HW/D OR HW/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) $h_o = TW$ or $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = [1+k_e+(29\ n^2L)/R^{1.33}]\ V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUSCRIPT DEFINITIONS:

a. Approximate
f. Culvert Face
hd. Design Headwater
hi. Headwater in Inlet Control
ho. Headwater in Outlet Control
i. Inlet Control Section
o. Outlet
sf. Streamed at Culvert Face
tw. Tailwater

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ______

SHAPE: ______

MATERIAL: ______ n ____

ENTRANCE: ______

Figure 5-26 Culvert Design Calculations Form

(HDS-5, 2001)

5.3.4 Design Procedures for Beveled-Edged Inlets

Introduction

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side-tapered, and slope-tapered inlets. Designers interested in using side- and slope-tapered inlets should consult the detailed design criteria and example designs outlined in HDS-5.

Design Figures

Four inlet control figures for culverts with beveled edges are found in Section I of HDS 5.

Table 5-9 Inlet Control Figures of HDS-5

Chart	Page	Use for
3	D-3A & B	circular pipe culverts with beveled rings
10	D-10-1 thru 10-3	90^{o} headwalls (same for 90^{o} wingwalls)
11	D-11-1 thru 11-4	skewed headwalls
12	D-12-1 thru 12-3	wingwalls with flare angles of 18 to 45 degrees

The following symbols are used in these figures:
B – Width of culvert barrel or diameter of pipe culvert
D – Height of box culvert or diameter of pipe culvert
Hf – Depth of pool or head, above the face section of invert
N – Number of barrels
Q – Design discharge

Design Procedure

The figures for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier. Note that Charts 10, 11, and 12 apply only to bevels having either a 33-degree angle (1.5:1) or a 45-degree angle (1:1).

For box culverts the dimensions of the bevels to be used are based on the culvert dimensions. The top bevel dimension is determined by multiplying the height of the culvert by a factor. The side bevel dimensions are determined by multiplying the width of the culvert by a factor. For a 1:1 bevel, the factor is 0.5 inch/ft. For a 1.5:1 bevel the factor is 1 inch/ft. For example, the minimum bevel dimensions for an 8 ft x 6 ft box culvert with 1:1 bevels would be:

- Top Bevel = d = 6 ft x 0.5 inch/ft = 3 inches
- Side Bevel = b = 8 ft x 0.5 inch/ft = 4 inches

The improved inlet design figures are based on research results from culvert models with barrel width-to-depth ratios (W/D) of from 0.5:1 to 2:1. For box culverts with more than one barrel, the figures are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size.

Multibarrel pipe culverts shall be designed as a series of single barrel installations since each pipe requires a separate bevel.

5.3.5 Skewed Inlets

Ideally, culverts should be placed in the natural channel allowing for the best alignment with natural flow and minimum excavation and channel work. Where location in the natural channel would require an inordinately long culvert, some stream modification may be in order. Such modifications to reduce skew and shorten culverts shall be designed carefully to avoid erosion and sedimentation problems.

Culvert locations normal to the roadway centerline are not recommended where severe or abrupt changes in channel alignment are required upstream or downstream of the culvert. Short-radius bends are subject to erosion on the concave bank and deposition on the inside of the bend. Such changes upstream of the culvert result in poor alignment of the approach flow to the culvert; this subjects the embankment to erosion, and increases the likelihood of deposition in the culvert entrance, barrel, and exit.

Reinforced concrete box (RCB) culverts may be rotated or skewed to improve the alignment of the culvert with the stream. A rotated RCB is defined as an RCB that intersects the roadway centerline at an angle other than 90º and has entrance and exit faces normal to the RCB centerline. A skewed RCB is defined as an RCB that intersects the roadway centerline at an angle other than 90º and has entrance and exit faces parallel to the roadway centerline. Ordinary design practice allows normal (90º crossing) box culverts to be rotated up to 15º. Skewed boxes with variable angles may be used at RCB installations.

It is recommended that Chart 11 for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in underdesign due to a greatly reduced capacity. Inlets skewed at an angle with the centerline of the stream shall be avoided whenever possible and shall not be used with side- or slope-tapered inlets. It is important to align culverts with streams in order to avoid erosion problems associated with changing the direction of the natural stream flow.

Channel changes in Kansas streams are regulated by the Division of Water Resources (DWR) of the Kansas Department of Agriculture. The designer should inform the DWR of proposed channel changes as early as feasible, and promptly provide the design information needed for the permit application. The timely submittal of the permit application is necessary to avoid delays in the project.

5.3.6 Weep Holes

Weep holes are sometimes used to relieve uplift pressure on headwalls. Filter materials shall be used in conjunction with the weep holes to prevent the formation of internal soil erosion "piping" channels through the fill embankment. The filter materials shall be designed as a drain filter so as not to become clogged and so that piping cannot occur through the pervious material and the weep hole.

5.3.7 Safety Considerations

Roadside safety shall be considered for culverts crossing under roadways. Consult local, State and Federal road design standards for guidance.

5.3.8 Debris Control

In areas with expected high debris accumulations that could affect the hydraulic performance of the culvert, it is recommended that the FHWA HEC-9 entitled "Debris Control Structures: Evaluation and Countermeasures, Third Edition (FHWA, 2005) be consulted.

5.3.9 Comprehensive Design Guidance

The Federal Highway Administration, Hydraulic Design Series No. 5, Hydraulic Design of Highway Culverts, Second Edition (2005) is available from:

http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=7&id=13

5.3.10 Software for Culvert Analysis

The calculation of culvert hydraulics can be extremely time-consuming when performed by hand. Nomograph solutions are prone to error. However, the HY-8 program created by FHWA can be acquired at no cost to speed these calculations. HY-8 uses the procedures documented in HDS-5, and provides a good interface for analyzing options in culvert design. The program will output head-discharge rating curves for the culvert and associated road overflow. HY-8 can be downloaded free of charge from the following website:

http://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/

Following are a series of screen shots from HY-8 displaying the input screen (Figure 5-27), culvert profile output showing EGL and HGL (Figure 5-28) and rating curve (Figure 5-29).

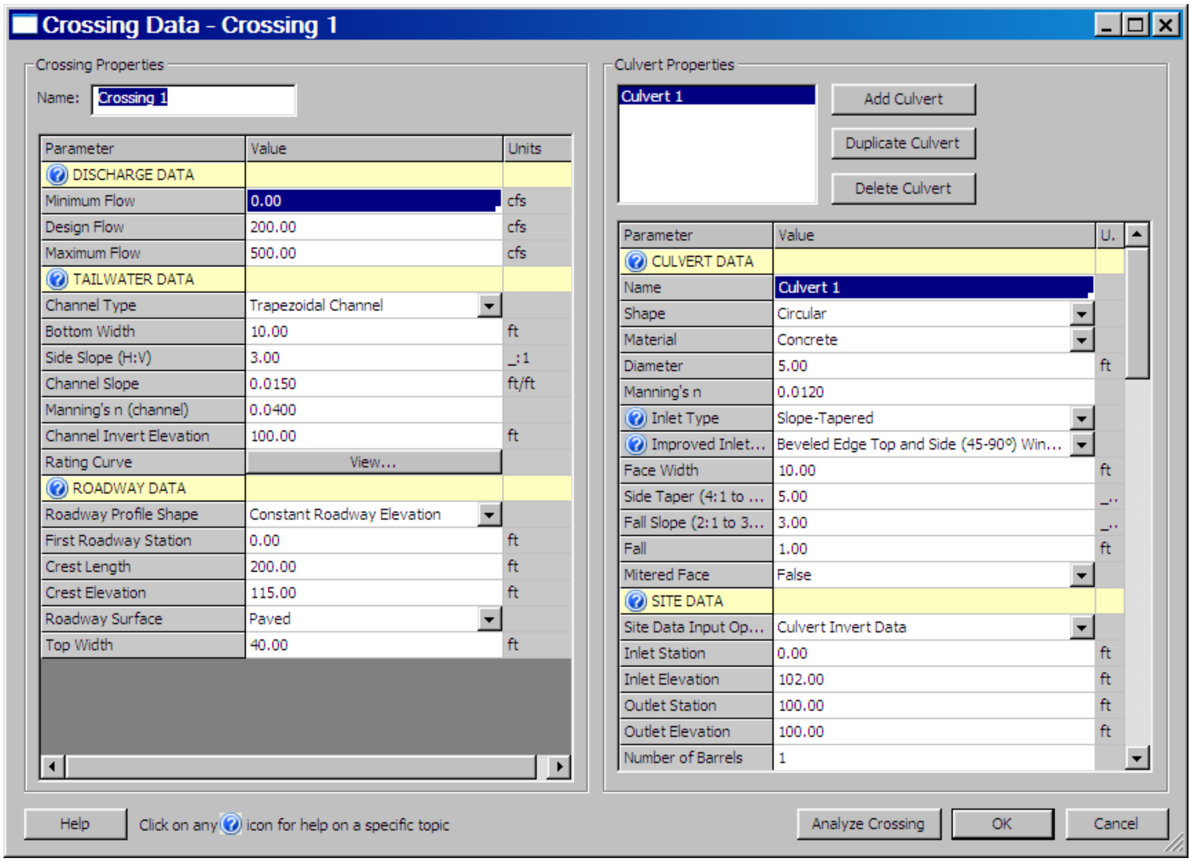


Figure 5-27 HY-8 Input Screen

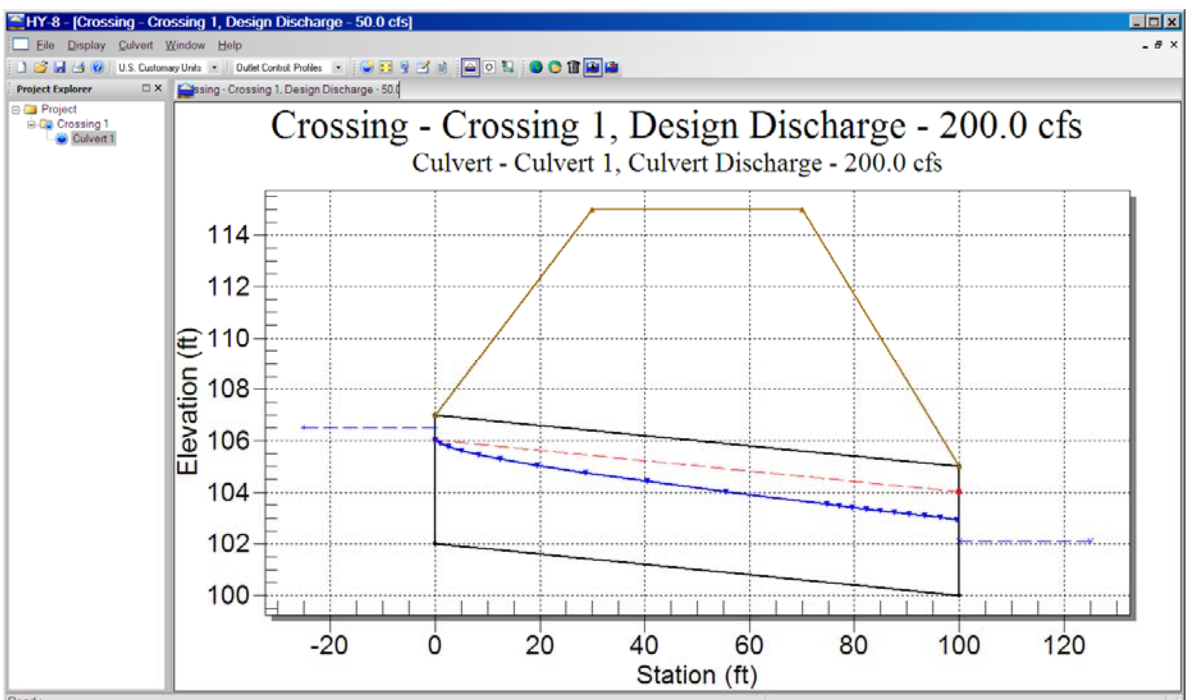


Figure 5-28 HY-8 Culvert Profile

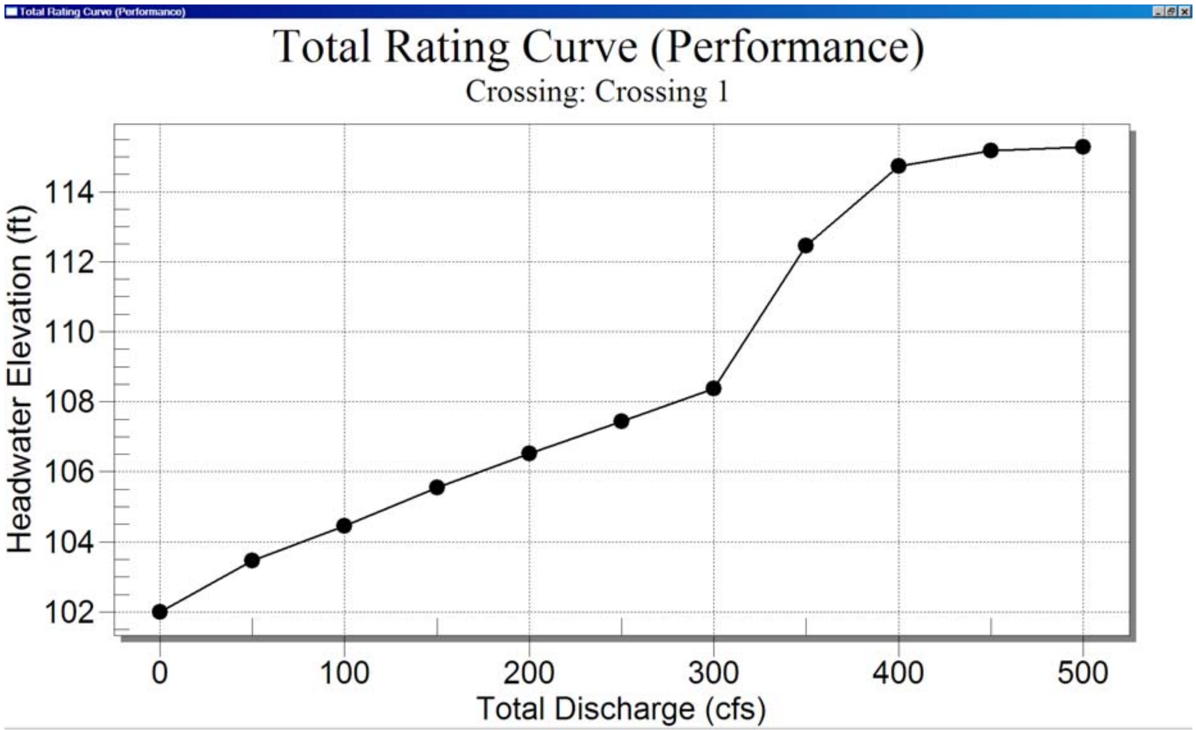


Figure 5-29 HY-8 Rating Curve

Proprietary software may also be used for culvert analysis. However, commercial third-party culvert software must be pre-approved by the City and County, not only on the basis of technical acceptance but also on the basis of the cost for the City and County to maintain a license of the product for review purposes.

5.4 Bridges

5.4.1 Introduction

This section provides an overview of the hydraulic design of bridges for new developments or redevelopments. For economy, engineers may design culverts to operate with the inlet submerged during flood flows, if conditions permit. Bridges, on the other hand, are not covered with embankment or designed to take advantage of submergence to increase hydraulic capacity, even though some are designed to be inundated under flood conditions. Bridges are usually selected instead of culverts if the discharge or the stream to be crossed is large. Both types of facilities (bridges and culverts) should be evaluated and a choice made based on performance and economics. If the stream crossing is wide with multiple concentrations of flow, a multiple opening facility may be in order.

5.4.2 General Criteria

The design of a bridge must take into account the many different engineering and technical aspects at the bridge site and adjacent areas. The following design criteria shall be followed for all bridge designs as applicable.

Hydraulic analysis of bridges shall be performed according to FEMA guidelines. All bridge design submittals to Valley Center and Sedgwick County shall be created and presented in a format that would be acceptable to FEMA, whether the anlayis will need to be submitted to FEMA or not. Hydraulic modelling shall be based on current effective FEMA models where applicable.

All bridges designs shall be reviewed and approved by the Kansas Division of Water Resources.

Bridges carrying roadways shall provide 1 foot of freeboard for the 100-year storm. Additionally, the 100-year frequency storm shall be analyzed through all bridges to ensure a minimum of 2 feet of freeboard for building structures (e.g., houses and commercial buildings). Pedestrian bridges and other low water crossings shall be permitted with DWR and may be exempt from freeboard requirements.

Table 5-10 gives values that shall be used for the contraction (K_c) and expansion (K_e) coefficients to be used for bridge hydraulic analysis (USACE HEC-RAS Manual, 2008).

Table 5-10 Loss Coefficients for Bridges

Transition Type	Contraction (K_c)	Expansion (K_e)
Gradual transition	0.1	0.3
Typical bridge	0.3	0.5
Severe transition	0.6	0.8

Scour analysis and protection shall be provided for all bridges according to guidance found in FHWA publications, HEC-18, HEC-20 and HEC-23.

5.4.3 Bridge Site Analysis

Crossing

The horizontal alignment of a highway at a stream crossing shall be taken into consideration when selecting the design and location of the waterway opening as well as the crossing profile. If practicable, the roadway shall be aligned so that the crossing will be normal to the stream flow direction (roadway centerline perpendicular to the streamline).

Often, this is not possible because of the highway or stream configuration. When a skewed structure is necessary, it shall be ensured that substructure fixtures such as foundations, columns, piers, and bent caps offer minimum resistance to the stream flow. Bent caps shall be oriented as near to the skew of the streamlines at flood stage as possible. Headers shall be skewed to minimize eddy-causing obstructions.

Factors Affecting Bridge Length

The discussions of bridge design herein assume normal cross sections and lengths (perpendicular to flow at flood stage). Usually one-dimensional flow is assumed, and cross sections and lengths are considered 90° to the direction of stream flow at flood stage. The following examples illustrate various factors that can cause a bridge opening to be larger than that required by hydraulic design.

- Bank protection may be placed in a certain location due to local soil instability or a high bank.
- Bridge costs may be less expensive than embankment costs.
- A roadway profile grade line might dictate an excessive freeboard allowance. For sloping abutments, a higher freeboard will result in a longer bridge.
- High potential for an upstream or downstream stream meander to migrate, or other channel instabilities may warrant a longer opening.

5.4.4 Bridge Hydraulic Analysis

As previously stated, all bridge projects for new developments or redevelopments must be designed according to FEMA hydraulic procedures. Hydraulic analysis will be performed using software such as HEC-2 or HEC-RAS. The hydraulic design must meet the requirements of the floodplain and/or stormwater management ordinances. The following text describes the hydraulics of flow through bridges, and presents a very general overview of the hydraulic design for bridges.

Flow Zones and Energy Losses

Flows through bridges must be modelled using FEMA standard methodologies with approved software such as HEC-RAS. Such analysis is required for all bridge design projects. Figure 5-30 shows a plan of typical cross section locations that establish three flow zones that must be considered when estimating the effects of bridge openings:

Zone 1 represents the area between the downstream face of the bridge and a cross section downstream of the bridge within which expansion of flow from the bridge is expected to occur. The distance over which this expansion occurs can vary depending on the flow rate and the floodplain characteristics. Generally, the expansion zone proceeds at a 2:1 to 3:1 flare away from the open area of the bridge. Section 1 represents the effective channel flow geometry at the end of the expansion zone, which is also called the “exit” cross section. Cross sections 2 and 3 are at the toe of roadway embankment and represent the portion of unconstricted channel geometry that approximates the effective flow areas near the bridge opening as shown in Figure 5-31.

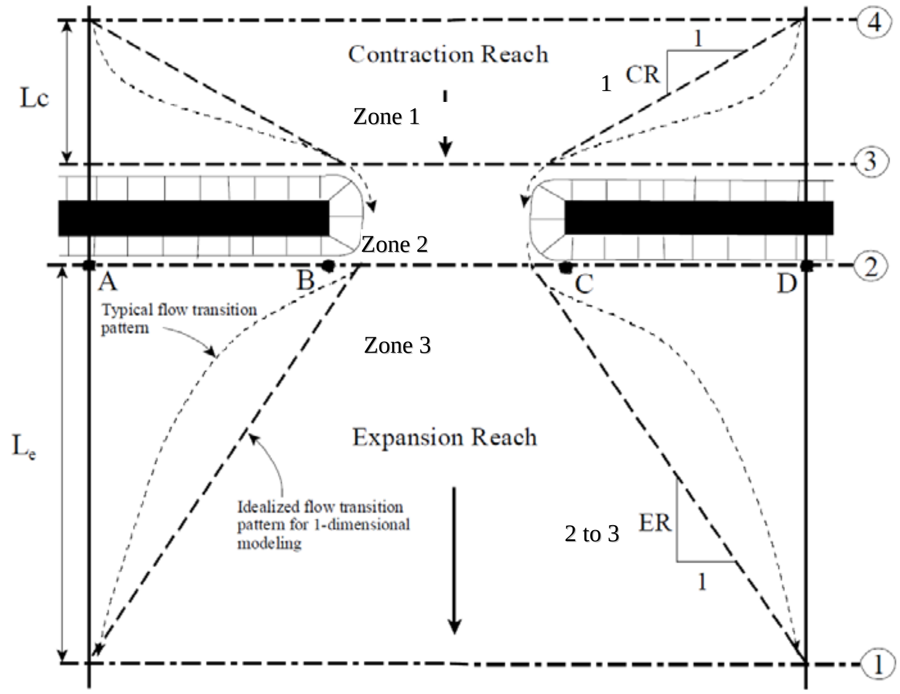


Figure 5-30 Flow Zones at Bridges
(USACE , HEC-RAS Hydraulic Reference Manual)

Zone 2 represents the area under the bridge opening through which friction, turbulence, and drag losses are considered. Generally for HEC-RAS analysis, the bridge opening is obtained by superimposing the bridge geometry on cross sections 2 and 3, unless channel conditions under the bridge differ significantly.

Zone 3 represents the area between the upstream face of the bridge and a cross section upstream where the contraction of flow starts. Contraction flare generally is modelled as proceeding away from the bridge opening at a 1:1 angle. Cross section 4 represents the effective channel flow geometry where contraction begins. This is sometimes referred to as the “approach” cross section.

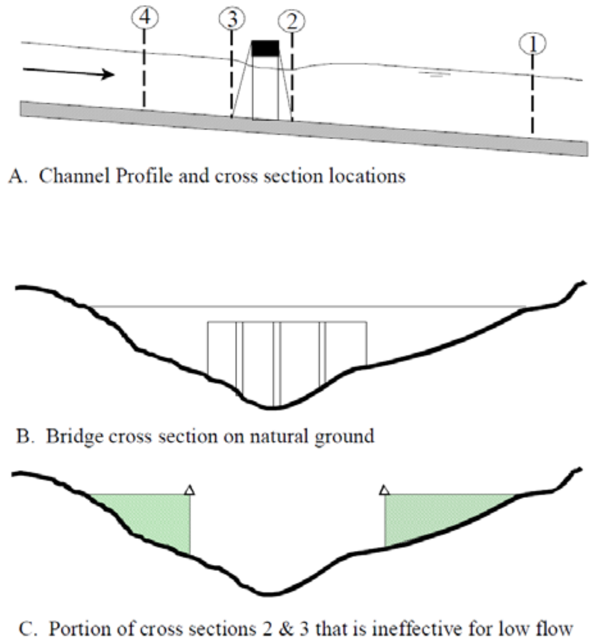


Figure 5-31 Effective Geometry for Bridge
(USACE , HEC-RAS Hydraulic Reference Manual)

Bridge Flow Class

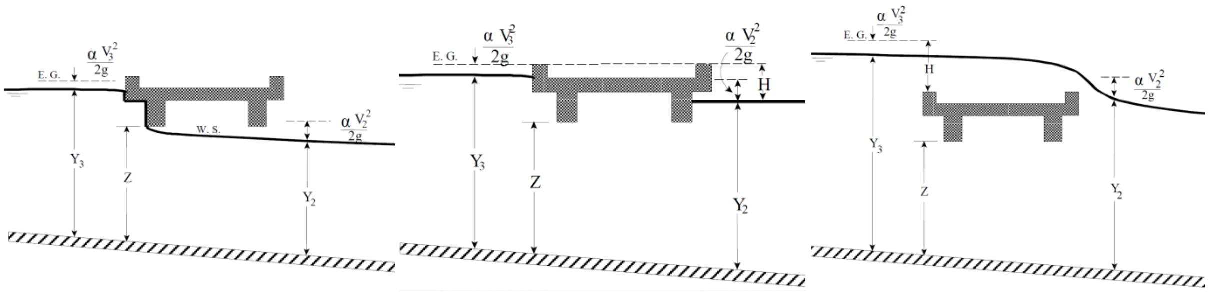
The losses associated with flow through bridges depend on the hydraulic conditions of low or high flow. Low flow describes hydraulic conditions in which the water surface between Zones 1, 2, and 3 is open to atmospheric pressure. That means the water surface does not impinge upon the superstructure. Low flow is divided into categories as described in Table 5-11. Type A is the most common in Kansas, although severe constrictions could result in Type B. Type C is usually limited to steep hills and mountainous regions not present in the region.

Table 5-11 Bridge Opening Low Flow Classes

Low Flow Class	Description
A	Subcritical flow through all Zones
B	Flow passes through critical depth within bridge opening
C	Supercritical flow through all Zones

High flow refers to conditions in which the water surface impinges on the bridge superstructure (Figure 5-32):

- When the water surface submerges the upstream low chord, but does not submerge the downstream low chord of the bridge, the flow condition is comparable to a pressure flow sluice gate.



Sluice Gate Flow **Orifice Flow** **Orifice and Weir Flow**

Figure 5-32 High Flow Classes
(USACE , HEC-RAS Hydraulic Reference Manual)

- When the water surface submerges the upstream and downstream low chords but does not exceed the elevation of critical depth over the road, the flow condition is comparable to orifice flow.
- If the water surface overtops the roadway, flow is usually orifice flow under the bridge and weir flow or open channel flow over the bridge (depending on tailwater conditions). However, under some conditions sluice gate flow and weir flow can occur.

The detailed analysis of bridge hydraulics is performed using computer software and is beyond the scope of this Manual. Additional information on bridge analysis techniques is provided in the HEC-RAS Hydraulic Reference Manual.

Scour

Water flowing through bridge openings has the tendency to create areas of scour due to the greater velocities at the bridge approach and exit, at the abutments and at the piers. These areas of scour can eventually undermine bridge stability if allowed to progress. Figure 5-33 provides an example of the effects of bridge scour. Figure 5-34 gives a generalized representation of the hydraulic forces that act to create scour around bridge piers.

Figure 5-33 Effects of Bridge Scour

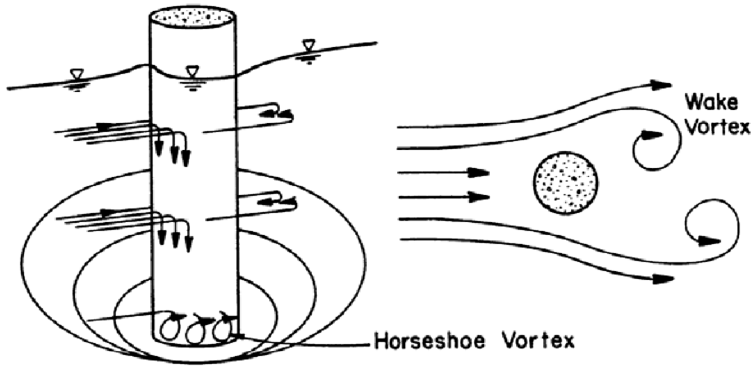


Figure 5-34 Creation of Pier Scour
(FHWA HEC-23)

Bridge design must account for and abate the effects of scour based on the guidance found in FHWA Hydraulic Engineering Circulars 18, 20, and 23. Scour calculations must include contraction scour, abutmet scour, and pier scour as outlined in the KDOT Design Manual: Volume 3, Bridge Section (2008). All bridge designs must ensure that the abutments and piers are not structurally compromised by the expected scour.

Scour calculations can be integrated into the same HEC-RAS model used for bridge hydraulic calculations. HEC-RAS implements many of the scour equations found in the FHWA publications. An example of scour shown in HEC-RAS is shown in Figure 5-35.

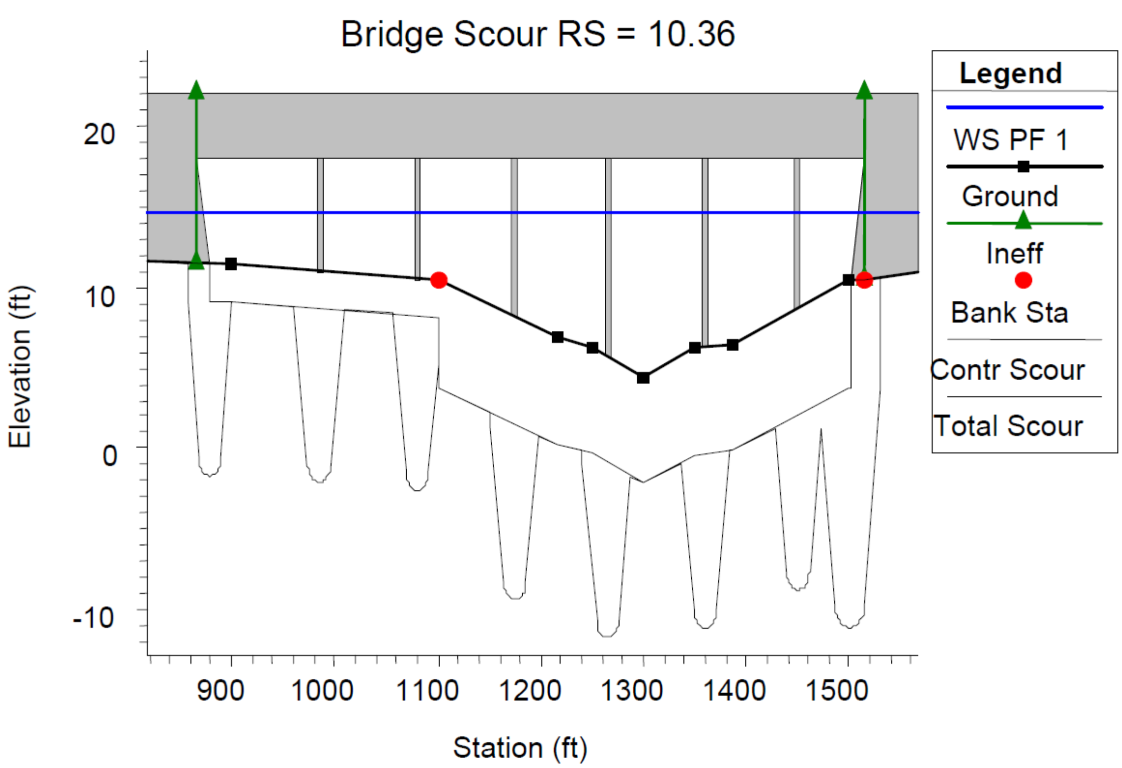


Figure 5-35 HEC-RAS Bridge Scour Example
(USACE , HEC-RAS Hydraulic Reference Manual)

5.4.5 Design Procedures

The following is a general bridge hydraulic design procedure that incorporates the topics discussed above.

Step 1 Determine the most efficient alignment of proposed roadway, attempting to minimize skew at the proposed stream crossing.

Step 2 Determine design discharge from hydrologic studies or available data (City, County, FEMA, USACE, KDOT, or similar sources).

Step 3 If available, obtain the effective FEMA hydraulic model. If an effective FEMA model or other model is not available, a water surface profile analysis for the stream must be prepared. The HEC-RAS computer model is routinely used to compute water surface profiles.

Step 4 Using FEMA guidelines, compute an existing conditions water surface profile for the appropriate storms as outlined in Section 5.4.2. Compute a profile for the fully-developed watershed to use as a baseline for design of a new bridge/roadway crossing.

Step 5 Use the design discharge to compute an approximate opening that will be needed to pass the design storm (for preliminary sizing, use a normal-depth design procedure, or simply estimate a required trapezoidal opening).

Step 6 Prepare a bridge crossing data set in the hydraulic model to reflect the preliminary design opening, which includes the required freeboard and any channelization upstream or downstream to transition the floodwaters through the proposed structure.

Step 7 Compute the proposed bridge flood profile and design parameters (velocities, flow distribution, energy grade, etc.). Review for criteria on velocities and freeboard, and revise model as needed to accommodate design flows.

Step 8 Review the velocities and model scour requirements downstream, through the structure, and upstream.

Step 9 Revise design as necessary to prevent structural impacts of scour per KDOT, KSDA and local standards.

Step 10 Finalize the design size and erosion control features to prevent impacts on other properties, meet all FEMA guidelines, and satisfy other local criteria.

Step 11 Exceptions/Other Issues

- Conditional Letter of Map Amendment (CLOMR) may be needed for new or modified crossings of streams studied by FEMA.

- If applicable, coordinate with USACE Regulatory Permit requirements.
- Coordinate with DWR for any necessary water structures permits.
- Design shall be for fully developed watershed conditions. If the available discharges are from FEMA existing conditions hydrology, new hydrology must be developed for built-out conditions. (Note that for formal no-rise, CLOMR and LOMR submittals to FEMA, the flows are usually for existing conditions. Therefore, analyses for both existing and fully developed conditions would be required in those cases.)
- Freeboard criteria may require an unusually expensive bridge or impracticable roadway elevation. A reasonable variance in criteria **may** be available from the Director.

5.4.6 Software for Bridge Analysis

The calculation of bridge hydraulics can be extremely time-consuming when performed by hand. The HEC-RAS program created by the Corps of Engineers can be acquired at no cost to speed these calculations, along with culvert analysis and many types of open channel flow calculations. HEC-RAS can be downloaded free of charge from the following website:

http://www.hec.usace.army.mil/software/hec-ras/hecras-download.html

5.5 Open Channels

5.5.1 Introduction

Open channel systems are an integral part of stormwater drainage designs, particularly for development sites utilizing preferred site design practices and open channel structural controls. Examples of open channels include drainage ditches, grassed channels and swales, enhanced swales, riprap channels and concrete-lined channels.

The purpose of this section is to provide an overview of open channel design criteria and methods, including the use of channel design nomographs. These procedures are intended for development-scale and site-scale channels used to convey local stormwater flows, where the channel is expected to flow under hydraulically “normal” flow conditions (EGL and channel bottom approximately parallel). For gradually varied flow conditions where the channel may flow with significant drawdown or backwater conditions, a water surface profile must be performed. Please note that the procedures provided herein are not intended for major flood control channels. The details of major flood control channel design are beyond the scope of this Manual.

The three main classifications of open channel types according to channel linings are vegetated, flexible, and rigid. Vegetated linings include grass, sod, and wetland channels. Stone riprap and some forms of flexible man-made linings or gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

Vegetative Linings: Vegetation, where practical, is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, provides habitat, and provides water quality benefits (see Section 3.2.4 and Section 3.2.5 for more details on using enhanced swales and grass channels for water quality purposes).

Conditions under which vegetation may not be acceptable include but are not limited to:

- High velocities;
- Standing or continuously flowing water;
- Lack of regular maintenance necessary to prevent growth of taller or woody vegetation;
- Lack of nutrients and inadequate topsoil; and
- Excessive shade.

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of healthy vegetation. If low flows are prevalent, a hard lined base flow channel may be needed. Channels shall be wide enough to accommodate maintenance equipment.

Flexible Linings: Rock riprap, including rubble and gabion baskets, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity and is commonly used in areas of high flow and steep grade. These linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance. However, they may require the use of a filter fabric depending on the underlying soils, and the growth of grass, weeds, and trees may present maintenance problems.

Rigid Linings: Rigid linings are generally constructed of concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel headcutting. This type of lining comes with a higher initial cost and can be degraded by environmental factors.

5.5.2 General Criteria

The following criteria shall be followed for open channel design:

- Where practicable, a primary and auxiliary channel shall be employed. The purpose of the primary channel is to carry the design flood discharge. The purpose of the auxiliary channel is to contain frequent flow, provide self-cleaning velocities, minimize the portion of channel bottom that is normally wet, and reduce bank erosion caused by the natural tendency of stream flow to meander from bank to bank.

- The primary channel shall provide a minimum of one foot of freeboard for the design storm.
- The primary channel and overbanks shall convey the 100-year storm flow such that structures are protected with a minimum 2 feet of freeboard.
- The auxiliary channel shall be sized to convey the 1-year storm with a velocity of at least 2.5 ft/s, where possible.
- The auxiliary channel shall be centered in the primary channel wherever sufficient bottom width is available, or shifted to one side where the auxiliary channel slope is continuous with the primary channel slope; the minimum cross-slope on the bench shall be 2%.
- Where the primary channel width will not accommodate an auxiliary channel, the primary channel shall feature a "V" bottom using 2% cross slope.

- Channel side slopes shall be no steeper than 4:1.
- Hydraulic analysis utilizing Manning's equation shall be used to calculate water surface profiles in open channels and roadside ditches. Manning's roughness values shall be taken from Appendix A based on the expected depth of flow.
- Velocities or shear stresses may not be increased in existing channels.
- Channel erosion shall be prevented by ensuring that shear stresses or velocities are less than the permissible values for the 2, 5 and 10-year storms, plus the design storm.
- Additional channel erosion protection shall be provided at bends where shear stresses would exceed the permissible shear shown for the 2, 5 and 10-year storms, plus the design storm.
- If relocation of a stable stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions insofar as practicable. Energy dissipation may be necessary when existing conditions cannot be duplicated.

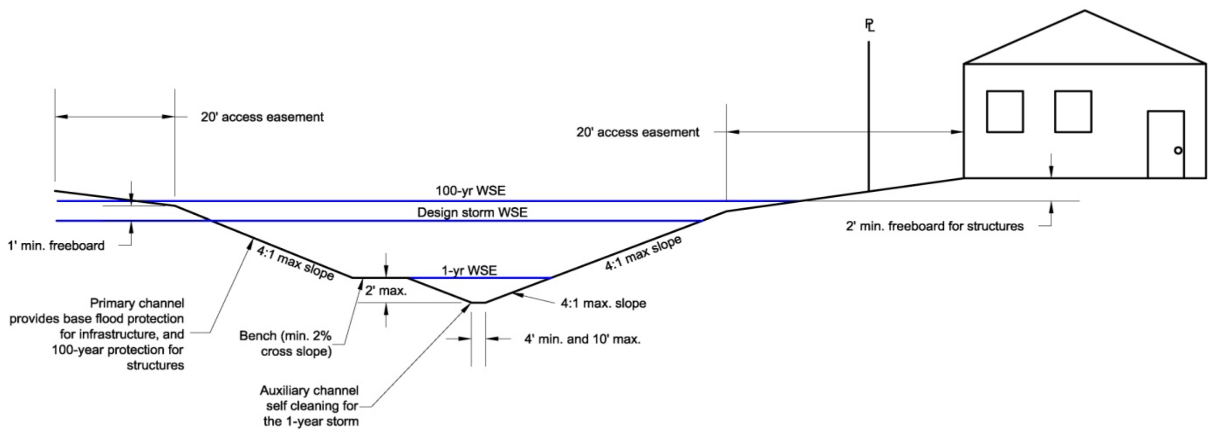


Figure 5-36 Channel Geometry Criteria

5.5.3 Channel Capacity

5.5.3.1 Design Charts

The following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Administration has prepared numerous design charts to aid in the design of rectangular, trapezoidal, and triangular open channel cross sections. In addition, design charts for grass-lined channels have been developed. Examples of these charts and instructions for their use are given in Section 5.5.3.3.

5.5.3.2 Manning's Equation

Manning's Equation, presented in three forms below, shall be used for evaluating uniform flow conditions in open channels (HEC-15, 2005):

Equation 5-22 $$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

Equation 5-23 $$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

Equation 5-24 $$S = \left[\frac{Qn}{1.49 A R^{2/3}} \right]^2$$

where:

V	=	average channel velocity, ft/s
Q	=	discharge rate for design conditions, cfs
n	=	Manning's roughness coefficient
A	=	cross-sectional area, ft^2
R	=	hydraulic radius = A/P, ft
P	=	wetted perimeter, ft
S	=	slope of the energy grade line, ft/ft

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are assumed to be equal. For a more comprehensive discussion of open channel theory and design, see Hydraulic Design of Flood Control Channels, USACE, 1994.

The wetted perimeter (P) and hydraulic radius (R) can be calculated from geometric dimensions of standard trapezoidal channel sections. Irregular channel cross sections, however, must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coefficients. Hydraulic analysis programs such as HEC-RAS can greatly speed such calculations.

5.5.3.3 Direct Solutions

When the hydraulic radius, cross-sectional area, roughness coefficient and slope are known, discharge can be calculated directly from Equation 5-23. Nomographs for obtaining direct solutions to Manning's Equation are presented in Figure 5-37 and Figure 5-38.

The following steps are used for the general solution nomograph in Figure 5-37:

Step 1 Determine open channel geometry, including slope in ft/ft, hydraulic radius in ft, and Manning's n value.

Step 2 Connect a line between the Manning's n scale and slope scale and note the point of intersection on the turning line.

Step 3 Connect a line from the hydraulic radius to the point of intersection obtained in Step 2.

Step 4 Extend the line from Step 3 to the velocity scale to obtain the velocity in ft/s.

The trapezoidal channel nomograph solution to Manning's Equation in Figure 5-38 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known, according to the following steps.

Given Q, find d:

Step 1 Determine input data, including slope in ft/ft, Manning's n value, bottom width in ft, and side slope in ft/ft.

Step 2 Given the design discharge, find the product of Q times n, connect a line from the slope scale to the Qn scale, and find the point of intersection on the turning line.

Step 3 Connect a line from the turning point to the b scale and find the intersection with the z = 0 scale.

Step 4 Project horizontally from the point located in Step 3 to the appropriate z value and find the value of d/b.

Step 5 Multiply the value of d/b obtained in Step 4 by the bottom width b to find the depth of uniform flow, d.

Given d, find Q:

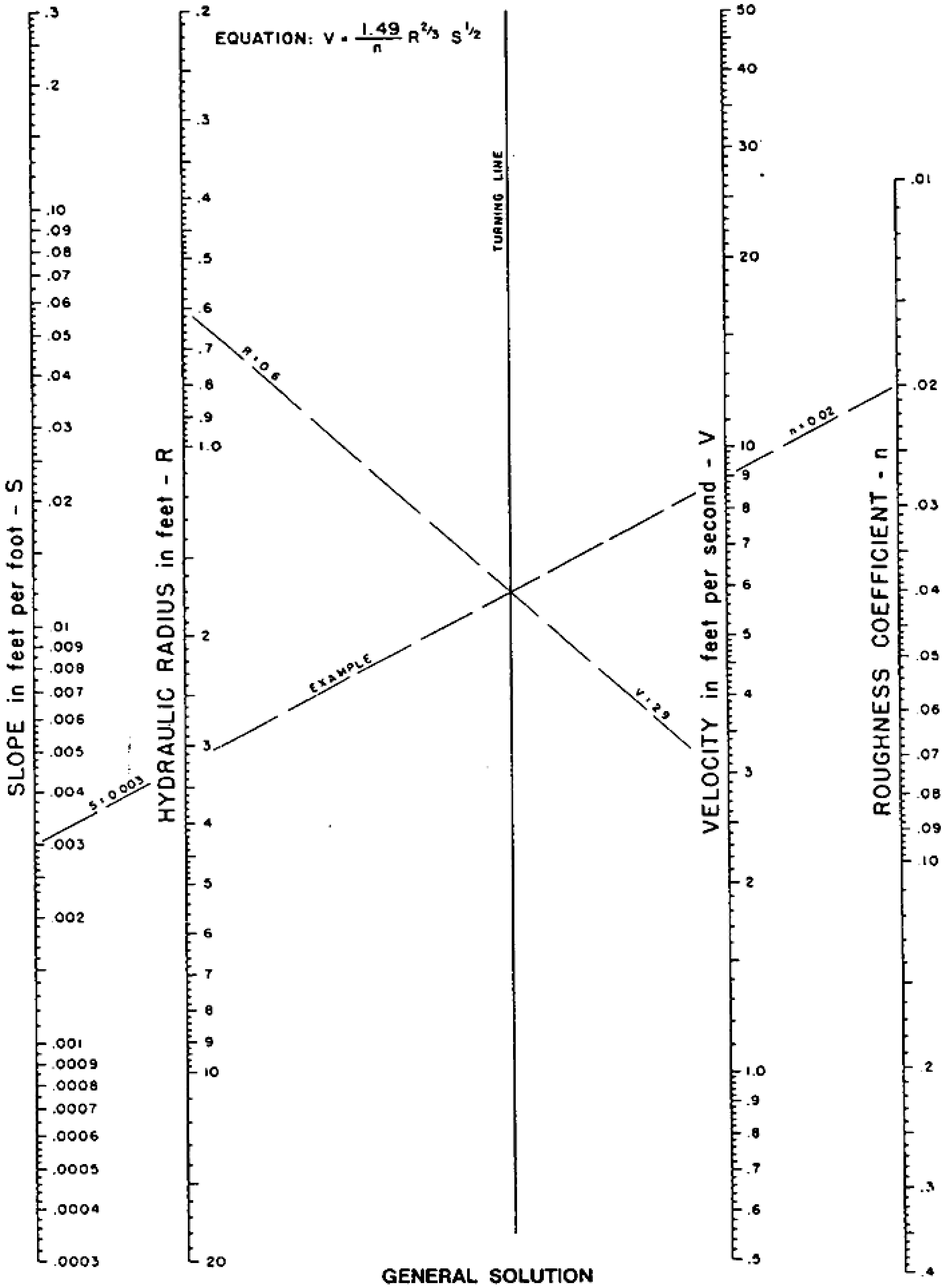
Step 1 Determine input data, including slope in ft/ft, Manning's n value, bottom width in ft, and side slope in ft/ft.

Step 2 Given the depth of flow, find the ratio d divided by b and project a horizontal line from the d/b ratio at the appropriate side slope, z, to the z = 0 scale.

Step 3 Connect a line from the point located in Step 2 to the b scale and find the intersection with the turning line.

Step 4 Connect a line from the point located in Step 3 to the slope scale and find the intersection with the Qn scale.

Step 5 Divide the value of Qn obtained in Step 4 by the n value to find the design discharge, Q.



Reference: USDOT, FHWA, HDS-3 (1961).

Figure 5-37 Nomograph for the Solution of Manning's Equation

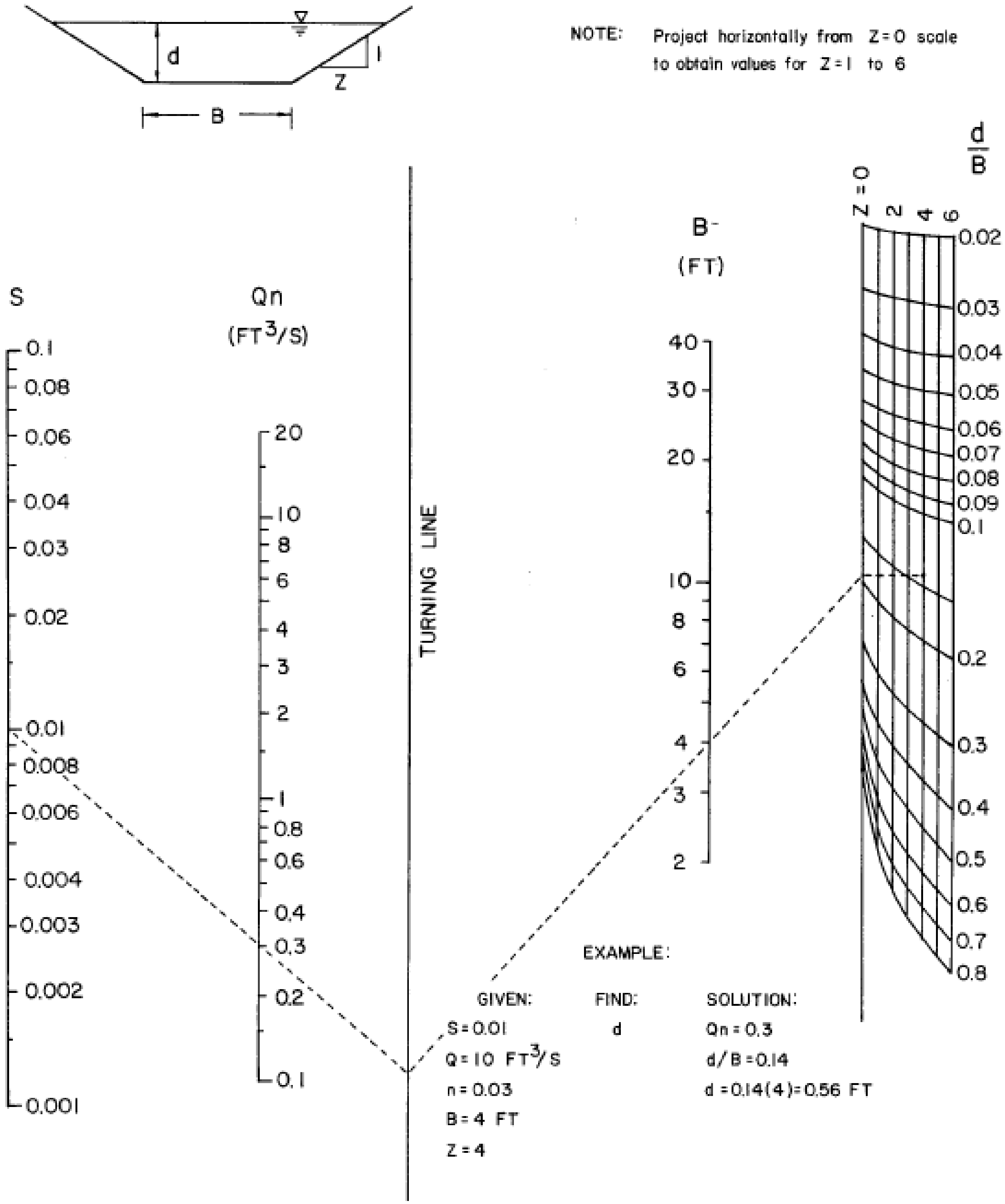


Figure 5-38 Solution of Manning's Equation for Trapezoidal Channels
(HEC-22, 2001)

5.5.3.4 Trial and Error Solutions

A trial and error procedure for solving Manning's equation can be used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process, Manning's equation can be arranged as shown in Equation 5-25 (HEC-15, 2005).

Equation 5-25 $$AR^{2/3} = \frac{Qn}{1.49S^{1/2}}$$

where:

A	=	cross-sectional area, ft
R	=	hydraulic radius = A/P, ft
Q	=	discharge rate for design conditions, cfs
n	=	Manning's roughness coefficient
S	=	slope of the energy grade line, ft/ft

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of Equation 5-25 is satisfied such that the design flow is conveyed for the slope and selected channel cross section. This trial and error calculation can be easily solved using a simple spreadsheet.

5.5.4 Channel Stability

For the design of grassed channels, stability against erosion will be evaluated based on flow shear stress and flow velocity. Shear stress will be used as the primary indicator of stability against erosion; however, in no case may the average flow velocity exceed the maximum permissible velocity.

For the analysis of existing unlined channels, and for riprapped channels, stability against erosion will be based on permissible velocities.

The maximum shear stress in a channel may be approximated by Equation 5-26 (HEC-15, 2005).

Equation 5-26 $$\tau_d = \gamma * d * S$$

where:

τ_d	=	maximum shear stress, lb/ft^2
γ	=	specific weight of water = 62.4 lb/ft^3
d	=	maximum depth, ft
S	=	slope of EGL, ft/ft

The permissible shear stress for a grass channel is the maximum shear stress that the channel can sustain without erosion. Table 5-12 provides permissible shear stresses for the design of grassed channels and unlined channels. The permissible shear stress for a grass lining depends on the hydraulic resistance of the lining, as measured by Manning's n, and the erodibility of the underlying soil.

Table 5-12 Permissible Shear Stresses and Velocities

Lining Category	Lining Type	Permissible Shear Stress, τ_p (lb/ft^2)[1]	Permissible Velocity (ft/s)[2]
Grass (good stand)	Grass with easily erodible soil (sands and silts)	$\tau_p = 310\ n^2$	5
	Grass with average soil erodibility (silty clays)	$\tau_p = 1250\ n^2$	7
	Grass with erosion-resistant soil (clays)	$\tau_p = 2190\ n^2$	7.5
Unlined	Fine sand	---	2
	Coarse sand	---	4
	Sandy silt	---	2
	Silt clay	---	3.5
	Clay	---	6

where: n = Manning's roughness
[1] KDOT Drainage Design Manual 2008-07
[2] EM 1110-2 1601, USACE

The equation for permissible velocity in riprapped channels is provided in Equation 5-27 and Equation 5-28 (HDC 712-1, USACE).

Equation 5-27 $$V = C\left[2g\left(\frac{\gamma_S - \gamma_W}{\gamma_W}\right)\right]^{1/2}\left(D_{50}\right)^{1/2}$$

Equation 5-28 $V = 12.3\left(D_{50}\right)^{1/2}$ (for typical conditions as defined below)

where:

V	=	velocity, ft/s
C	=	Isbash constant = 1.2
g	=	gravity = 32.2 ft/s^2
γ_s	=	unit weight of riprap, may assume 165 lb/ft^3
γ_w	=	unit weight of water = 62.4 lb/ft^3
D_{50}	=	median equivalent spherical diameter of riprap, ft

5.5.5 Channel Lining Design

Procedure

This procedure is intended for channels used to convey local storm flows, where the channel is expected to flow under hydraulically "normal" flow conditions. For gradually varied flow conditions where the channel may flow with significant drawdown or backwater conditions, a water surface profile must be performed and the velocities and shear stresses resulting from that analysis are to be used. Also, as previously stated, this procedure is not for major flood control channels. The details of major flood control channel design are beyond the scope of this Manual.

Step 1 Determine or select a channel cross-section (bottom width, side slopes, and depth) and bottom slope that are compatible with the overall design requirements.

Step 2 Determine the design storm recurrence interval that governs the channel capacity.

Step 3 Compute the design flow for the channel capacity according to the hydrologic procedures presented in Chapter 4.

Step 4 Compute the normal depth for the design flow. If the normal depth is at least one foot below the bank-full depth, the channel has sufficient capacity for the design flow. Otherwise, increase the bottom width and/or the depth of the channel and repeat the capacity check.

Step 5 Check the capacity of the channel to prevent flooding of nearby structures as follows: Compute the discharge for the 100-year recurrence interval by hydrologic analysis. Next, compute the normal depth at this discharge and the corresponding water-surface elevation. (This may include out-of-banks flow.) If this elevation is equal to or less than two feet below the lowest opening of the structure, the channel has sufficient capacity. Otherwise, increase the bottom width and/or the depth of the channel and repeat this check.

Step 6 Check the stability of the channel. Compute the normal depth for the 2, 5 and 10-year storms. Compute shear stresses (for grassed channels) and average velocities for each event, and compare with the permissible values. Adjust the design to ensure permissible values are not exceeded.

Step 7 Check for self-cleaning velocities. Compute the normal depth for the 1-year storm, and the corresponding average flow velocity. If the velocity is less than 2.5 fps, adjust the design.

Step 8 Repeat steps 1 through 7 until all requirements are met.

5.5.6 Flow in Bends

Flow around a bend creates secondary currents, which impose higher shear stresses on the channel sides and bottom compared to a straight reach as shown in Figure 5-39. At the beginning of the bend, the maximum shear stress is near the inside and moves toward the outside as the flow leaves the bend. The increased shear stress caused by a bend persists downstream of the bend.

The maximum shear stress in a bend is a function of the ratio of channel curvature to the top (water surface) width, R_C/T. As R_C/T decreases, that is as the bend becomes sharper, the maximum shear stress in the bend tends to increase.

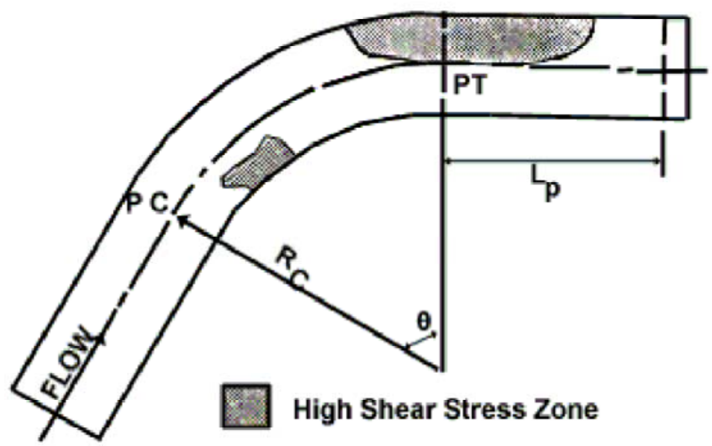


Figure 5-39 Flow in Channel Bends
(FHWA, HEC-15)

Equation 5-29 gives the maximum shear stress in a bend (HEC-15, 2005).

Equation 5-29 $\tau_b = K_b \tau_d$

where,

τ_b	=	side shear stress on the channel, lb/ft^2
K_b	=	ratio of channel curvature to bottom shear stress
τ_d	=	shear stress in channel at maximum depth, lb/ft^2

K_b can be determined from the following equations, depending on the ratio of channel curvature per (HEC-15, 2005).

Table 5-13 Ratio of Channel Bend to Bottom Shear Stress, K_b

K_b	Channel Curvature Constraint
2.00	$R_C/T \le 2$
Equation 5-30 $2.38 - 0.206\left(\frac{R_c}{T}\right) + 0.0073\left(\frac{R_c}{T}\right)^2$	$2 < R_C/T < 10$
1.05	$10 \le R_C/T$

The length downstream to the point that curvature-induced stresses dissipate (HEC-15, 2005) is given by Equation 5-31.

Equation 5-31 $$L_p = 0.6\left(\frac{R^{7/6}}{n}\right)$$

where,

L_p	=	length of protection, ft
R	=	hydraulic radius of the channel, ft
n	=	Manning's roughness for lining material in the bend

Example Problem – Channel Design

Design a grass channel in silty clay soil to convey flow past a new shopping mall. The hydrologic calculations have shown that the channel will carry 10 cfs for the 1-year storm and 100 cfs for the 10-year storm. The channel slope is 1.3%. Design a channel to meet the local criteria.

Determine the design storm.

The design storm is the 10-year, since the design is for a high value business area per Table 5-1.

Determine Manning's roughness for the channel.

Look in "artificial channels" portion Appendix A to find a roughness value of 0.045 for flow in grassed swales with depth between 0.5 and 2.0 feet.

Estimate the size of an auxiliary channel to convey the 1-year storm using Equation 5-25.

Set auxiliary channel bottom width to the minimum of 4'. Set side slope to the maximum steepness of 4:1. Using the channel slope and roughness value, iterate Equation 5-25 until a flow depth is found for the auxiliary channel. The iteration process seeks to balance the depth-variable terms left of the equality sign and the depth-independent terms to the right. The correct flow depth is found once they balance. This process is shown in the upper portion of the example spreadsheet within Figure 5-40.

Auxiliary channel depth is found to be 0.66 ft.

Estimate the size of a primary channel to carry the design storm.

Estimate a primary channel bottom width of 14'. Choose a side slope of 4:1. Once again, iterate through Equation 5-25 until a depth of flow for the compound channel is found in the lower portion of Figure 5-40. The overall flow area, wetted perimeter, and hydraulic radius must be calculated to include both the auxiliary and primary channels for this step.

The primary channel depth is found to be 1.12 ft. This provides a total flow depth of 0.66 + 1.12 = 1.78 ft. A total channel depth of 2.8 feet will provide the minimum freeboard of 1' for the design storm.

Determine the actual channel shear for the 10-year storm using Equation 5-26:

$$\tau_d = \gamma d S$$

$$\tau_d = 62.4(1.78)0.013 = 1.44 \text{ lb/ft}^2$$

Determine the permissible channel shear per Table 5-12.

$$\tau_p = 1250n^2$$

$$\tau_p = 1250(0.045)^2 = 2.53 \text{ lb/ft}^2$$

Determine the side shear stress on the channel assume an R_c of 20 ft and a T of 6 ft.

To determine K_b use Table 5-13. Since Rc/T is between 10 and 2 Equation 5-30 is used to calculate K_b.

$$K_b = 2.38 - 0.206\left(\frac{20}{6}\right) + 0.0073\left(\frac{20}{6}\right)^2 = 1.77$$

The side shear stress can be calculated using Equation 5-29:

$$\tau_b = K_b \tau_d = (1.77)(1.44) = 2.56 \text{ ft/lb}^2$$

Added lining may be needed to resist the bend stresses. Determine the distance stream of the bend that the added shear stress will dissipated.

To determine the length of protection needed first find the hydraulic radius of the entire channel. The hydraulic radius is found by using the calculations in Equation 5-40.

$$R = \frac{(A)}{(WP)} = \frac{(25.22)}{(23.44)} = 1.08$$

The protection length can be found by using Equation 5-31.

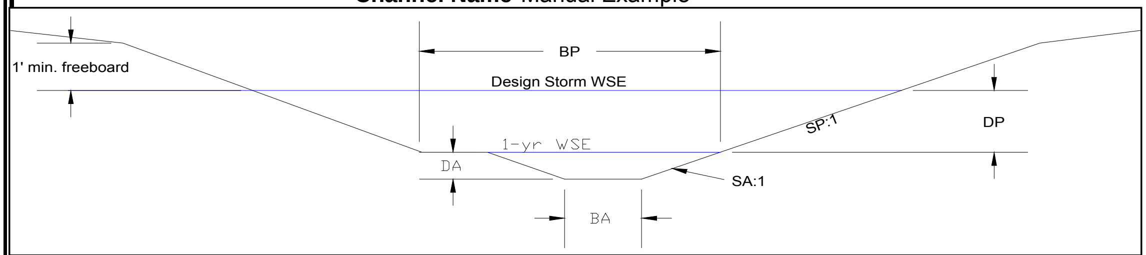
$$L_p = 0.6\left(\frac{R^{7/6}}{n}\right) = 0.6\frac{(1.08)^{7/6}}{(0.045)} = 14.58 \text{ ft}$$

The maximum permissible velocity in Table 5-12 is 7 fps. The average velocity for the total channel is 5.4 fps.

Check for self-cleaning velocity. The velocity in the auxiliary channel for the 1-year flow is 2.3 cfs, which is slightly less than 2.5 fps, suggesting a small adjustment to the auxiliary channel geometry or channel slope is needed.

Compound Channel Sizing Spreadsheet

Channel Name Manual Example



	Channel Slope (ft/ft)	0.013

$$AR^{2/3} = \frac{Qn}{1.49S^{1/2}}$$

Auxiliary Channel Sizing for the 1-year Storm using Manning's Equation

Match these per equation above

Auxiliary Channel Data		
BA	Aux. B (ft)	4
SA	Aux. Side Slope:1	4
	Aux. Q (cfs)	10
	Auxiliary n	0.045

Aux. Qn/(1.49S$^{1/2}$)	Aux. Depth	Aux. A	Aux. WP	Aux. R	Aux. AR$^{2/3}$
2.65	0	0.00	4.00	0.00	0.00
2.65	0.2	0.96	5.65	0.17	0.29
2.65	0.5	3.00	8.12	0.37	1.54
2.65	1	8.00	12.25	0.65	6.02
2.65	0.7	4.76	9.77	0.49	2.95
2.65	0.6	3.84	8.95	0.43	2.18
2.65	0.65	4.29	9.36	0.46	2.55
2.65	0.67	4.48	9.52	0.47	2.71
2.65	0.66	4.38	9.44	0.46	2.63
2.65	0.665	4.43	9.48	0.47	2.67
2.65	0.663	4.41	9.47	0.47	2.65
2.65		0.00	4.00	0.00	0.00
2.65		0.00	4.00	0.00	0.00
Chosen Aux. Depth	0.663	4.41	9.47	0.47	2.65
Aux. Velocity (ft/s)	2.27				
Aux. Shear (lb/ft^2)	0.54				

Primary Channel Sizing for the Design Storm using Manning's Equation

Match these per equation above

Primary Channel Data		
BP	Prim. B (ft)	14
SP	Prim. Side Slope: 1	4
	Total. Q (cfs)	100
	Primary n	0.045

Total Qn/(1.49S$^{1/2}$)	Prim. Depth	Prim. A	Prim. WP	Total A	Total WP	Total R	Total AR$^{2/3}$
26.49	0	0.00	4.70	4.41	14.16	0.31	2.03
26.49	2	44.00	21.19	48.41	30.66	1.58	65.65
26.49	1	18.00	12.94	22.41	22.41	1.00	22.41
26.49	1.2	22.56	14.59	26.97	24.06	1.12	29.10
26.49	1.1	20.24	13.77	24.65	23.23	1.06	25.64
26.49	1.15	21.39	14.18	25.80	23.65	1.09	27.34
26.49	1.12	20.70	13.93	25.11	23.40	1.07	26.32
26.49	1.13	20.93	14.01	25.34	23.48	1.08	26.66
26.49	1.125	20.81	13.97	25.22	23.44	1.08	26.49
26.49		0.00	4.70	4.41	14.16	0.31	2.03
26.49		0.00	4.70	4.41	14.16	0.31	2.03
Chosen Prim. Depth	1.125	20.81	13.97	25.22	23.44	1.08	26.49
Total Depth	1.788						
Prim. Velocity (ft/s)	3.96						
Prim. Shear (lb/ft^2)	1.45						

Note: Total wetted perimeter and area includes that from the auxiliary channel.
Note: Total flow includes that from the auxiliary channel

Figure 5-40 Channel Sizing Example Spreadsheet

Flow around bends also creates localized increases and decreases in flow depth due to centrifugal force. For subcritical flow, the water surface elevation on the outside of the curve will increase and the depth on the inside of the curve will decrease an equal amount (approximately). The total difference in water surface elevation between the inside and outside of the curve, called "superelevation, may be estimated with Equation 5-32 (HEC-22, 2001).

Equation 5-32 $$d = \frac{v^2 * T}{g * R_c}$$

where:

d	=	difference in water surface elevation across top of channel, feet
v	=	average flow velocity, fps
T	=	water surface top width, feet
g	=	gravitational acceleration, 32.2 ft/sec^2
R_c	=	radius of center of channel, feet

Thus, the change in water surface elevation when compared with the nominally computed elevation is plus d/2 on the outside of the curve, and minus d/2 on the inside.

For supercritical flow, the elevation changes are more complex. In addition to superelevation, "standing waves" will also develop. The analysis of standing waves is beyond the scope of this Manual. However, the increase (at the outside of the curve) and decrease (at the inside of the curve) is approximately two times the values that would be computed for subcritical flow (EM-1110-2-1601, USACE, 1994).

5.5.7 Critical Flow Calculations

Background

In the design of open channels, it is important to calculate the critical depth in order to determine if the flow in the channel will be subcritical or supercritical. If the flow is subcritical, the flows through channel transitions are relatively tranquil and wave action is minimal. In subcritical flow, the depth at any point is influenced by a downstream control, which may be either the critical depth at a control structure (such as a weir) or the water surface elevation in a pond or downstream channel. In supercritical flow, the depth at any point is influenced by a control upstream, usually critical depth. Supercritical flows have relatively shallow depths and high velocities. Standing waves may develop and hydraulic jumps are possible under these conditions. Channel stabilization may require extensive measures.

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth (King & Brater, 1963) is expressed in Equation 5-33.

Equation 5-33 $$\frac{Q^2}{g} = \frac{A^3}{T}$$

where:

Q	=	discharge rate for design conditions, cfs
g	=	acceleration due to gravity = 32.2 ft/s^2
A	=	cross-sectional area, ft^2
T	=	top width of water surface, ft

Note: A trial and error procedure is needed to solve Equation 5-33.

Semi-Empirical Equations

Semi-empirical equations (as presented in Table 5-14) or section factors (as presented in Figure 5-41) can be used to simplify trial and error critical depth calculations. The following equation is used to determine critical depth with the critical flow section factor, Z (King & Brater, 1963):

Equation 5-34 $$Z = \frac{Q}{g^{0.5}}$$

where:

Z	=	critical flow section factor (not to be confused with Z used to denote side slope in some figures, such as Figure 5-41
Q	=	discharge rate for design conditions, cfs
g	=	acceleration due to gravity = 32.2 ft/s^2

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel (King & Brater, 1963):

Equation 5-35 $$Fr = \frac{v}{\sqrt{gd}}$$

where:

Fr	=	Froude number
v	=	velocity of flow, ft/s
g	=	acceleration of gravity = 32.2 ft/sec^2
d	=	hydraulic depth = A/T, ft
A	=	cross-sectional area of flow, ft^2
T	=	top width of flow, ft

If Fr is greater than 1.0, flow is supercritical; if it is less than 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

Example Problem – Flow Calculations

Find the critical depth and Froude number at the 1-yr water surface elevation for the previous example. Note to calculate the critical depth for the design storm requires computing the depth in a compound channel which beyond the scope of this manual.

Table 5-14 is used to calculate the critical depth.

$$0.5522Q/b^{2.5} = 0.5522(10/4^{2.5}) = 0.17$$

This is within the range of applicability for estimated the depth in a trapezoidal channel. From Table 5-14 the critical depth can be calculated.

$$d_c = 0.81[10^2/((32.2)(4^{0.75}4^{1.25}))]^{0.27} - 4/((30)(4)) = 0.49 \text{ ft}$$

To determine the Froude number Equation 5-35 is used.

$$Fr = \frac{v}{\sqrt{gd}} = \frac{2.27}{\sqrt{(32.2)(4.41/9.30)}} = 0.58$$

The flow is subcritical because Fr < 1.

Table 5-14 Critical Depth Equations for Uniform Flow in Selected Channel Cross Sections

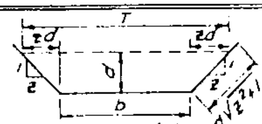
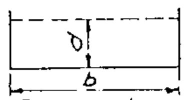
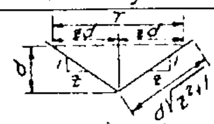
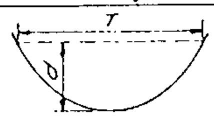
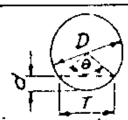
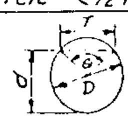
Channel Type[1]	Semi-Empirical Equations[2] for Estimating Critical Depth	Range of Applicability
1. Rectangular[3]	$d_c = [Q^2/(gb^2)]^{1/3}$	All rectangular sections
2. Trapezoidal[3]	$d_c = 0.81[Q^2/(gz^{0.75}b^{1.25})]^{0.27} - b/30z$	$0.1 < 0.5522\ Q/b^{2.5} < 0.4$ For $0.5522\ Q/b^{2.5} < 0.1$, use rectangular channel equation
3. Triangular[3]	$d_c = [(2Q^2)/(gz^2)]^{1/5}$	All triangular sections
4. Circular[4]	$d_c = 0.325(Q/D)^{2/3} + 0.083D$	$0.3 < d_c/D < 0.9$
5. General[5]	$(A^3/T) = (Q^2/g)$	All sections

where:

d_c	=	critical depth (ft)
Q	=	design discharge (cfs)
g	=	acceleration due to gravity (32.3 ft/s^2)
b	=	bottom width of channel (ft)
z	=	side slopes of a channel (horizontal to vertical)
D	=	diameter of circular conduit (ft)
A	=	cross-sectional area of flow (ft^2)
T	=	top width of water surface (ft)

[1] See Figure 5-41 for channel sketches
[2] Assumes uniform flow with the kinetic energy coefficient equal to 1.0
[3] Reference: French (1985)
[4] Reference: USDOT, FHWA, HDS-4 (1965)
[5] Reference: Brater and King (1976)

If the water surface profile in a channel transitions from supercritical flow to subcritical flow, a hydraulic jump must occur. The location and length of the hydraulic jump and its sequent (downstream) depth are critical to proper design of free flow conveyances. To determine the location of a hydraulic jump, the standard step method is used to compute the water surface profiles, and specific force (momentum principle) and specific energy relationships are used. For computational methods, including length and sequent depth, refer to Chow, 1959, and Mays, 1999. The HEC-RAS computer program can be used to compute water surface profiles for both subcritical and supercritical flow regimes.

Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width T	Critical Depth Factor, Z
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$	$\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$
 Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2}\,zd^{2.5}$
 Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$	$\frac{2}{9}\sqrt{6}\,Td^{1.5}$
 Circle - <½ full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$
 Circle - >½ full [3]	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$

[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $P = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert θ in degrees in above equations

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

Reference: USDA, SCS, NEH-5 (1956).

Figure 5-41 Open Channel Geometric Relationships for Various Cross Sections

5.5.8 Gradually Varied Flow

Many computer programs are available for computation of water surface profiles. The most general and widely used programs are HEC-2 and HEC-RAS, both developed by the U.S. Army Corps of Engineers, and EPA-SWMM developed by the Environmental Protection Agency. These programs can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the profiles can be computed manually using the Direct Step method (Chow, 1959). In the Direct Step method an increment of water depth is chosen, and the distance over which the depth change occurs is computed. This method is often used in association with culvert hydraulics. It is most accurate when the slope and depth increments are small. It is appropriate for prismatic channel sections that occur in most conduits, and can be useful when estimating both supercritical and subcritical profiles. For supercritical flow, the water surface profile is computed upstream-to-downstream. For subcritical flow, the water surface profile is computed downstream-to-upstream.

For an irregular nonprismatic channel, the Standard Step method is used. It is a more tedious and iterative process than the direct step method. The use of HEC-RAS is recommended for Standard Step calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity of the waterway. Channel cross sections will be required at each location along the waterway where there are hydraulically significant changes in channel shape or dimensions, slope, or roughness. These sections are in addition to any sections necessary to define obstructions such as culverts, bridges, dams, energy dissipation features, or aerial crossings (pipelines). Sections should usually be no more than 4 to 5 channel widths apart or 100 feet apart for ditches or streams and 500 feet apart for floodplains, unless the channel is very regular.

5.6 Outlet Structures

Outlet structures are used to regulate the flow of water. They are primarily used in stormwater management facilities that pool water. Chapter 3 discusses many of these facilities. Chapter 4 provides methods for incorporating outlet structures into overall calculations of water quality volume extended detention, channel protection volume extended detention and flood flow detention. This section outlines the governing equations for outlet structures and provides guidance on detailed design.

5.6.1 General Criteria

- All outlet structures shall be of cast-in-place or precast concrete. No outlet structures of earth, corrugated metal, riprap, grouted riprap, or plastics are allowed.
- All outlet rating curves shall be created using the equations presented in this section.
- All extended detention outlets shall be provided with outlet protection from clogging.

- All riser overflows shall have trash racks sized according to guidance provided in Section 5.6.4.
- All ponds shall be designed such that the secondary outlet can convey the 100-year storm with a minimum of one foot of freeboard with all primary outlets clogged.

5.6.2 Primary Outlets

Primary outlets provide the critical function of the regulation of flow for structural stormwater controls. There are several different types of outlets such as single stage outlet structures or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the stormwater facility outlet can be designed as a simple orifice, pipe or culvert. For multistage control structures, the inlet is designed with multiple outlets.

A stage-discharge curve is developed for the full range of flows that the structure may experience. Typically, the outlets are housed in a riser structure connected to a single outlet conduit. An alternative approach would be to provide several pipe or culvert outlets at different levels in the facility that are either discharged separately or are combined to discharge at a single location.

This section provides an overview of outlet structure hydraulics and design for stormwater storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

5.6.2.1 Outlet Structure Functions

There are two main outlet types that serve different purposes:

- Water quality and channel protection flows are normally handled with smaller, more protected outlet structures such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs.
- Larger flows, such as flood flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a weir/spillway. The orifices and weirs can be of different heights and configurations to control multiple design flows.

Water quality and channel protection outlets are typically found at or near the bottom of a riser, while flood protection outlets are found higher up.

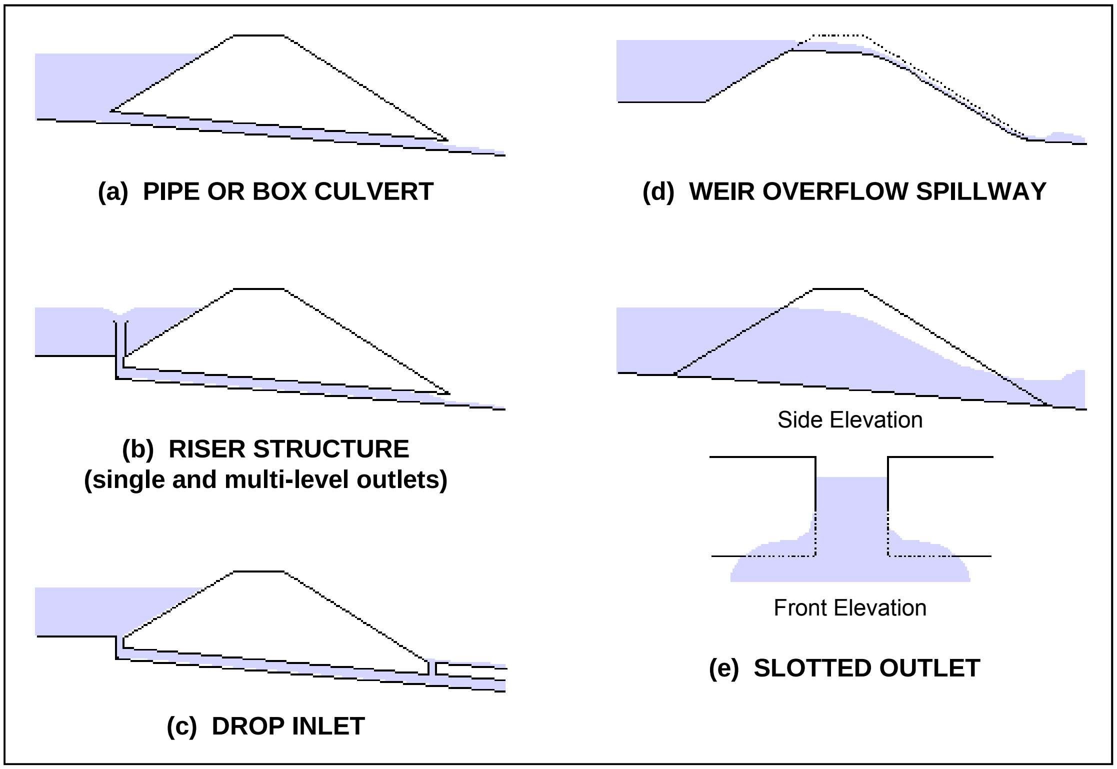


Figure 5-42 Typical Primary Outlets

5.6.2.2 Outlet Structure Types

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in stormwater facility design:

- Orifices;
- Pipes / culverts;
- Sharp-crested weirs;
- Broad-crested weirs;
- V-notch weirs;
- Proportional weirs;
- Combination outlets.

Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice. For a single orifice the discharge can be determined using the standard orifice equation below, Equation 5-36 (King & Brater, 1963).

Equation 5-36 $Q = CA(2gH)^{0.5}$

where:

Q	=	the orifice flow discharge, cfs
C	=	discharge coefficient
A	=	cross-sectional area of orifice or pipe, ft^2
g	=	acceleration due to gravity = 32.2 ft/s^2
D	=	diameter of orifice or pipe, ft
H	=	effective head on the orifice, from the center of orifice to the water surface, ft

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation between the headwater and tailwater surfaces.

When the orifice wall material is thinner than the orifice diameter, with sharp edges, a coefficient of 0.6 should be used. When the material is equal to or thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges of the latter are rounded, a coefficient of 0.92 can be used.

Pipes and Culverts

Discharge pipes are often used as outlet structures for stormwater control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse-slope underwater pipe is often used for water quality or channel protection outlets.

Outlet pipes shall be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the culvert nomographs and procedures given in Section 5.3.3 or using approved software such as EPA-SWMM, HY-8, or approved commercial programs.

Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear as it overflows, the overflow is termed a sharp-crested weir. If the sides of the weir also cause the through flow to contract, it is termed an end-contracted sharp-crested weir. Sharp-crested weirs have stable stage-discharge relationships and are often used as measurement devices. Equation 5-37 is the discharge equation for a sharp-crested weir without end contractions. This equation can also be used for the overflow of circular pipe risers using the riser circumference as the weir length (HEC-22, 2001):

Equation 5-37 $Q = \left[3.27 + 0.4\frac{H}{H_c}\right] * LH^{1.5}$

where:

Q	=	discharge, cfs
H	=	head above weir crest excluding velocity head, ft

H_C = height of weir crest above channel bottom, ft
L = horizontal weir length, ft

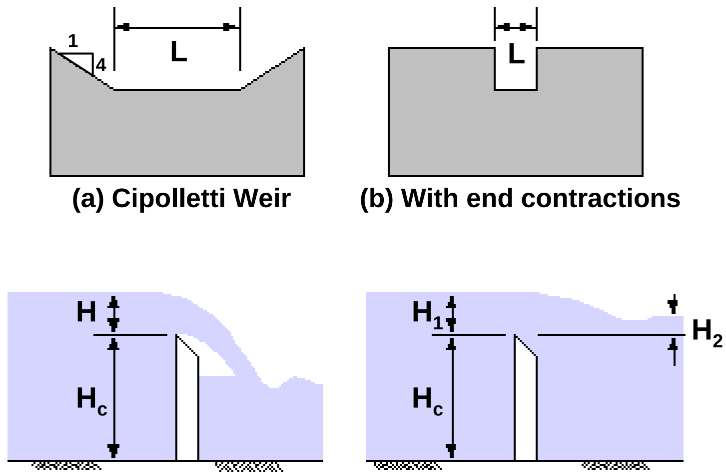


Figure 5-43 Sharp-Crested Weir

A sharp-crested weir with two end contractions is illustrated in item (b) of Figure 5-43. The discharge equation for this configuration presented in Equation 5-38 (Chow, 1959).

Equation 5-38 $$Q = \left[3.27 + \frac{0.4H}{H_C}\right](L - 0.1N)H^{1.5}$$

where:
Q = discharge, cfs
H = head above weir crest excluding velocity head, ft
H_C = height of weir crest above channel bottom, ft
L = horizontal weir length, ft
N = number of end contractions (0, 1 or 2)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result is that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is provided in Equation 5-39 (HEC-22, 2001).

Equation 5-39 $$Q_s = Q_f\left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385}$$

where:
Q_S = submerged flow, cfs
Q_f = free flow from weir equations, cfs
H_1 = upstream head above crest, ft
H_2 = downstream head above crest, ft

The discharge equation for the Cipolletti weir (configured to compensate for end contractions) is given in Equation 5-40.

Equation 5-40 $Q = 3.367LH^{1/2}$

where:

Q	=	discharge, cfs
L	=	length of horizontal portion of Cippolletti weir, cfs
H	=	head above weir crest excluding velocity head, ft

Broad-Crested Weirs

A weir in the form of a relatively long overflow section or a raised channel control crest section is a broad-crested weir. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. The equation for the broad-crested weir is Equation 5-41 (King & Brater, 1963).

Equation 5-41 $Q = CLH^{1.5}$

where:

Q	=	discharge, cfs
C	=	broad-crested weir coefficient
L	=	broad-crested weir length perpendicular to flow, ft
H	=	head above weir crest, ft

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 5-15.

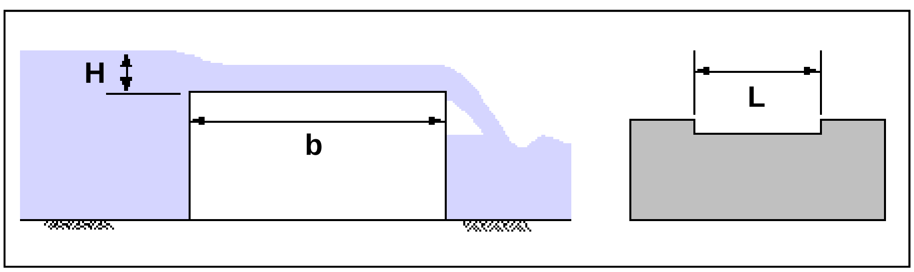


Figure 5-44 Broad-Crested Weir

Table 5-15 Broad-Crested Weir Coefficient (C) Values

Measured Head (H)* In feet	Weir Crest Breadth (b) in feet 0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

* Measured at least 2.5H upstream of the weir.
Source: Brater and King (1963)

V-Notch Weirs

The discharge through a V-notch weir (Figure 5-45) can be calculated from Equation 5-42 (King & Brater, 1963).

Equation 5-42

$$Q = 2.5 \tan\left(\frac{\theta}{2}\right) H^{2.5}$$

where:

Q	=	discharge (cfs)
θ	=	angle of V-notch (degrees)
H	=	head on apex of notch (ft)

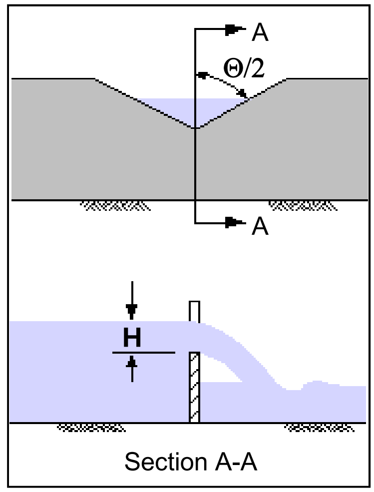


Figure 5-45 V-Notch Weir

Proportional Weirs

Although it may be more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. A typical proportional weir is shown in Figure 5-46. Design Equations 5-43 and 5-44 are for proportional weirs (HEC-22, 2001).

Equation 5-43 $$Q = 4.97a^{0.5}b\left(H - \frac{a}{3}\right)$$

Equation 5-44 $$\frac{x}{b} = 1 - \left(\frac{1}{3.17}\right)\left(\arctan\left(\frac{y}{a}\right)^{0.5}\right)$$

where:

Q = discharge, cfs

Dimensions a, b, H, x, and y are shown in Figure 5-46.

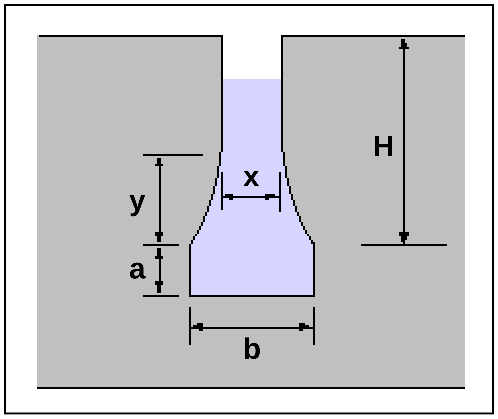


Figure 5-46 Proportional Weir Dimensions

Combination Outlets

Combinations of orifices, weirs, and pipes can be used to provide multi-stage outlet control for different purposes within a storage facility (i.e., water quality protection volume, channel protection volume, and flood control).

There are generally two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs, or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure 5-47 shows an example of a riser designed for a wet extended detention pond.

Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

The use of a combination outlet requires the construction of a composite stage-discharge curve (as shown in Figure 5-48) suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed later in this section.

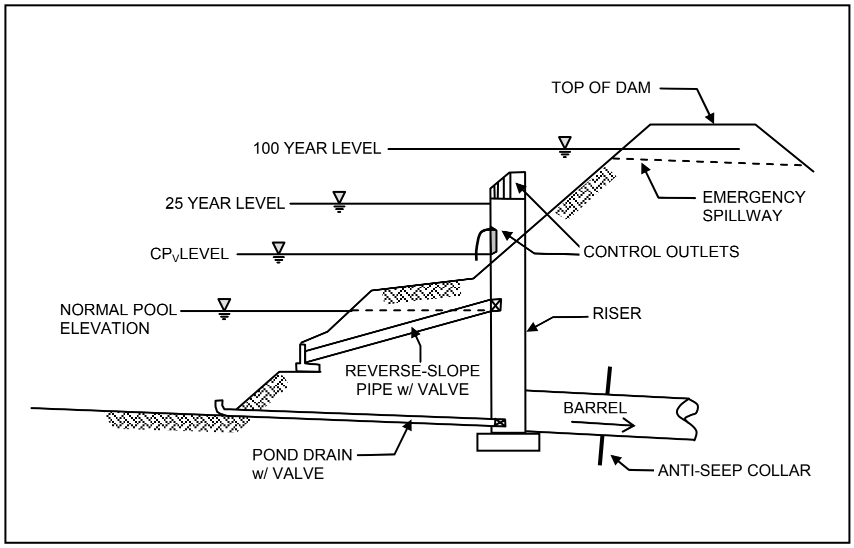


Figure 5-47 Schematic of Combination Outlet Structure

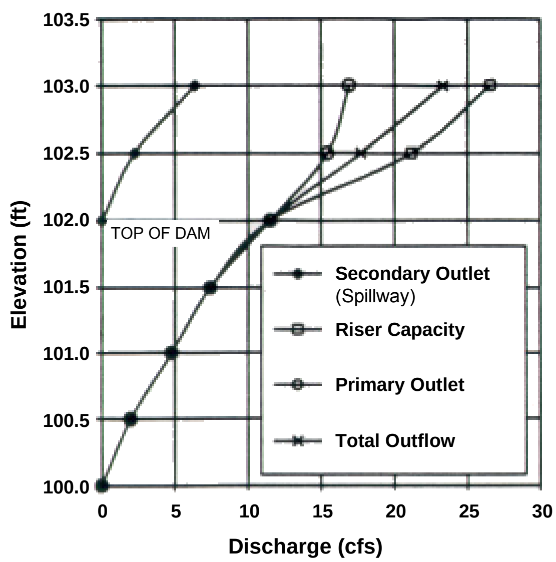


Figure 5-48 Composite Stage-Discharge Curve

5.6.3 Extended Detention Outlet Protection

Small low flow orifices such as those used for small storm detention and extended detention applications can easily clog, preventing the structural control from meeting its design purpose(s) and potentially causing adverse impacts. Therefore, orifices shall be adequately protected from clogging. There are a number of different anti-clogging designs, including:

- The use of a reverse slope pipe attached to a riser for a stormwater pond or wetland with a permanent pool (see Figure 5-47). The inlet is submerged a minimum of 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.

- The use of a hooded outlet for a stormwater pond or wetland with a permanent pool (see Figure 5-49 and Figure 5-50).
- Internal orifice protection through the use of an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket (see Figure 5-51). Internal orifice size requirements may be attained by the use of adjustable gate valves to achieve an equivalent orifice diameter.

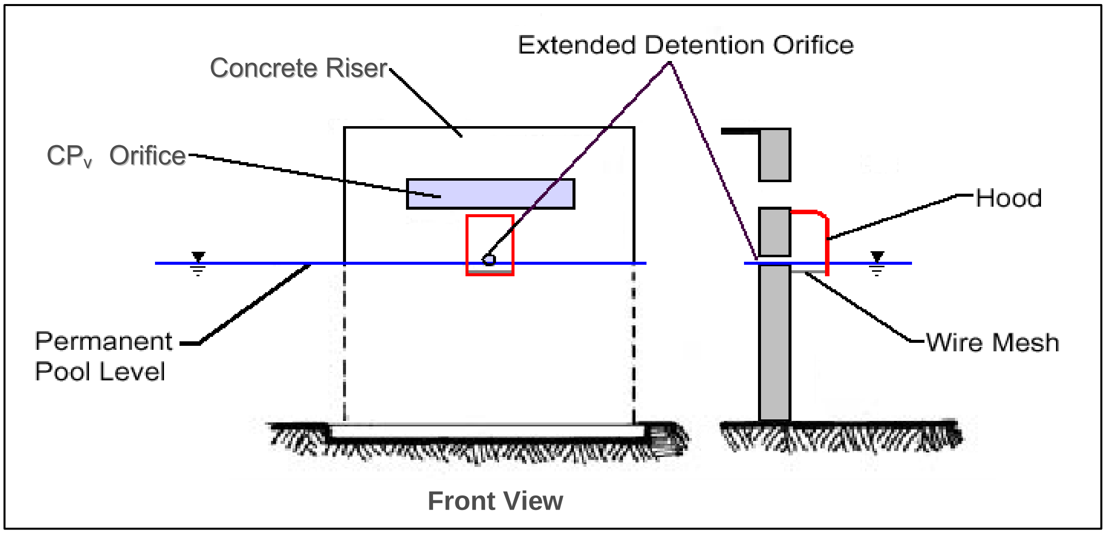


Figure 5-49 Hooded Outlet

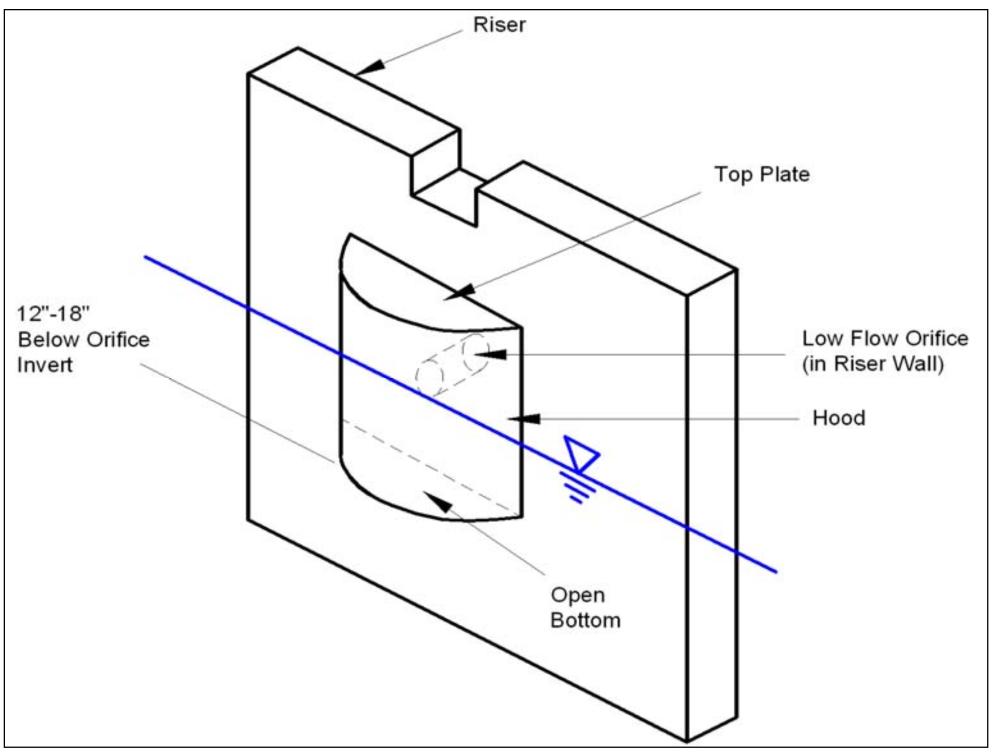


Figure 5-50 Orifice Hood

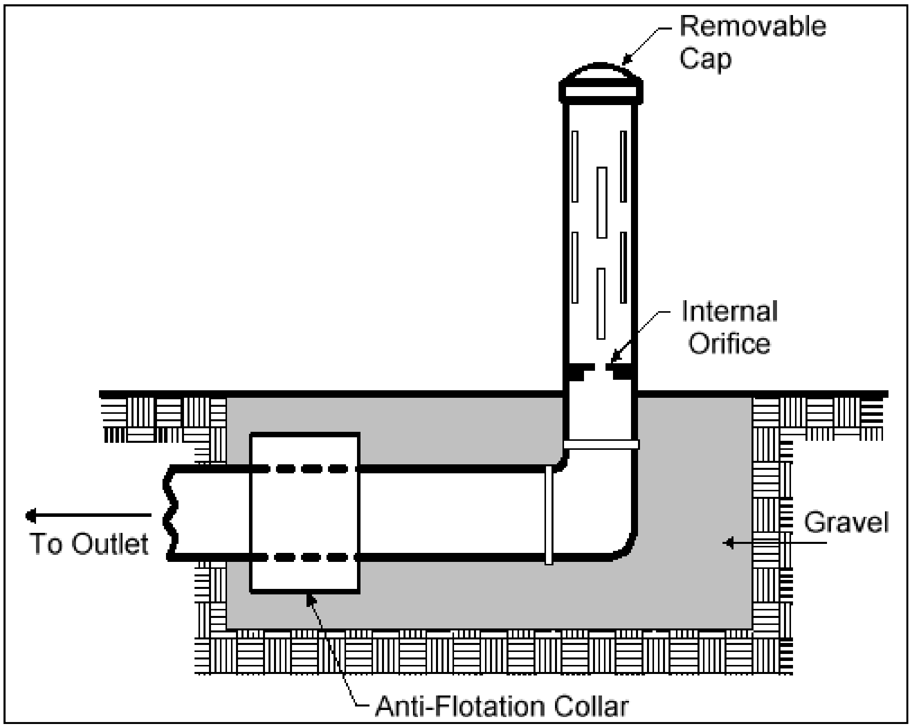


Figure 5-51 Internal Control for Orifice Protection

5.6.4 Trash Racks and Safety Grates

Introduction

The susceptibility of larger inlets to clogging by debris and trash must be considered when estimating their hydraulic capacities. In most instances trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions:

- Keeping debris away from the entrance to the outlet works where they will not clog the critical portions of the structure.
- Capturing debris in such a way that relatively easy removal is possible.
- Ensuring that people and animals are kept out of confined conveyance and outlet areas.
- Providing a safety system that prevents anyone from being drawn into the outlet and allows them to climb to safety.

When designed properly, trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985; Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety and size of outlet.

An example of trash racks used on a riser outlet structure is shown in Figure 5-52. The inclined vertical bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

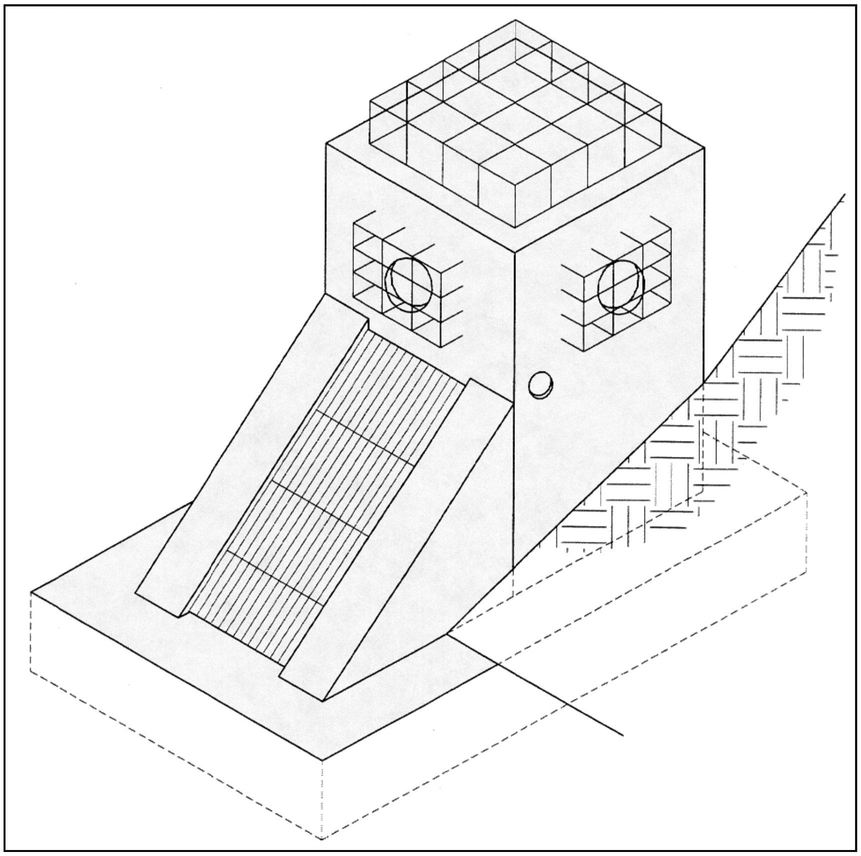


Figure 5-52 Example of Various Trash Racks Used on a Riser Outlet Structure
(VDCR, 1999)

Trash Rack Design

Trash racks must be large enough so that partial clogging will not adversely restrict flows reaching the control outlet. The surface area of all trash racks should be maximized and the trash racks shall be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The control for the outlet shall not shift to the rack, nor shall the rack cause the headwater to rise above planned levels. The bar opening spacing shall be less than the pipe diameter, and in no case greater than 6 inches.

However, where a small orifice is provided, a separate trash rack for that outlet shall be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars shall be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

Figure 5-53 shall be used to size trash rack openings based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

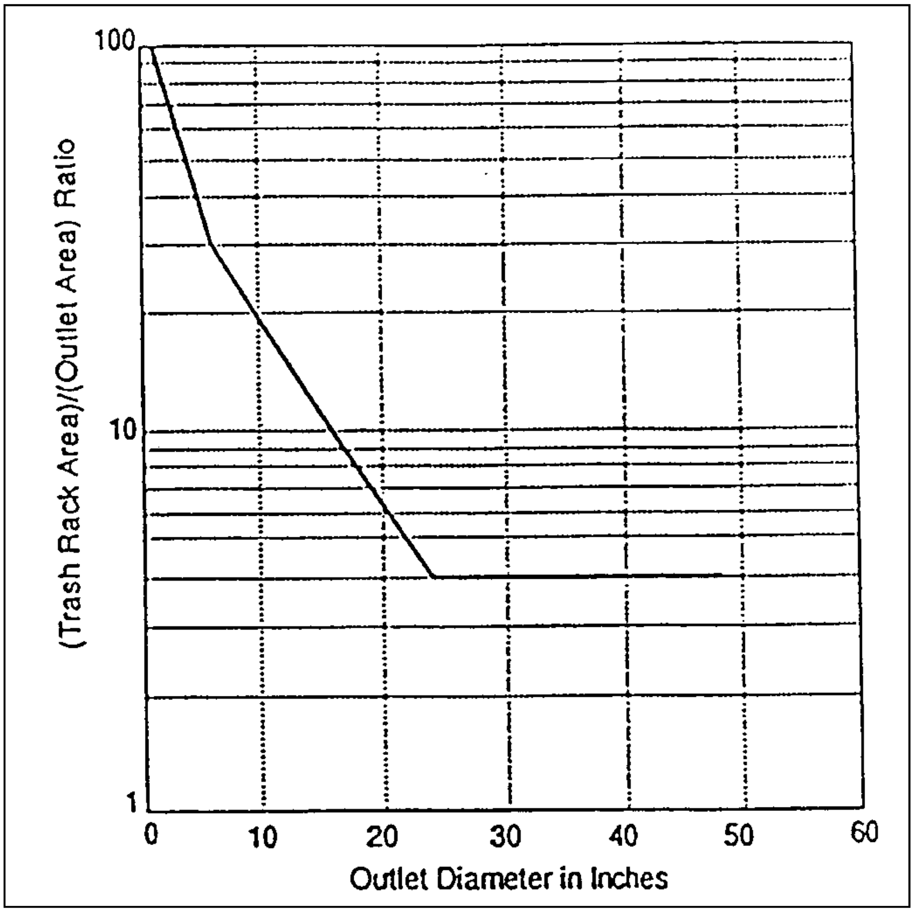


Figure 5-53 Minimum Rack Size vs. Outlet Diameter
(UDCFD, 1992)

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level — the slower the approach flow, the flatter the angle.

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks shall have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest orifice of a multistage riser, the riser outlet must be set far enough below the lowest orifice to ensure that sediment is flushed through the riser.

Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack. Alternately, debris for culvert openings can be caught upstream from the opening by using barriers such as a chain safety net (USBR, 1978; UDFCD, 1999).

Example Problem

Find the trash rack open area:

Outlet Diameter = 20 in
Outlet Area = 2.18 ft^2

From Figure 5-53:
(Trash Rack Area)/(Outlet Area) ratio = 6

Therefore the required track rack opening area = (2.18)(6) = 13 ft^2
See (UDCFD, 1992) for more examples.

5.6.5 Secondary Outlets

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for the maximum design storm for a storage facility. Figure 5-54 shows an example of an emergency spillway.

Emergency spillway designs are open channels, usually rectangular or trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 5-54). The emergency spillway is proportioned to pass flows for the storage facility maximum design flood (the 100-year flood in Wichita and Sedgwick County) without overtopping the embankment. The 100-year flood discharge, assuming blockage of all primary outlets, must be conveyed with 1 foot of freeboard to the top of the dam. The concrete discharge channel of the spillway shall also have a minimum of 1 foot of freeboard. Flow in the emergency spillway is open channel flow. Normally, it is assumed that critical depth occurs at the control section.

Figure 5-54 Emergency Spillway

5.7 Energy Dissipation

5.7.1 Introduction

The outlets of pipes and lined channels are points of critical erosion potential. Stormwater transported through man-made conveyance systems at design capacity often reaches a velocity that exceeds the capacity of the receiving channel to resist erosion. To prevent scour at stormwater outlets, protect the outlet structure, and minimize the potential for downstream erosion, a flow transition structure shall be used to absorb the initial impact of flow and reduce the flow velocity to a non-erosive value.

Energy dissipators are engineered devices such as riprap aprons or concrete baffles placed at the outlet of stormwater conveyances for the purpose of reducing the velocity, energy and turbulence of the discharged flow.

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 oriented in a downstream direction relative to the streambed. This will reduce turbulence and the potential for excessive erosion.

5.7.2 General Criteria

Erosion problems at culvert, storm sewer, and spillway outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.

Energy dissipators shall be employed whenever the shear of flow leaving a stormwater management facility exceeds the permissible shear, or the velocity exceeds the permissible velocity for the downstream channel system, for the 2, 5 and 10-year storms. Energy dissipator designs will vary based on discharge specifics and tailwater conditions.

In addition, protection against damage to any component of a dam must be provided for all flows up to and including the 100-year maximum design event. This usually requires the installation of protective riprap on the downstream face of the dam and adjacent areas where flow velocities or high turbulence will occur. Riprap in those areas may be sized using Equation 5-27 with a C value of 0.86 instead of 1.2 (HDC 712-1, USACE).

Pipe and culvert outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence. For most designs, the following outlet protection devices and energy dissipators provide sufficient protection at a reasonable cost:

- Riprap apron;
- Riprap outlet basins;
- Grade Control Structures.

This section focuses on the design on these measures. The reader is referred to the Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, Hydraulic Design of Energy Dissipators for Culverts and Channels, for the design procedures for other energy dissipators.

5.7.3 Design Guidelines

If outlet protection is required, choose an appropriate type. Suggested outlet protection facilities and applicable flow conditions (based on Froude number and dissipation velocity) are described below:

- Riprap aprons may be used when the outlet Froude number (Fr) is less than or equal to 2.5. In general, riprap aprons prove economical for transitions from culverts to overland sheet flow at terminal outlets, but may also be used for transitions from culvert sections to stable channel sections. Stability of the surface at the termination of the apron shall be considered.
- Riprap outlet basins may also be used when the outlet Fr is less than or equal to 2.5. They are generally used for transitions from culverts to stable channels. Since riprap outlet basins function by creating a hydraulic jump to dissipate energy, performance is impacted by tailwater conditions.

When outlet protection facilities are selected, appropriate design flow conditions and site-specific factors affecting erosion and scour potential, construction cost, and long-term durability shall be considered. Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is otherwise expected to occur per the guidance found in Section 5.5.4.

5.7.4 Riprap Aprons

Description

A riprap-lined apron is a commonly used practice for energy dissipation because of its relatively low cost and ease of installation. A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow.

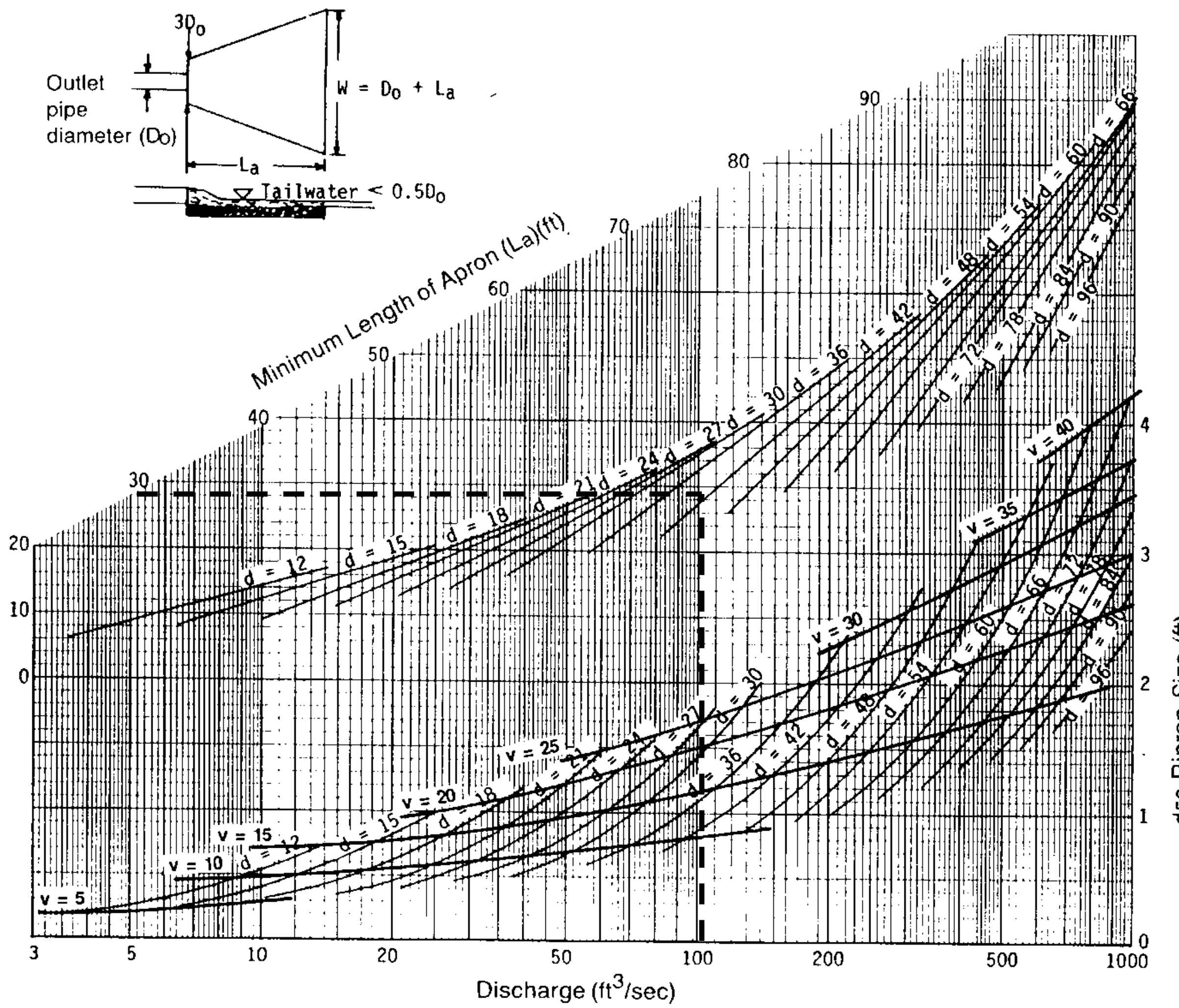
Design Procedure

The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, d_{50}. If tailwater conditions are unknown, or if both minimum and maximum conditions may occur, the apron shall be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, they can be used for partially full pipes and box culverts. The design procedure consists of the following steps:

Step 1 If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 5-55 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 5-56 shall be used.

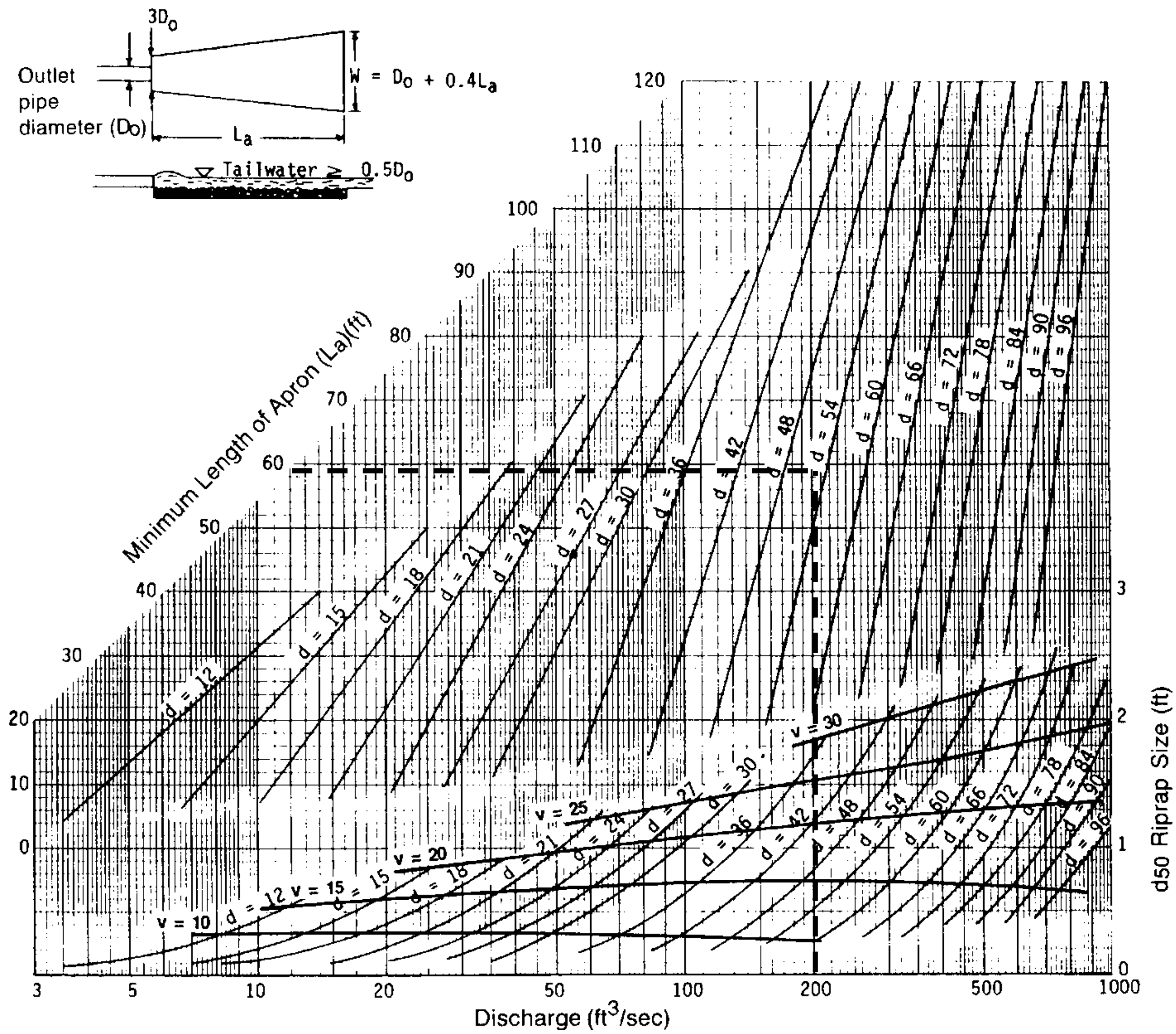
Step 2 Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curve. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 5-57.

- For pipes flowing full use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, L_a, and median riprap diameter, d_{50}, from the appropriate curves.
- For pipes flowing partially full use the depth of flow, d, in feet, and velocity, v, in ft/s. On the lower portion of the appropriate figure, find the intersection of the d and v curves, and then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, La from the scale on the left.
- For box culverts use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, and then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth, d. Find the minimum apron length, L_a, using the scale on the left.



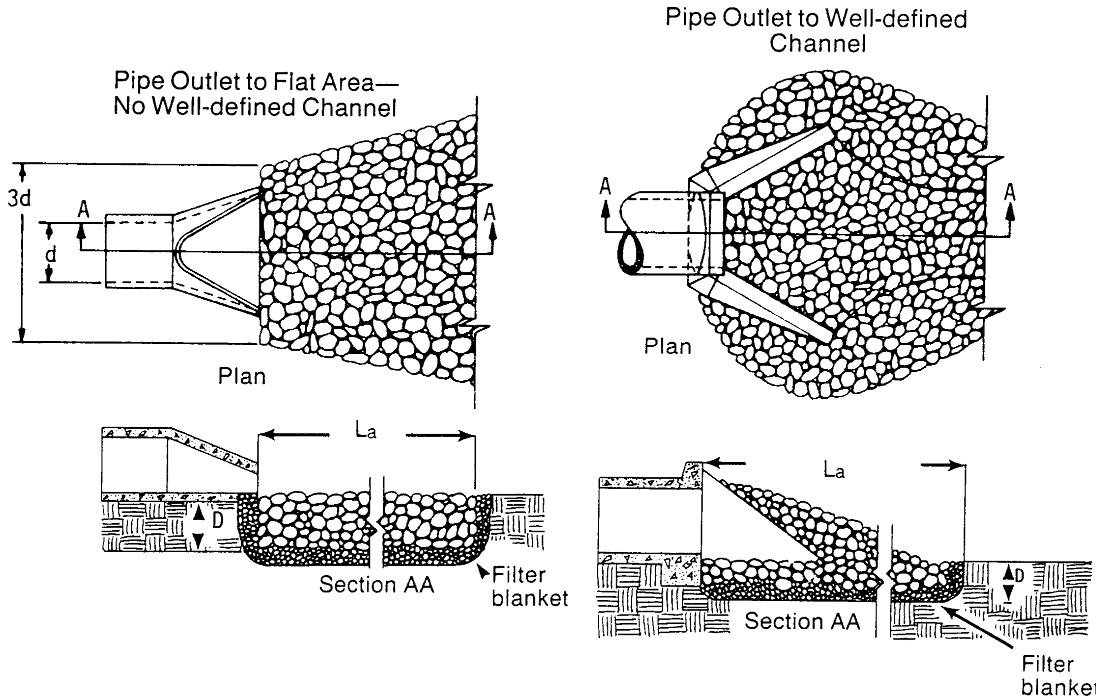
Curves may not be extrapolated.

Figure 5-55 Design of Riprap Apron under Minimum Tailwater Conditions
(USDA, SCS, 1975)



Curves may not be extrapolated.

Figure 5-56 Design of Riprap Apron under Maximum Tailwater Conditions
(USDA, SCS, 1975)



NOTES:

1. L_a is the length of the riprap apron.
2. D = 1.5 times the maximum stone diameter but not less than 6 inches.
3. In a well defined channel extend the apron up the channel banks to an elevation of 6 inches above the maximum tailwater depth or to the top of the bank, whichever is less.
4. A filter blanket or filter fabric shall be installed between the riprap and soil foundation.

Figure 5-57 Riprap Apron

(Manual for Erosion and Sediment Control in Georgia, 1996)

Design Considerations

The following items shall be considered during riprap apron design:

- The maximum stone diameter should be 1.5 times the median riprap diameter.
- The riprap thickness should be 1.5 times the maximum stone diameter or 6 inches, whichever is greater. (Apron thickness may be reduced to 1.5 x d_{50} when an appropriate filter fabric is used under the apron.)
- The apron width at the discharge outlet shall be at least equal to the pipe diameter or culvert width, d_w. Riprap shall extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 3:1 and a height not less than the pipe diameter or culvert height, and shall taper to the flat surface at the end of the apron.
- If there is a well-defined channel, the apron length shall be extended as necessary so the downstream apron width is equal to the channel width. The sidewalls of the channel shall not be steeper than 3:1.

- If the ground slope downstream of the apron is steep, channel erosion may occur. The apron shall be extended as necessary until the slope is gentle enough to prevent further erosion.

Example Problem – Riprap Apron Design #1

A flow of 280 cfs discharges from a 66-in pipe with a tailwater of 2 ft above the pipe invert. Find the required design dimensions for a riprap apron.

Determine 0.5 D_o.

D_o = 66 in = 5.5 ft; therefore, 0.5 D_o = 2.75 ft.
Since TW = 2 ft is less than 2.75 ft, use Figure 5-55 for minimum tailwater conditions.

Determine the apron length (L_a) and median stone size (d_{50}) using Figure 5-55.

Start at discharge = 280 cfs and read right to d_{50} of 1.2 ft.
Continue up to the intersection with the d = 66 in curve, and read left to L_a = 38 ft.

Determine the downstream apron width.

The downstream apron width equals the pipe diameter plus the apron length:
$W = D_o + L_a = 5.5 + 38 = 43.5$ ft

Determine the maximum riprap diameter.

Maximum riprap diameter is 1.5 times the median stone size:
$d_{max} = 1.5\ (d_{50}) = 1.5\ (1.2) = 1.8$ ft

Determine the riprap depth.

Riprap depth is 1.5 times the maximum riprap diameter
Riprap depth = 1.5 (d_{max}) = 1.5 (1.8) = 2.7 ft.

Example Problem – Riprap Apron Design #2

A concrete box culvert 5.5 ft high and 10 ft wide conveys a flow of 600 cfs at a depth of 5.0 ft. Tailwater depth is 5.0 ft above the culvert outlet invert. Find the design dimensions for a riprap apron.

Determine 0.5 D_o.

D_o = 0.5 (5.0) = 2.5 ft.
Since TW = 5.0 ft is greater than 2.5 ft, use Figure 5-56 for maximum tailwater conditions.

Determine the outlet velocity

$v = Q/A = 600/[(5)(10)] = 12$ ft/s

Determine the apron length (L_a) and median stone size (d_{50}) using Figure 5-56.

Start at the intersection of the curves, D_o = 60 in and v = 12 ft/s.
Read right to d_{50} = 0.4 ft.
Reading up to the intersection with d = 60 in, read left to L_a = 40 ft.

Determine the downstream apron width.

Apron width downstream is equal to the pipe diameter plus 0.4 times apron length
$W = D_o + 0.4\ L_a = 10 + 0.4\ (40) = 26$ ft.

Determine the maximum riprap diameter.

$d_{max} = 1.5\ (d_{50}) = 1.5\ (0.4) = 0.6$ ft.

Determine the riprap depth.

Riprap depth = $1.5\ (d_{max}) = 1.5\ (0.6) = 0.9$ ft.

5.7.5 Riprap Basins

Description

Another method to reduce the exit velocities from stormwater outlets is through the use of a riprap basin. A riprap outlet basin is a preshaped scourhole lined with riprap that functions as an energy dissipator by forming a hydraulic jump.

Basin Features

General details of the basin recommended in this section are shown in Figure 5-58. Principal features of the basin are:

- The basin is preshaped and lined with riprap of median size (d_{50}).
- The floor of the riprap basin is constructed at an elevation of h_s below the culvert invert. The dimension h_s is the approximate depth of scour that would occur in a thick pad of riprap of size d_{50} if subjected to design discharge. The ratio of h_s to d_{50} of the material shall be between 2 and 4.
- The length of the energy dissipating pool is 10 x h_s or 3 x W_o, whichever is larger. The overall length of the basin is 15 x h_s or 4 x W_o, whichever is larger.

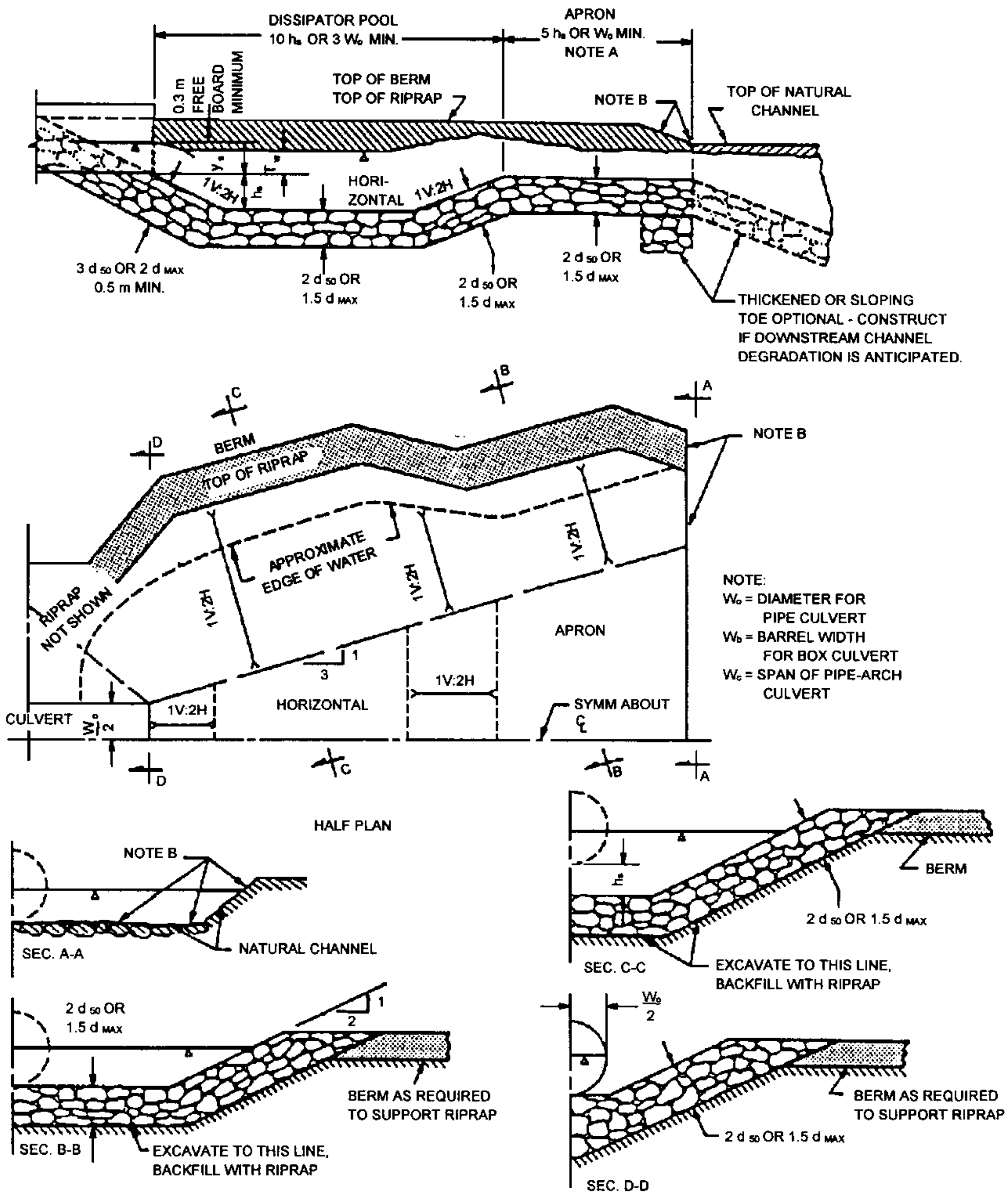
Design Procedure

The following procedure shall be used for the design of riprap basins.

- Estimate the flow properties at the brink (outlet) of the culvert. Establish the outlet invert elevation such that $TW/y_o \leq 0.75$ for the design discharge.

- For subcritical flow conditions (culvert set on mild or horizontal slope) use Figure 5-59 or Figure 5-60 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.
- For channel protection, compute the Froude number for brink conditions with $y_e = (A/2)^{1.5}$. Select d_{50}/y_e appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25 < d_{50}/y_e < 0.45$). Obtain h_s/y_e from Figure 5-61, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if h_s/d_{50} falls out of this range.
- Size basin as shown in Figure 5-58.

Material, construction techniques, and design details for riprap shall be in accordance with specifications in the Federal Highway publication HEC No. 11 entitled Use of Riprap For Bank Protection.



NOTE A - IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A-A SUCH THAT Q/(CROSS SECTION AREA AT SEC. A-A) = SPECIFIED EXIT VELOCITY.

NOTE B - WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A-A.

Figure 5-58 Details of Riprap Outlet Basin
(Source: HEC-14, 2006)

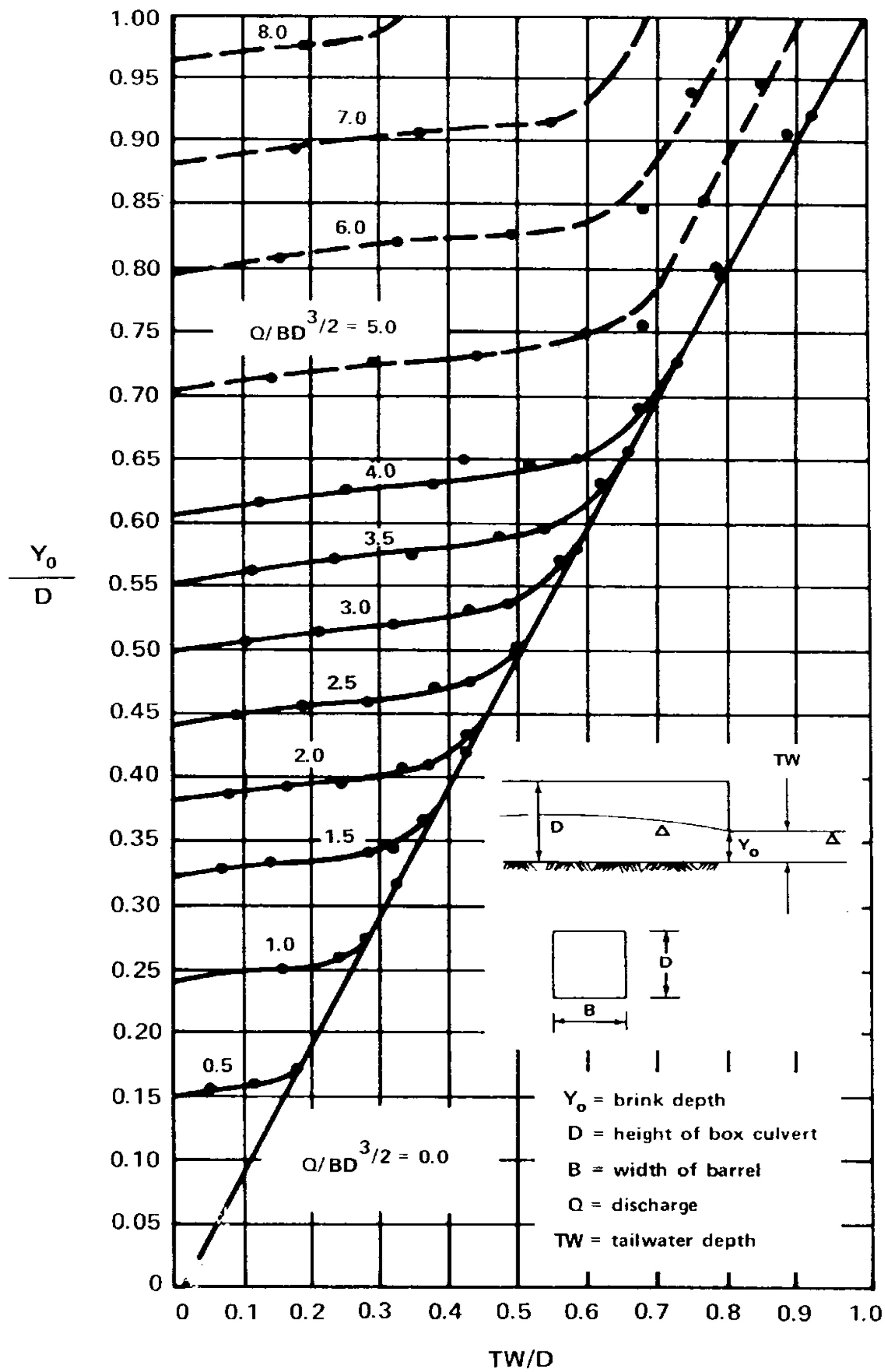


Figure 5-59 Dimensionless Rating Curves for Rectangular Culvert Outlets on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 2006)

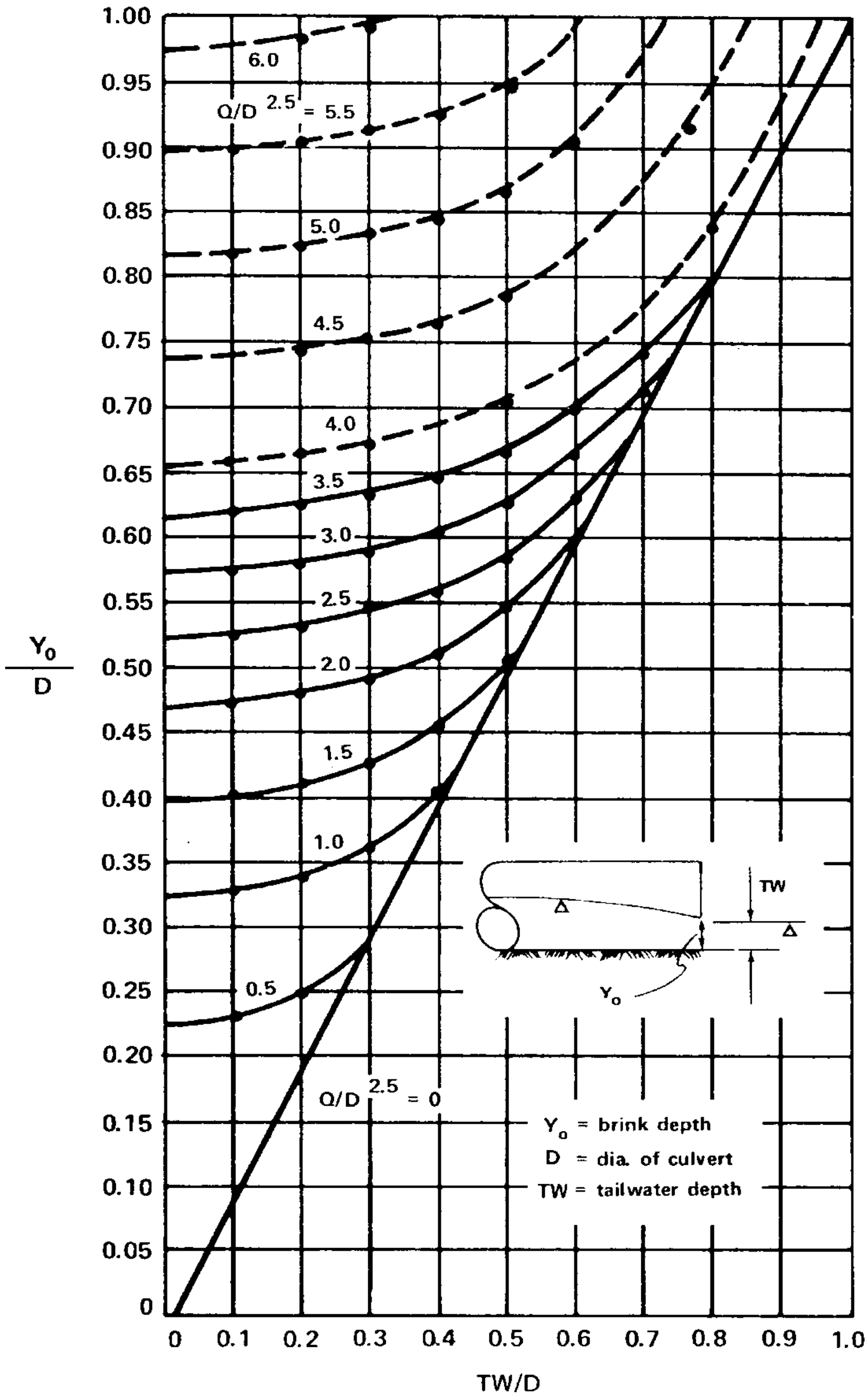


Figure 5-60 Dimensionless Rating Curves for Circular Culvert Outlets on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 2006)

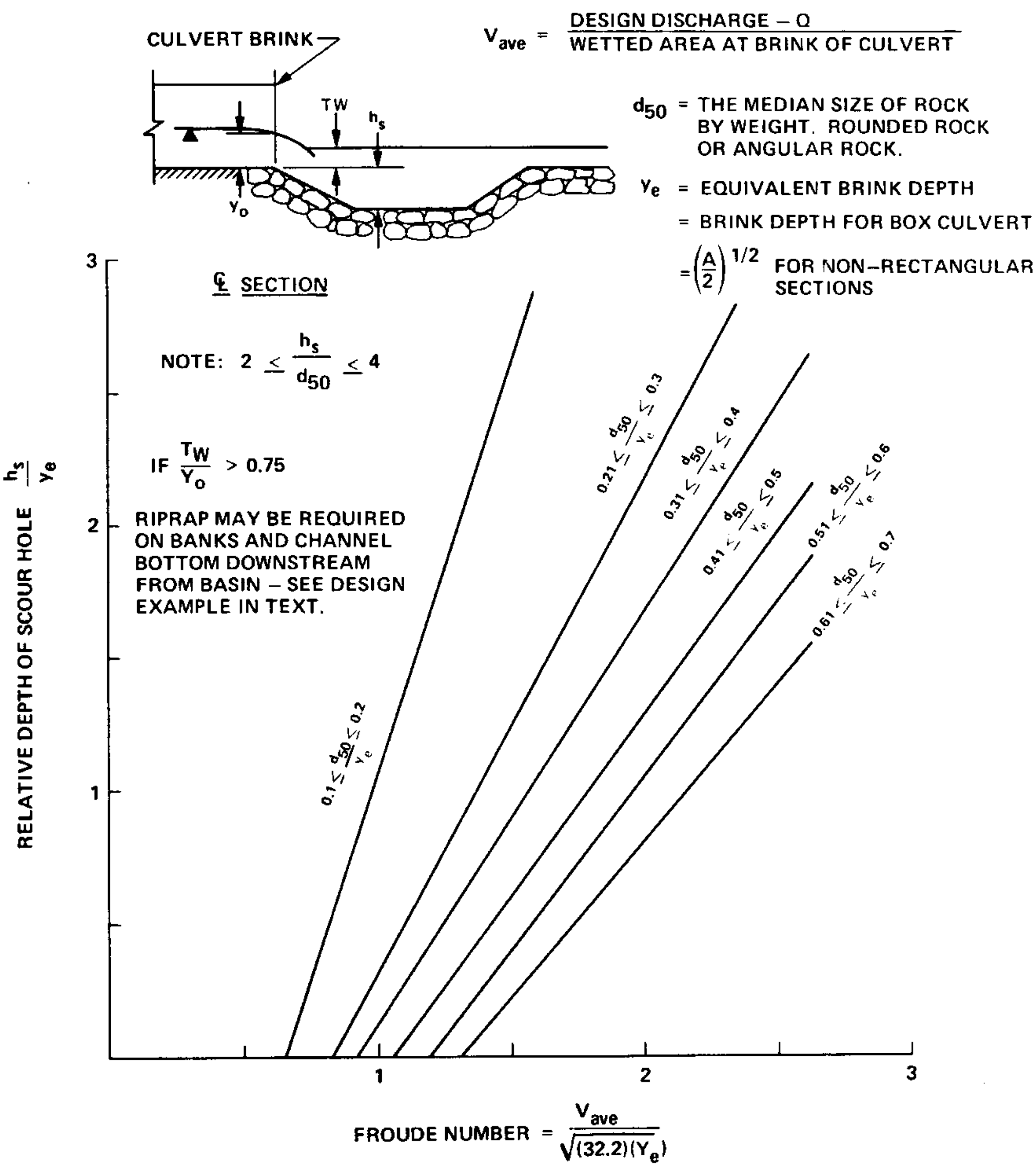


Figure 5-61 Depth of Scour Hole vs Froude Number at Brink of Culvert (with respect to Size of Riprap)
(USDOT, FHWA, HEC-14, 2006)

Design Considerations

Riprap basin design shall include consideration of the following:

The dimensions of a scourhole in a basin constructed with angular rock can be approximately the same as the dimensions of a scourhole in a basin constructed of rounded material when rock size and other variables are similar.

When the ratio of tailwater depth to brink depth, TW/y_o, is less than 0.75 and the ratio of scour depth to size of riprap, h_s/d_{50}, is greater than 2.0, the scourhole should function very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunges into the hole, a jump forms against the downstream extremity of the scourhole, and flow is generally well dispersed leaving the basin.

The mound of material formed on the bed downstream of the scourhole contributes to the dissipation of energy and reduces the size of the scourhole; that is, if the mound from a stable scoured basin is removed and the basin is again subjected to design flow, the scourhole will enlarge.

For high tailwater basins (TW/y_o greater than 0.75), the high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin and diffuses similarly to a concentrated jet diffusing in a large body of water. As a result, the scourhole is much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

It should be recognized that there is a potential for limited degradation to the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2(d_{50})$ thickness of riprap, the heavy layer of riprap adjacent to the channel prism, and the apron riprap in the downstream portion of the basin, such damage should be superficial.

See Standards in FHWA HEC No. 11 for details on riprap materials and use of filter fabric.

Stability of the surface at the outlet of a basin shall be considered using the methods for open channel flow as outlined in Section 5.5.5.

Example Problem – Riprap Basin Design #1

Size a riprap basin to protect against scour for the following conditions:

Box culvert - 8 ft by 6 ft (invert = 112.2’)	Design Discharge Q = 800 cfs
Supercritical flow in culvert	Normal flow depth = brink depth
Y_o = 4 ft	Tailwater depth TW = 2.8 ft

Determine $y_{e.}$

$y_o = y_e$ for rectangular section; therefore, with y_o given as 4 ft, y_e = 4 ft.

Determine culvert exit velocity.

V_o = Q/A = 800/(4 x 8) = 25 ft/s

Determine exit Froude number and check that it is less than 2.5. Froude numbers greater than 2.5 must utilize dissipators more structurally sound than riprap basins.

Froude Number = Fr = $V/(g \times y_e)^{0.5}$
Fr = $25/(32.2 \times 4)^{0.5}$ = 2.20, therefore Fr < 2.5 OK

Determine TW/y_e and check that it is less than 0.75 so that the scourhole does not grow beyond the dissipator by jet action.

TW/y_e = 2.8/4.0 = 0.7, therefore, TW/y_e < 0.75 OK

Choose median size of stone (d_{50}) such that h_s/d_{50} is between 2 and 4.

Try d_{50}/y_e = 0.45, d_{50} = 0.45 x 4 = 1.80 ft
From Figure 5-61, h_s/y_e = 1.6
h_s = 4 x 1.6 = 6.4 ft
h_s/d_{50} = 6.4/1.8 = 3.6 ft, therefore 2 < h_s/d_{50} < 4 OK

Determine dissipator pool length.

L_s = 10 x h_s = 10 x 6.4 = 64 ft
L_s min = 3 x W_o = 3 x 8 = 24 ft; therefore, use L_s = 64 ft

Determine riprap basin length.

L_B = 15 x h_s= 15 x 6.4 = 96 ft
L_B min = 4 x W_o = 4 x 8 = 32 ft; therefore, use L_B = 96 ft

Determine riprap pool invert.

Based on Figure 5-58, pool invert = culvert invert – h_s
Pool invert = 112.2 – 6.4 = 105.8

Determine riprap thickness.

Thickness of riprap on the approach = 3 x d_{50} = 3 x 1.8 = 5.4 ft
Thickness of riprap on the remainder = 2 x d_{50} = 2 x 1.8 = 3.6 ft

Other basin dimensions per Figure 5-58.

Example Problem – Riprap Basin Design #2

Size a riprap basin to protect against scour for the following conditions:

6' circular culvert - (invert = 120.2')	Design Discharge Q = 135 cfs
Subcritical flow in culvert	Normal flow depth in pipe = 4.5'
Tailwater depth TW = 2.0 ft	Normal velocity in channel = 5.9 ft/s

Determine y_o.

$Q/D^{2.5} = 135/6^{2.5} = 1.53$
$TW/D = 2.0/6 = 0.33$
From Figure 5-60, $y_o/D = 0.45$
$y_o = 0.45 \times 6 = 2.7$ ft

Check TW/y_o.

$TW/y_o = 2.0/2.7 = 0.74$, therefore $TW/y_o < 0.75$ OK

Determine brink area for $y_o/D = 0.45$ from Figure 5-14 for depth = 45% of full.

Brink area = 42% of full area = $0.42^*(\pi D^2/4) = 0.42^*(3.14^*6^2/4) = 11.9\ ft^2$

Determine pipe exit velocity.

$V_o = Q/A = 135/11.9 = 11.3$ ft/s

Determine y_e.

$y_e = (A/2)^{1/2} = (11.9/2)^{1/2} = 2.4$ ft

Determine Froude number (Fr).

$Fr = V_o/(32.2^*y_e)^{1/2} = 11.3/(32.2^*2.4)^{1/2} = 1.3$, therefore Fr < 2.5 OK

Choose median size of stone (d_{50}) such that h_s/d_{50} is between 2 and 4.

Try $d_{50}/y_e = 0.25$
$d_{50} = (d_{50}/y_e)^*y_e = 0.25^*2.4 = 0.6$ ft
From Figure 5-61 with Fr = 1.3 and $d_{50}/y_e = 0.25$, the value of $h_s/y_e = 0.8$
$h_s = (h_s/y_e)^*y_e = 0.8^*2.4 = 1.9$ ft
$h_s/d_{50} = 1.9/0.6 = 3.2$, therefore $2 < h_s/d_{50} < 4$ OK

Determine dissipator pool length.

$L_s = 10 \times h_s = 10 \times 1.9 = 19$ ft
L_s min $= 3 \times W_o = 3 \times 6 = 18$ ft; therefore, use $L_s = 19$ ft

Determine riprap basin length.

$L_B = 15 \times h_s = 15 \times 1.9 = 28.5$ ft

L_B min = 4 x W_o = 4 x 6 = 24 ft; therefore, use L_B = 28.5 ft

Determine riprap pool invert.

Based on Figure 5-58, pool invert = culvert invert – h_s
Pool invert = 120.2 – 1.9 = 118.3

Determine riprap thickness.

Thickness of riprap on the approach = 3 x d_{50} = 3 x 0.6 = 1.8 ft
Thickness of riprap on the remainder = 2 x d_{50} = 2 x 0.6 = 1.2 ft

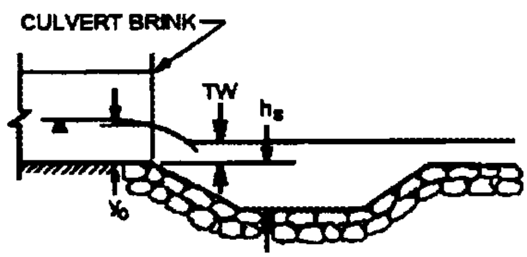
Other basin dimensions per Figure 5-58.

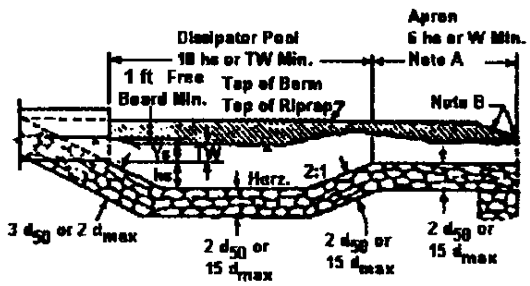
RIPRAP BASIN

Project No. ______________________________

Designer ______________________________ Date ________

Reviewer ______________________________ Date ________





DESIGN VALUES	TRIAL 1	FINAL TRIAL
Equi. Depth, d_E		
D_{50}/d_E		
D_{50}		
Froude No., Fr		
h_s/d_E		
h_s		
h_s/D_{50}		
$2 < h_s/D_{50} < 4$		

BASIN DIMENSIONS		FEET	
Pool length is the larger of:	$10h_s$		
	$3W_o$		
Basin length is the larger of:	$15h_s$		
	$4W_o$		
Approach Thickness	$3D_{50}$		
Basin Thickness	$2D_{50}$		

TAILWATER CHECK	
Tailwater, TW	
Equivalent depth, d_E	
TW/d_E	
IF $TW/d_E > 0.75$, calculate riprap downstream	
$D_E = (4A_c/\pi)^{0.5}$	

DOWNSTREAM RIPRAP

L/D_E	L	V_L/V_o	V_L	D_{50}

Figure 5-62 Riprap Basin Design Form
(USDOT, FHWA, HEC-14, 2006)

5.7.6 Grade Control Structures

When channels are relocated through easily erodible soils and stream gradients are increased, it may be unusually difficult to meet the stability requirements. This can cause bank instability, increased upstream scouring, and sloughing of natural slopes. Valley Center and Sedgwick County require that streambed stability be maintained. This can be accomplished by grade stabilization structures; in essence a series of low-head weirs. The effect of these weirs is to reduce the effective energy grade line in all places except for the stabilized drop over the weir. This flatter energy grade line can then be used in channel permissible shear calculations to reduce the need for armoring of the entire channel.

If designed and constructed with ecological considerations in mind, these structures can double as habitat enhancement devices. If improperly planned however, they can actually degrade habitat values. The most productive method of installing these structures is to use low weirs that pool water just a short distance (approximately 100 feet) upstream. A plunge pool will form just below the structures, and a riffle area should develop below this pool. The next structure should be located downstream a sufficient distance to avoid frequent impounding of the riffle area below the pool at the base of the upstream weir.

Specific construction requirements and techniques can be obtained from the NRCS or other agencies upon request. In addition, “natural channel design” methods may be utilized for this type of control. The intent of this general discussion of grade stabilization structures is to promote consideration of such measures early in the planning process.

5.8 Level Spreaders

5.8.1 Introduction

A level spreader is a mechanism used to disperse concentrated runoff uniformly over the ground surface as sheet flow. The purpose of this practice is to convert concentrated, potentially erosive flow to sheet flow and release it uniformly over a stabilized area or filter strip. The resultant sheet flow enhances pollutant filtering and runoff infiltration and reduces the potential for erosion.

The level spreader is a relatively low cost structure that is used for two primary applications: to disperse shallow concentrated or channelized stormwater runoff from impervious areas to a filter strip, water quality or other buffer, or other vegetated area; or, outlet diversion (release small volumes of concentrated flow from diversions when conditions are suitable). To accomplish these purposes, a high degree of care must be taken to construct the spreader lip so it is completely level. Level spreaders must be constructed correctly since any depressions in the spreader lip will concentrate the flow, resulting in a loss of adequate dispersion of runoff. Improperly designed or constructed level spreaders can reduce the effectiveness of filter strips and buffer areas to remove pollutants, and can increase the potential for erosion in vegetated areas to which the level spreader discharges.

5.8.2 General Criteria

All level spreaders shall conform to the design standards listed below.

For impervious surface runoff applications:

- The capacity for the level spreader is determined in the design of the filter strip to which it discharges. For filter strip design guidance, refer to Section 3.2.
- The spreader shall run linearly along the entire length of the filter strip (or buffer area) to which it discharges. In most cases, the spreader will be the same length as the contributing impervious surface. The ends of the spreader shall be tied into higher ground to prevent flow around the spreader.

For diversion outlet applications:

The capacity of the spreader shall be determined using the peak flow from the 10-year, 24-hour storm. The drainage area shall be restricted so that maximum 10-year design flows into the spreader will not exceed 30 cfs.

For all level spreader applications:

- The minimum depth shall be 6 inches and the minimum width shall be 6 feet for the lower side slope. Side slopes shall be 4:1 (horizontal to vertical) or flatter.
- The grade of the spreader shall be 0%.
- The appropriate length, width, and depth of spreader shall be selected from Table 5-16.
- It will be necessary to construct a 20 foot transition section in the diversion channel so the width of the channel will smoothly meet the width of the spreader to ensure uniform outflow.
- The last 20 feet of the diversion channel shall provide a smooth transition from the channel grade to the level spreader and shall have slope less than or equal to 1%.

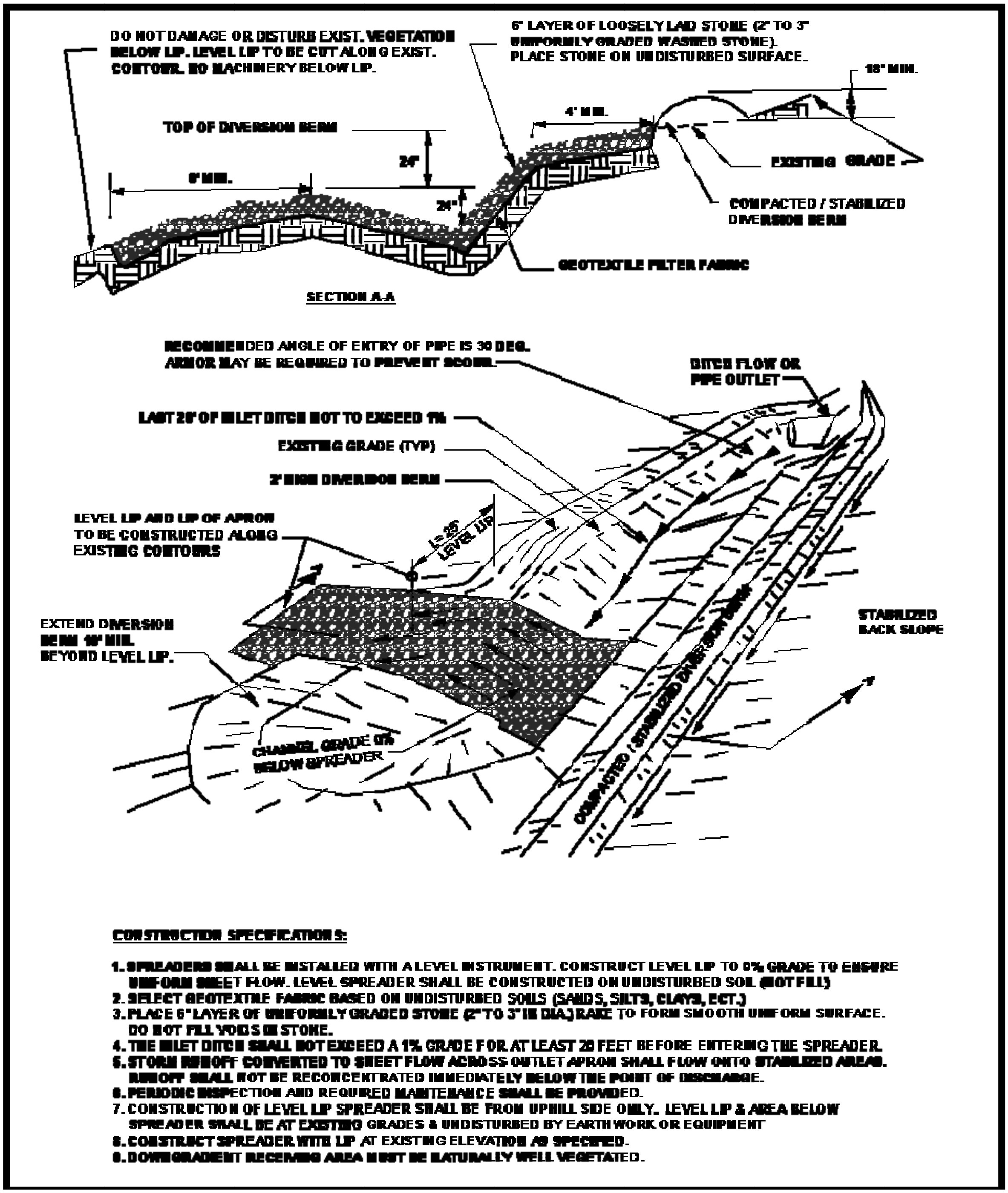


Figure 5-63 Level Spreader

(Maine Erosion and Sediment Control BMP Manual, 2003)

- The receiving area below the level spreader shall be protected from harm during construction. Minor disturbed areas shall be stabilized with vegetative measures. A temporary stormwater diversion may be necessary until the level spreader has fully stabilized.

- Level spreaders must blend smoothly into the downstream receiving area without any sharp drops or irregularities, to avoid channelization, turbulence and hydraulic "jumps."

Table 5-16 Level Flow Spreader Dimensions

Design Flow (cfs)	Minimum Entrance Width (ft)	Minimum Depth (ft)	Minimum End Width (ft)	Minimum Length (ft)
0-10	10	0.5	3	10
10-20	16	0.6	3	20
20-30	24	0.7	3	30

Source: NCDENR, 2006

- Level spreaders shall be constructed on undisturbed soil where possible. If fill is used, it shall be constructed of material compacted to 95% of standard proctor test levels for the area not considered the seedbed.
- Immediately after level spreader construction, seed and mulch the entire disturbed area of the spreader.

The level spreader lip shall be protected with erosion resistant material to prevent erosion and allow vegetation to be established. Alternatively, the lip may be constructed of concrete on a prepared foundation to ensure that it is level and erosion resistant.