DESIGN GUIDE – Introduction to Highway Hydraulics Part 2 of 2

Florida Board of Professional Engineers

Approved Course No. 0010329

4 PDH Hours

A test is provided to assess your comprehension of the course material – 24 questions have been chosen from each of the above sections. You will need to answer at least 17 out of 24 questions correctly (>70%) in order to pass the overall course. You can review the course material and re-take the test if needed.

You are required to review each section of the course in its entirety. Because this course information is part of your Professional Licensure requirements it is important that your knowledge of the course contents and your ability to pass the test is based on your individual efforts.

Course Description:

This course is based entirely on the information published in a technical report prepared by National Highway Institute (NHI) and Office of Bridge Technology (HIBT). This course is 1 of a 2 Part Series that will provide an introduction to highway hydraulics. Inside the report is a discussion of the design highway drainage facilities and should be particularly useful for designers and engineers without extensive drainage training or experience. A review of fundamental hydraulic concepts is provided, including continuity, energy, momentum, hydrostatics, and weir flow followed by a more detailed review of open channel and closed conduit applications.

PART 1 of the Series will cover Chapters 1 through 4 of the technical report

PART 2 of the Series will cover Chapters 5 through 13 of the technical report (this course)

How to reach Us …

If you have any questions regarding this course or any of the content contained herein you are encouraged to contact us at Easy-PDH.com. Our normal business hours are Monday through Friday, 10:00 AM to 4:00 PM; any inquiries will be answered within 2 days or less. Contact us by:

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Refer to Course No. 0010329

DESIGN GUIDE – Introduction to Highway Hydraulics Part 2 of 2

How the Course Works…

What do you want To do?	LOOK For This!
Search for Test Questions and the relevant review section	Q1 Search the PDF for: Q1 for Question 1, Q2 for Question 2, Q3 for Question 3, Etc… (Look for the icon on the left to keep you ON Target!)

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24 QUESTIONS

Q1:	**The capacity of a drainage channel depends upon all of the following EXCEPT:**

(A)	Shape
(B)	Slope
(C)	Configuration
(D)	Size

Q2:	**Roadside channels are commonly used with uncurbed roadways to:**

(A)	Convey runoff from the highway pavement
(B)	Slow the convenance of runoff from the highway pavement
(C)	Convey runoff from areas that drain toward the highway
(D)	A and C

Q3:	**Which type of lining material is able to conform to changes in channel shape and maintains the overall integrity of the channel:**

(A)	Flexible linings
(B)	Conforming linings
(C)	Elastic linings
(D)	Polymeric linings

Q4:	**Which Hydraulic Engineering Circular Number details the tractive force channel design procedure for vegetative linings:**

(A)	HEC-10
(B)	HEC-11
(C)	HEC-15
(D)	HEC-21

Q5:	**It is important to remove water on pavement in an expeditious and efficient manner for the following:**

(A)	Water slows traffic
(B)	Water contributes to hydroplaning
(C)	Water lessens visibility from splash and spray
(D)	All of the above

Q6:	**The triangular shaped area defined by the curb, gutter and the spread onto the pavement creates:**

(A)	An Open channel section for conveying runoff
(B)	A holding area for runoff
(C)	A counter measure for reducing spread onto the pavement
(D)	A and C

Q7:	Potential clogging can occur in sag locations and this type of inlet is not recommended:

(A)	Curb open inlet
(B)	Combination inlet
(C)	Grate Inlet
(D)	Slotted drain inlet

Q8:	Which Hydraulic Engineering Circular Number provides detailed information on bridge deck drainage design:

(A)	HEC-10
(B)	HEC-11
(C)	HEC-15
(D)	HEC-21

Q9:	Flow conditions in a closed conduit can occur as ALL of the following EXCEPT:

(A)	Vacuum induced Flow
(B)	Open Channel flow
(C)	Gravity full flow
(D)	Pressure flow

Q10:	All of the following Hydraulic structures along a closed conduit contribute to form losses EXCEPT:

(A)	Access holes
(B)	Bends
(C)	Inlet grating
(D)	Pipe transition

Q11:	Flows entering a structure from an inlet can be treated as what type of flow for calculation of additional energy losses:

(A)	Inlet flow
(B)	Plunging flow
(C)	Converging flow
(D)	Expansion flow

Q12:	For storm drains smaller than about 48 inches access is required about how many feet:

(A)	Every 200 feet
(B)	Every 300 feet
(C)	Every 400 feet
(D)	Every 600 feet

Q13:	For self cleaning purposes, storm drains should be designed at full flow to have a velocity of (feet per second):

(A)	1.5
(B)	3.0
(C)	3.5
(D)	4.0

Q14:	The starting point for determination of a hydraulic grade line should be:

(A)	Outlet and work up stream
(B)	Outlet and work down stream
(C)	Inlet and work up stream
(D)	Inlet and work down stream

Q15:	Typical pipe materials used in storm drains include ALL of the following EXCEPT:

(A)	Reinforced concrete pipe
(B)	Corrugated metal pipe
(C)	Cast iron pipe
(D)	Plastic pipe

Q16:	Commonly used culvert shapes include:

(A)	Circular
(B)	Elongated
(C)	Elliptical
(D)	A and C

Q17:	What can be done to improve the discharge capacity of a culvert:

(A)	Gradual transition at the outlet of a culvert
(B)	Gradual transition at the inlet of a culvert
(C)	Sharp transition at the outlet of a culvert
(D)	Sharp transition at the inlet of a culvert

Q18:	Inlet control for a culvert occurs when WHAT is capable of conveying more flow than the inlet will accept:

(A)	Culvert barrel
(B)	Culvert grating
(C)	Culvert side wall
(D)	Culvert invert elevation

Q19:	Headwater depth is defined as:

(A)	Depth of water at culvert exit
(B)	Depth of water at culver entrance
(C)	Depth of water at largest culvert cross section
(D)	Depth of water at smallest culvert cross section

Q20:	Culvert inlet improvements that can reduce contraction of flow include:

(A)	Bevel-edged inlets
(B)	Side tapered outlets
(C)	Slope tapered outlets
(D)	Inlet gratings

Q21:	Erosion risks may be greater at flow rates less than the design discharge due to:

(A)	depth of ponding at the inlet is greater
(B)	depth of ponding at the inlet will be less
(C)	Higher Scour potential
(D)	Exposed culvert edges

Q22:	What type of structure is commonly used at culvert outfalls:

(A)	Stilling wells
(B)	Drop structures
(C)	Riprap stilling basins
(D)	Impact basin

Q23:	When placing an unformed slab on a slope the concrete should not be stiffer than what slump (inches)

(A)	1.0
(B)	1.5
(C)	2.0
(D)	2.5

Q24:	Refer to table B-1, DRAINAGE DESIGN CHARTS AND TABLES - the Rational Method Runoff Coefficient value for unimproved areas is:

(A)	0.10 to 0.25
(B)	0.10 to 0.30
(C)	0.25 to 0.35
(D)	0.30 to 0.50

End of Test Questions

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Hydraulic Design Series No. 4

Introduction to Highway Hydraulics

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16. Abstract

Hydraulic Design Series No. 4 provides an introduction to highway hydraulics. Hydrologic techniques presented concentrate on methods suitable to small areas, since many components of highway drainage (culverts, storm drains, ditches, etc.) service primarily small areas. A brief review of fundamental hydraulic concepts is provided, including continuity, energy, momentum, hydrostatics, weir flow and orifice flow. The document then presents open channel flow principles and design applications, followed by a parallel discussion of closed conduit principles and design applications. Open channel applications include discussion of stable channel design and pavement drainage. Closed conduit applications include culvert and storm drain design. Examples are provided to help illustrate important concepts. An overview of energy dissipators is provided and the document concludes with a brief discussion of construction, maintenance and economic issues.

As the title suggests, Hydraulic Design Series No. 4 provides only an introduction to the design of highway drainage facilities and should be particularly useful for designers and engineers without extensive drainage training or experience. More detailed information on each topic discussed is provided by other Hydraulic Design Series and Hydraulic Engineering Circulars.

This publication is an update of the third edition. Revisions were necessary to reflect new information given in the third edition of HEC-14 (Hydraulic Design of Energy Dissipators for Culverts and Channels), the third edition of HEC-15 (Design of Roadside Channels with Flexible Linings), and the third edition of HEC-22 (Urban Drainage Design Manual).

17. Key Words **Hydrology, hydraulics, highway drainage, open channels, roadside ditches, pavement drainage, inlets, closed conduits, culverts, storm drains, energy dissipators**		18. Distribution Statement **This document is available to the public from the National Technical Information Service, Springfield, Virginia 22161**	
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CHAPTER 5

OPEN-CHANNEL APPLICATIONS – STABLE CHANNEL DESIGN

5.1 GENERAL DESIGN CONCEPTS

Q1

The capacity of a drainage channel depends upon its shape, size, slope, and roughness. For a given channel, the capacity becomes greater when the grade or the depth of flow is increased. The channel capacity decreases as the channel surface becomes rougher. For example, a riprap-lined ditch has only about half the capacity of a concrete-lined ditch of the same size, shape, and slope because of the differences in channel roughness. A rough channel is sometimes an advantage on steep slopes where it is desirable to keep velocities from becoming too high.

The most efficient shape of channel is that of a semi-circle, but hydraulic efficiency is not the sole criterion. In addition to performing its hydraulic function, the drainage channel should be economical to construct and require little maintenance during the life of the roadway. Channels should also be safe for vehicles accidentally leaving the traveled way, pleasing in appearance, and dispose of collected water without damage to the adjacent property. Most of these additional requirements for drainage channels reduce the hydraulic capacity of the channel. The best design for a particular section of highway is a compromise among the various requirements, sometimes with each requirement having a different influence on the design. Figure 5.1 illustrates the preferred geometric cross section for ditches with gradual slope changes in which the front and back slopes are traversable (AASHTO 2002). This figure is applicable for rounded ditches with bottom widths of 2.4 m (8 ft) or more, and trapezoidal ditches with bottom widths equal to or greater than 1.2 m (4 ft).

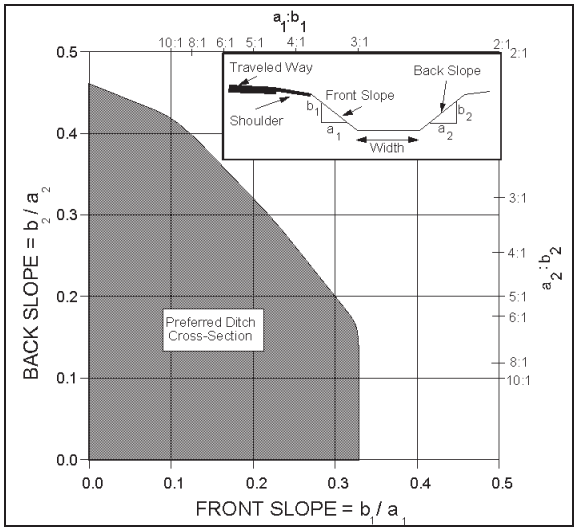

Figure 5.1. Preferred ditch cross section geometry (AASHTO 2002).

The width of the right-of-way usually allows little choice in the alignment or in the grade of the channel, but as far as practicable, abrupt changes in alignment or in grade should be avoided. A sharp change in alignment presents a point of attack for the flowing water and abrupt changes in grade cause deposition of transported material when the grade is flattened or scour when grade is steepened.

It is unnecessary to standardize the design of roadway drainage channels for any length of the highway. Not only can the depth and breadth of the channel be varied with variation in the amounts of runoff, channel grade, and distance between lateral outfall culverts, but the dimensions can be varied by the use of different types of channel lining. Nor is it necessary to standardize the lateral distance between the channel and the edge of pavement. Often liberal offsets can be obtained where cuts are slight and where cuts end and fills begin.

Systematic maintenance is essential to any drainage channel. Without proper maintenance, a well-designed channel can become an unsightly gully. Maintenance methods should be considered in the design of drainage channels so that the channel sections will be suitable for the methods and equipment that will be used for their maintenance (see Chapter 12).

5.2 STABLE CHANNEL DESIGN CONCEPTS

Q2

Roadside channels are commonly used with uncurbed roadways to convey runoff from the highway pavement and from areas that drain toward the highway. A channel section may also be used with curbed highway sections to intercept off-pavement drainage in order to minimize deposition of sediment and other debris on the roadway and to reduce the amount of water that must be carried by the roadway section. The gradient of roadside channels typically parallels the grade of the highway. Even at relatively mild highway grades, highly erosive hydraulic conditions can exist in adjacent roadside channels. Consequently, designing a stable conveyance becomes a critical component in the design of roadside channels.

The need for erosion prevention is not limited to the highway drainage channels; it extends throughout the right-of-way and is an essential feature of adequate drainage design. Erosion and maintenance are minimized largely by the use of flat sideslopes rounded and blended with natural terrain, drainage channels designed with due regard to location, width, depth, slopes, alignment, and protective treatment, proper facilities for groundwater interception, dikes, berms, and other protective devices, and protective ground covers and planting.

The discussion in this chapter is limited to providing erosion control in drainage channels by proper design, including the selection of an economical channel lining. Lining as applied to drainage channels includes vegetative coverings. The type of lining should be consistent with the degree of protection required, overall cost, safety requirements, and esthetic considerations. Control of erosion caused by overland or sheet flow is not discussed.

5.3 LINING MATERIALS

Q3

Lining materials may be classified as flexible or rigid. Flexible linings are able to conform to changes in channel shape and can sustain such changes while maintaining the overall integrity of the channel. In contrast, rigid linings cannot change shape and tend to fail when a portion of the channel lining is damaged. Channel shape may change due to frost-heave, slumping, piping, etc. Typical flexible lining materials include grass and riprap (Figures 5.2 and 5.3), while a typical rigid lining material is concrete (Figure 5.4). Flexible linings are generally less expensive, have a more natural appearance, and are typically more environmentally acceptable. However, flexible linings are limited in the erosive forces they can sustain without damage to the channel and lining. A rigid lining can typically provide higher capacity and greater erosion resistance and in some cases may be the only feasible alternative.

Figure 5.2. Vegetative channel lining.

Figure 5.3. Riprap channel lining.

Figure 5.4. Rigid concrete channel lining.

Flexible linings can be either long-term, transitional or temporary. Long-term flexible linings are used where the channel requires protection against erosion for the life of the channel. Long-term lining materials include vegetation, cobbles, rock riprap, wire-enclosed riprap, and turf reinforcement. Transitional flexible linings are used to provide erosion protection until a long-term lining, such as grass, can be established. Temporary channel linings are used without vegetation to line channels that might be part of a construction site or some other short-term channel situation. Turf reinforcement can serve either a transitional or long-term function by providing additional structure to the soil/vegetation matrix. Typical turf reinforcement materials include gravel/soil mixes and turf reinforcement mats (TRM's). A TRM is a non-degradable rolled erosion control product (RECP) processed into a three-dimensional matrix. A TRM is stiffer, thicker and denser than an erosion control blanket (ECB), which is typically a degradable RECP composed of an even distribution of natural or polymer fibers bound together to form a continuous mat. Open-weave textiles (OWT) are a degradable RECP composed of natural or polymer yarns more loosely woven into a matrix. RECP's are laid in the channel and secured with staples or stakes (Figure 5.5).

Figure 5.5. Installed TRM channel lining.

Construction of rigid concrete linings requires specialized equipment and costly materials. As a result the cost of rigid linings is typically high. Prefabricated linings can be a less expensive alternative if shipping distances are not excessive. Interlocking concrete paving blocks are a typical prefabricated lining.

In general, when a lining is needed, the lowest cost lining that affords satisfactory protection should be used. In humid regions, this is often vegetation used alone or in combination with other types of linings. Thus, a channel might be grass-lined on the flatter slopes and lined with more resistant material on the steeper slopes. In cross section, the channel might be lined with a highly resistant material within the depth required to carry floods occurring frequently and lined with grass above that depth for protection from the rare floods.

5.4 STABLE CHANNEL DESIGN PROCEDURE

Stable channel design can be based on the concepts of static or dynamic equilibrium. Static equilibrium exists when the channel boundaries are essentially rigid and the material forming the channel boundary effectively resists the erosive forces of the flow. Under such conditions the channel remains essentially unchanged during the design flow and the principles of rigid boundary hydraulics can be applied. Dynamic equilibrium exists when the channel boundary is moveable and some change in the channel bed and/or banks occurs. A dynamic system is considered stable as long as the net change does not exceed acceptable levels.

Designing a stable channel under dynamic equilibrium conditions must be based on the concepts of sediment transport. For most highway drainage channels bed and bank instability and/or possible lateral migration cannot be tolerated and stable channel design must be based on the concepts of static equilibrium, including the use of a lining material if necessary to achieve a rigid boundary condition.

Two methods have been developed and are commonly applied to design static equilibrium channel conditions: the permissible velocity approach and the permissible tractive force (shear stress) approach. Under the permissible velocity approach the channel is assumed stable if the adopted mean velocity is lower than the maximum permissible velocity for the given channel boundary condition. Similarly, the tractive force approach requires that the shear stresses on the channel bed and banks do not exceed the allowable amounts for the given channel boundary. Permissible velocity procedures were first introduced around the 1920s and have been developed and widely used by the Soil Conservation Service, now the Natural Resource Conservation Service. Tractive force procedures based on shear stress concepts (see Chapter 4) originated largely through research by the Bureau of Reclamation in the 1950s. Based on the actual physical processes involved in maintaining a stable channel, specifically the stresses developed at the interface between flowing water and materials forming the channel boundary, the tractive force procedure is a more realistic model and was adopted as the preferred design procedure for flexible linings in Hydraulic Engineering Circular Number 15, entitled "Design of Roadside Channels with Flexible Linings" (HEC-15), which is the primary reference for stable channel design (Kilgore and Cotton 2005).

The definition and equation for computing the tractive force was provided in Chapter 4 (Equation 4.13). This equation defines the average tractive force on the channel. The maximum shear stress along the channel bottom may be estimated by substituting the flow depth, y, for the hydraulic radius, R, in Equation 4.13. To be consistent with HEC-15, the flow depth will be represented by "d" in the following discussion, and the resulting equation is:

$$\tau_d = \gamma\, d\, S \tag{5.1}$$

where:

τ_d	=	Shear stress in channel at maximum depth, N/m^2 (lb/ft^2)
γ	=	Specific weight of water
d	=	Maximum depth of flow in channel for the design discharge, m (ft)
S	=	Slope of channel, m/m (ft/ft)

Flexible linings act to reduce the shear stress on the underlying soil surface. Therefore, the erodibility of the underlying soil is a key factor in the performance of flexible linings. Erodibility of non-cohesive soils (plasticity index less than 10) is mainly due to particle size, while cohesive soils is a function of cohesive strength and soil density. Vegetative and RECP lining performance relates to how well they protect the underlying soil from shear stress, and so these lining types do not have permissible shear stresses independent of soil type.

When the permissible shear stress is greater than or equal to the computed shear stress, the lining is considered acceptable

$$\tau_p \geq SF\tau_d \tag{5.2}$$

where:

τ_p = Permissible shear stress for the channel lining, N/m^2 (lb/ft^2)
SF = Safety factor

The safety factor provides for a measure of uncertainty and failure tolerance, and typically ranges from 1.0 to 1.5.

The basic procedure for designing a flexible lining consists of the following steps, which are summarized in Figure 5.6 (from HEC-15).

Step 1. Determine a design discharge, Q, and select the channel slope and channel shape.

Step 2. Select a trial lining type. Initially, the designer may need to determine if a long-term lining is needed and whether or not a temporary or transitional lining is required. For determining the latter, the trial lining type could be chosen as the native material (unlined), typically bare soil. For example, it may be determined that the bare soil is insufficient for a long-term solution, but vegetation is a good solution. For the transitional period between construction and vegetative establishment, analysis of the bare soil will determine if a temporary lining is prudent.

Step 3. Estimate the depth of flow, d_i in the channel and compute the hydraulic radius, R. The estimated depth may be based on physical limits of the channel, but this first estimate is essentially a guess. Iterations on Steps 3 through 5 may be required.

Step 4. Estimate Manning's n and the discharge implied by the estimated n and flow depth values. Calculate the discharge (Q_i).

Step 5. Compare Q_i with Q. If Q_i is within 5 percent of the design, Q, then proceed on to Step 6. If not, return to Step 3 and select a new estimated flow depth, d_{i+1}. This can be estimated from the following equation or any other appropriate method.

$$d_{i+1} = d_i \left(\frac{Q}{Q_i} \right)^{0.4}$$

Step 6. Calculate the shear stress at maximum depth, τ_d (Equation 5.1), determine the permissible shear stress, τ_p, according to the methods described in HEC-15 and select an appropriate safety factor.

Step 7. Compare the permissible shear stress to the calculated shear stress from Step 6 using Equation 5.2. If the permissible shear stress is adequate then the lining is acceptable. If the permissible shear is inadequate, then return to Step 2 and select an alternative lining type with greater permissible shear stress. As an alternative, a different channel shape may be selected that results in a lower depth of flow.

The selected lining is stable and the design process is complete. Other linings may be tested, if desired, before specifying the preferred lining.

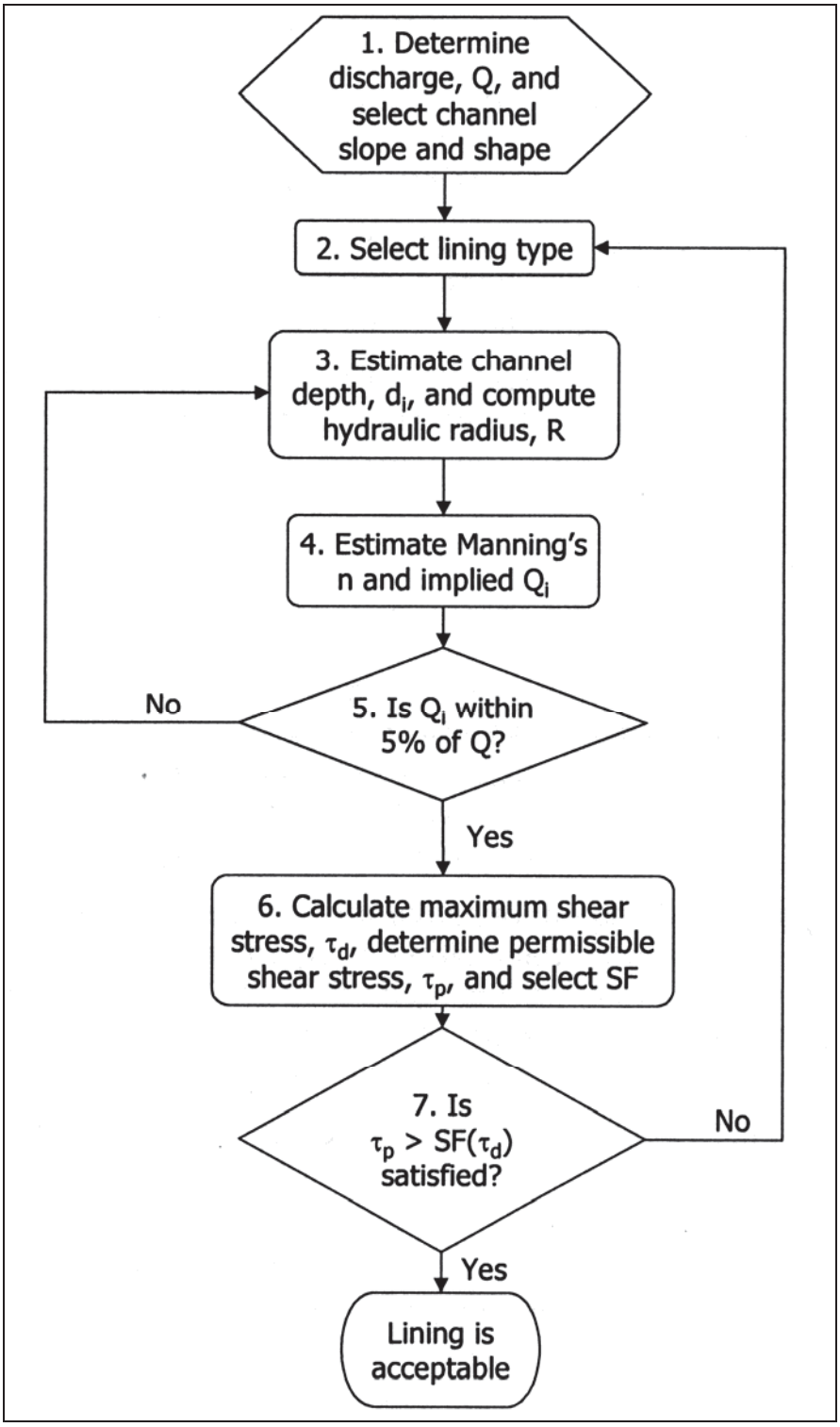

Figure 5.6. Flexible channel lining design flow chart (from HEC-15).

5.5 GRASS LINED STABLE CHANNEL DESIGN EQUATIONS

Q4

HEC-15 details the tractive force stable channel design procedure for vegetative linings, RECP's, riprap/cobble, and gabion linings. Information on special considerations for steep-slope riprap design and design of composite linings is also included. For purposes of illustration, the equations necessary to design a vegetative lining on a cohesive soil are given below. To design other lining types, refer to HEC-15.

The Manning's roughness coefficient for grass linings varies depending on grass properties and shear stress given that the roughness changes as the grass stems bend under flow. The equation describing the n value for grass linings is

$$n = \alpha\, C_n\, \tau_o^{-0.4} \tag{5.3}$$

where:

τ_o = Average boundary shear stress, N/m^2 (lb/ft^2) based on Equation 4.13
α = Unit conversion constant, 1.0 (0.213)
C_n = Grass roughness coefficient (Typical values in Table 5.1; site-specific values can be calculated using HEC-15)

Table 5.1. Grass Roughness Coefficient, C_n.

Stem Height m (ft)	Excellent	Very Good	Good	Fair	Poor
0.075 (0.25)	0.168	0.157	0.142	0.122	0.111
0.150 (0.50)	0.243	0.227	0.205	0.177	0.159
0.225 (0.75)	0.301	0.281	0.254	0.219	0.197

The soil grain roughness, n_s, is 0.016 when D_{75} < 1.3 mm (0.05 in). For larger grained soils the soil grain roughness is

$$n_s = \alpha\, (D_{75})^{1/6} \tag{5.4}$$

where:

n_s = Soil grain roughness (D_{75} > 1.3 mm (0.05 in))
D_{75} = Soil size where 75 percent of the material is finer, mm (in)
α = Unit conversion constant, 0.015 (0.026)

The permissible shear stress for cohesive soils is:

$$\tau_{p,\,soil} = [c_1\,(PI)^2 + c_2\,(PI) + c_3][c_4 + c_5\,(e)]^2\, c_6 \tag{5.5}$$

where:

$\tau_{p,\,soil}$ = Soil permissible shear stress, N/m^2 (lb/ft^2)
PI = Plasticity index
e = Void ratio
c_i = Coefficients as defined by Table 5.2

Table 5.2. Coefficients for Permissible Soil Shear Stress.								
ASTM Soil Classification[1]	Applicable Range	C_1	C_2	C_3	C_4	C_5	C_6 (SI)	C_6 (English)
GM	$10 \le PI \le 20$ $20 \le PI$	1.07	14.3	47.7 0.076	1.42 1.42	-0.61 -0.61	4.8×10^{-3} 48.	10^{-4} 1.0
GC	$10 \le PI \le 20$ $20 \le PI$	0.0477	2.86	42.9 0.119	1.42 1.42	-0.61 -0.61	4.8×10^{-2} 48.	10^{-3} 1.0
SM	$10 \le PI \le 20$ $20 \le PI$	1.07	7.15	11.9 0.058	1.42 1.42	-0.61 -0.61	4.8×10^{-3} 48.	10^{-4} 1.0
SC	$10 \le PI \le 20$ $20 \le PI$	1.07	14.3	47.7 0.076	1.42 1.42	-0.61 -0.61	4.8×10^{-3} 48.	10^{-4} 1.0
ML	$10 \le PI \le 20$ $20 \le PI$	1.07	7.15	11.9 0.058	1.48 1.48	-0.57 -0.57	4.8×10^{-3} 48.	10^{-4} 1.0
CL	$10 \le PI \le 20$ $20 \le PI$	1.07	14.3	47.7 0.076	1.48 1.48	-0.57 -0.57	4.8×10^{-3} 48.	10^{-4} 1.0
MH	$10 \le PI \le 20$ $20 \le PI$	0.0477	14.3	10.7 0.058	1.38 1.38	-0.373 -0.373	4.8×10^{-2} 48.	10^{-3} 1.0
CH	$20 \le PI$			0.097	1.38	-0.373	48.	1.0

[1]NOTE: Typical Names

GM Silty gravels, gravel-sand silt mixtures
GC Clayey gravels, gravel-sand-clay mixtures
SM Silty sands, sand-silt mixtures
SC Clayey sands, sand-clay mixtures
ML Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
CL Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
MH Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
CH Inorganic clays of high plasticity, fat clays

The combined effects of the soil permissible shear stress and the effective shear stress transferred through the vegetative lining results in a permissible shear stress for the given conditions. The resulting equation is:

$$\tau_p = \frac{\tau_{p,soil}}{(1 - C_f)} \left(\frac{n}{n_s} \right)^2 \tag{5.6}$$

where:

τ_p = Permissible shear stress, N/m^2 (lb/ft^2)
$\tau_{p,soil}$ = Soil permissible shear stress, N/m^2 (lb/ft^2), see Equation 5.5
n_s = Soil grain roughness, see Equation 5.4
n = Overall lining roughness, see Equation 5.3
C_f = Grass cover coefficient as defined by Table 5.3

Table 5.3. Cover Factor Values for Uniform Stands of Grass.					
Growth Form	Cover Factor, C_f				
	Excellent	Very Good	Good	Fair	Poor
Sod	0.98	0.95	0.90	0.84	0.75
Bunch	0.55	0.53	0.50	0.47	0.41
Mixed	0.82	0.79	0.75	0.70	0.62

When the permissible shear stress is greater than or equal to the computed shear stress, as defined by Equation 5.2, the lining is considered acceptable. Example 5.1 illustrates the calculation procedure for a grass lined channel in a cohesive soil.

EXAMPLE PROBLEM 5.1 – Grass Lining Design (SI Units)

Evaluate a grass lining for a roadside channel given the following channel shape, soil conditions, grade, and design flow. It is expected that the grass lining will be maintained in good condition in the spring and summer months, which are the main storm seasons.

Given:

Shape:	Trapezoidal, B = 0.9 m, Z = 3
Soil:	Clayey sand (SC classification), PI = 16, e = 0.5, D_{75} < 1.3 mm
Grass:	Sod, height = 0.075 m
Grade:	3.0 percent
Flow:	0.5 m^3/s

Solution:

The solution is accomplished using the step-by-step procedure given in Section 5.4.

Step 1. Channel slope, shape, and discharge have been given.

Step 2. A vegetative lining on a clayey sand soil will be evaluated.

Step 3. Initial depth is estimated at 0.30 m

From the geometric relationship of a trapezoid (see Section 4.3.2):

$$A = Bd + Zd^2 = 0.9(0.3) + 3(0.3)^2 = 0.540 \text{ m}^2$$

$$P = B + 2d\,(Z^2 + 1)^{1/2} = 0.9 + 2\,(0.3)\,(3^2 + 1)^{1/2} = 2.80 \text{ m}$$

$$R = A/P = 0.54/2.8 = 0.193 \text{ m}$$

Step 4. To estimate n, the applied shear stress on the grass lining is given by Equation 4.13.

$$\tau_o = \gamma\, R\, S_o = 9810(0.193)(0.03) = 56.8 \text{ N/m}^2$$

Determine a Manning's n value from Equation 5.3. From Table 5.1,

$$C_n = 0.142$$

$$n = \alpha\, C_n\, \tau_o^{-0.4} = 1.0(0.142)(56.8)^{-0.4} = 0.028$$

The discharge is calculated using the Manning's equation (Equation 4.5):

$$Q = (K_u/n)\, A\, R^{2/3}\, S_f^{1/2} = (1.0/0.028)(0.540)(0.193)^{2/3}\,(0.03)^{1/2} = 1.12 \text{ m}^3/\text{s}$$

Step 5. Since this value is more than 5 percent different from the design flow, we need to go back to Step 3 to estimate a new flow depth.

Step 3. (2nd iteration). Estimate a new depth:

$$d_{i+1} = d_i\,(Q / Q_i)^{0.4} = 0.3\,(0.5 / 1.12)^{0.4} = 0.22 \text{ m}$$

So, try d = 0.22 m

Revise the hydraulic radius

$$A = Bd + Zd^2 = 0.9(0.22) + 3(0.22)^2 = 0.343 \text{ m}^2$$

$P = B + 2d(Z^2 + 1)^{1/2} = 0.9 + 2(0.22)\ (3^2 + 1)^{1/2} \quad = 2.29$ m

$R = A/P = 0.343/2.29 = 0.150$ m

Step 4. (2nd iteration). To estimate n, the applied shear stress on the grass lining is given by Equation 4.13.

$\tau_o = \gamma\ RS_o = 9810(0.150)(0.03) = 44.2\ N/m^2$

Determine a Manning's n value from Equation 5.3. From Table 5.1,

$C_n = 0.142$

$n = \alpha\ C_n\ \tau_o^{-0.4} = 1.0\ (0.142)(44.2)^{-0.4} = 0.031$

The discharge is calculated using the Manning's equation (Equation 4.5):

$= (K_u/n)\ A\ R^{2/3}\ S_f^{1/2} = (1/0.031)\ (0.343)\ (0.150)^{2/3}\ (0.03)^{1/2} = 0.54\ m^3/s$

Step 5. (2nd iteration). Since this value is more that 5 percent different from the design flow we need to go back to Step 3 to estimate a new flow depth.

Step 3. (3rd iteration). Estimate a new depth:

$d_{i+1} = d_i\ (Q\ /\ Q_i)^{0.4} = 0.22\ (0.5\ /\ 0.54)^{0.4} = 0.21$ m

So, try d = 0.21 m

Revise the hydraulic radius

$A = Bd + Zd^2 = 0.9(0.21) + 3(0.21)^2 = 0.321\ m^2$

$P = B + 2d(Z^2 + 1)^{1/2} = 0.9 + 2(0.21)\ (3^2 + 1)^{1/2} \quad = 2.23$ m

$R = A/P = 0.321/2.23 = 0.144$ m

Step 4. (3rd iteration). To estimate n, the applied shear stress on the grass lining is given by Equation 4.13.

$\tau_o = \gamma\ RS_o = 9810(0.144)(0.03) = 42.4\ N/m^2$

Determine a Manning's n value from Equation 5.3. From Table 5.1,

$C_n = 0.142$

$n = \alpha C_n\ \tau_o^{-0.4} = 1.0(0.142)(42.4)^{-0.4} = 0.032$

The discharge is calculated using the Manning's equation (Equation 4.5):

$Q = (K_u/n)\ A\ R^{2/3}\ S_f^{1/2} = (1/0.032)\ (0.321)\ (0.144)^{2/3}\ (0.03)^{1/2} = 0.48\ m^3/s$

Step 5. (3rd iteration). Since this value is within 5 percent of the design flow, we can proceed to Step 6.

Step 6. The maximum shear on the channel bottom is.

$\tau_d = \gamma d S_o = 9810(0.21)(0.03) = 61.8\ N/m^2$

Determine the permissible soil shear stress from Equation 5.5.

$\tau_{p,soil} = (c_1\ PI^2 + c_2\ PI + c_3)\ (c_4 + c_5\ e)^2\ c_6$

$\tau_{p,soil} = [1.07(16)^2 + 14.3(16) + 47.7]\ [1.42 – 0.61\ (0.5)]^2\ (0.0048) = 3.28\ N/m^2$

Equation 5.6 gives the permissible shear stress on the vegetation. The value of

grass cover factor, C_f, is found in Table 5.3, and the soil grain roughness n_s, is 0.016 since D_{75} < 1.3 mm.

$$\tau_p = ((\tau_{p,\,soil}) / (1 - C_f))\ (n / n_s)^2 = (3.28 / (1 - 0.9))\ (0.032 / 0.016)^2 = 131\ N/m^2$$

The safety factor for this channel is taken as 1.0.

Step 7. The grass lining is acceptable since the maximum shear on the vegetation (61.8 N/m^2) is less than the permissible shear of 131 N/m^2.

EXAMPLE PROBLEM 5.1 Grass Lining Design (English Units)

Evaluate a grass lining for a roadside channel given the following channel shape, soil conditions, grade, and design flow. It is expected that the grass lining will be maintained in good condition in the spring and summer months, which are the main storm seasons.

Given:

Shape:	Trapezoidal, B = 3.0 ft, Z = 3
Soil:	Clayey sand (SC classification), PI = 16, e = 0.5, D_{75} < 0.05 in
Grass:	Sod, height = 0.25 ft
Grade:	3.0 percent
Flow:	17.5 ft^3/s

Solution:

The solution is accomplished using the step-by-step procedure given in Section 5.4.

Step 1. Channel slope, shape, and discharge have been given.

Step 2. A vegetative lining on a clayey sand soil will be evaluated.

Step 3. Initial depth is estimated at 1.0 ft

From the geometric relationship of a trapezoid (see Section 4.3.2):

$$A = Bd + Zd^2 = 3.0(1.0) + 3(1.0)^2 = 6.00\ ft^2$$

$$P = B + 2d\ (Z^2 + 1)^{1/2} = 3.0 + 2(1.0)\ (3^2 + 1)^{1/2} = 9.32\ ft$$

$$R = A/P = 6.00/9.32 = 0.64\ ft$$

Step 4. To estimate n, the applied shear stress on the grass lining is given by Equation 4.13

$$\tau_o = \gamma\ R\ S_o = 62.4(0.64)(0.03) = 1.21\ lb/ft^2$$

Determine a Manning's n value from Equation 5.3. From Table 5.1,

$$C_n = 0.142$$

$$n = \alpha\ C_n\ \tau_o^{-0.4} = 0.213(0.142)(1.21)^{-0.4} = 0.028$$

The discharge is calculated using the Manning's equation (Equation 4.5):

$$Q = (K_u/n)\ A\ R^{2/3}\ S_f^{1/2} = (1.49/0.028)\ (6.0)\ (0.644)^{2/3}\ (0.03)^{1/2} = 41.2\ ft^3/s$$

Step 5. Since this value is more than 5 percent different from the design flow, we need to go back to Step 3 to estimate a new flow depth.

Step 3. (2nd iteration). Estimate a new depth:

$d_{i+1} = d_i\ (Q / Q_i)^{0.4} = 1.0\ (17.5 / 41.2)^{0.4} = 0.71$ ft

So, try d = 0.71 ft

Revise the hydraulic radius

$A = Bd + Zd^2 = 3.0(0.71) + 3(0.71)^2 = 3.64\ ft^2$

$P = B + 2d(Z^2 + 1)^{1/2} = 3.0 + 2(0.71)\ (3^2 + 1)^{1/2} = 7.49$ ft

$R = A/P = 3.64/7.49 = 0.486$ ft

Step 4. (2nd iteration). To estimate n, the applied shear stress on the grass lining is given by Equation 2.3.

$\tau_o = \gamma\ RS_o = 62.4(0.486)(0.03) = 0.91\ lb/ft^2$

Determine a Manning's n value from Equation 5.3. From Table 5.1,

$C_n = 0.142$

$n = \alpha\ C_n\ \tau_o^{-0.4} = 0.213(0.142)(0.91)^{-0.4} = 0.031$

The discharge is calculated using the Manning's equation (Equation 4.5):

$Q = (K_u/n)\ A\ R^{2/3}\ S_f^{1/2} = (1.49/0.031)\ (3.64)\ (0.486)^{2/3}\ (0.03)^{1/2} = 18.7\ ft^3/s$

Step 5. Since this value is more than 5 percent different from the design flow, we need to go back to Step 3 to estimate a new flow depth.

Step 3. (3rd iteration). Estimate a new depth:

$d_{i+1} = d_i\ (Q / Q_i)^{0.4} = 0.71\ (17.5 / 18.7)^{0.4} = 0.69$ ft

So, try d = 0.69 ft

Revise the hydraulic radius

$A = Bd + Zd^2 = 3.0(0.69) + 3(0.69)^2 = 3.50\ ft^2$

$P = B + 2d(Z^2 + 1)^{1/2} = 3.0 + 2(0.69)(3^2 + 1)^{1/2} = 7.36$ ft

$R = A/P = 3.50/7.36 = 0.476$ ft

Step 4. (3rd iteration). To estimate n, the applied shear stress on the grass lining is given by Equation 4.13.

$\tau_o = \gamma\ RS_o = 62.4(0.476)(0.03) = 0.89\ lb/ft^2$

Determine a Manning's n value from Equation 5.3. From Table 5.1,

$C_n = 0.142$

$n = \alpha\ C_n\ \tau_o^{-0.4} = 0.213(0.142)(0.89)^{-0.4} = 0.032$

The discharge is calculated using the Manning's equation (Equation 4.5):

$Q = (K_u/n)\ A\ R^{2/3}\ S_f^{1/2} = (1.49/0.032)\ (3.50)\ (0.476)^{2/3}\ (0.03)^{1/2} = 17.2\ ft^3/s$

Step 5. (3rd iteration). Since this value is within 5 percent of the design flow, we can proceed to Step 6.

Step 6. The maximum shear on the channel bottom is.

$\tau_d = \gamma d S_o = 62.4(0.69)(0.03) = 1.29\ lb/ft^2$

Determine the permissible soil shear stress from Equation 5.5.

$\tau_{p,soil} = (c_1PI^2 + c_2PI + c_3)\ (c_4 + c_5e)^2\ c_6$

$\tau_{p,soil} = (1.07(16)^2 + 14.3(16) + 47.7)(1.42 – 0.61(0.5))^2\ (0.0001) = 0.068\ lb/ft^2$

Equation 5.6 gives the permissible shear stress on the vegetation. The value of grass cover factor, C_f, is found in Table 5.3, and the soil grain roughness is 0.016 since $D_{75} < 0.05$ in.

$\tau_p = ((\tau_{p,\ soil}) / (1 - C_f))\ (n / n_s)^2 = (0.068 / (1 - 0.9))\ (0.032 / 0.016)^2 = 2.7\ lb/ft^2$

The safety factor for this channel is taken as 1.0.

Step 7. Grass lining is acceptable since the maximum shear on the vegetation (1.29 lb/ft^2) is less than the permissible shear of 2.7 lb/ft^2.

CHAPTER 6

OPEN-CHANNEL APPLICATIONS-PAVEMENT DRAINAGE DESIGN

6.1 BASIC CONCEPTS

Q5

Water on the pavement will slow traffic and contribute to accidents from hydroplaning and loss of visibility from splash and spray. Hence, removal of water on pavement in an expeditious and efficient manner is important. Pavement drainage design will provide for effective removal of water from the roadway surface.

Pavement drainage design is typically based on a design discharge and an allowable spread of water across the pavement. Spread on traffic lanes can be tolerated more frequently and to greater widths where traffic volumes and speeds are low. In contrast, high speed, high volume facilities, such as freeways, should be designed to minimize or eliminate spread of water on the traffic lanes during the design event.

Standard roadway geometric design features greatly influence pavement drainage design. These features include curbs, gutter configuration, longitudinal and lateral pavement slope, shoulders and parking lanes. Curbing at the right edge of pavements is normal practice for low-speed, urban highway facilities. Gutters adjacent to the curb combined with a portion of the shoulder or roadway pavement, depending on allowable spread, are used to carry runoff. Water flowing on a pavement with a curb is essentially a special case of open-channel flow in a shallow, triangular-shaped cross section. Pavement drainage to a gutter or swale adjacent to the roadway surface is another case of open-channel flow, again typically occurring in a wide shallow cross section.

Longitudinal grade influences the spread of water onto the pavement. Curbed pavements typically require a minimum slope of 0.3 percent to promote drainage. Minimum grades can be maintained in very flat terrain by use of rolling profiles or by warping the cross slope to achieve a rolling gutter profile. Pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. Adequate cross slope will reduce water depth on the pavement and, therefore, is an important countermeasure against hydroplaning.

In areas where vegetative cover cannot be used to prevent erosion damage to high fills, shoulders are designed to serve as a gutter with a curb constructed at the outer edge to confine the water to the shoulder. Water collected in the gutter is discharged down the slope through down drains. The curb may be made of bituminous or portland cement concrete.

The primary design reference for highway pavement drainage design is HEC-22 (Brown et al. 2008). The following information has been extracted from that document and provides an overview of pavement drainage design concepts and considerations.

6.2 FLOW IN GUTTERS AND SWALES

Q6

The triangular shaped area defined by the curb, gutter and the spread onto the pavement creates an open-channel flow section for conveying runoff (Figure 6.1). Gutters adjacent to the curb can have a steeper cross slope from the pavement. This steeper gutter section can be an effective countermeasure for reducing spread on the pavement.

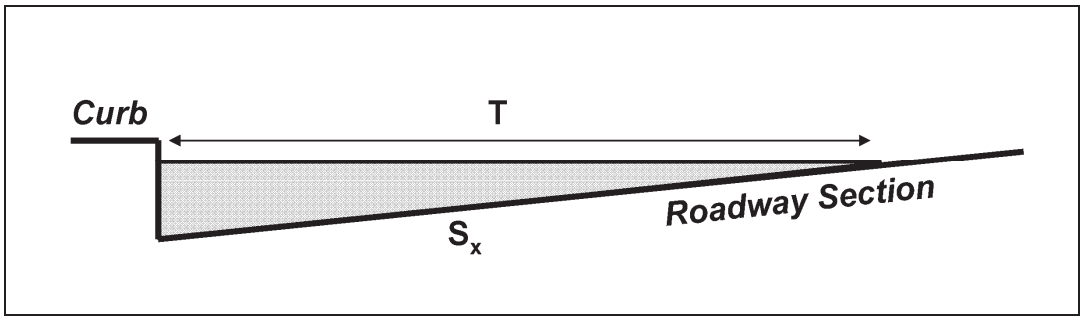

Figure 6.1. Definition sketch - triangular section.

Modification of Manning's equation (Equation 4.5) is necessary for use in computing flow in triangular channels because the hydraulic radius in the equation does not adequately describe the gutter cross section, particularly where the top width of the water surface may be more than 40 times the depth at the curb. To compute gutter flow Manning's equation is integrated for an increment of width across the section, and the resulting equation in terms of cross slope and spread is:

$$Q = (K_u/n)\, S_x^{5/3}\, S^{1/2}\, T^{8/3} \tag{6.1}$$

where:

Q	=	Discharge in m^3/s (ft^3/s)
S_x	=	Cross slope in m/m (ft/ft)
S	=	Longitudinal slope in m/m (ft/ft)
T	=	Spread in m (ft)
K_u	=	Units conversion factor equal to 0.376 (0.56 in English units)

Example 6.1 illustrates the application of this equation. Note that nomographs for solution of Equation 6.1, for both uniform and for the more complex geometry created by composite cross slopes, are provided in HEC-22 (Brown et al. 2008). Tabulated flow capacity for standard highway cross sections may also be available from local and regional design guides.

Swale sections at the edge of the roadway pavement or shoulder offer advantages over curbed sections where curbs are not needed for traffic control. These advantages include less hazard to traffic than a near-vertical curb and hydraulic capacity that is not dependent on spread on the pavement. Swale sections are particularly appropriate where curbs are used to prevent water from eroding fill slopes. Swale sections are typically circular or v-shape. HEC-22 also provides nomograph solution of Manning's equation for shallow swale sections (Brown et al. 2008).

EXAMPLE PROBLEM 6.1 (SI Units)

Given: A gutter with the following dimensions and conditions

$T = 2.5$ m
$S_x = 0.02$
$S = 0.01$
$n = 0.016$

Find: Flow in gutter at design spread

Solution: (Use the modified Manning's equation)

$$Q = (K_u/n)\ S_x^{5/3}\ S^{1/2}\ T^{8/3}$$

$$Q = (0.376/0.016)\ (0.02)^{5/3}\ (0.01)^{1/2}\ (2.5)^{8/3}$$

$$Q = 0.040\ m^3/s$$

EXAMPLE PROBLEM 6.1 (English Units)

Given: A gutter with the following dimensions and conditions

$T = 8.0$ ft
$S_x = 0.02$
$S = 0.01$
$n = 0.016$

Find: Flow in gutter at design spread

Solution: (Use the modified Manning's equation)

$$Q = (K_u/n)\ S_x^{5/3}\ S^{1/2}\ T^{8/3}$$

$$Q = (0.56/0.016)\ (0.02)^{5/3}\ (0.01)^{1/2}\ (8.0)^{8/3}$$

$$Q = 1.32\ ft^3/s$$

6.3 PAVEMENT DRAINAGE INLETS

When the capacity of the curb/gutter/pavement section has been exceeded, typically as a result of spread considerations, runoff must be diverted from the roadway surface. A common solution is often interception of all or a portion of runoff by drainage inlets that are connected to a storm drain pipe. Inlets used for intercepting runoff from highway surfaces can be divided into four major classes: (1) curb-opening inlets, (2) grate inlets, (3) slotted drains, and (4) combination inlets (Figures 6.2 and 6.3). Each class has many variations in design and may be installed with or without a depression of the gutter.

Inlet capacity is a function of a variety of factors, including type of inlet, grate design, location (on grade or in a sag location), gutter design, debris clogging, etc. Inlets on continuous grade operate as weir flow, while inlets in sag locations will initially operate as weir flow but will transition to orifice flow as depth increases (see discussion of weirs and orifices in Chapter 3).

Orifice flow begins at depths dependent on the grate size, the curb opening height, or the slot width of the inlet, depending on the type of inlet/grate. At depths between those where weir flow occurs and those where orifice flow occurs, flow is in a transition stage and may be ill-defined and poorly behaved.

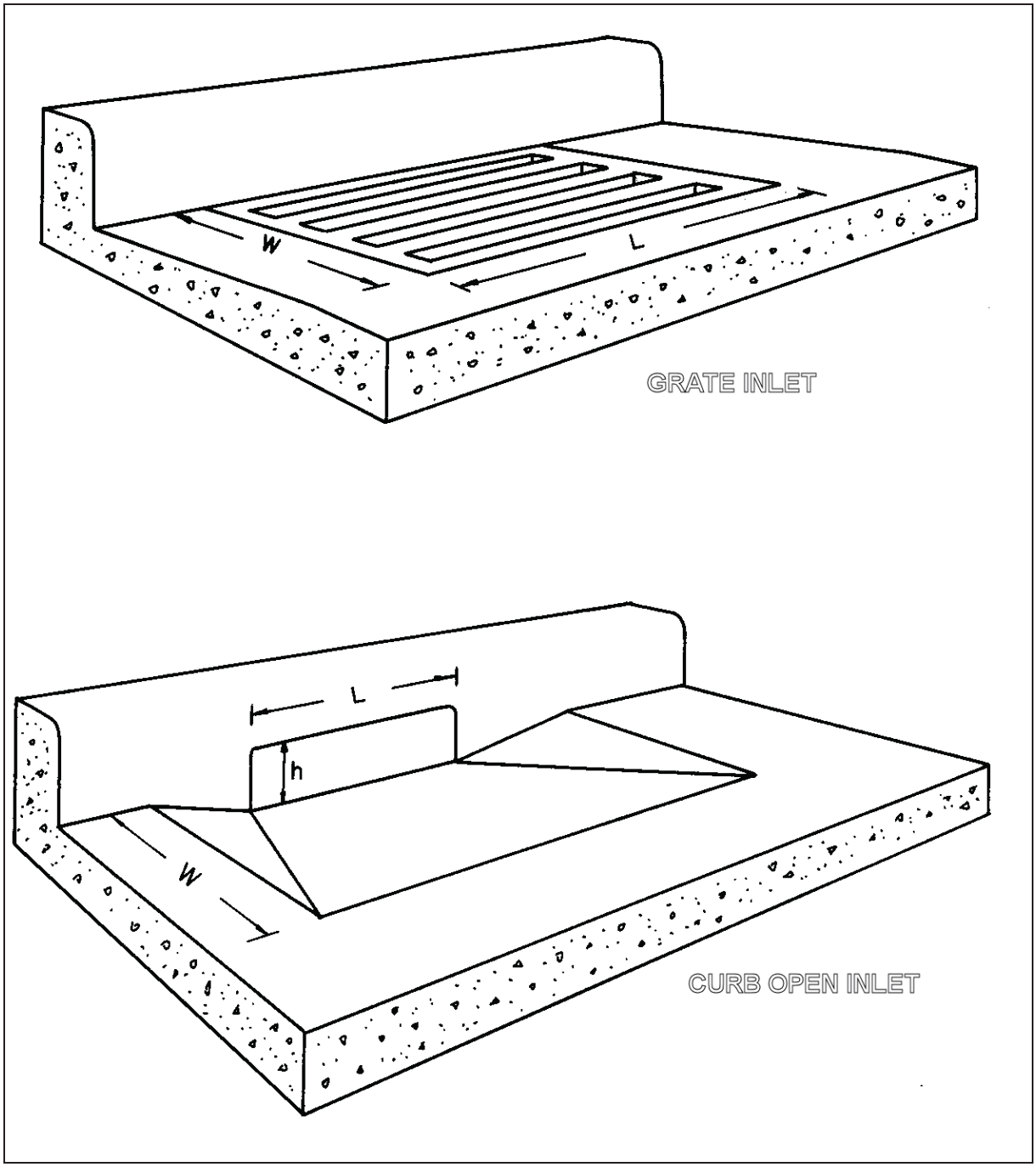

Figure 6.2. Perspective views of grate and curb-opening inlets.

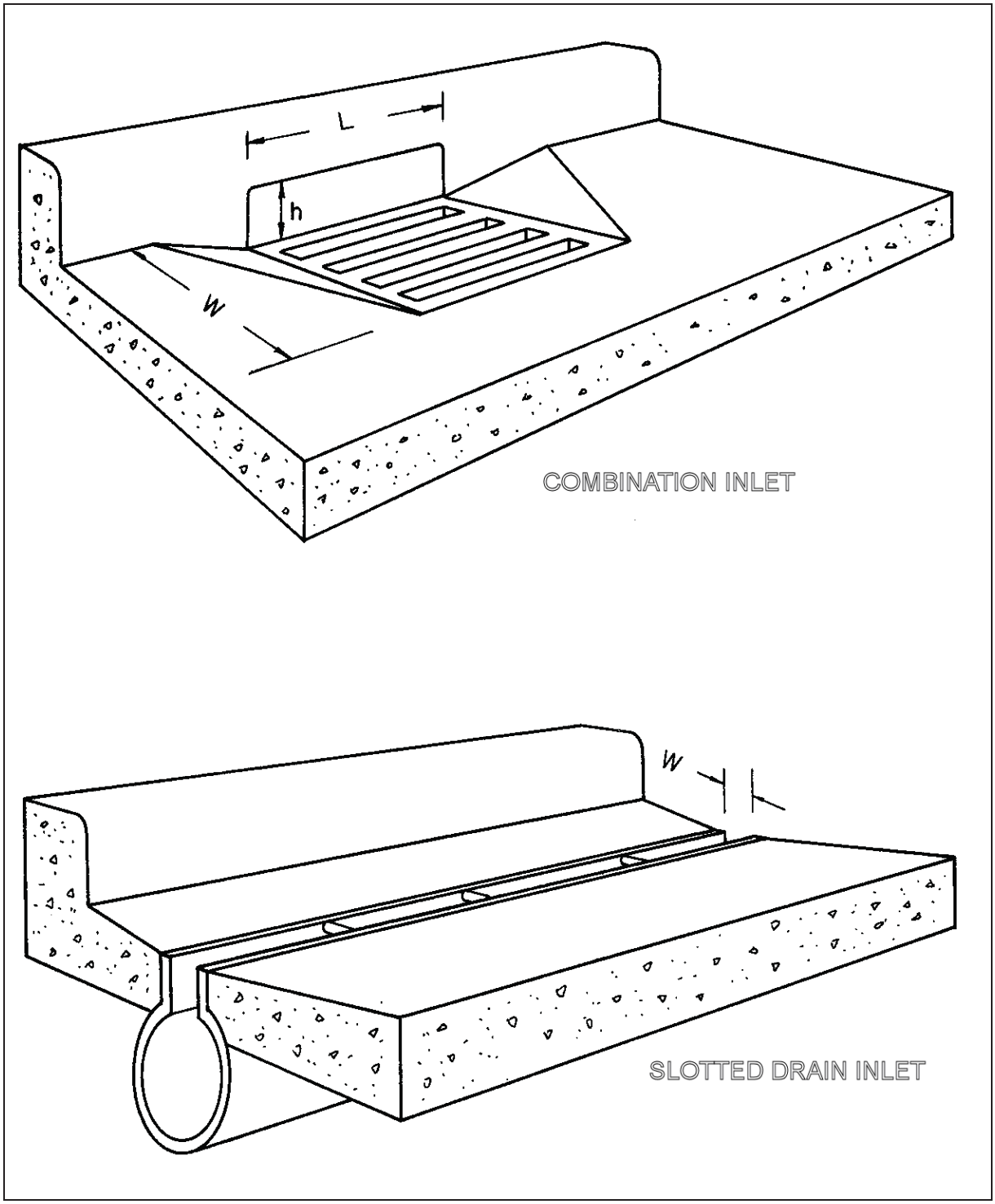

Figure 6.3. Perspective views of combination inlet and slotted drain inlet.

Q7

Efficiency of inlets in passing debris is critical in sag locations since all runoff that enters the sag must pass through the inlet, otherwise a hazardous ponding condition can result. Grate inlets alone are not recommended in sag locations because of potential clogging.

Inlet capacity is typically defined by design charts developed for standard inlet configurations from laboratory testing. As an example, the design chart for total interception by curb-inlets and slotted drains is shown in Figure 6.4 for SI units and Figure 6.5 for English units. Figure 6.6 can be used to evaluate the interception efficiency for the same inlets when less than total interception occurs. Example Problem 6.2 illustrates the use of these design charts. HEC-22 provides other design charts for a wide range of inlet and grate types typically used in highway engineering (Brown et al. 2008).

EXAMPLE PROBLEM 6.2 (SI Units)

Design the length of curb opening inlet required to intercept 0.07 m^3/s flowing along a street with a cross slope of 0.02 and a longitudinal slope of 0.03. Assume an n value of 0.016. Also determine the discharge intercepted if a 3.0 m curb inlet is used, and the amount of bypass flow to the next inlet.

Given:

Sx	=	0.02
S	=	0.030
Q_i	=	0.07 m^3/s
n	=	0.016

Find:

(1) Required length for total interception by a curb-opening inlet.

(2) Discharge intercepted by a 3.0 m curb-opening inlet, and the amount of bypass flow at the next inlet.

Solution:

(1) From Figure 6.4, for the given conditions a curb opening inlet 11.7 m long would intercept the total design flow of 0.07 m^3/s.

(2) If a 3.0 m curb-opening inlet is used, only a portion of the design flow will be intercepted. Figure 6.6 defines the interception efficiency for curb-opening inlets based on the length for total interception (L_t).

Therefore, given a 11.7 m length for total interception from item (1), and an a curb-opening length of only 3.0 m, the ratio of L/L_t is

$$L / L_t = 3.0/11.7 = 0.26$$

From Figure 6.6 the efficiency, E, is then 0.41, and the discharge intercepted by a 3.0 m curb-opening inlet is:

$Q_{intercepted} = EQ = 0.41\ (0.07) = 0.029\ m^3/s$

The amount of bypass flow to the next inlet is

$Q_{bypass} = 0.07 - 0.029 = 0.041\ m^3/s$

EXAMPLE PROBLEM 6.2 (English Units)

Design the length of curb opening inlet required to intercept 2.29 ft^3/s flowing along a street with a cross slope of 0.02 and a longitudinal slope of 0.03. Assume an n value of 0.016. Also determine the discharge intercepted if a 10 ft curb inlet is used, and the amount of bypass flow to the next inlet.

Given:

Sx	=	0.02
S	=	0.03
Q	=	2.29 ft^3/s
n	=	0.016

Find:

(1) Required length for total interception by a curb-opening inlet.

(2) Discharge intercepted by a 10-foot long curb-opening inlet, and the amount of bypass flow at the next inlet.

Solution:

(1) From Figure 6.5, for the given conditions a curb opening inlet 37.1 ft long would intercept the total design flow of 2.29 ft^3/s.

(2) If a 10.0 ft curb-opening inlet is used, only a portion of the design flow will be intercepted. Figure 6.6 defines the interception efficiency for curb-opening inlets based on the length for total interception (L_t).

Therefore, given a 37.1 ft length for total interception from item (1), and a curb-opening length of only 10 ft, the ratio of L/L_t is:

$L / L_t = 10 / 37.1 = 0.27$

From Figure 6.6 the efficiency, E, is then 0.43, and the discharge intercepted by a 10 ft curb-opening inlet is:

$Q_{intercepted} = EQ = 0.43\ (2.29) = 0.99$

The amount of bypass flow to the next inlet is:

$Q_{bypass} = 2.29 - 0.99 = 1.30\ ft^3/s$

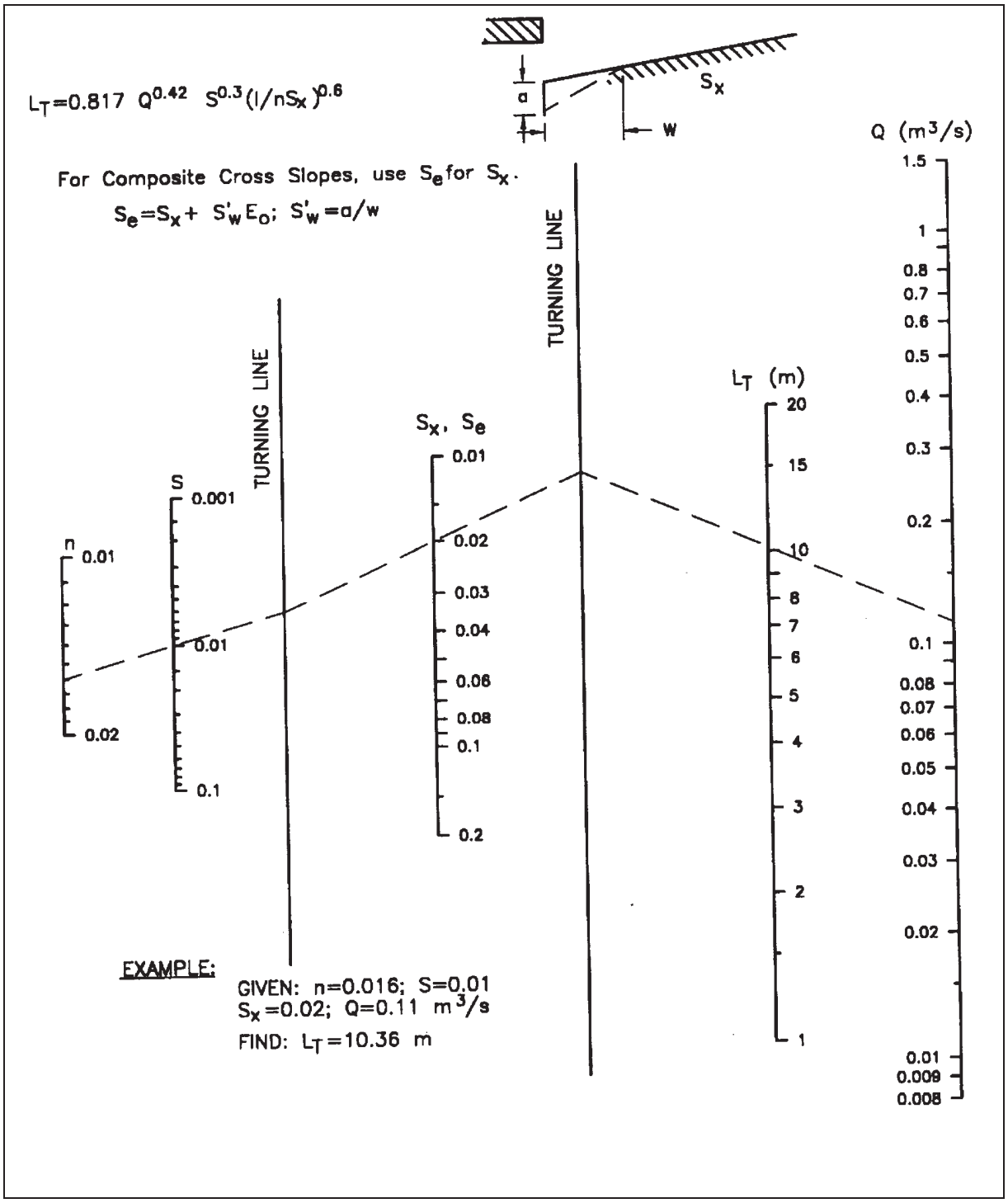

Figure 6.4. SI chart for curb-opening and slotted-drain inlet length for total interception.

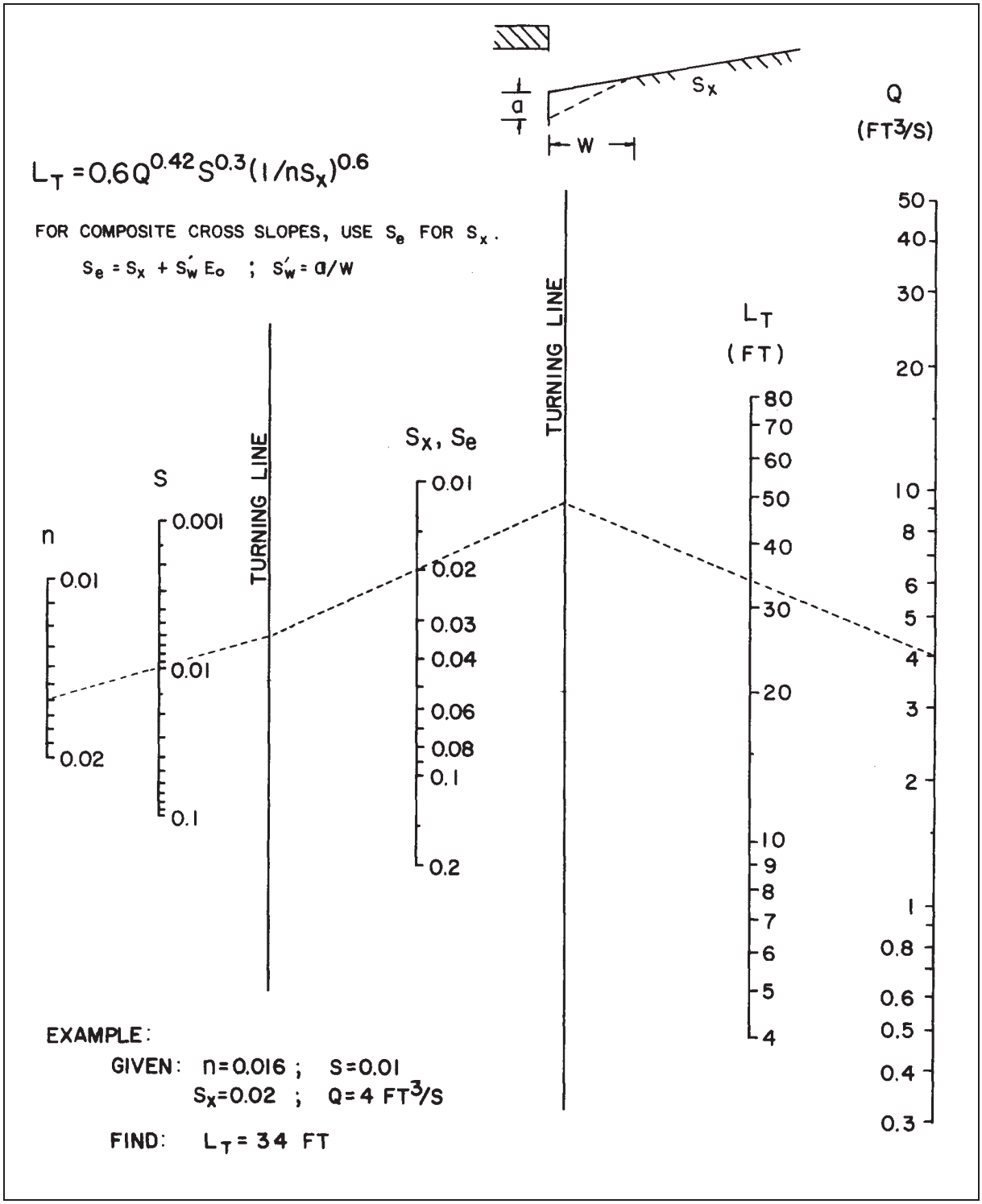

Figure 6.5. English chart for curb-opening and slotted-drain inlet length for total interception.

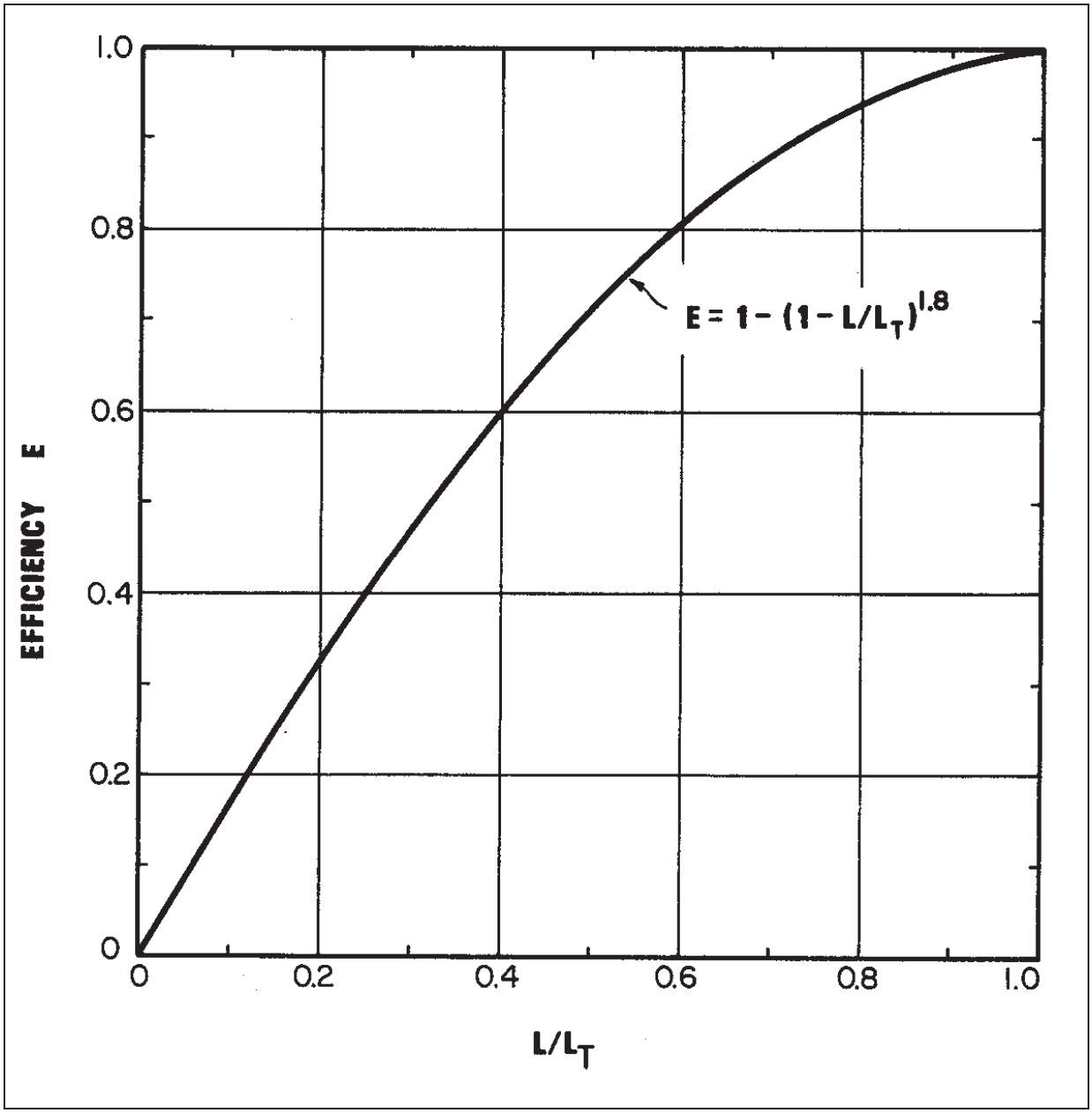

Figure 6.6. Curb-opening and slotted-drain inlet interception efficiency (from HEC-22, Brown et al. 2008).

6.4 MEDIAN, EMBANKMENT, AND BRIDGE INLETS

Q8

Medians are commonly used to separate opposing lanes of traffic on divided highways. Median areas should preferably not drain across traveled lanes, and often times the inside lanes and shoulder of multi-lane highways will drain to the median area where a center swale collects the runoff. Based on capacity or erosion considerations, it is sometimes necessary to place inlets in medians to remove some or all the runoff that has been collected. Medians may be drained by drop (grate) inlets similar to those used for pavement drainage (Figure 6.7).

Effective bridge deck drainage is important for several reasons, including hydroplaning, ice formation, and susceptibility of the deck structural and reinforcing steel to corrosion from deicing salts. While bridge deck drainage is accomplished in the same manner as any other curbed roadway section (Figure 6.8), bridge decks are often less effectively drained because of lower cross slopes, uniform cross slopes for traffic lanes and shoulders, parapets that collect debris, and drainage inlets that are relatively small and susceptible to clogging. Because of the limitations of bridge deck drainage, roadway drainage should be intercepted where practical before it reaches a bridge. HEC-21 provides detailed information on bridge deck drainage design (FHWA 1993).

Drainage inlets used to intercept runoff upgrade or downgrade of bridges, or runoff that might endanger an embankment fill slope, differ from other pavement drainage inlets. First, the economies achieved by system design are not possible because a series of inlets are not used. Second, total or near total interception is necessary and third, a closed storm drain system is often not available to dispose of the intercepted flow. Intercepted flow is usually discharged into open chutes or pipe downdrains terminating at the toe of the fill slope (Figure 6.9).

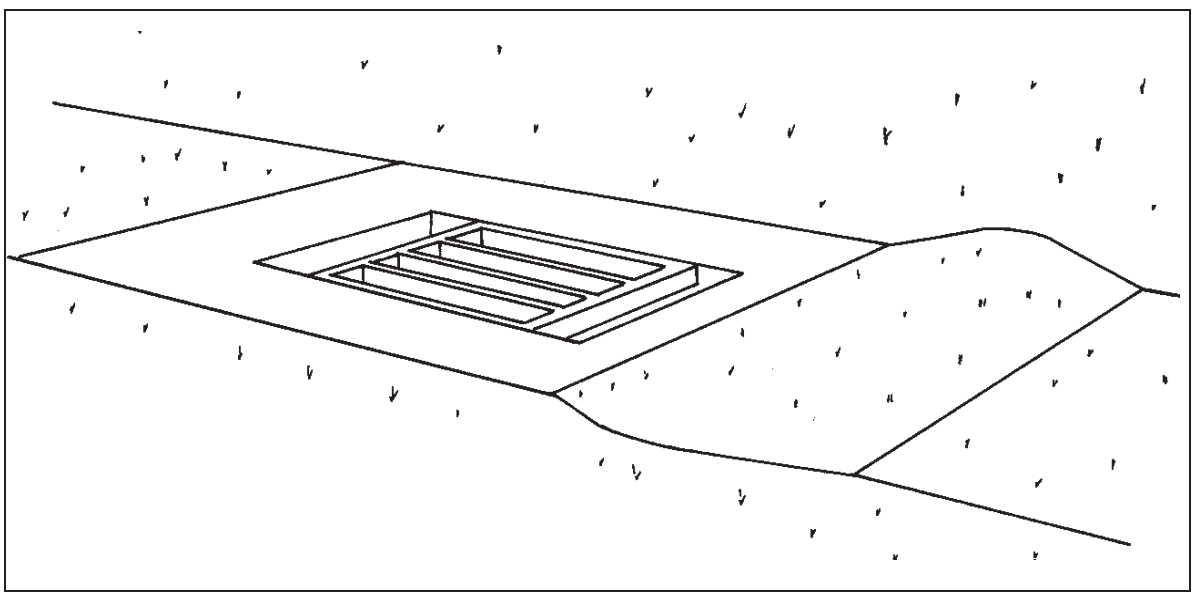

Figure 6.7. Median drop inlet.

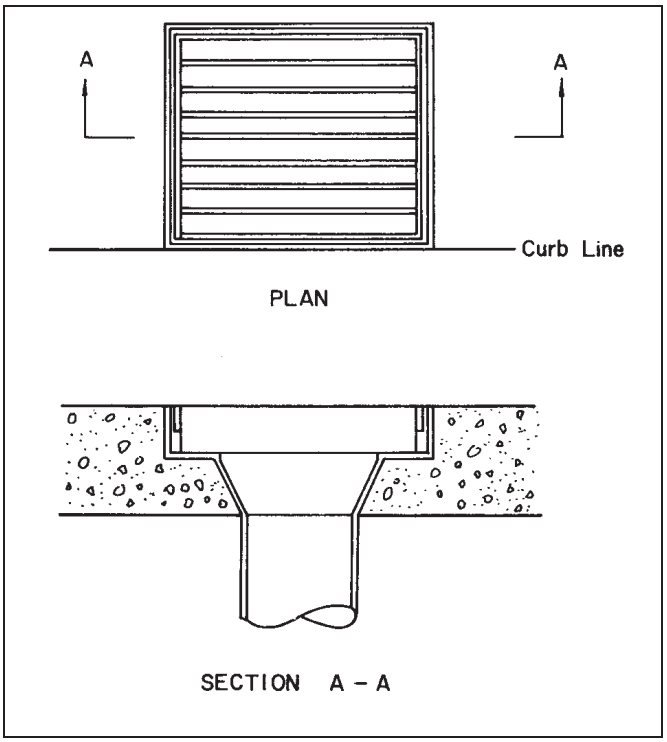

Figure 6.8. Bridge inlet.

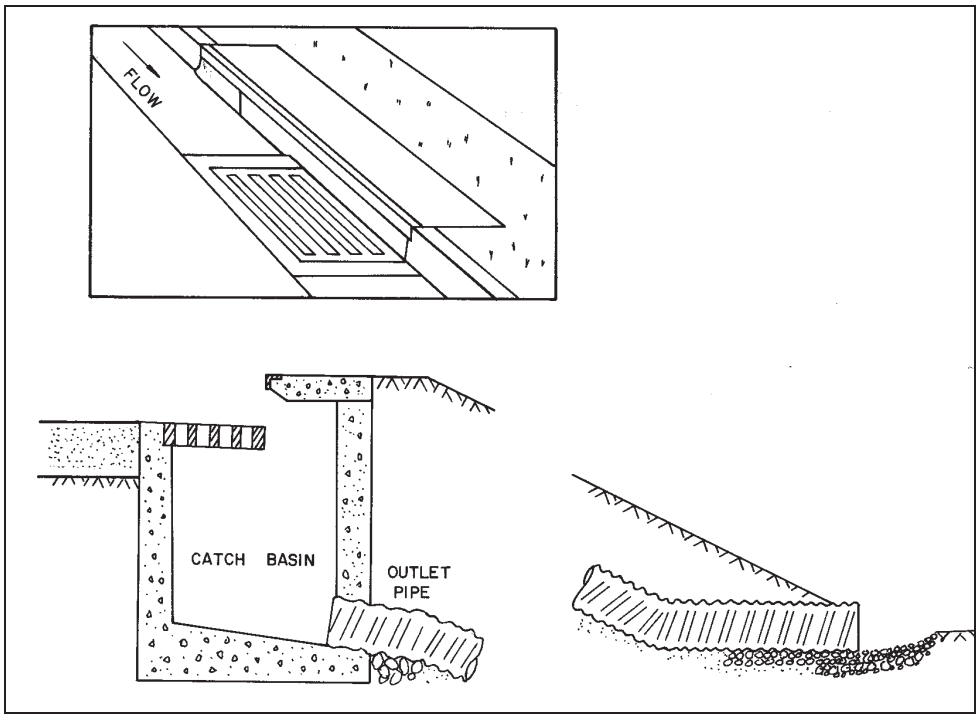

Figure 6.9. Embankment inlet and downdrain.

CHAPTER 7

CLOSED-CONDUIT FLOW

7.1 TYPES OF FLOW IN CLOSED CONDUITS

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Flow conditions in a closed conduit can occur as open-channel flow, gravity full flow or pressure flow. In open-channel flow the water surface is exposed to the atmosphere, which can occur in either an open conduit or a partially full closed conduit. Analysis of open-channel flow in a closed conduit is no different than any other type of open-channel flow, and all the concepts and principles discussed in Chapter 4 are applicable. Gravity full flow occurs at that condition where the conduit is flowing full, but not yet under any pressure. Pressure flow occurs when the conduit is flowing full and under pressure.

Due to the additional wetted perimeter and increased friction that occurs in a gravity full pipe, a partially full pipe will actually carry greater flow. For a circular conduit the peak flow occurs at 93 percent of the height of the pipe, and the average velocity flowing one-half full is the same as gravity full flow (Figure 7.1).

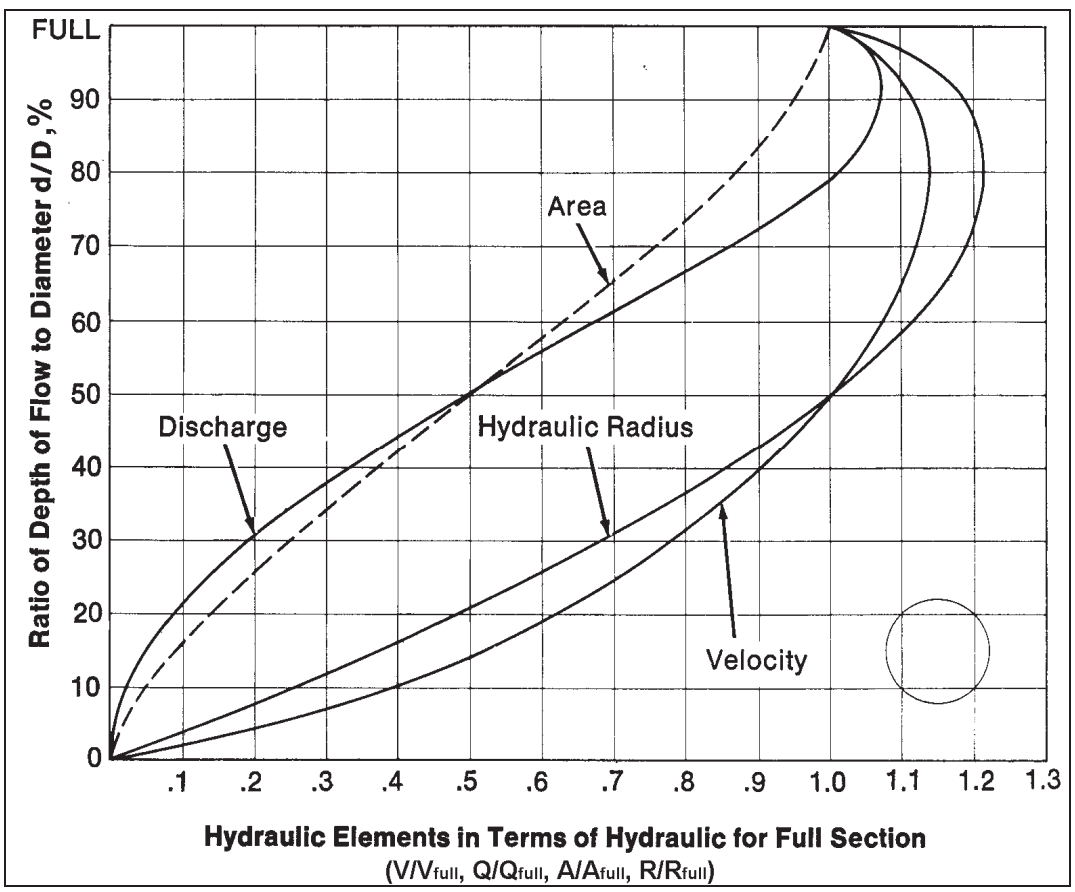


Figure 7.1. Part-full flow relationships for circular pipes.

Gravity full flow condition is usually assumed for purposes of storm drain design. Manning's equation (Equation 4.5) for circular section flowing full can be rewritten as:

$$Q = (K_u/n)\, D^{8/3}\, S^{1/2} \tag{7.1}$$

where:

Q	=	Discharge, m^3/s (ft^3/s)
n	=	Manning's coefficient
D	=	Pipe diameter, m (ft)
S	=	Slope, m/m (ft/ft)
K_u	=	0.312 (0.46)

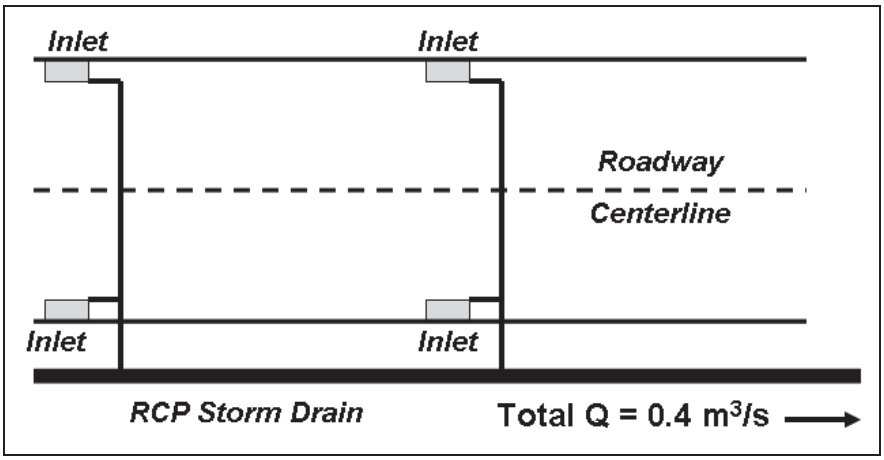
This equation allows for a direct computation of the required pipe diameter. Note that the computed diameter must be increased in size to a larger nominal dimension in order to carry the design discharge without creating pressure flow. The standard SI nominal sizes based on current English unit nominal sizes are given in Table 7.1.

Table 7.1. Nominal Pipe Sizes.

Nominal Size as Manufactured in English Units		Nominal Size Converted to SI Metric Units
Pipe Diameter		Pipe Diameter
Inches	Feet	(mm)
18	1.5	450
24	2.0	600
30	2.5	750
36	3.0	900
42	3.5	1,050
48	4.0	1,200
54	4.5	1,350
60	5.0	1,500
66	5.5	1,650
72	6.0	1,800
78	6.5	1,950
84	7.0	2,100
90	7.5	2,250
96	8.0	2,400
102	8.5	2,550
108	9.0	2,700
114	9.5	2,850
120	10.0	3,000
126	10.5	3,150
132	11.0	3,300
138	11.5	3,450
144	12.0	3,600

EXAMPLE PROBLEM 7.1 (SI Units)

Given: Pavement runoff is collected by a series of combination inlets. During the design event, the total discharge intercepted by all inlets is 0.4 m^3/s. A concrete storm drain pipe (n = 0.013) is to be placed on a grade parallel to the roadway grade, which is 0.005 m/m.



Find: The required storm drain pipe diameter and the full flow velocity.

1. Use the full flow equation (which gives pipe diameter in m)

   $Q = (K_u/n)\ D^{8/3}\ S^{1/2}$ where $K_u = 0.312$ for SI units

   $0.4 = (0.312/0.013)\ (D^{8/3})\ (0.005)^{1/2}$

   $D^{8/3} = 0.236$

   $D = 0.58$ m or 580 mm

2. Based on Table 7.1, the next larger nominal pipe size is 600 mm.

3. Under our design conditions, a 600 mm would be flowing slightly less than full. Based on the part-full flow relationships (Figure 7.1) the velocity does not change significantly from half-full to full. However, for tc calculation, the part-full velocity should be used.

   To calculate the part-full velocity, nomographs or trial-and-error solution can be used. Alternatively, the part-full flow relationships can be used. The full-flow discharge and velocity of a 600-mm concrete pipe are:

   $Q = (0.312/0.013)\ (0.60)^{8/3}\ (0.005)^{1/2} = 0.43\ m^3/s$

   $V = Q/A = 0.43 / \{[\pi\ (0.6)^2] / 4\} = 1.52$ m/s

   The ratio of part-full to full-flow discharge is

   $Q / Q_f = 0.40 / 0.43 = 0.93$

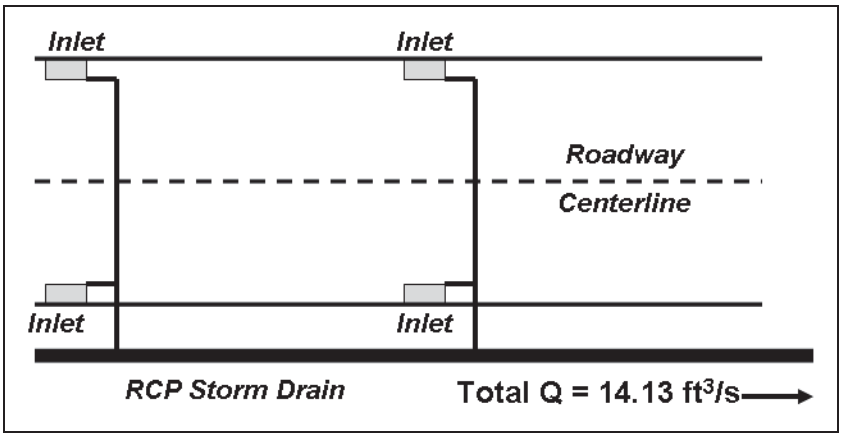
   The corresponding velocity ratio is 1.13 (Figure 7.1).

   Therefore;

   $V / V_f = 1.13$ and $V = 1.13\ (1.52) = 1.72$ m/s

EXAMPLE PROBLEM 7.1 (English Units)

Given: Pavement runoff is collected by a series of combination inlets. During the design event, the total discharge intercepted by all inlets is 14.13 ft^3/s. A concrete storm drain pipe (n = 0.013) is to be placed on a grade parallel to the roadway grade, which is 0.005 ft/ft.



Find: The required storm drain pipe diameter and the full flow velocity.

1. Use the full flow equation (which gives pipe diameter in ft)

   $Q = (K_u/n)\ D^{8/3}\ S^{1/2}$ where $K_u = 0.46$

   $14.13 = (0.46/0.013)\ (D^{8/3})\ (0.005)^{1/2}$

   $D^{8/3} = 5.64$

   $D = 1.91$ ft

2. Based on Table 7.1, the next larger nominal pipe size is 24 inches or 2 ft.

3. Under our design conditions, a 24 inch would be flowing slightly less than full. Based on the part-full flow relationships (Figure 7.1) the velocity does not change significantly from half-full to full. However, for tc calculation, the part-full velocity should be used.

   To calculate the part-full velocity, nomographs or trial-and-error solution can be used. Alternatively, the part-full flow relationships can be used. The full-flow discharge and velocity of a 24-inch concrete pipe is:

   $Q = (0.46/0.013)\ (2)^{8/3}\ (0.005)^{1/2} = 15.89\ ft^3/s$

   $V = Q / A = 15.89 / 3.14 = 5.06$ ft/s

   The ratio of part-full to full-flow discharge is:

   $Q / Q_f = 14.13 / 15.89 = 0.89$

The corresponding velocity ratio is 1.12 (Figure 7.1).

Therefore

$$V / V_f = 1.12 \text{ and } V = 1.12\ (5.06) = 5.66 \text{ ft/s}$$

7.2 ENERGY EQUATION

The energy equation was reviewed in Chapter 3 (Equation 3.2). In very simple terms the equation states that the energy head at any cross section must equal that in any other downstream section plus the intervening energy losses. The energy head is divided into three components: velocity head, pressure head, and elevation head. The energy grade line (EGL) represents the total energy at any given cross section. Energy losses are classified as friction losses and form losses (see Section 7.3).

The hydraulic grade line (HGL) is below the EGL by the amount of the velocity head. In open-channel flow the HGL is equal to the water surface elevation in the channel, while in pressure flow the HGL represents the elevation water would rise to in a stand pipe connected to the conduit. For example, in a storm drain designed for pressure flow the HGL should be lower than the roadway elevation, otherwise water in the storm drain will rise up through inlets and access hole covers and flood the roadway. Similarly, if an open-channel flow condition in a storm drain is supercritical, care must be taken to ensure that a hydraulic jump does not occur which might create pressure flow and a HGL above the roadway elevation.

7.3 ENERGY LOSSES

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When using the energy equation all energy losses should be accounted for. Energy losses can be classified as friction losses or form losses. Friction losses are due to forces between the fluid and boundary material, while form losses are the result of various hydraulic structures along the closed conduit. These structures, such as access holes, bends, contractions, enlargements and transitions, will each cause velocity headlosses and potentially major changes in the energy grade line and hydraulic grade line across the structure. The form losses are often called "minor losses," because they are typically much less than friction losses.

7.3.1 Calculating Friction Losses

Friction losses are calculated as:

$$h_f = L\ S_f \tag{7.2}$$

where:

L	=	Length of the conduit
S_f	=	Friction slope (energy grade line slope)

Uniform flow conditions are typically assumed so that the friction slope can be calculated from either Manning's equation or the Darcy-Weisbach equation. Rewriting Manning's equation for S_f:

$$S_f = \left(\frac{Qn}{K_u AR^{2/3}} \right)^2 \tag{7.3}$$

The Darcy-Weisbach equation for open-channel flow:

$$S_f = \frac{f}{4R} \frac{V^2}{2g} \tag{7.4}$$

and for pressure flow in circular conduit:

$$h_f = \frac{fL}{D} \frac{V^2}{2g} \tag{7.5}$$

Manning's equation is more commonly used by practicing engineers, even though the Darcy-Weisbach equation is a theoretically better equation since it is dimensionally correct and applicable for any fluid over a wide range of conditions. However, the possibilities for greater accuracy with the Darcy-Weisbach equation are limited by determination of the Darcy f and a generally more complicated application than the Manning's equation. Typical Manning's n values for closed-conduit flow are given in Appendix B, Table B.3.

No matter which formula is used, judgment is required in selecting roughness coefficients. Roughness coefficients are primarily defined by the type of pipe material; however, many other factors can modify the value based on pipe material. Other important factors include the type of joint used, poor alignment and grade due to settlement or lateral soil movement, sediment deposits and flow from laterals disturbing flow in the mainline.

7.3.2 Calculating Form Losses

Form losses occur when flow passes through structures such as access holes, junctions, bends, contractions, enlargements and transitions. These structures can cause major losses in both the energy grade line and the hydraulic grade line across the structure, and if not accounted for in design, the capacity of the conduit may be restricted.

Form losses may be evaluated by several methods. The simplest method is based on a coefficient times the velocity head, with different coefficients tabulated for entrance/exit losses, bends, expansions, contractions, etc. The general form of the equation is:

$$h_L = K \frac{V^2}{2g} \tag{7.6}$$

HEC-22 (Brown et al. 2008) provides K values for these various situations.

7.3.3 Junction Losses

A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of an access hole structure. Headloss through a junction can be evaluated using Equation 7.6 with a suitable K value. A more sophisticated approach recommended by FHWA is based on pressure and momentum concepts, specifically that the sum of all forces acting at a junction must equal the sum of all momentums, expressed as:

$$H_j = \frac{(Q_o\ V_o) - (Q_i\ V_i) - (Q_1\ V_1 \cos\theta)}{0.5\ g\ (A_o + A_i)} + h_i - h_o \tag{7.7}$$

where:

H_j	=	Junction headloss, m (ft)
Q_o, Q_i, Q_1	=	Outlet, inlet, and lateral flows, respectively, m^3/s (ft^3/s)
V_o, V_i, V_1	=	Outlet, inlet, and lateral velocities, respectively, m^3/s (ft^3/s)
h_o, h_i	=	Outlet inlet velocity heads, m (ft)
A_o, A_i	=	Outlet and inlet cross-sectional areas, m^2 (ft^2)
θ	=	Angle of lateral with respect to centerline of outlet pipe, degrees
g	=	Gravitational acceleration, 9.81 m/s^2 (32.2 ft/s^2)

7.3.4 Calculating Access Hole Losses

A more complex situation exists where an access hole or inlet exists at the junction between inflow and outflow pipes. The simplest method is based on Equation 7.6 with an access hole K value. For example, a typical K value for a straight through access hole with no change in pipe size is 0.15. FHWA refers to this approach as the approximate method and considers it appropriate only for preliminary design estimates, and should not be used when making energy grade line (EGL) calculations.

For many years, FHWA has been developing and refining more complex approaches for estimating losses in access holes and inlets. The recommended FHWA method for accurately calculating access hole head loss classifies access holes and their hydraulic conditions in a manner analogous to inlet control and full flow for culverts as detailed in HEC-22 (Brown et al. 2008). The three fundamental steps in the FHWA method are:

STEP 1: Determine an initial access hole energy level (E_{ai}) based on outlet control (partial and full flow) or inlet control (weir and orifice) equations for the outflow pipe.

STEP 2: Adjust the initial access hole energy level based on benching, inflow angle(s), and plunging flows to compute the final calculated energy level (E_a).

STEP 3: Calculate the exit loss from each inflow pipe and estimate the energy gradeline (EGL_o), which will then be used to continue calculations upstream.

Figure 7.2 summarizes the primary variables used in the procedure, which is described step-by-step in the following section.

STEP 1: Initial Access Hole Energy Level

The initial energy level in the access hole structure (E_{ai}) is calculated as the maximum of three possible conditions; these determine the hydraulic regime within the structure. The three conditions considered for the outflow pipe are:

1. Outlet control condition
   - Outlet control full flow condition - this is a common occurrence when a storm drain system is surcharged by high tailwater and may also occur if flow in the pipe is limited by pipe capacity.
   - Outlet control partial flow condition – considered when the outflow pipe is flowing partially full and in subcritical flow.

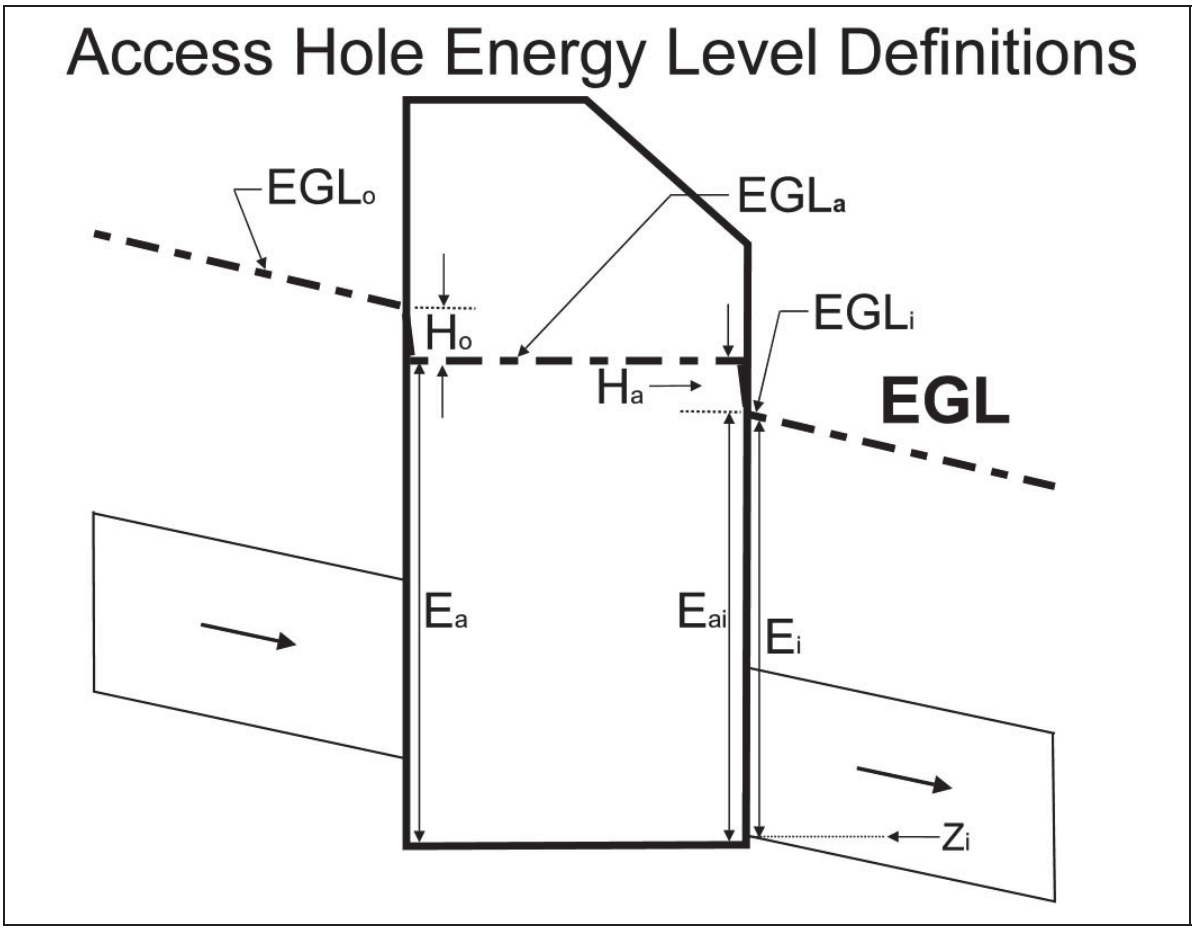

Figure 7.2. Definition sketch for access holes.

2. Inlet control (submerged) condition – considered possibly to occur if the opening in the access hole structure to the outlet pipe is limiting and the resulting water depth in the access hole is sufficiently high that flow through the opening to the outflow pipe is treated as an orifice.
3. Inlet control (unsubmerged) condition – considered possibly to occur if the flow control is also limited by the opening, but the resulting water level in the access hole requires treating the opening to the outflow pipe as a weir.

Algebraically, the initial energy level may be described as:

$$E_{ai} = \max (E_{aio}, E_{ais}, E_{aiu}) \tag{7.8}$$

where:

E_{aio} = Estimated access hole energy level for outlet control (full and partial flow)
E_{ais} = Estimated access hole energy level for inlet control (submerged)
E_{aiu} = Estimated access hole energy level for inlet control (unsubmerged)

Estimated Energy Level for Outlet Control: Partial Flow and Full Flow

In the outlet control condition, discharge out of the access hole is limited by the downstream storm drain system such that the outflow pipe is either flowing full or partially full in subcritical flow. The initial structure energy level (E_{aio}) estimate is:

$$E_{aio} = E_i + H_i \tag{7.9}$$

where:

E_i = Outflow pipe specific energy head (calculated using Equation 7-10)
H_i = Entrance loss assuming outlet control (calculated using Equation 7-11)

$$E_i = EGL_i - Z_i \tag{7.10}$$

$$H_i = K_i \, (V^2 / 2g) \tag{7.11}$$

where:

K_i = Entrance loss coefficient = 0.2, dimensionless

Estimated Energy Level for Inlet Control: Submerged

Inlet control calculations employ a dimensionless ratio adapted from the analysis of culverts referred to as the discharge intensity. The discharge intensity is described by the Discharge Intensity (DI) parameter, which is the ratio of discharge to pipe dimensions:

$$DI = Q / [A \, (g \, D_o)^{1/2}] \tag{7.12}$$

where:

A = Area of outflow pipe, m^2 (ft^2)
D_o = Diameter of outflow pipe, m (ft)

The submerged inlet control condition uses an orifice analogy to estimate the energy level, E_{ais}

$$E_{ais} = D_o \, (DI)^2 \tag{7.13}$$

Derivation of Equation 7-13 used data with discharge intensities less than or equal to 1.6.

Estimated Energy Level for Inlet Control: Unsubmerged

Laboratory analyses describe that unsubmerged inlet control conditions are associated with discharge intensities (DI) in a 0.0 to 0.5 range (this is not to suggest that the equation is limited to this range). The unsubmerged inlet control condition uses a weir analogy to estimate the energy level (E_{aiu}):

$$E_{aiu} = 1.6 \, D_o \, (DI)^{2/3} \tag{7.14}$$

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STEP 2: Adjustments for Benching, Angled Inflow, and Plunging Inflow

The initial structure energy level calculated in STEP 1 is used as a basis for estimating additional losses for: (1) discharges entering the structure at angles other then 180 degrees; (2) benching configurations (benching helps streamline the invert of an access hole and reduces energy loss); and (3) plunging flows entering the structure at elevations above the water depth in the access hole (Flows entering a structure from an inlet can be treated as plunging flows).

The effects of these conditions may be estimated and applied to the initial access hole energy level using the principle of superposition. This additive approach avoids a problem experienced in other methods where extreme values of energy losses are obtained when a single multiplicative coefficient takes on an extreme value. The equation is:

$$E_a = E_{ai} + H_B + H_\theta + H_P \qquad (7.15)$$

where:

H_B = Additional energy loss for benching (floor configuration)
H_θ = Additional energy loss for angled inflows other than 180 degrees
H_P = Additional energy loss for plunging flows

HEC-22 details the equations necessary to account for these additional complexities in the configuration of an access hole. Note that E_a represents the level of the energy gradeline in the access hole. Should E_a be less than the outflow pipe energy head (E_i), then E_a should be set equal to E_i. The water depth in the access hole (y_a) is approximately equal to E_a.

Knowing the access hole energy level (E_a) and assuming the access hole invert (z_a) is the same elevation as the outflow pipe invert (z_i) allows determination of the access hole energy gradeline (EGL_a):

$$EGL_a = E_a + Z_a \qquad (7.16)$$

STEP 3: Inflow Pipe Exit Losses

The final step is to calculate the energy gradeline into each inflow pipe. The FHWA method considers two cases: (1) plunging inflow pipe(s) and (2) non-plunging inflow pipe(s).

Non-Plunging Inflow Pipe

The first case is for non-plunging inflow pipes, that is, those pipes with a hydraulic connection to the water in the access hole. Inflow pipes operating under this condition are identified when the access hole energy gradeline (E_a) is greater than the inflow pipe invert elevation (Z_o). In this case, the inflow pipe energy head (EGL_o) is equal to:

$$EGL_o = EGL_a + H_o \qquad (7.17)$$

where:

H_o = Inflow pipe exit loss, calculated using Equation 7.18

Exit loss is calculated in the traditional manner using the inflow pipe velocity head since a condition of supercritical flow is not a concern on the inflow pipe. The equation is as follows:

$$H_o = K_o (V^2 /2g) \quad (7.18)$$

where:

K_o = Exit loss coefficient = 0.4, dimensionless

Plunging Inflow Pipe

The second case is for an inflow pipe in a plunging condition. For pipes that are plunging, the inflow pipe energy gradeline (EGL_o) is taken as the energy gradeline calculated from the inflow pipe hydraulics. EGL_o is independent of access hole water depth and losses.

Continuing Computations Upstream

For either the nonplunging or plunging cases, the resulting energy gradeline is used to continue computations upstream to the next access hole. The three step procedure of estimating: (1) entrance losses, (2) additional losses, and (3) exit losses is repeated at each access hole. The application of this procedure is included in the storm drain design example in Chapter 8 (Example 8.1).

CHAPTER 8

CLOSED CONDUIT APPLICATIONS – STORM DRAIN DESIGN

8.1 DESIGN APPROACH

The design of a storm drain system is not a complicated process, but can involve detailed calculations that are often completed in an iterative manner. Major steps in storm drain design are:

1. Preliminary Design
2. Computation of the Hydraulic and Energy Grade Lines (HGL and EGL)
3. Adjustment of inlet sizes/locations and pipe size/location to correct HGL/EGL problems and/or optimize the design

The following sections briefly discuss each major component in the storm drain design process. Note that detention ponds are often an integral part of a storm drainage system. Temporary storage or detention/retention of excess storm water runoff can be used to control the quality and/or quantity of storm water released downstream. For a detailed discussion of detention and retention facilities, see HEC-22 (Brown et al. 2008).

8.2 PRELIMINARY DESIGN

8.2.1 Plan and Profile Layout

The first step in storm drain design is to develop a preliminary storm drain layout, including inlet, access hole and pipe locations. This is usually completed on a plan view map that shows the roadway, bridges, adjacent land use conditions, intersections, and under/ overpasses. Other utility locations and situations should also be identified and shown, including surface utilities, underground utilities and any other storm drain systems. Storm drain alignment within the road right-of-way is usually influenced, if not dictated, by the location of other utilities. These other utilities, which may be public or private, may cause interference with the alignment or elevation of the proposed storm drain.

Generally, a storm drain should be kept as close to the surface as minimum cover and/or hydraulic requirements allow to minimize excavation costs. Another location control is the demand of traffic and the need to provide for traffic flow during construction including the possible use of detours. Providing curved storm drain alignments may be cost effective and should be considered for large pipe sizes, especially when headlosses are a concern. The deflection angle is divided by the allowable deflection per joint to determine the number of pipe sections required to create a given curve.

Tentative inlets, junctions and access locations should be identified based primarily on experience factors. Initially estimated type and location of inlets will provide the basis for hydrologic calculations and pipe sizing and will be adjusted as required during the design process. Ultimately, inlets must be provided based on spread criteria and/or intersection requirements. Generally, all flow approaching an intersection should be intercepted, as cross gutters are not practical in highway applications.

Q12 Access is required for inspection and maintenance of storm drain systems. For storm drains smaller than about 1.2 m (48 in.) access is required about every 120 m (400 ft), while for larger sizes the spacing can be 180 m (600 ft) and larger. Junctions are also required at the confluence of two or more storm drains, where pipe size changes, at sharp curves or angle points (greater than 10°), or at abrupt grade changes.

8.2.2 Pipe Sizing

Given the preliminary layout, it is possible to begin the hydrologic and hydraulic analysis necessary to size the storm drain system. These calculations begin at the upstream end and work downstream. In contrast, the hydraulic and energy grade line calculations described below in Section 8.3 begin at the downstream end and work upstream.

The first step is to calculate the discharge contributing to each inlet location and to size the inlet. Based on the small incremental drainage areas involved, the Rational Method is typically used (Chapter 2). Given the discharge at each selected inlet and considering spread criteria it may be necessary to relocate an inlet or to incorporate additional inlets.

After adjusting inlet locations the storm drain laterals and main line can be sized. Laterals are sized based on the discharge used to size the inlets. However, the discharge for the main line is not simply the sum of the incremental discharges at each inlet. Recalling that in the Rational Method the rainfall intensity used should be based on the longest time of concentration to the design point, the discharge for the main line should be determined based on longest time of concentration from various upstream approach branches and the corresponding accumulated CA values. This procedure satisfies the assumptions and stipulations for use of the Rational Method.

Preliminary pipe size is then calculated based on a full flow assumption given the discharge and pipe slope. Pipe slope is typically established in preliminary design based on the roadway grade and the need to avoid other existing utilities or storm drains. When pipe sizes are increased in a downstream direction, it is generally preferable to match the crown elevation, rather than the invert elevation. A better design approach is for the crown of the downstream pipe to drop by the headloss across the structure. A crown drop is recommended even when pipe size stays the same. For purposes of preliminary design the headloss for all structures, including access holes, may be estimated using the approximate method based on Equation 7.6. Alternatively, some states use a standard amount for the crown drop, typically 0.03 m (0.10 ft).

Q13 Generally, storm drains should be designed to provide a velocity of at least 1 m/s (3 ft/s) when the conduit is full to ensure that the pipe is self cleaning. Most state highway agencies consider a 10-year frequency storm as the minimum for design of storm drains. Storm drains in sag locations are typically checked for the 50-year frequency storm.

Both minimum and maximum cover limits must be considered in the design of storm drainage systems. Minimum cover limits are established to ensure the conduits structural stability under live and impact loads. With increasing fill heights, dead load becomes the controlling factor. For highway applications, a minimum cover depth of 0.9 m (3.0 ft) should be maintained where possible. In cases where this criteria cannot be met, the storm drains should be evaluated to determine if they are structurally capable of supporting imposed loads.

8.3 COMPUTATION OF HYDRAULIC AND ENERGY GRADE LINES

The EGL and HGL should be calculated to evaluate overall system performance and ensure that at the design discharge the storm drain system does not inundate or adversely affect inlets, access holes or other appurtenances. This calculation begins at the downstream end (outfall) of the system and works upstream.

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

Given the outfall HGL/EGL, the calculation is based on an application of the energy equation in an upstream direction on a reach-by-reach basis. Reaches are defined between hydraulic structures and/or grade breaks. Many storm drain systems are designed to function in a subcritical flow regime. In subcritical flow, pipe and access hole losses are summed to determine the upstream EGL levels. If supercritical flow occurs, pipe and access losses are not carried upstream. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream pipe section to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime.

The design procedure is usually based on the assumption of a uniform hydraulic gradient within a conduit reach. Greater accuracy could be achieved with water surface profile computations, but such accuracy is seldom necessary.

Friction losses are calculated assuming full flow. An appropriate equation for this calculation is the full flow version of the Manning's equation (Equation 7.1), which defines the friction slope given the design discharge and preliminary pipe diameter. Note that if the calculated friction slope is steeper than the pipe slope, pressure flow conditions will exist. If the friction slope is less than the pipe slope, partial flow may occur (also depends on downstream tailwater). At the location where the pipe becomes unsealed (transitions to partial flow) normal depth calculations may be used to estimate hydraulic conditions.

Form losses are calculated for each hydraulic structure based on the methods described in Chapter 7. For access holes the use of the approximate method (based on Equation 7.6) is acceptable only for preliminary design. For HGL/EGL calculations the more accurate FHWA method for access holes, as described in Section 7.3.3, should always be used.

8.4 OPTIMIZATION OF SYSTEM

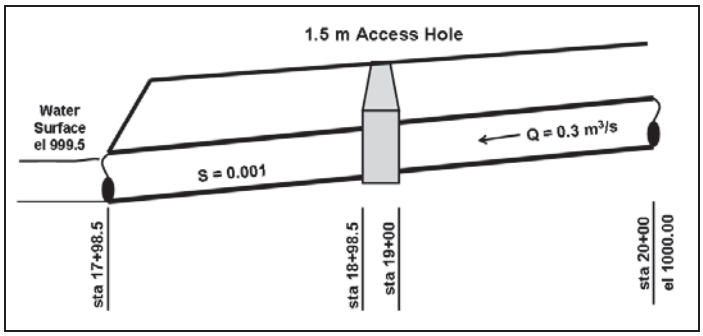
Having completed the above calculations, initial design should be evaluated using a higher check flood and adjusted to reduce cost and risk. For example, if the HGL is too high in a given reach, the pipe size will have to be increased which will require recalculation of the HGL. As designed the system will operate basically at or near gravity full flow; however, if surcharging (pressure flow) is acceptable, the pipe sizes can be reduced and the system reanalyzed. The initial design should be evaluated using a higher check flood and adjusted to reduce cost and risk, if necessary.

8.5 STORM DRAIN DESIGN USING COMPUTER SOFTWARE

Storm drain analysis and design can be completed using any one of a number of software packages. To ensure results are consistent with local design standards, the user should verify the design methods and assumptions being used, particularly those for calculating energy losses through structures.

EXAMPLE PROBLEM 8.1 (SI Units)

Given: An RCP (n = 0.013) storm drain must be designed to carry 0.3 m^3/s on a 0.001 slope. The storm drain will consist of 200 m of pipe with a 1.5 m (diameter) access hole in the middle. The invert of the pipe at the upstream end is 1000.00 m at Station 20+00. The storm drain will discharge into a channel where the water surface elevation is 999.5 m. Complete preliminary design to define pipe size, crown drop and invert elevations. Calculate the crown drop based on headloss, but assume a minimum value of 0.03 m. Complete final design to evaluate the HGL and EGL profiles.



Find: Pipe size, crown drop and invert elevations based on preliminary design.
HGL/EGL based on final design.

Preliminary Design

Preliminary design starts at the upstream end and works downstream.

1. Use the full-flow Manning's equation to size the pipe:

$$Q = (K_u/n)\, D^{8/3}\, S^{1/2} \qquad \text{where } K_u = 0.312 \text{ for SI Units}$$

$$D = \left[\frac{Qn}{0.312\ S^{1/2}}\right]^{3/8}; \qquad D = \left[\frac{(0.3)(0.013)}{(0.312)\ (0.001)^{1/2}}\right]^{3/8} = 0.71\text{m}$$

Therefore, based on nominal pipe sizes and to ensure open channel flow, use a 750 mm pipe.

2. The invert at the upstream end was given as 1000 m. The elevation at the upstream side of the access hole (Sta 19+00) will be:

$$Z = 1000.00 - 100\ (0.001) = 999.90 \text{ m}$$

3. The elevation on the downstream side of the access hole (Sta 18+98.50) should incorporate the crown drop based on the approximate energy loss through the structure.

   The approximate energy loss through the access hole is based on Equation 7.6

   $h_L = K\,[V^2 / (2g)]$

   The K value for a straight through access hole is 0.15.

   The velocity in the part full pipe can be estimated using the hydraulic elements graph (Figure 7.1). The full flow capacity of a 750 mm RCP pipe on a 0.001 slope is:

   $Q_{full} = (K_u/n)\, D^{8/3}\, S^{1/2}$ where $K_u = 0.312$ for SI units

   $Q_{full} = (0.312/0.013)\,(0.75)^{8/3}\,(0.001)^{1/2} = 0.35\ m^3/s$

   And the full flow velocity is:

   $Q = V\,A, \quad V = Q / A$

   $A = [\pi\,(D)^2] / 4 = [\pi\,(0.75)^2] / 4 = 0.44\ m^2$

   $V = 0.35 / 0.44 = 0.80\ m/s$

   The Q/Q_{full} ratio is 0.3/0.35 = 0.86, and from the Figure 7.1, the V/V_{full} ratio will be about 1.12. Therefore, the part-full velocity is:

   $V = 1.12\,(V_{full}) = 1.12\,(0.80) = 0.90\ m/s$

   and headloss through the access hole is:

   $h_L = K[V^2 / (2g)] = 0.15\,[0.90^2 / (2(9.81))] = 0.006\ m$

   Consequently, the elevation on the downstream side of the access hole based on a minimum crown drop of 0.03 m is:

   $Z = 999.90 - 0.03 = 999.87$

4. The elevation at the outfall (Sta 17+98.50) is then

   $Z = 999.87 - 100(0.001) = 999.77$

5. Summary of Preliminary design data

   Storm drain diameter = 750 mm

Pipeline invert elevations

Location	Station	Elevation
Outfall	17+98.50	999.77
Downstream side of access hole	18+98.50	999.87
Upstream side of access hole	19+00	999.90
Upstream end of storm drain	20+00	1000.00

Q14

Evaluation of HGL/EGL

HGL/EGL evaluation starts at the outlet and works upstream. The calculation is based on the energy equation applied from a known downstream location to an unknown upstream location, accounting for the intervening energy losses.

1. Starting HGL

The starting point for the hydraulic grade line determination should be either the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, $(d_c + D)/2$, whichever is greater. The water surface elevation in the detention pond at the outlet (tailwater) was given as 999.5 m.

The critical depth for a 750 mm pipe carrying 0.3 m^3/s is 0.3 m (Figure 4.10a) and so

$$(d_c + D)/2 = (0.3 + 0.75)/2 = 0.53 \text{ m}$$

Adding this depth to the invert elevation

$$999.77 + 0.53 = 1000.30$$

which is greater than the tailwater elevation and will be used as the starting point for the HGL calculation.

2. HGL/EGL at Station 17+98.50

The first energy loss that occurs is the expansion loss, or exit loss, as the flow exits the storm drain into the tailwater region. To illustrate the concepts involved in applying the energy equation in storm drain design, the calculation will maintain the three components of energy (velocity head, pressure head and elevation head).

First, apply the energy equation from the tailwater to just inside the pipe at Station 17+98.50.

$$\frac{V_{17+98.50}^2}{2g} + \frac{P_{17+98.50}}{\gamma} + Z_{17+98.50} = \frac{V_{tw}^2}{2g} + \frac{P_{tw}}{\gamma} + Z_{tw} + h_L$$

The exit loss is

$$h_L = K\,[V^2 / (2g)]$$

For the exit, the value of K is 1.0 and the loss is

$$h_L = 1.0\ (0.9)^2 / [(2)(9.81)] = 0.04 \text{ m}$$

As determined above, the starting HGL was 1000.30. Remembering that the HGL equals the sum of the pressure head plus the elevation head and assuming negligible velocity in the tailwater ($V_{tw} = 0$):

$$\frac{(0.9)^2}{2(9.81)} + \frac{P_{17+98.50}}{\gamma} + 999.77 = \frac{(0)^2}{2g} + 1000.30 + 0.04$$

$$\frac{P_{17+98.50}}{\gamma} = 0.53 \text{ m}$$

Therefore, the EGL at 17+98.50 is (velocity head + pressure head + elevation head)

$$0.04 + 0.53 + 999.77 = 1000.34 \text{ ft}$$

3. Apply the energy equation from Station 17+98.50 to the downstream side of the access hole at Station 18+98.50

$$\frac{V^2_{18+98.50}}{2g} + \frac{P_{18+98.50}}{\gamma} + Z_{18+98.50} = \frac{V^2_{17+98.50}}{2g} + \frac{P_{17+98.50}}{\gamma} + Z_{17+98.50} + h_f$$

The headloss is due to friction.

$$h_f = LS_f = L\left(\frac{Qn}{K_u D^{8/3}}\right)^2 \quad \text{where } K_u = 0.312$$

Therefore, the friction headloss from Station 17+98.50 to 18+98.50 is

$$h_f = 100\left[\frac{(0.3)\ (0.013)}{0.312\ (0.75)^{8/3}}\right]^2 = 0.07 \text{ m}$$

$$\frac{(0.9)^2}{2(9.81)} + \frac{P_{18+98.50}}{\gamma} + 999.87 = \frac{(0.9)^2}{2(9.81)} + 0.53 + 999.77 + 0.07$$

Therefore $\frac{P_{18+98.50}}{\gamma} = 0.50 \text{ m}$

If the pipe is not flowing full, check for supercritical flow. The Q/Q_{full} ratio was previously calculated as 0.86 and from Figure 7.1 the d/D ratio is 0.70. Therefore, the normal depth is 0.70 (0.75) = 0.53 m. Compared to the previously calculated critical depth of 0.3 m, the flow is subcritical.

And the EGL at 18+99.50 is (velocity head + pressure head + elevation head)

$$0.04 + 0.50 + 999.87 = 1000.41 \text{ m}$$

4. Energy Loss through the Access Hole

STEP 1: Initial Access Hole Energy Level

The initial energy level in the access hole structure (E_{ai}) is calculated as:

$$E_{ai} = \max (E_{aio}, E_{ais}, E_{aiu})$$

where:

E_{aio} = Estimated access hole energy level for outlet control (full and partial flow)
E_{ais} = Estimated access hole energy level for inlet control (submerged)
E_{aiu} = Estimated access hole energy level for inlet control (unsubmerged)

Estimated Energy Level for Outlet Control: Partial Flow and Full Flow

$$E_{aio} = E_i + H_i$$

where:

E_i = Outflow pipe specific energy head (calculated using Equation 7-10)
H_i = Entrance loss assuming outlet control (calculated using Equation 7-11)

$$E_i = EGL_i - Z_i = 1000.41 – 999.87 = 0.54$$

$$H_i = K_i [V^2 / (2g)] = 0.2 [0.9^2 / (2g)] = 0.01$$

$$E_{aio} = E_i + H_i = 0.54 + 0.01 = 0.55$$

Estimated Energy Level for Inlet Control: Submerged

$$E_{ais} = D_o (DI)^2$$

$$DI = Q / [A (g D_o)^{1/2}]$$

where:

A = Area of outflow pipe, m^2
D_o = Diameter of outflow pipe, m

$$A = 3.14 (0.75^2) / 4 = 0.44$$

$$DI = 0.3 / [0.44 ((9.81) (0.75))^{1/2}] = 0.25$$

$$E_{ais} = 0.75 (0.25)^2 = 0.05$$

Estimated Energy Level for Inlet Control: Unsubmerged

$$E_{aiu} = 1.6 D_o (DI)^{2/3} = 1.6 (0.75) (0.25)^{2/3} = 0.47$$

Therefore, the resulting initial energy level is

$$E_{ai} = \max (E_{aio}, E_{ais}, E_{aiu}) = \max (0.55, 0.05, 0.47) = 0.55 \text{ m}$$

STEP 2: Adjustments for Benching, Angled Inflow, and Plunging Inflow

The initial structure energy level calculated in STEP 1 is used as a basis for estimating additional losses for: (1) discharges entering the structure at angles other then 180 degrees; (2) benching configurations; and (3) plunging flows entering the structure at elevations above the water depth in the access hole (Flows entering a structure from an inlet can be treated as plunging flows). The equation is:

$$E_a = E_{ai} + H_B + H_\theta + H_P$$

where:

H_B = Additional energy loss for benching (floor configuration)
H_θ = Additional energy loss for angled inflows other than 180 degrees
H_P = Additional energy loss for plunging flows

Since we do not have any benching, angled flows or plunging flows, $E_a = E_{ai} = 0.55$. Assuming the access hole invert (z_a) is the same elevation as the outflow pipe invert (z_i) allows determination of the access hole energy gradeline (EGL_a):

$$EGL_a = E_a + Z_a = 0.55 + 999.87 = 1000.42$$

STEP 3: Inflow Pipe Exit Losses

The final step is to calculate the energy gradeline into each inflow pipe. The FHWA method considers two cases: (1) plunging inflow pipe(s) and (2) non-plunging inflow pipe(s). For pipes that are plunging, the inflow pipe energy gradeline (EGL_o) is taken as the energy gradeline calculated from the inflow pipe hydraulics. Non-plunging pipes have a hydraulic connection to the water in the access hole and are identified when the access hole energy gradeline is greater than the inflow pipe invert elevation. This situation exists in our case and the inflow pipe energy head (EGL_o) is defined as:

$$EGL_o = EGL_a + H_o$$

where:

H_o = Inflow pipe exit loss, calculated using Equation 7.19

$$H_o = K_o [V^2 /(2g)] = 0.4 [0.9^2 / (2g)] = 0.02$$

And so,

$$EGL_o = 1000.42 + 0.02 = 1000.44 \text{ at Station 19+00.}$$

HEC-22 identifies a number of special cases that should be considered to verify this result. In our example, Case D applies (see HEC-22, Section 7.5) and the adjusted energy grade line is:

$$EGL_o = V^2 / (2g) + \text{normal depth} + Z$$

The previously calculated normal depth was 0.53 m. The adjusted EGL_o is:

$$EGL_o = V^2/(2g) + \text{normal depth} + Z = (0.9^2/2g) + 0.53 + 999.9 = 1000.47$$

5. Apply the energy equation from Station 19+00 to 20+00

$$\frac{V_{20+00}^2}{2g} + \frac{P_{20+00}}{\gamma} + Z_{20+00} = \frac{V_{19+00}^2}{2g} + \frac{P_{19+00}}{\gamma} + Z_{19+00} + h_L$$

The headloss is due to friction, and since the discharge and pipe are the same as the reach from 17+98.50 to 18+98.50, the friction loss is again 0.07 m.

$$\frac{(0.9)^2}{2g} + \frac{P_{20+00}}{\gamma} + 1000.0 = 1000.47 + 0.07$$

Therefore $\frac{P_{20+00}}{\gamma} = 0.50 \text{ m}$

Since the computed pressure head is less than the normal depth of 0.53 m, use the normal depth in the energy equation.

And the EGL at Station 20+00 is

$$0.04 + 0.53 + 1000.00 = 1000.57$$

6. Calculate the HGL at each section

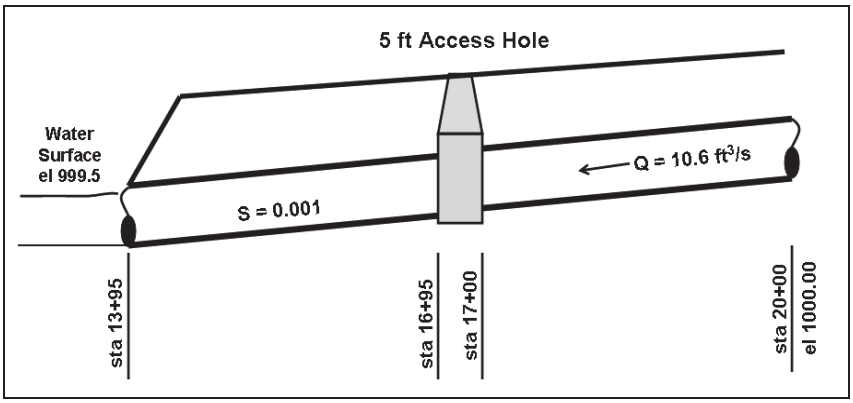
$$HGL = EGL - V^2/(2g)$$

Since the discharge, slope and pipe size do not change, the velocity head is the same through the network (0.04 m).

Location	Station	Elevation	EGL	HGL
Outfall	17+98.50	999.77	1000.34	1000.30
Downstream side of access hole	18+98.50	999.87	1000.41	1000.37
Upstream side of access hole	19+00	999.90	1000.47	1000.43
Upstream end of storm drain	20+00	1000.00	1000.57	1000.53

EXAMPLE PROBLEM 8.2 (English Units)

Given: An RCP (n = 0.013) storm drain must be designed to carry 10.6 ft^3/s on a 0.001 slope. The storm drain will consist of 600 ft of pipe with a 5 ft (diameter) access hole in the middle. The invert of the pipe at the upstream end is 1000.00 ft at Station 20+00. The storm drain will discharge into a channel where the water surface elevation is 999.5 ft. Complete preliminary design to define pipe size, crown drop and invert elevations. Calculate the crown drop based on headloss, but assume a minimum value of 0.1 ft. Complete final design to evaluate the HGL and EGL profiles.



Find: Pipe size, crown drop and invert elevations based on preliminary design.
HGL/EGL based on final design.

Preliminary Design

Preliminary design starts at the upstream end and works downstream.

1. Use the full-flow Manning's equation to size the pipe:

$$Q = (K_u/n)\, D^{8/3}\, S^{1/2} \qquad \text{where } K_u = 0.46 \text{ for English Units}$$

$$D = \left[\frac{Q\,n}{0.46\;S^{1/2}}\right]^{3/8}; \qquad D = \left[\frac{(10.6)(0.013)}{(0.46)\;(0.001)^{1/2}}\right]^{3/8} = 2.3\,\text{ft}$$

Therefore, based on nominal pipe sizes and to ensure open channel flow, use a 30-inch pipe.

2. The invert at the upstream end was given as 1000.00 ft. The elevation at the upstream side of the access hole (Sta 17+00) will be:

$$Z = 1000.00 - 300\,(0.001) = 999.70 \text{ ft}$$

3. The elevation on the downstream side of the access hole (Sta 16+95) should incorporate the crown drop based on the approximate energy loss through the structure.

The approximate energy loss through the access hole is based on Equation 7.6:

$$h_L = K\,[V^2 / (2g)]$$

The K value for a straight through access hole is 0.15.

The velocity in the part full pipe can be estimated using the hydraulic elements graph (Figure 7.1). The full flow capacity of a 30-in RCP pipe on a 0.001 slope is:

$$Q_{full} = (K_u/n)\, D^{8/3}\, S^{1/2} \qquad \text{where } K_u = 0.46 \text{ for English units}$$

$$Q_{full} = (0.46/0.013)\,(2.5)^{8/3}\,(0.001)^{1/2} = 12.9 \text{ ft}^3/\text{s}$$

And the full flow velocity is

$Q = V A, \quad V = Q / A$

$A = [\pi (D)^2] / 4 = [\pi (2.5)^2] / 4 = 4.91 ft^2$

$V = 12.9 / 4.91 = 2.63$ ft/s

The Q/Q_{full} ratio is 10.6/12.9 = 0.82, and from the Figure 7.1, the V/V_{full} ratio will be about 1.12. Therefore, the part-full velocity is:

$V = 1.12 (V_{full}) = 1.12 (2.63) = 3.0$ ft/s

and headloss through the access hole is

$h_L = K [V^2 / (2g)] = 0.15 \{(3.0)^2 / [2 (32.2)]\} = 0.021$ ft

Consequently, the elevation on the downstream side of the access hole based on a minimum crown drop of 0.10 ft is

$Z = 999.7 – 0.10 = 999.60$

4. The elevation at the outfall (Sta 13+95) is then

$Z = 999.60 – 300(0.001) = 999.30$

5. Summary of Preliminary design data

Storm drain diameter = 30 inch

Pipeline invert elevations

Location	Station	Elevation
Outfall	13+95	999.30
Downstream side of access hole	16+95	999.60
Upstream side of access hole	17+00	999.70
Upstream end of storm drain	20+00	1000.00

Evaluation of HGL/EGL

HGL/EGL evaluation starts at the outlet and works upstream. The calculation is based on the energy equation applied from a known downstream location to an unknown upstream location, accounting for the intervening energy losses.

1. Starting HGL

The starting point for the hydraulic grade line determination should be either the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, $(d_c + D)/2$, whichever is greater. The water surface elevation in the detention pond at the outlet (tailwater) was given as 999.50 ft.

The critical depth for a 30-inch pipe carrying 10.6 cfs is 1.1 ft (Figure 4.10b) and so

$$(d_c + D)/2 = (1.1+2.5)/2 = 1.8 \text{ ft}$$

Adding this depth to the invert elevation

$$999.30 + 1.8 = 1001.10$$

which is greater than the tailwater elevation and will be used as the starting point for the HGL calculation.

2. HGL/EGL at Station 13+95

The first energy loss that occurs is the expansion loss, or exit loss, as the flow exits the storm drain into the tailwater region. To illustrate the concepts involved in applying the energy equation in storm drain design, the calculation will maintain the three components of energy (velocity head, pressure head and elevation head).

First, apply the energy equation from the tailwater to just inside the pipe at Station 13+95.

$$\frac{V_{13+95}^2}{2g} + \frac{P_{13+95}}{\gamma} + Z_{13+95} = \frac{V_{tw}^2}{2g} + \frac{P_{tw}}{\gamma} + Z_{tw} + h_L$$

The exit loss is:

$$h_L = K\,[V^2/(2g)]$$

For the exit, the value of K is 1.0 and the loss is:

$$h_L = 1.0\,[(3.0)^2/((2)(32.2))] = 0.14 \text{ ft}$$

As determined above, the starting HGL was 1001.10. Remembering that the HGL equals the sum of the pressure head plus the elevation head and assuming negligible velocity in the tailwater ($V_{tw} = 0$).

$$\frac{(3.0)^2}{2(32.2)} + \frac{P_{13+95}}{\gamma} + 999.30 = \frac{(0)^2}{2g} + 1001.10 + 0.14$$

Therefore $\frac{P_{13+95}}{\gamma} = 1.8 \text{ ft}$

Therefore, the EGL at 13+95 is (velocity head + pressure head + elevation head).

$$0.14 + 1.8 + 999.30 = 1001.24 \text{ ft}$$

3. Apply the energy equation from Station 13+95 to the downstream side of the access hole at Station 16+95.

$$\frac{V_{16+95}^2}{2g} + \frac{P_{16+95}}{\gamma} + Z_{16+95} = \frac{V_{13+95}^2}{2g} + \frac{P_{13+95}}{\gamma} + Z_{13+95} + h_f$$

The headloss is due to friction.

$$h_f = LS_f = L\left(\frac{Qn}{K_u D^{8/3}}\right)^2 \quad \text{where } K_u = 0.46$$

Therefore, the friction headloss from Station 13+95 to 16+95 is:

$$h_f = 300\left[\frac{(10.6)\ (0.013)}{0.46\ (2.5)^{8/3}}\right]^2 = 0.20\ \text{ft}$$

$$\frac{(3.0)^2}{2(32.2)} + \frac{P_{16+95}}{\gamma} + 999.60 = \frac{(3.0)^2}{2(32.2)} + 1.8 + 999.30 + 0.20$$

Therefore $\frac{P_{16+95}}{\gamma} = 1.7\ \text{ft}$

If the pipe is not flowing full, check for supercritical flow. The Q/Q_{full} ratio was previously calculated as 0.82 and from Figure 7.1, the d/D ratio is 0.68. Therefore, the normal depth is 0.68 (2.5) = 1.7 ft. Compared to the previously calculated critical depth of 1.1 ft, the flow is subcritical.

And the EGL at 16+95 is (velocity head + pressure head + elevation head)

0.14 + 1.7 + 999.60 = 1001.44 ft

4. Energy Loss through the Access Hole

STEP 1: Initial Access Hole Energy Level

The initial energy level in the access hole structure (E_{ai}) is calculated as

$$E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu})$$

where:

E_{aio} = Estimated access hole energy level for outlet control (full and partial flow)
E_{ais} = Estimated access hole energy level for inlet control (submerged)
E_{aiu} = Estimated access hole energy level for inlet control (unsubmerged)

Estimated Energy Level for Outlet Control: Partial Flow and Full Flow

$$E_{aio} = E_i + H_i$$

where:

E_i = Outflow pipe specific energy head (calculated using Equation 7-10)
H_i = Entrance loss assuming outlet control (calculated using Equation 7-11)

$E_i = EGL_i - Z_i = 1001.44 – 999.60 = 1.84$

$H_i = K_i (V^2 /2g) = 0.2 (3.0^2/2g) = 0.03$

$E_{aio} = E_i + H_i = 1.84 + 0.03 = 1.87$

Estimated Energy Level for Inlet Control: Submerged

$E_{ais} = D_o (DI)^2$

$DI = Q / [A (g D_o)^{1/2}]$

where:

A = Area of outflow pipe, ft^2
D_o = Diameter of outflow pipe, ft

$A = 3.14 (2.5^2) / 4 = 4.91 ft^2$

$DI = 10.6 / [4.91 ((32.2) (2.5))^{1/2}] = 0.24$

$E_{ais} = 2.5 (0.24)^2 = 0.14$

Estimated Energy Level for Inlet Control: Unsubmerged

$E_{aiu} = 1.6 D_o (DI)^{2/3} = 1.6 (2.5) (0.24)^{2/3} = 1.54$

Therefore, the resulting initial energy level is

$E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu}) = \max (1.87, 0.14, 1.54) = 1.87$ ft

STEP 2: Adjustments for Benching, Angled Inflow, and Plunging Inflow

The initial structure energy level calculated in STEP 1 is used as a basis for estimating additional losses for: (1) discharges entering the structure at angles other then 180 degrees; (2) benching configurations; and (3) plunging flows entering the structure at elevations above the water depth in the access hole (Flows entering a structure from an inlet can be treated as plunging flows). The equation is:

$E_a = E_{ai} + H_B + H_\theta + H_P$

where:

H_B = Additional energy loss for benching (floor configuration)
H_θ = Additional energy loss for angled inflows other than 180 degrees
H_P = Additional energy loss for plunging flows

Since we do not have any benching, angled flows or plunging flows, $E_a = E_{ai} = 1.87$. Assuming the access hole invert (z_a) is the same elevation as the outflow pipe invert (z_i) allows determination of the access hole energy gradeline (EGL_a):

$$EGL_a = E_a + Z_a = 1.87 + 999.60 = 1001.47$$

STEP 3: Inflow Pipe Exit Losses

The final step is to calculate the energy gradeline into each inflow pipe. The FHWA method considers two cases: (1) plunging inflow pipe(s) and (2) non-plunging inflow pipe(s). For pipes that are plunging, the inflow pipe energy gradeline (EGL_o) is taken as the energy gradeline calculated from the inflow pipe hydraulics. Non-plunging pipes have a hydraulic connection to the water in the access hole and are identified when the access hole energy gradeline is greater than the inflow pipe invert elevation. This situation exists in our case and the inflow pipe energy head (EGL_o) is defined as:

$$EGL_o = EGL_a + H_o$$

where:

H_o = Inflow pipe exit loss, calculated using Equation 7.18

$$H_o = K_o (V^2 /2g) = 0.4 (3.0^2 / 2g) = 0.06$$

And so,

$$EGL_o = 1001.47 + 0.06 = 1001.53 \text{ at Station } 17+00$$

HEC-22 identifies a number of special cases that should be considered to verify this result. In our example, Case D applies (see HEC-22, Section 7.5) and the adjusted energy grade line is

$$EGL_o = (V^2/2g) + \text{normal depth} + Z$$

The previously calculated normal depth was 1.7 ft. The adjusted EGL_o is:

$$EGL_o = (V^2/2g) + \text{normal depth} + Z = (3.0^2/2g) + 1.7 + 999.70 = 1001.54$$

5. Apply the energy equation from Station 17+00 to 20+00

$$\frac{V_{20+00}^2}{2g} + \frac{P_{20+00}}{\gamma} + Z_{20+00} = \frac{V_{17+00}^2}{2g} + \frac{P_{17+00}}{\gamma} + Z_{17+00} + h_L$$

The headloss is due to friction, and since the discharge and pipe are the same as the reach from 13+95 to 16+95, the friction loss is again 0.20 ft.

$$\frac{(3.0)^2}{2g} + \frac{P_{20+00}}{\gamma} + 1000.00 = 1001.54 + 0.20$$

Therefore $\frac{P_{20+00}}{\gamma} = 1.60$ ft

Since the computed pressure head is less than the normal depth of 1.7 ft, use the normal depth in the energy equation.

And the EGL at Station 20+00 is:

$$0.14 + 1.70 + 1000.00 = 1001.84 \text{ ft}$$

6. Calculate the HGL at each section

$$HGL = EGL - V^2 / (2g)$$

Since the discharge, slope and pipe size do not change, the velocity head is the same through the network (0.14 ft).

Location	Station	Elevation	EGL	HGL
Outfall	13+95	999.30	1001.24	1001.10
Downstream side of access hole	16+95	999.60	1001.44	1001.30
Upstream side of access hole	17+00	999.70	1001.54	1001.40
Upstream end of storm drain	20+00	1000.00	1001.84	1001.70

CHAPTER 9

CLOSED-CONDUIT APPLICATIONS – CULVERT DESIGN

9.1 GENERAL DESIGN CONCEPTS

Typical closed-conduit facilities in highway drainage include culverts and storm drains. A storm drain facility can be a much more extensive closed-conduit system than a cross drainage system such as a culvert. In some respects, a storm drain is simply a long culvert. Storm drain systems consist of inlets connected to an underground pipe and an outlet facility. Storm drain systems are often used when the capacity of the roadway (established by the allowable spread) is exceeded or for the collection and diversion of median drainage when the capacity of the swale is exceeded. A storm drain system may also be used in high gradient situations where erosion control is a concern.

Q15

Culverts are commonly used for cross drainage and can range in size from a single small culvert draining an isolated depression to multiple barrel designs and/or very large culverts for passing major stream channels under a roadway. Small culverts are also used for downdrains to protect fill slopes or to divert roadway water from a bridge deck.

Typical pipe materials used in storm drains include reinforced concrete pipe (RCP), corrugated metal pipe (CMP) and plastic pipe. These same materials are common for culverts, however, culverts are available in a variety of cross section shapes and often a shape other than circular is desirable. Conduit and culvert material are typically available in standard (nominal) sizes. Conduit size should not be decreased in the downstream direction, even if hydraulic calculations suggest this is possible due to maintenance issues such as deposition and clogging.

Energy dissipation is often required at the outlet of a storm drain or culvert to prevent erosion. Chapter 10 provides information on energy dissipators based on HEC-14 (Thompson and Kilgore 2006). Debris control structures may be required at the entrances of some culverts. HEC-9 provides guidance on debris control structures for culvert and bridges (Bradley et al. 2005). Maintenance is required for any closed-conduit facility. Sediment deposition within the conduit and debris removal at the entrances are typical maintenance items.

9.2 CULVERT DESIGN APPROACH

A culvert is a conduit that conveys flow through a roadway embankment or past some other type of flow obstruction. Culverts are typically constructed of concrete (reinforced and nonreinforced), corrugated metal (aluminum or steel) and plastic in a variety of cross sectional shapes. The most common cross sectional shapes for culverts are illustrated in Figure 9.1a and typical entrance conditions are shown in Figure 9.1b. The selection of culvert material depends on structural strength, hydraulic roughness, durability, and corrosion and abrasion resistance.

Flow conditions in a culvert may occur as open-channel flow, gravity full flow or pressure flow, or in some combination of these conditions. A complete theoretical analysis of the hydraulics of culvert flow is time-consuming and difficult. Flow conditions depend on a complex interaction of a variety of factors created by upstream and downstream conditions, barrel characteristics and inlet geometry. For purposes of design, standard procedures and nomographs have been developed to simplify the analysis of culvert flow. These procedures are detailed in Hydraulic Design Series Number 5 (HDS-5) entitled "Hydraulic Design of Highway Culverts." (Normann et al. 2005). The following information summarizes the basic design concepts and principles for culverts.

Q16

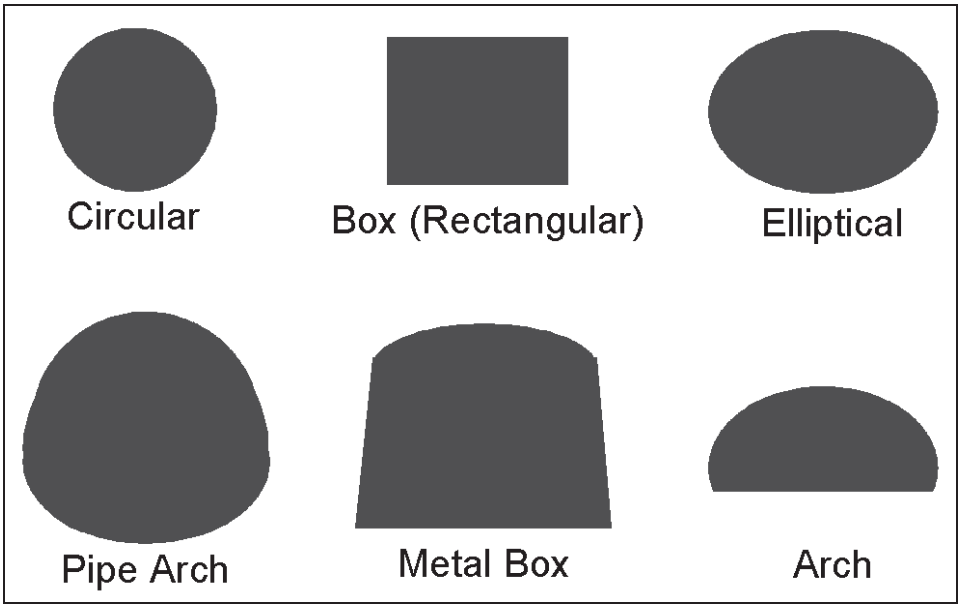


Figure 9.1a. Commonly used culvert shapes.

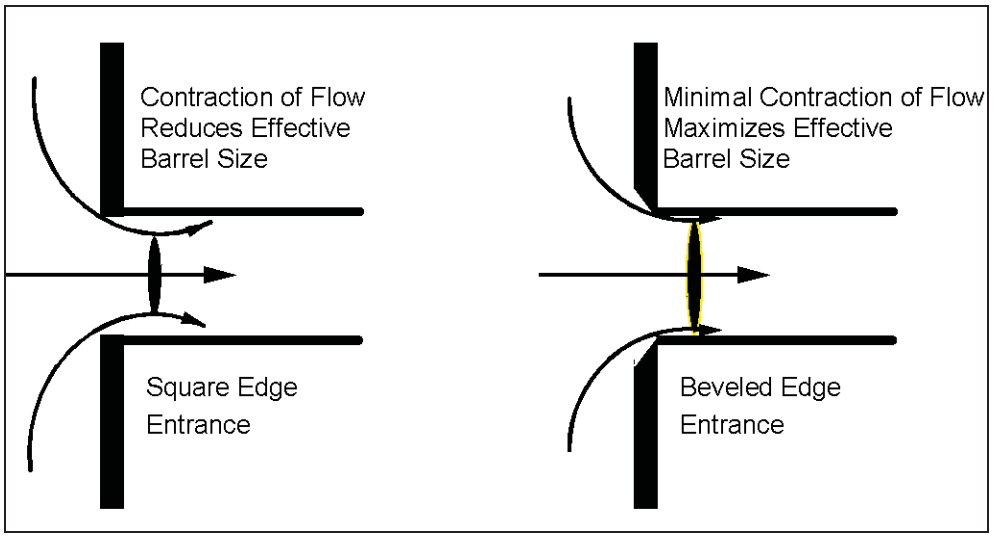


Figure 9.1b. Entrance contraction (schematic).

9.3 TYPES OF CULVERT INLETS AND OUTLETS

Q17

A culvert typically represents a significant contraction of flow over conditions in the upstream and downstream channels and often is a hydraulic control point in the channel. Provision of a more gradual flow transition at the inlet of a culvert can improve the discharge capacity of the culvert by reducing the energy losses associated with flow contraction. Culvert inlets are available in a variety of configurations and may be prefabricated or constructed in place. Commonly used inlet configurations include projecting culvert barrels, cast-in-place concrete headwalls, precast or prefabricated end sections, and culvert ends mitered to conform to the fill slope (Figure 9.2). Structural stability, aesthetics, erosion control, fill retention, economics, safety, and hydraulic performance are considerations in the selection of an inlet.

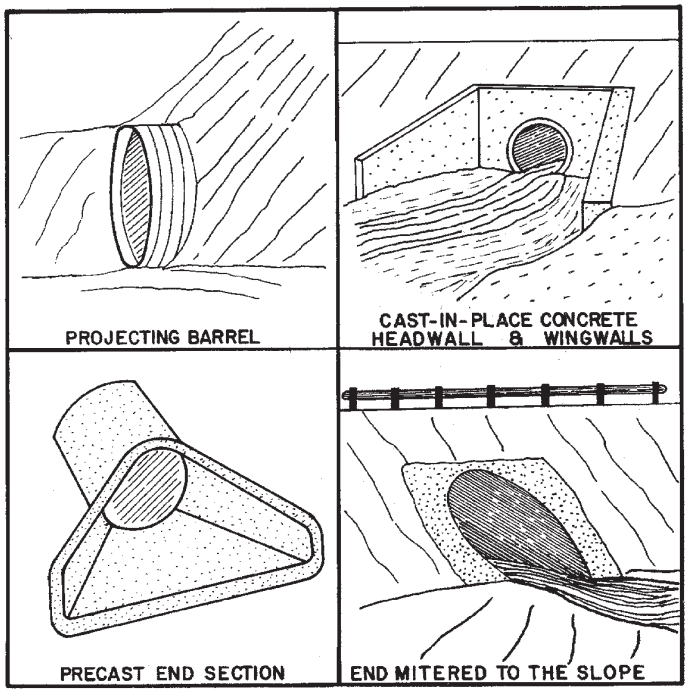


Figure 9.2. Four standard inlet types (schematic).

Hydraulic performance is improved by use of beveled edges rather than square edges, as illustrated in Figure 9.1b. Side-tapered and slope-tapered inlets, commonly referred to as improved inlets, can significantly increase culvert capacity. Figure 9.3 illustrates side-tapered and slope-tapered inlet conditions. A side-tapered inlet provides a more gradual contraction of flow and reduces energy losses. A slope-tapered inlet, or depressed inlet, increases the effective head on the control section and improves culvert efficiency.

Culvert outlet configuration can be similar to any of the typical inlet configurations; however, hydraulic performance of a culvert is influenced more by tailwater conditions in the downstream channel than by the type of outlet. Outlet design is important for transitioning flow back into the natural channel, since outlet velocities are typically high and can cause scour of the downstream streambed and bank.

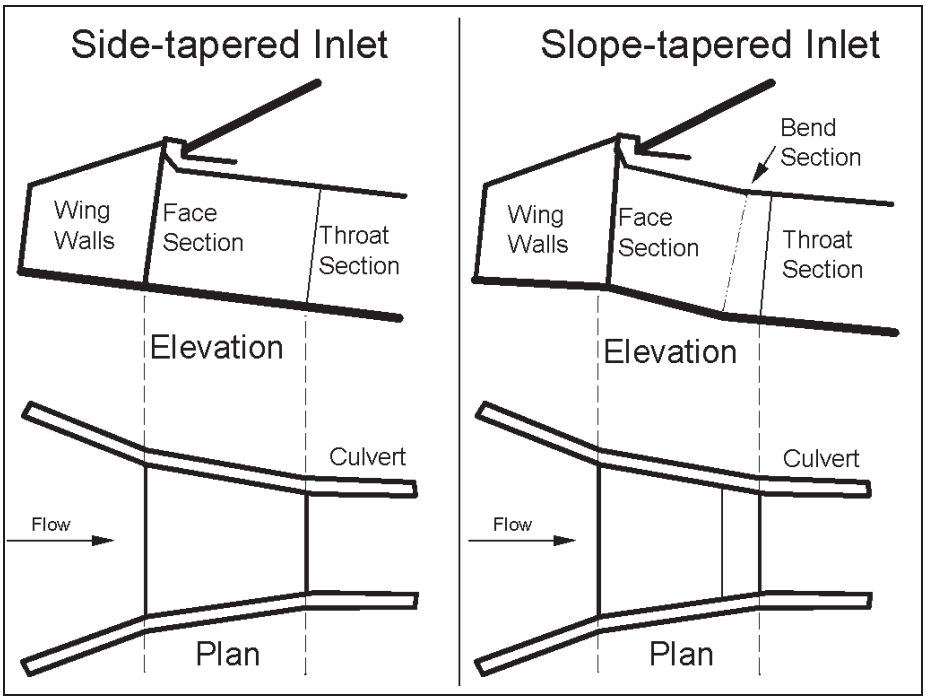


Figure 9.3. Side- and slope-tapered inlets.

9.4 CULVERT FLOW CONDITIONS

A culvert may flow full over all its length or partially full. Full flow throughout a culvert is rare, and generally some portion of the barrel flows partly full. A water surface profile analysis is the only way to determine accurately how much of the barrel flows full. Pressure flow conditions in a culvert can be created by either high downstream or upstream water surface elevations. Regardless of the cause, the capacity of a culvert operating under pressure flow is affected by up- and downstream conditions and by the hydraulic characteristics of the culvert.

Partly full flow, or open-channel flow, in a culvert may occur as subcritical, critical or supercritical flow. Gravity full flow, where the pipe flows full with no pressure and the water surface just touches the crown of the pipe, is a special case of free surface flow and is analyzed in the same manner as open-channel flow.

9.5 TYPES OF FLOW CONTROL

Q18

Based on a variety of laboratory tests and field experience, two basic types of flow control have been defined for culverts: (1) inlet control, and (2) outlet control. Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. The hydraulic control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location and the flow regime immediately downstream is supercritical. Hydraulic characteristics downstream of the inlet do not affect culvert capacity. Upstream water surface elevation and inlet geometry are the primary factors influencing culvert capacity.

Figure 9.4 illustrates typical inlet control conditions. The type of flow depends on the submergence of the inlet and outlet ends of the culvert; however, in each case the control section is at the inlet end of the culvert. For low headwater conditions the entrance of the culvert operates as a weir, and for headwaters submerging the entrance the entrance operates as an orifice. Figure 9.4a depicts a condition where neither the inlet nor the outlet end of the culvert are submerged. The flow passes through critical depth just downstream of the culvert entrance and the flow in the barrel is supercritical. The barrel flows partly full over its length, and the flow approaches normal depth at the outlet end.

Figure 9.4b shows that submergence of the outlet end of the culvert does not assure outlet control. In this case, the flow just downstream of the inlet is supercritical and a hydraulic jump forms in the culvert barrel.

Figure 9.4c is a more typical design situation. The inlet end is submerged and the outlet end flows freely. Again, the flow is supercritical and the barrel flows partly full over its length. Critical depth is located just downstream of the culvert entrance, and the flow is approaching normal depth at the downstream end of the culvert.

Figure 9.4d is an unusual condition illustrating the fact that even submergence of both the inlet and the outlet ends of the culvert does not assure full flow. In this case, a hydraulic jump will form in the barrel. The median inlet provides ventilation of the culvert barrel. If the barrel were not ventilated, sub-atmospheric pressures could develop which might create an unstable condition during which the barrel would alternate between full flow and partly full flow.

Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert under outlet control. All the geometric and hydraulic characteristics of the culvert play a role in determining culvert capacity. Figure 9.5 illustrates typical outlet control conditions. Condition 9.5a represents the classic full flow condition, with both inlet and outlet submerged. The barrel is in pressure flow throughout its length.

Condition 9.5b depicts the outlet submerged with the inlet unsubmerged. For this case, the headwater is shallow so that the inlet crown is exposed as the flow contracts into the culvert.

Condition 9.5c shows the entrance submerged to such a degree that the culvert flows full throughout its entire length while the exit is unsubmerged. This is a rare condition, it requires an extremely high headwater to maintain full barrel flow with no tailwater. Outlet velocities are unusually high under this condition.

Condition 9.5d is more typical. The culvert entrance is submerged by the headwater and the outlet end flows freely with a low tailwater. For this condition, the barrel flows partly full over at least part of its length (subcritical flow) and the flow passes through critical depth just upstream of the outlet.

Condition 9.5e is also typical, with neither the inlet nor the outlet end of the culvert submerged. The barrel flows partly full over its entire length, and the flow profile is subcritical.

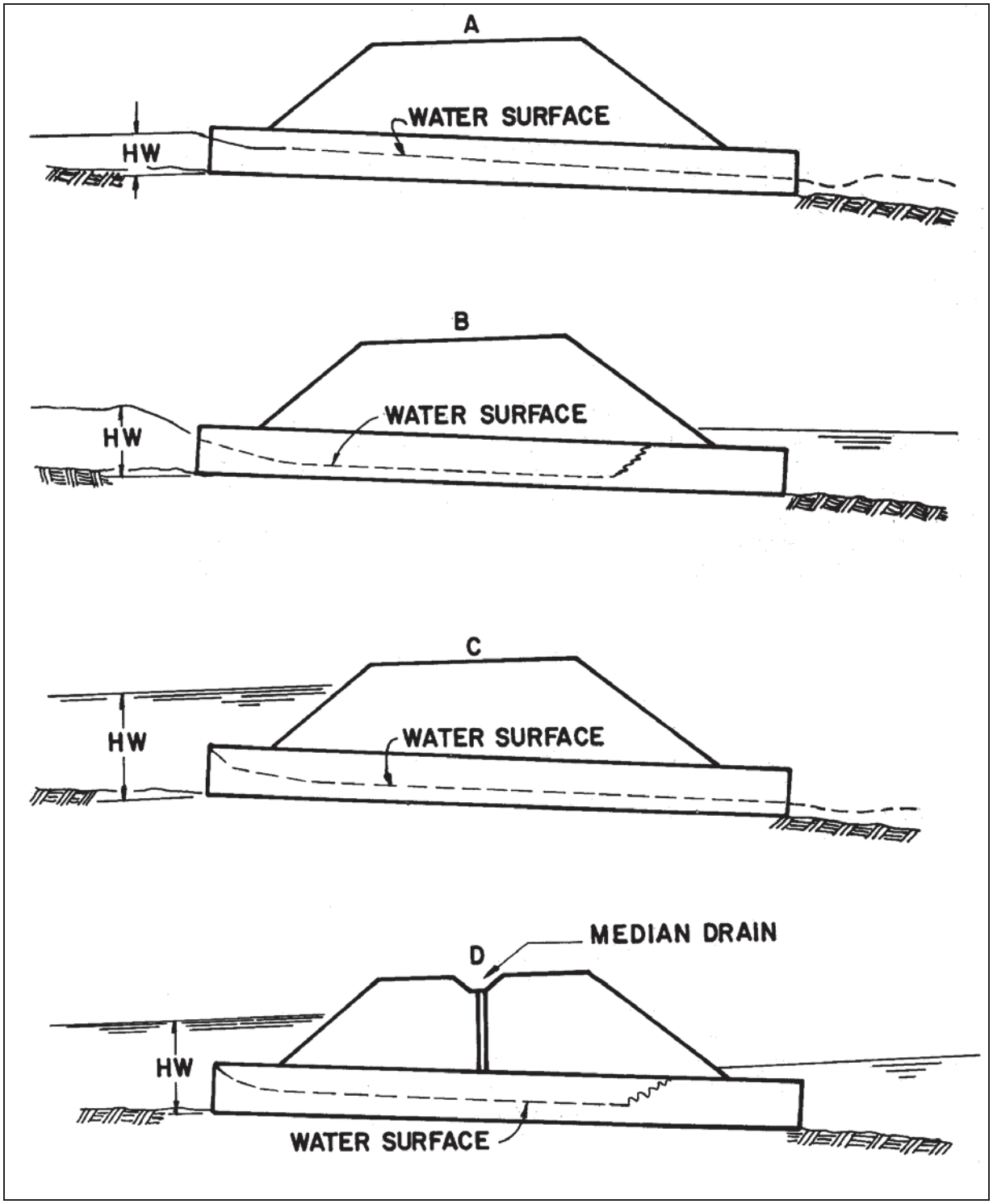


Figure 9.4. Types of inlet control.

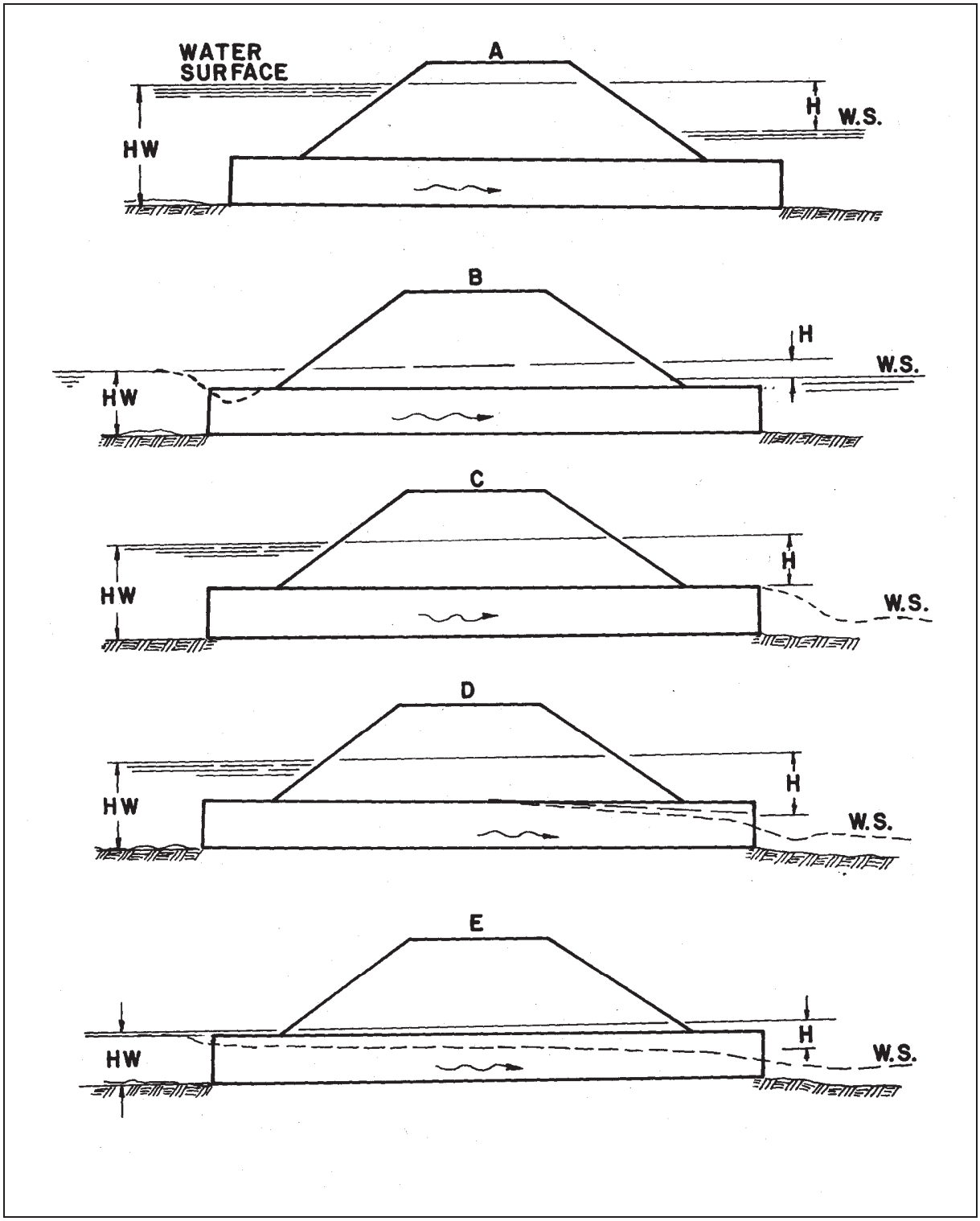


Figure 9.5. Types of outlet control.

9.6 HEADWATER AND TAILWATER CONSIDERATIONS

Q19

Energy is required to force flow through the constricted opening represented by a culvert. This energy occurs as an increased water surface elevation on the upstream side of the culvert. The headwater depth (HW) is defined as the depth of water at the culvert entrance. In areas with flat ground slope or high fills a considerable amount of ponding may occur upstream of the culvert. If significant, this ponding can attenuate flood peaks and may justify a reduction in the required culvert size.

Tailwater is defined as the depth of water downstream of the culvert, measured from the outlet invert. Tailwater is an important factor in determining culvert capacity under outlet control conditions. Tailwater conditions are most accurately estimated by water surface profile analysis of the downstream channel; however, when appropriate, tailwater conditions may be estimated by normal depth approximations.

9.7 PERFORMANCE CURVES

A performance curve is a plot of headwater depth or elevation versus flow rate. A performance curve can be used to evaluate the consequences of higher flow rates, such as the potential for overtopping the roadway if the design event is exceeded or to evaluate the benefits of inlet improvements. In developing a performance curve both inlet and outlet control curves must be plotted, since the dominant control is hard to predict and may shift over a range of flow rates.

Figure 9.6 illustrates a typical culvert performance curve. Below a headwater elevation of 4.3, the culvert operates under inlet control suggesting that inlet improvements might increase the culvert capacity and take better advantage of the culvert barrel capacity. A culvert that operates with inlet control over the range of design conditions could also be designed with additional barrel roughness to reduce outlet velocities, should downstream erosion be a concern.

9.8 CULVERT DESIGN METHOD

The basic design method is based on the location of the control (inlet or outlet). Although control may oscillate from inlet to outlet, the concept of "minimum performance" is applied meaning that while the culvert may operate more efficiently at times, it will never operate at a lower performance than calculated. The design procedure then is to assume a pipe size and material and calculate the headwater elevation for both inlet and outlet control. The higher of the two is designated as the controlling headwater elevation. The controlling headwater elevation is compared to the desired design headwater, usually governed by overtopping considerations, to determine if the assumed culvert size is acceptable.

Outlet velocity should then be considered to evaluate the need for outlet protection. If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. Velocity at normal depth is assumed to be the outlet velocity. If the controlling headwater is based on outlet control, determine the area of flow at the outlet based on the barrel geometry and the following: (1) critical depth if the tailwater is below critical depth, (2) tailwater depth if the tailwater is between critical depth and the top of the barrel, and (3) height of the barrel if the tailwater is above the top of the barrel.

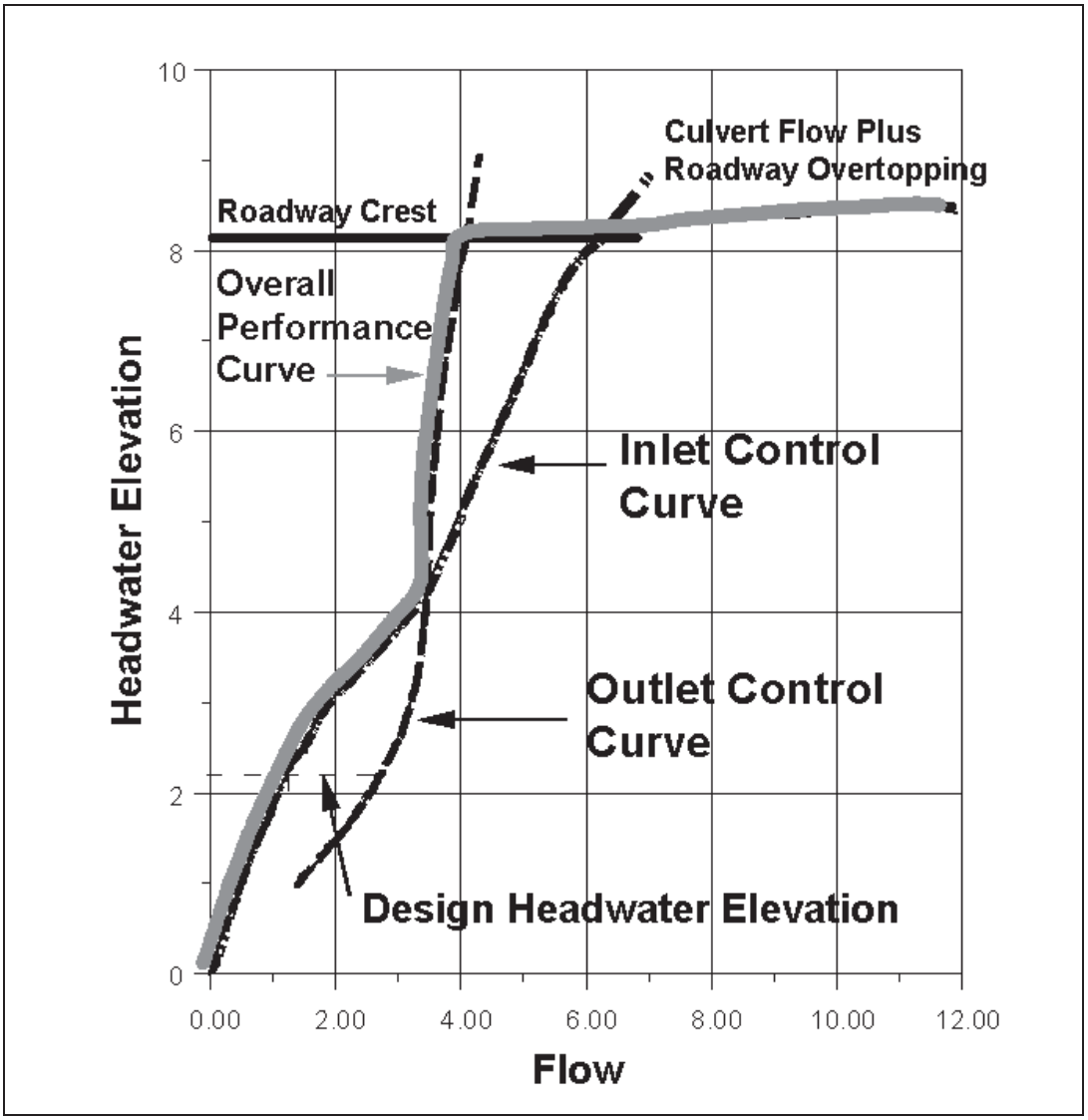


Figure 9.6. Culvert performance curve.

Evaluation of headwater conditions and outlet velocity is repeated until an acceptable culvert configuration is determined. To facilitate the design process, a Culvert Design Form is provided in HDS-5. Once the barrel is selected, it must be fitted into the roadway cross section. The culvert barrel must have adequate cover, the length should be close to the approximate length, and the headwalls and wingwalls must be dimensioned.

An exact theoretical analysis of culvert flow is extremely complex because the flow is usually nonuniform with regions of both gradually varied and rapidly varied flow. An exact analysis would involve backwater and drawdown calculations, energy and momentum balance and applications of the results of hydraulic model studies. Flow conditions in a given culvert will change as the flow rate and tailwater elevations change, and hydraulic jumps often form inside or downstream of the barrel.

To avoid the analytical complexities created by this wide range of flow conditions, HDS-5 (Normann et al. 2005) provides a culvert design method based on design charts and nomographs. These same procedures are implemented by the FHWA computer program HY-8. The design equations used to develop the nomograph and HY-8 procedures were based on extensive research. This research included quantifying empirical coefficients for various culvert conditions. Inlet and outlet control nomographs for reinforced concrete pipe (RCP) are provided in Figures 9.7 and 9.8, respectively. HDS-5 (Normann et al. 2005) provides nomographs for other pipe materials and shapes and a number of examples on the application of the design method. While it is possible to use the design method nomographs in HDS-5 and particularly the HY-8 computer program, without a thorough understanding of culvert hydraulics, this is not recommended.

EXAMPLE PROBLEM 9.1 (SI and English Units)

Given: A culvert at a new roadway crossing must be designed to pass the 25-year flood. Hydrologic analysis indicates a peak flow rate of 6.0 m^3/s (212 ft^3/s). The approximate culvert length is 60 m (197 ft) and the natural stream bed slope approaching the culvert is 1 percent. The elevation of the culvert inlet invert is 600 m (1968.5 ft) and the roadway elevation is 603 m (1978.35 ft). To provide some capacity in excess of the design flood, the desired headwater elevation should be at least 0.5 m (1.64 ft) below the roadway elevation. The tailwater for the 25-year flood is 1 m (3.28 ft).

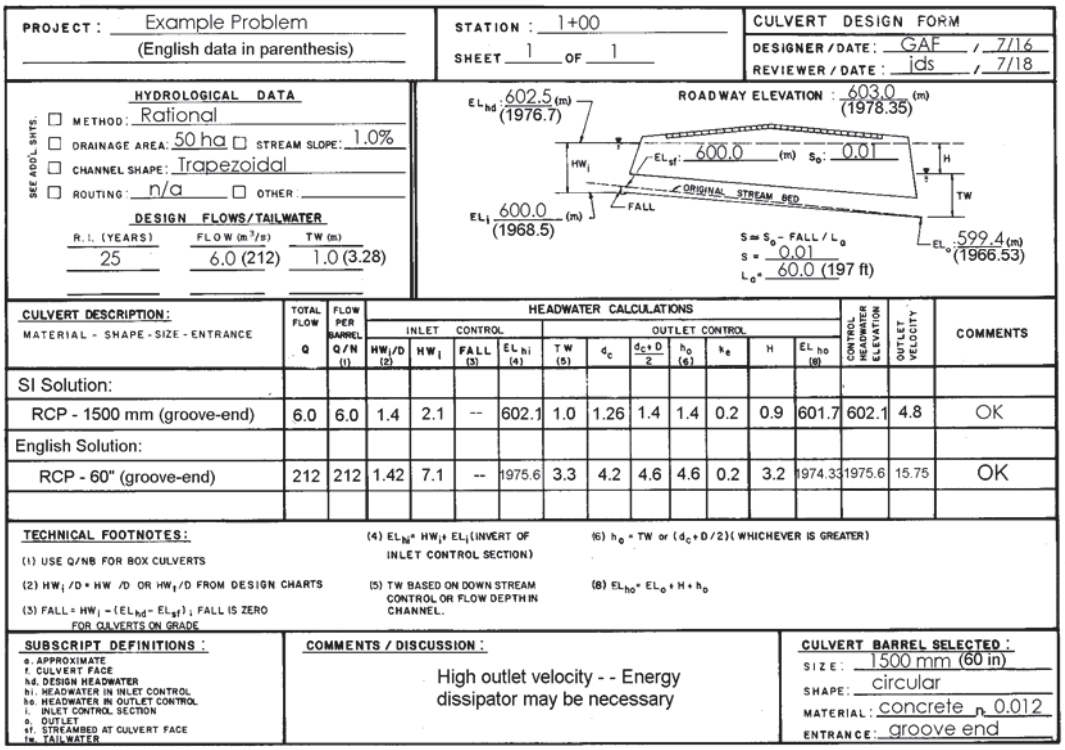
Find: The size of RCP culvert necessary for the 25-year flood.

1. The design will be completed using the nomographs in Figures 9.7 and 9.8. The Culvert Design Form will be used to facilitate the trial and error design process. The Culvert Design Form provides a summary of all the pertinent design data, and a small sketch with important dimensions and elevations.

2. The critical depth required as part of the outlet control computation was evaluated using Figures 4.10a and b.

3. The outlet velocity can be computed by calculating the full flow discharge (Equation 7.1) and full flow velocity (from continuity) and then using the part-full flow relationships (Figure 7.1) to find V/V_f ratio given Q/Q_f.

4. The completed Culvert Design Form (see following page) indicates that a 1,500-mm (60 in.) RCP with a projecting groove end entrance, operating under inlet control, will result in a headwater elevation that is 0.9 m (2.95 ft) below the roadway. The computed outlet velocity is relatively high and protection should be provided at the outlet.



PROJECT: Example Problem (English data in parenthesis)

STATION: 1+00 SHEET 1 OF 1

CULVERT DESIGN FORM
DESIGNER / DATE: GAF / 7/16
REVIEWER / DATE: jds / 7/18

HYDROLOGICAL DATA
SEE ADD'L. SHTS.
☐ METHOD: Rational
☐ DRAINAGE AREA: 50 ha ☐ STREAM SLOPE: 1.0%
☐ CHANNEL SHAPE: Trapezoidal
☐ ROUTING: n/a ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (m^3/s)	TW (m)
25	6.0 (212)	1.0 (3.28)

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	h_e	H	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
SI Solution:																
RCP - 1500 mm (groove-end)	6.0	6.0	1.4	2.1	--	602.1	1.0	1.26	1.4	1.4	0.2	0.9	601.7	602.1	4.8	OK
English Solution:																
RCP - 60" (groove-end)	212	212	1.42	7.1	--	1975.6	3.3	4.2	4.6	4.6	0.2	3.2	1974.33	1975.6	15.75	OK

TECHNICAL FOOTNOTES:
(1) USE Q/NB FOR BOX CULVERTS
(2) $HW_i/D = HW/D$ OR HW_f/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)
(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:
e. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:
High outlet velocity - - Energy dissipator may be necessary

CULVERT BARREL SELECTED:
SIZE: 1500 mm (60 in)
SHAPE: circular
MATERIAL: concrete n 0.012
ENTRANCE: groove end

9.9 IMPROVED INLET DESIGN

Culvert outlet control capacity is governed by headwater depth, tailwater depth, entrance configuration and barrel characteristics. The entrance condition is defined by the barrel cross-sectional area, shape and edge condition, while the barrel characteristics are area, shape, slope, length and roughness. Inlet improvements on culverts functioning under outlet control will reduce entrance losses, but these losses are only a small portion of the total headwater requirement. Therefore, only minor modifications of the inlet geometry which result in little additional cost are justified.

Q20

Culvert inlet control capacity is governed only by entrance configuration and headwater depth. Barrel characteristics and tailwater depth are normally of little consequence since culverts with inlet control typically flow only partly full. Entrance improvements can result in full, or nearly full flow, thereby increasing culvert capacity significantly.

As discussed in Section 9.3 inlet improvements consist of bevel-edged inlets, side-tapered inlets and slope-tapered inlets. Beveled edges reduce the contraction of flow by effectively enlarging the face of the culvert. Bevels are plane surfaces, but rounded edges that approximate a bevel and the socket end of RCP are also effective. Bevels are recommended on all headwalls.

A second degree of improvement is a side-tapered inlet. Tapered inlets improve culvert performance by providing a more efficient control section (the throat). The inlet has an enlarged face area with the transition to the culvert barrel accomplished by tapering the sidewalls. The inlet face has the same height as the barrel, and its top and bottom are extensions of the top and bottom of the barrel. The intersection of the sidewall taper and barrel is defined as the throat section. Two control sections occur on a side-tapered inlet: at the face and throat. Throat control reduces the contraction at the throat.

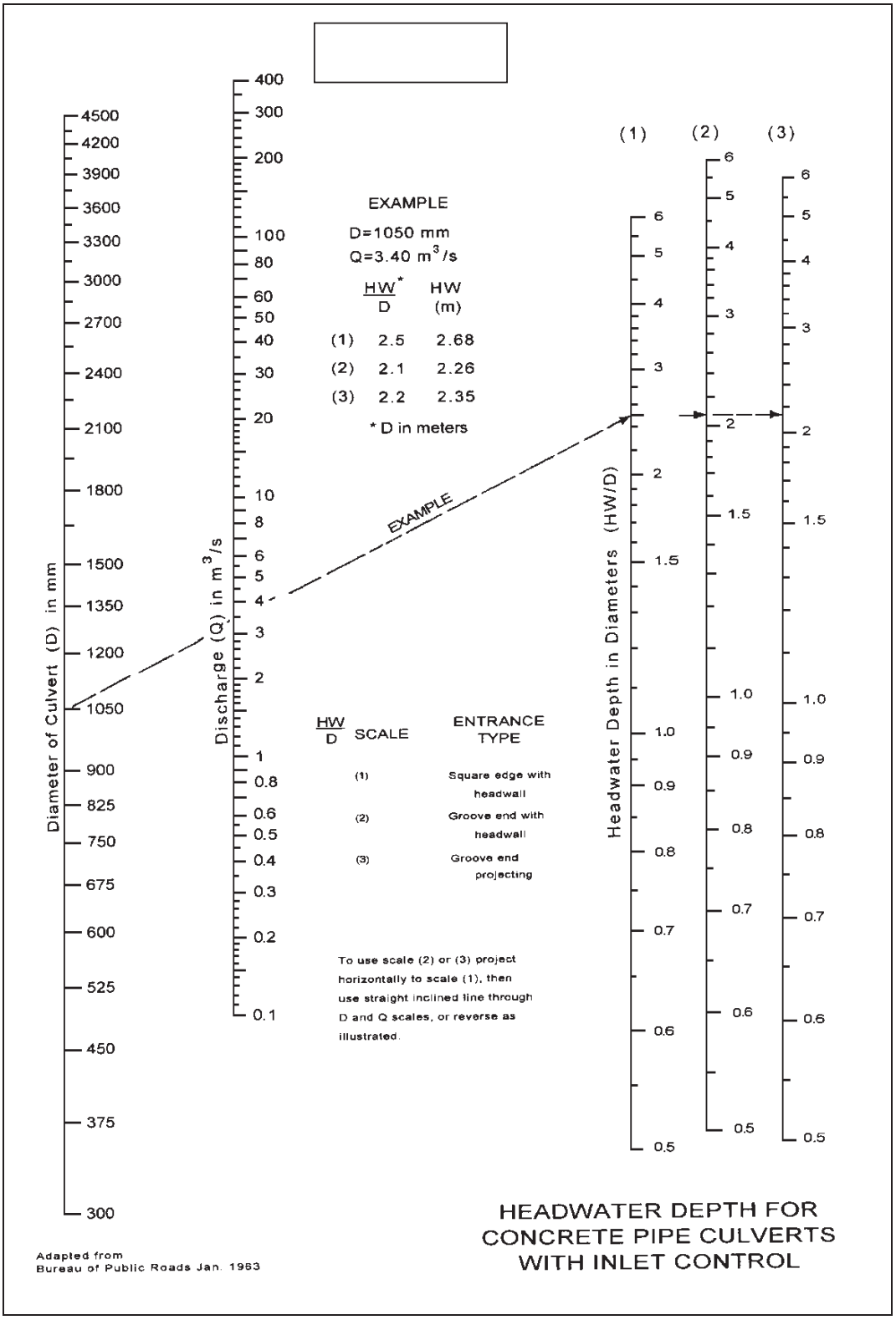


Figure 9.7a. RCP inlet control culvert nomograph - SI units (from HDS-5).

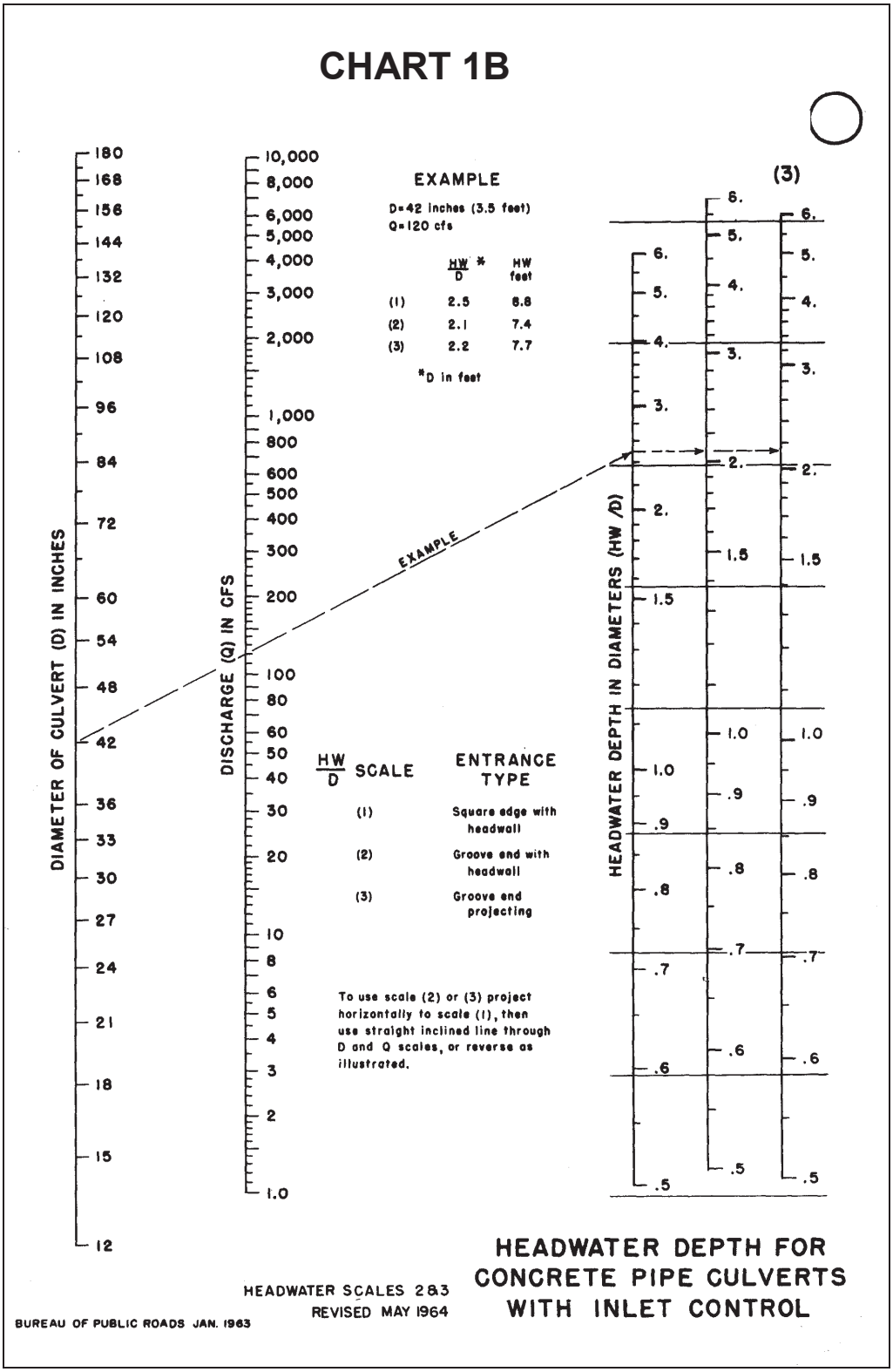


Figure 9.7b. RCP inlet control culvert nomograph - English units.

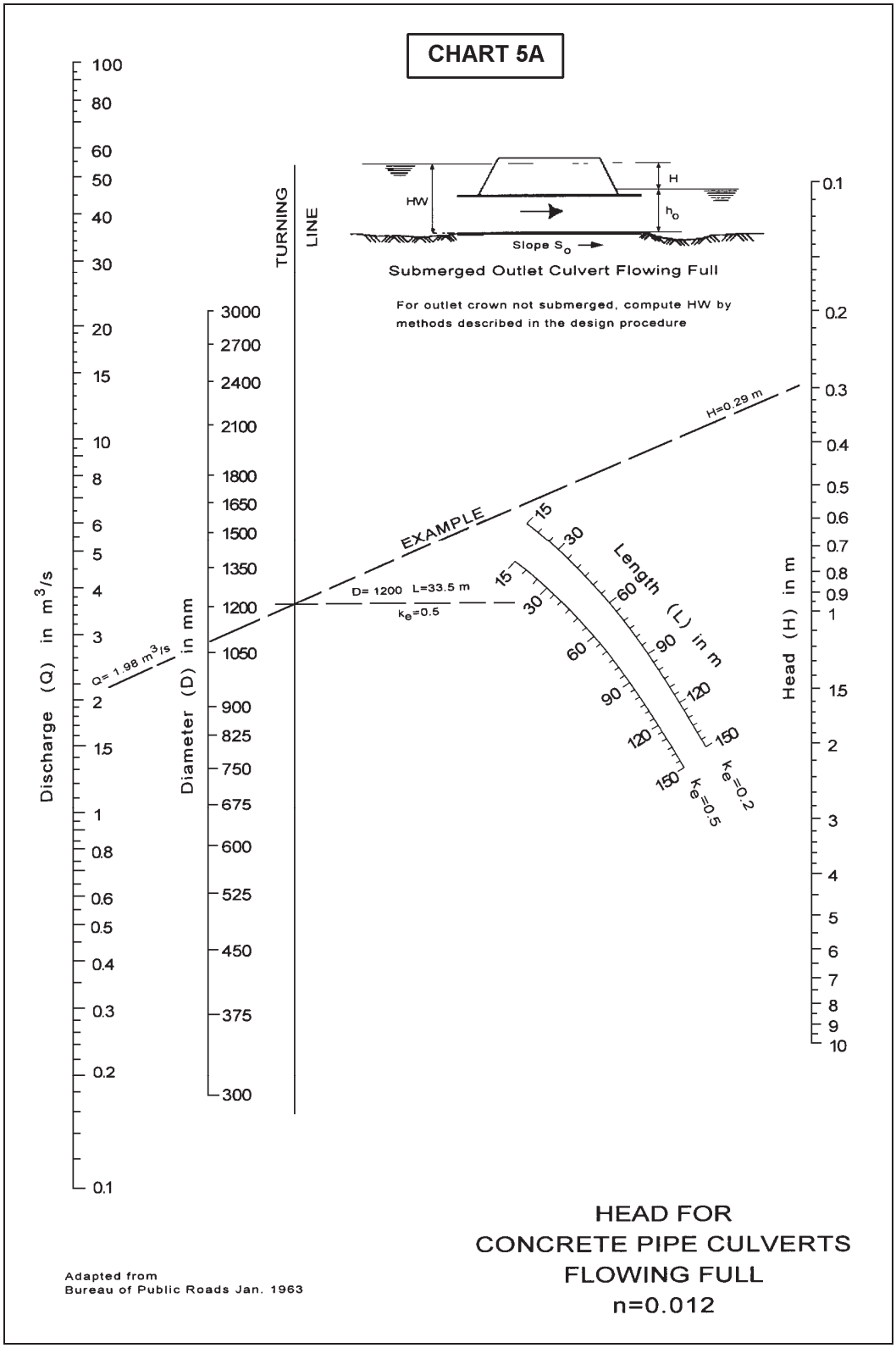


Figure 9.8a. RCP outlet control culvert nomograph - SI units (from HDS-5, Normann et al. 2005).

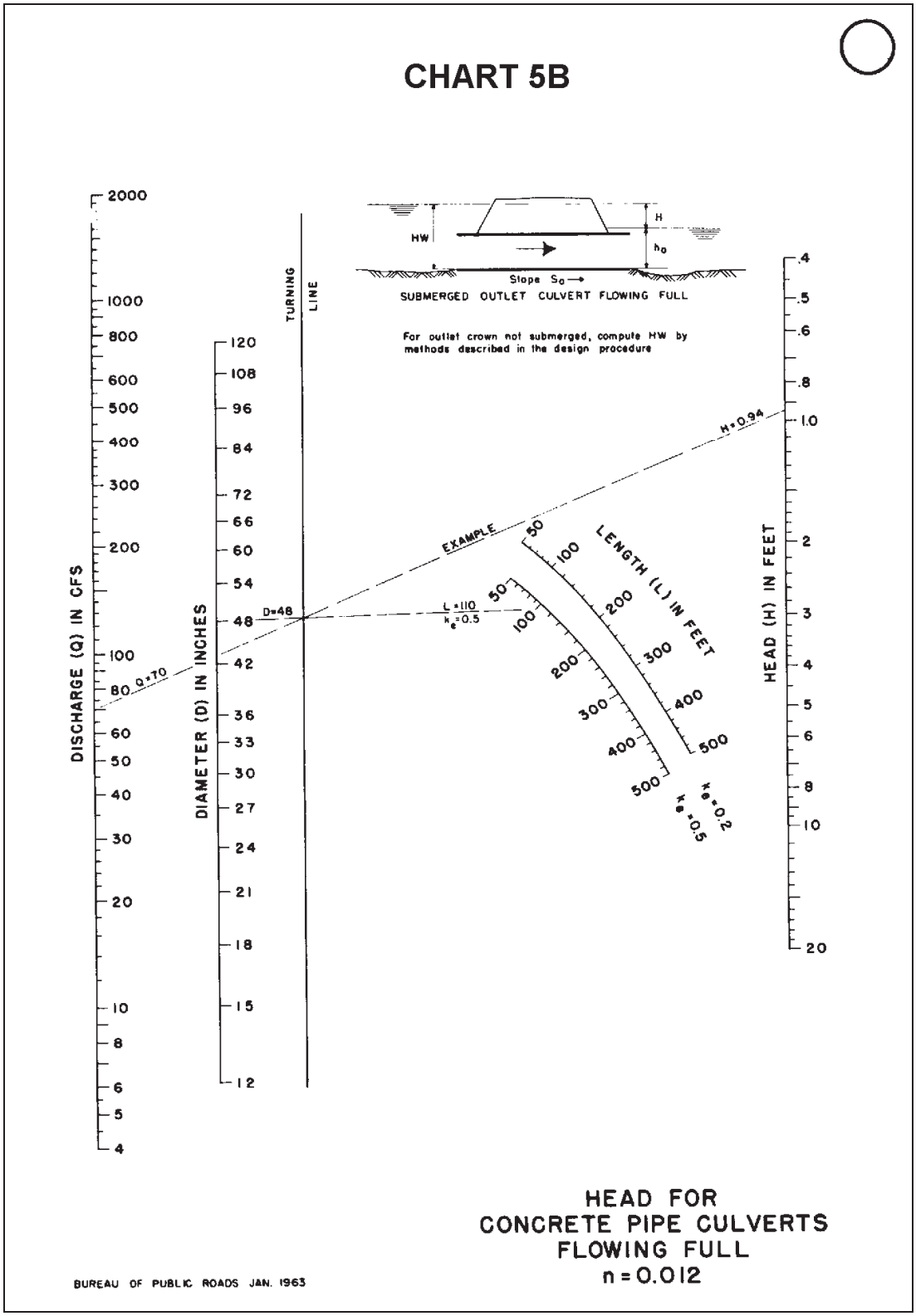


Figure 9.8b. RCP outlet control culvert nomograph - English units (from HDS-5, Normann et al. 2005).

A third degree of improvement is a slope-tapered inlet. The advantage of a slope-tapered inlet over a side-tapered inlet without a depression is that more head is applied at the control (throat) section. Both face and throat control are possible in a slope-tapered inlet; however, since the major cost of a culvert is in the barrel portion and not the inlet structure, the inlet face should be designed with greater capacity at the allowable headwater elevation than the throat. This will ensure flow control will be at the throat and more of the potential capacity of the barrel will be used.

9.10 CULVERT DESIGN USING HY 8

The FHWA culvert program HY-8 is an interactive culvert analysis program that uses the HDS-5 analysis methods. The program will compute the culvert hydraulics and water surface profiles for circular, rectangular, elliptical, pipe arch, metal box and user-defined geometry. Additionally, improved inlets can be specified and the user can analyze inlet and outlet control for full and partially full culverts, analyze the tailwater in trapezoidal and coordinate defined downstream channels, analyze flow over the roadway embankment, and balance flows through multiple culverts.

CHAPTER 10

ENERGY DISSIPATOR DESIGN

10.1 GENERAL DESIGN CONCEPTS

Highways can be very vulnerable to the erosive forces at work in the natural drainage network. Design concepts must be used which address the effects of interception and concentration of flow and constriction of natural waterways to ensure that there is no increased erosion potential resulting from the highway. Energy dissipators should be considered part of the larger design system which includes the culvert and channel protection requirements (upstream and downstream) and may include a debris control structure. While energy dissipators are most often considered for outlet treatment of culverts they may also be applicable for erosion protection at the outlet of storm drains or other high velocity channel outlets. The interrelationship of these various components must be considered in designing any one part of the system. For example, energy dissipator requirements may be reduced, increased or possibly eliminated by changes in the culvert design; and downstream channel conditions (velocity, depth and channel stability) will impact the selection and design of appropriate energy dissipation devices.

Throughout the design process, the designer should keep in mind that the objective of using an energy dissipator is to protect the highway structure and adjacent area from excessive damage due to erosion. One way to accomplish this objective is to return flow to the downstream channel in a condition that approximates the natural flow regime. Note that this also implies guarding against employing energy dissipation devices that reduce flow conditions substantially below the natural or normal channel conditions. If an energy dissipator is necessary, the first step should be consideration of possible ways of modifying the outlet velocity or erosion potential. This could include modifying the culvert barrel. If an internal modification is not cost effective or is hydraulically unacceptable, the designer must begin the process of selecting and designing an appropriate external energy dissipation device. The following sections summarize some of the factors involved in designing an energy dissipator. For a comprehensive treatment of energy dissipator design see HEC-14 (Thompson and Kilgore 2006).

10.2 EROSION HAZARDS

Q21

Erosion at a culvert inlet is not typically a major problem. At the design discharge, water will normally pond at the inlet, and the only significant increases in velocity will occur upstream of the culvert a distance about equal to the height of the culvert. The average velocity near the inlet may be approximated by dividing the flow rate by the area of the culvert opening. The risk of erosion approaching the inlet should be based on this velocity estimate. Note that the erosion risk may be greater at flow rates less than the design discharge, since depth of ponding at the inlet will be less and greater velocities may occur. This is especially true in channels with steep slopes and high velocity flow.

Most inlet failures have occurred on large flexible-type pipe culverts with projected or mitered entrances without headwalls or other entrance protection. Projecting inlets can bend or buckle from buoyant forces. Mitered entrance edges can be bent in from hydraulic forces. To aid in preventing these types of failures, protective features should include concrete headwalls and/or slope paving.

Erosion at culvert outlets is a common problem. Determination of the flow condition, scour potential and channel erodibility should be standard procedure in the design of all highway

culverts. Ultimately, the only safe procedure is to design on the basis that erosion at a culvert outlet and downstream channel will occur and must be protected against.

10.3 CULVERT OUTLET VELOCITY AND VELOCITY MODIFICATION

The continuity equation (Equation 3.1) can be used in all situations to compute culvert outlet velocity, either within the barrel or at the outlet. Given the design discharge, the only other information needed is the flow area, and it is a function of the type of control (outlet or inlet).

Culvert outlet velocity is one of the primary indicators of erosion potential. Outlet velocities are seldom less than 3 m/s (10 ft/s) and will be as large as 10 m/s (30 ft/s) for culverts on mild slopes or even greater for culverts on steep slopes. If the velocity is higher than in the downstream channel, measures to modify or reduce velocity within the culvert barrel should be considered. However, the degree of velocity reduction is typically limited and must be balanced against the increased costs generally involved.

10.3.1 Culverts on Mild Slopes

For culverts on mild slopes operating under outlet control with high tailwater (Figures 9.5a and 9.5b), the outlet velocity will be determined using the full area of the barrel. With this condition it is possible to reduce the velocity by increasing the culvert size. Note that with high tailwater conditions, erosion may not be a serious problem since the ponded water will act as an energy dissipator; however, it will be important to determine if tailwater will always control or if any of the other conditions shown on Figure 9.5 might occur.

When the discharge is high enough to produce a critical depth equal to the crown of the culvert barrel (Figure 9.5c), full flow will again occur and the outlet velocity will be based on the area of the barrel. As before, the barrel size can be increased to achieve a reduction in velocity, but it will be necessary to evaluate if the increased size results in a flow depth below the crown, indicating less than full flow at the outlet. When this occurs, the area used in the continuity equation should be based on the actual flow area.

When culverts discharge with critical depth occurring near the outlet (Figures 9.5d and 9.5e), increasing the barrel size will typically not significantly reduce the outlet velocity. Similarly, increasing the resistance factor will not affect outlet velocity since critical depth is not a function of n.

10.3.2 Culverts on Steep Slopes

For culverts flowing on steep slopes with no tailwater (Figures 9.4a and 9.4c) the outlet velocity can be determined from normal depth calculations. With normal depth conditions on a steep slope, increasing the barrel size may slightly decrease the outlet velocity; however, calculations show that in reality, the slope is the driving force in establishing the normal depth. The velocity will not be significantly altered by even doubling the culvert size/width. Thus, such an approach is not cost effective. Some reduction in outlet velocity can be obtained by increasing the number of barrels, but this is also generally not cost effective.

Increasing the barrel resistance can significantly reduce outlet velocity and is an important factor in velocity reduction for culverts on steep slopes. The objective is to force full flow near the outlet without creating additional headwater. HEC-14 (Thompson and Kilgore 2006) discusses various methods of creating additional roughness, from changing pipe material to baffles and roughness rings, and details the appropriate design procedures.

10.4 HYDRAULIC JUMP ENERGY DISSIPATORS

The hydraulic jump is a natural phenomenon which occurs when supercritical flow changes to subcritical flow (see Chapter 4). This abrupt change in flow condition is accomplished by considerable turbulence and loss of energy, making the hydraulic jump an effective energy dissipation device. To define better the location and length of a hydraulic jump, standard design structures have been developed to force the hydraulic jump to occur. These structures typically use blocks, sills or other roughness elements to impose exaggerated resistance to flow. A comprehensive reference on forced hydraulic jump energy dissipators is USBR (1964). Forced hydraulic jump structures applicable in highway engineering include the Colorado State University (CSU) rigid boundary basin, USBR type IV basin and the St. Anthony Falls basin.

The CSU rigid boundary basin was developed from model study tests of basins with abrupt expansions (Figure 10.1); however, the configuration recommended for use is a combination flared-abrupt expansion basin. The roughness elements are symmetrical about the basin centerline and the spacing between the elements is approximately equal to the element width. Alternate rows of roughness elements are staggered. Riprap may be needed for a short distance downstream of the basin.

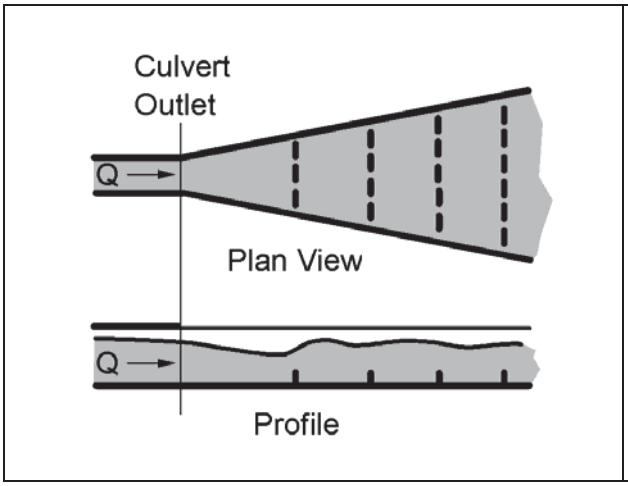


Figure 10.1a. Schematic of CSU rigid boundary basin.

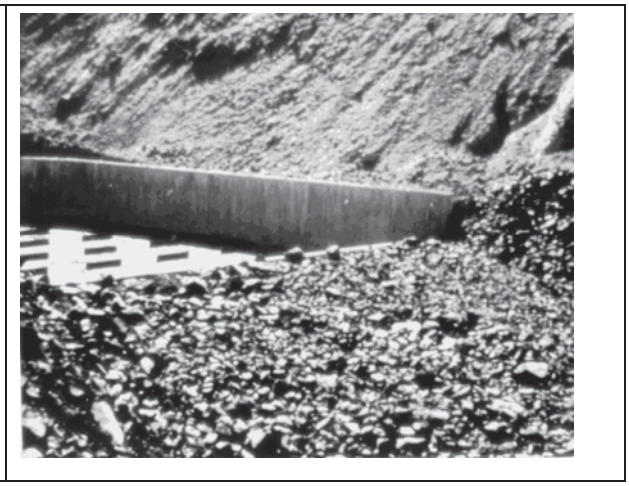


Figure 10.1b. CSU rigid boundary basin.

The St. Anthony Falls (SAF) stilling basin is a more generalized design that uses special appurtenances, chute blocks and baffle or floor blocks to force the hydraulic jump to occur (Figure 10.2). It is recommended for Froude Numbers between 1.7 and 17. Similar to the CSU basin, the design criteria were developed from model study test results.

10.5 IMPACT BASINS

As the name implies, impact basins are designed with part of the structure physically blocking the free discharge of water. The action of water impacting on the structure dissipates energy and modifies the downstream flow regime. Impact basins include the Contra Costa Energy Dissipator, Hook type energy dissipator, and the USBR Type VI Stilling Basin.

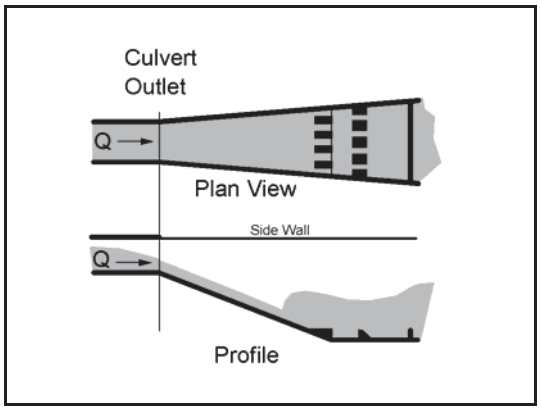


Figure 10.2a. Schematic of SAF stilling basin.

Figure 10.2b. SAF stilling basin.

The impact basin most commonly used in highway engineering is the USBR Type VI (Figure 10.3). The structure is contained in a relatively small box-like structure which requires no tailwater for successful performance. The shape of the basin evolved from extensive tests and resulted in a design based around a vertical hanging baffle. Energy dissipation is initiated by flow striking the vertical hanging baffle and being deflected upstream by the horizontal portion of the baffle and by the floor, creating horizontal eddies. Notches in the baffle provide a self cleaning feature after prolonged nonuse of the structure. If the basin is full of sediment, the notches provide concentrated jets of water for cleaning, and if the basin is completely clogged the full discharge can be carried over the top of the baffle. Use of the basin is limited to installations where the velocity at the entrance of the basin does not exceed 15 m/s (50 ft/s) and discharge is less than 11 m^3/s (400 ft^3/s).

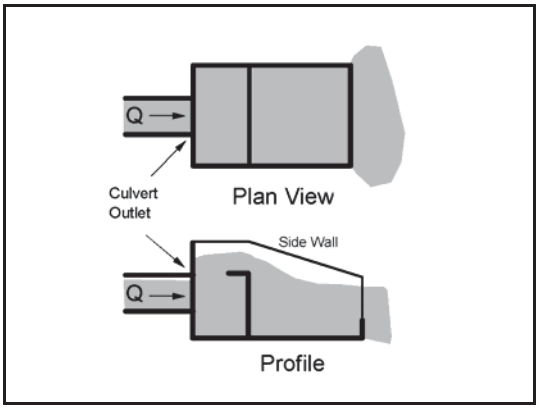


Figure 10.3a. Schematic of USBR Type VI.

Figure 10.3b. Baffle-wall energy dissipator, USBR Type VI.

10.6 DROP STRUCTURES WITH ENERGY DISSIPATION

Drop structures are commonly used for flow control and energy dissipation. Reducing channel slope by placing drop structures at intervals along the channel changes a continuous steeper sloped channel into a series of milder sloped reaches with vertical drops. Instead of slowing down and transferring high erosion producing velocities into lower nonerosive velocities, drop structures control the slope of the channel so that high velocities never develop. The kinetic energy or velocity gained by the water as it drops over the crest of each structure is dissipated by specially designed aprons or stilling basins.

Energy dissipation occurs through impact of the falling water on the floor, redirection of the flow, and turbulence. The stilling basin used to dissipate excess energy can vary from a simple concrete apron to an apron with flow obstructions such as baffle blocks, sills, or abrupt rises. The length of the concrete apron required can be shortened by addition of these appurtenances. Figure 10.4 illustrates a straight drop stilling basin with floor blocks and an end sill.

10.7 STILLING WELLS

Stilling wells dissipate kinetic energy by forcing flow to travel vertically upward to reach the downstream channel. The stilling well most commonly used in highway engineering is the Corps of Engineers Stilling Well (Figure 10.5). This stilling well has application where debris is not a serious problem. It will operate with moderate to high concentrations of sand and silt, but is not recommended for areas where quantities of large floating or rolling debris are expected unless suitable debris-control structures are used. Its greatest application in highway engineering is at the outfalls of storm drains and pipe down drains where little debris is expected. It is recommended that riprap or other types of channel protection be provided around the stilling well outlet.

10.8 RIPRAP STILLING BASINS

Q22

Riprap stilling basins are commonly used at culvert outfalls (Figure 10.6). The design procedure for riprap energy dissipators was developed from model study tests. The results of this testing indicated that the size of the scour hole at the outlet of a culvert was related to the size of the riprap, discharge, brink depth and tailwater depth. The mound of rock material that often forms on the bed downstream of the scour hole contributes to dissipation of energy and reduces the size of scour hole. The general design guidelines for riprap stilling basins include preshaping the scour hole and lining it with riprap.

10.9 ENERGY DISSIPATOR DESIGN USING HY-8

Energy dissipator design for culvert outlets based on HEC-14 can be completed with HY-8. A performance curve is necessary to perform the energy dissipator design and analysis.

Figure 10.4. Straight drop spillway stilling basin.

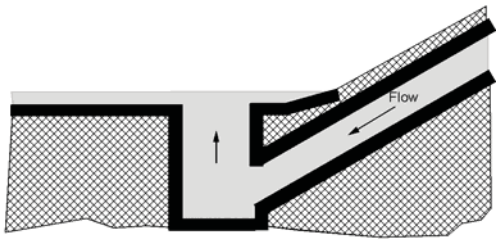


Figure 10.5a. Schematic of COE stilling well.

Figure 10.5b. COE stilling well.

Figure 10.6. Riprapped culvert energy basin.

CHAPTER 11

DRAINAGE SYSTEM CONSTRUCTION

11.1 GENERAL

Construction methods depend upon the equipment used and the expertise of the contractor and are outside the scope of this publication. This chapter will discuss a few of the procedures that should be followed to construct satisfactory highway drainage facilities.

Drainage facilities should be constructed early in the grading operations and any necessary erosion protection should be provided before potential damage occurs. Effective drainage during construction frequently eliminates costly delays as well as later failures that might result from a saturated subgrade. Slopes should be protected from erosion as early as practicable in order to minimize damage and lessen the discharge of eroded soil into existing and newly constructed drainage facilities.

11.2 SUPERVISION

Proper design of drainage facilities will not produce an adequate drainage system without careful supervision during construction. Supervision of drainage structure construction means not only seeing that the construction complies with the plans and specifications, but that any omissions in the plans are corrected. Drainage facilities should be shown on the construction plans together with sufficient hydraulic design data, such as drainage area and design discharge, so that the necessary information is available to solve unforeseen future drainage problems.

11.3 EXCAVATION

Drainage facilities are usually placed from the outlet toward the higher end so that the channel will drain during construction. Dikes for intercepting channels are preferably built from material excavated from the adjacent cuts without disturbing the natural soil at the channel location.

11.4 GRASS-LINED CHANNELS

A type of grass should be selected that is adapted to the locality and to the site conditions. Grass lining is most quickly attained by sodding. Upper parts of the channel may be sprigged or seeded if the cost of sod makes this necessary, but the time of the year and the likelihood of damaging rains occurring before the seedlings become established, should be considered.

Seeding can be protected by mulch, temporary cover grasses, geotextiles, etc. Sod strips perpendicular to the channel centerline at regular intervals have also been used to protect the intervening seeded area. Sod might also be used in the channel bottom and up part of the sideslope for immediate protection with the remainder of channel slope seeded.

11.5 CONCRETE-LINED CHANNELS AND CHUTES

Q23

Use of rigid linings is discouraged; however, if used, concrete channel linings can be cast-in-place, shotcrete, or precast. Soil-cement linings have been successful at some locations. Concrete linings must be placed on a firm well-drained foundation to prevent cracking or failure of the lining. Soil of low density should be thoroughly compacted or removed and replaced with suitable material. Where the soil is deep loess, concrete or other type rigid linings might not be suitable. Expansive clays are extremely hazardous to rigid-type linings because their movement buckles the linings as well as producing an unstable support.

When placing an unformed slab on a slope, a tendency exists to use a stiff concrete mix that will not slough; however, experience indicates that placement of such low-slump concrete without thorough vibration usually results in considerable honeycombing on the underside (USBR 1987). To avoid such results, the concrete should not be stiffer than a 63 mm (2.5 in.) slump. Concrete of this consistency will barely stay on a steep slope. After spreading, the concrete should be thoroughly vibrated, preferably just ahead of a weighted steel-faced slipform screen working up the slope (USBR 1987).

The linings of channels that carry high-velocity flow should be poured as nearly monolithic as possible, without expansion joints or weepholes, and using as few construction joints as possible. Construction joints should be made watertight. Longitudinal and transverse reinforcing steel should be used throughout to control cracking with the longitudinal steel carried through the construction joints. The lining should be anchored to the slope as necessary by reinforced cutoff walls to prevent sliding.

Proper curing of the concrete lining is important, particularly in warm, dry, windy weather, to prevent the early drying of corners, edges, and surfaces. A well-moistened subgrade and wet burlap in contact with the exposed concrete surfaces is excellent for curing purposes.

The edges of newly constructed channels should be protected by a strip of sod at the time of construction. The design of the lining edges should allow enough depth of soil to permit the growth of grass.

11.6 BITUMINOUS-LINED CHANNELS

A well-drained subgrade is necessary beneath a bituminous lining because the strength and weight of the lining is not sufficient to withstand high hydrostatic uplift pressure. Weed control measures are sometimes necessary before the lining is placed. These measures consist of careful grubbing of the subgrade followed by the application of soil sterilant.

11.7 RIPRAP-LINED CHANNELS

All stone used for channel linings or bank protection should be hard, dense, and durable. Most of the igneous and metamorphic rocks, many of the limestones, and some of the sandstones make excellent linings. Shale is not suitable, and limestones and sandstones that have shale seams are undesirable. Quarried stones, angular in shape, are preferred to rounder boulders or cobbles. HEC-15 (Kilgore and Cotton 2006) provides guidelines for riprap gradation, thickness and filter requirements for roadside channels and NCHRP Report 568 (Lagasse et al. 2006) for bank protection.

The stones should be placed on the filter blanket or prepared natural slope in a manner which will produce a reasonably well-graded mass of stone with the minimum practicable percentage of voids. Stone protection should be placed to its full course thickness at one operation and in such a manner as to avoid displacing the underlying material. Placing of stone protection in layers or by dumping into chutes or by similar methods likely to cause segregation should not be permitted. The larger stones should be well distributed and the entire mass of stones should roughly conform to the gradation specified. The stone protection should be so placed and distributed as to avoid large accumulations or areas composed largely of either the larger or smaller sizes of stone. The mass should be fairly compact, with all sizes of material placed in their proper proportions. Hand-placing and rearranging of individual stones by mechanical equipment may be required to the extent necessary to secure the results specified above. The desired distribution of the various sizes of stone throughout the mass might be obtained by selective loads during placing, or by a combination of these methods. Ordinarily, the stone protection should be placed in conjunction with the construction of the embankment with only sufficient delay in construction of the stone protection as may be necessary to prevent mixture of embankment and stone.

Hand-placed stone should be carefully laid to produce a more or less definite pattern with a minimum of voids and with the top surface relatively smooth. Joints should be staggered between courses. The stone used for hand-placed protection should be of better quality than the minimum quality suitable for dumped stone protection. Stones that are roughly square and of fairly uniform thickness are much easier to place than irregular stones. Stone of a flat stratified nature should be placed with the principal bedding planes normal to the slope. Openings to the subsurface should be filled with rock fragments; however, enough voids or openings should be left to drain the subsurface properly.

CHAPTER 12

DRAINAGE SYSTEM MAINTENANCE

12.1 GENERAL

Drainage facilities rapidly lose their effectiveness unless they are adequately maintained. Thus, a good maintenance program is of equal importance with the proper design and construction of the drainage system. In fact, knowledge of the equipment to be used in maintenance and the methods to be employed are a prerequisite to proper design.

Maintenance of vegetative cover on slopes and in drainage channels requires continued attention. The original treatment applied during construction will not last forever. Repeated applications of fertilizer, lime, or organic material at intervals are as necessary on the highway roadside as on the home lawn. Areas often may need reseeding or resodding to restore the vegetative cover. This should be done before serious erosion occurs.

Minor erosion damage within the highway right-of-way should be repaired immediately after it occurs and action taken to prevent a recurrence of the damage. Damage caused by light storms reveals the points of weakness in the drainage system. If these weaknesses are corrected when repairing the damage itself, the drainage system will likely carry the design discharge without damage. Deficiencies that are found in the drainage systems and the corrective action taken should be reported to the hydraulic or design engineer so that similar troubles will not occur on future construction. Reports on drainage works that function well during severe storms are equally valuable to the designer.

12.2 EFFECT OF MAINTENANCE ON FLOW CAPACITY

Maintenance of highway drainage facilities includes repairing erosion damage, mowing grass- lined channels, and removing any deposited sediment or debris. All these measures keep the capacity of the drainage system at the design level. If a channel or culvert contains brush, sediment, or debris, the flow capacity will be less than the design value. In a grass-lined channel, deposited sediment and debris may kill the vegetative lining with subsequent erosion damage during higher flood flows. In some situations, sediment traps and debris barriers might be constructed in order to collect the objectionable material for easy removal.

The effect of inadequate maintenance in a grass-lined channel can be illustrated by considering the Manning's equation and the effect of vegetation on n values. For a grass-lined channel that is regularly mowed, the n value would be relatively low, e.g., 0.035. In contrast, if the same channel is not mowed and grass/weeds are allowed to grow, the n value would be relatively high, e.g., 0.10. As a result, the channel will only carry 0.035/0.10 = 0.35, or about one-third the flow for which it was designed. The remainder of the design flow would overflow the channel and cause flooding or possible erosion.

CHAPTER 13

DRAINAGE SYSTEM ECONOMICS

13.1 GENERAL

Providing adequate drainage is essential to the existence of the highway. Economical drainage design is achieved through doing an adequate job at the lowest cost.

The lowest cost adequate drainage system maintains proper balance between first cost, flood damage, and maintenance cost, and has the capacity and protection to carry the runoff for which it was designed. Selection of the frequency of the design runoff is a matter of economics, while estimating the magnitude of the storm runoff for a selected frequency belongs in the field of hydrology. In this chapter, some of the factors to be considered in the economic selection of highway drainage facilities are discussed.

13.2 FREQUENCY OF THE DESIGN STORM

The average annual cost of a drainage facility is the sum of (1) the first cost divided by the expected life of the drainage facility, plus (2) the average annual maintenance cost, plus (3) the annual charge for possible damage from facility runoff exceeding the design capacity. The average annual maintenance cost might also include the annual charge for flood damage if a flood exceeding the design capacity has occurred; however, it is probably better to separate these costs. Damage to a facility designed to carry a 10-year runoff from a chance occurrence of, say, a 200-year runoff in a few years' record of maintenance expenditures would distort the average annual maintenance cost. The annual charge for possible flood damage should consider the frequency of the design storm and equals the cost of damage from runoff exceeding the design capacity divided by the return period, in years, of the design storm.

If these costs could be evaluated for various combinations of the component costs, the most economical drainage system could be determined as the one with the lowest average annual cost. The optimum frequency of the design storm would then be the frequency associated with the storm runoff that, in combination with other costs, produced the lowest average annual cost. However, the three cost items are interrelated and difficult to evaluate, particularly the item of damage by runoff that exceeds the design runoff. The cost of storm damage includes the cost of traffic interruption by floodwaters or washed-out highway, as well as the cost of repairing the damage to the highway and drainage system and the additional damage to the abutting property directly attributable to the presence of the highway. A further complication is the variation in damage due to the magnitude by which the runoff exceeds the design runoff.

Individual analysis for each small drainage system is impractical, if not impossible. The best solution appears to be a study of average conditions and selection of the frequency of design runoff to be used for various drainage structures according to the class of the highway. The designated frequencies might vary from state to state or even within a state composed of areas differing widely in topography or population density. Individual variations in the designated frequency of the design storm might be needed at locations where damage by flooding could be great, but the cost of a larger facility to carry a less frequent storm is moderate. Economic analysis as applied to drainage structure design is discussed in HEC-17 (Corry et al. 1981).

APPENDIX B

DRAINAGE DESIGN CHARTS AND TABLES

Q24

Table B.1. Values of the Rational Method Runoff Coefficient, C (after HDS-2).

Business:	
Downtown area	0.70-0.95
Neighborhood areas	0.50-0.70
Residential:	
Single-family areas	0.30-0.50
Multi-units, detached	0.40-0.60
Multi-units, attached	0.60-0.75
Suburban	0.25-0.40
Apartment dwelling areas	0.50-0.70
Industrial:	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30
Lawns:	
Sandy soil, flat, < 2%	0.05-0.10
Sandy soil, average, 2 to 7%	0.10-0.15
Sandy soil, steep, > 7%	0.15-0.20
Heavy soil, flat, < 2%	0.13-0.17
Heavy soil, average 2 to 7%	0.18-0.22
Heavy soil, steep, > 7%	0.25-0.35
Streets:	
Asphalt	0.70-0.95
Concrete	0.70-0.95
Brick	0.70-0.85
Drives and walks	0.70-0.85
Roofs	0.70-0.95
Rural:	
Meadow areas	0.10-0.40
Forested areas	0.10-0.30
Cultivated fields	0.20-0.40

Table B.2. Manning's Roughness Coefficients for Various Boundaries.

Rigid Boundary Channels	Manning's n
Very smooth concrete and planed timber	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Wood	0.014
Vitrified clay	0.015
Shot concrete, untroweled, and earth channels in best condition	0.017
Straight unlined earth canals in good condition	0.020
Mountain streams with rocky beds	0.040 -0.050
MINOR STREAMS (top width at flood stage < 30 m)	
Streams on Plain	
1. Clean, straight, full stage, no rifts or deep pools	0.025-0.033
2. Same as above, but more stones and weeds	0.030-0.040
3. Clean, winding, some pools and shoals	0.033-0.045
4. Same as above, but some weeds and stones	0.035-0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040-0.055
6. Same as 4, but more stones	0.045-0.060
7. Sluggish reaches, weedy, deep pools	0.050-0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075-0.150
Mountain Streams, no Vegetation in Channel, Banks Usually Steep, Trees and Brush Along Banks Submerged at High Stages	
1. Bottom: gavels, cobbles and few boulders	0.030-0.050
2. Bottom: cobbles with large boulders	0.040-0.070
Floodplains	
Pasture, No Brush	
1. Short Grass	0.025-0.035
2. High Grass	0.030-0.050
Cultivated Areas	
1. No Crop	0.020-0.040
2. Mature Row Crops	0.025-0.045
3. Mature Field Crops	0.030-0.050
Brush	
1. Scattered brush, heavy weeds	0.035-0.070
2. Light brush and trees in winter	0.035-0.060
3. Light brush and trees in summer	0.040-0.080
4. Medium to dense brush in winter	0.045-0.110
5. Medium to dense brush in summer	0.070-0.160

Table B.2. Manning's Roughness Coefficients for Various Boundaries (continued).

Rigid Boundary Channels	Manning's n
Trees	
1. Dense willows, summer, straight	0.110-0.200
2. Cleared land with tree stumps, no sprouts	0.030-0.050
3. Same as above, but with heavy growth of sprouts	0.050-0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080-0.120
5. Same as above, but with flood stage reaching branches	0.100-0.160
MAJOR STREAMS (Topwidth at flood stage > 30 m)	
The n value is less than that for minor streams of similar description, because banks offer less effective resistance.	
Regular section with no boulders or brush	0.025-0.060
Irregular and rough section	0.035-0.100
Alluvial Sand-bed Channels (no vegetation)	
Tranquil flow, Fr < 1	
Plane bed	0.014-0.020
Ripples	0.018-0.030
Dunes	0.020-0.040
Washed out dunes or transition	0.014-0.025
Plane bed	0.010-0.013
Rapid Flow, Fr > 1	
Standing waves	0.010-0.015
Antidunes	0.012-0.020
Overland Flow and Sheet Flow	
Smooth asphalt	0.011
Smooth concrete	0.012
Cement rubble surface	0.024
Natural range	0.13
Dense grass	0.24
Bermuda grass	0.41
Light underbrush	0.40
Heavy underbrush	0.80

Table B.3. Manning's n Values for Closed Conduits.

Description		Manning's n Range
Concrete pipe		0.011-0.013
Corrugated metal pipe or pipe-arch:		
Corrugated Metal Pipes and Boxes, Annular or Helical Pipe (Manning's n varies with barrel size)	68 by 13 mm (2-2/3 x 1/2 in.) corrugations	0.022-0.027
	150 by 25 mm (6 x 1 in.) corrugations	0.022-0.025
	125 by 25 mm (5 x 1in.) corrugations	0.025-0.026
	75 by 25 mm (3 x 1 in) corrugations	0.027-0.028
	150 by 50 mm (6 x 2 in.) structural plate corrugations	0.033-0.035
	230 by 64 mm (9 x 2-1/2 in.) structural plate corrugations	0.033-0.037
Corrugated Metal Pipes Helical Corrugations, Full Circular Flow	68 by 13 mm (2-2/3 x 1/2 in.) corrugations	0.012-0.024
Spiral Rib Metal Pipe	Smooth walls	0.012-0.013
Vitrified clay pipe		0.012-0.014
Cast-iron pipe, uncoated		0.013
Steel pipe		0.009-0.013
Brick		0.014-0.017
Monolithic concrete:		
1. Wood forms, rough		0.015-0.017
2. Wood forms, smooth		0.012-0.014
3. Steel forms		0.012-0.013
Cemented rubble masonry walls:		
1. Concrete floor and top		0.017-0.022
2. Natural floor		0.019-0.025
Laminated treated wood		0.015-0.017
Vitrified clay liner plates		0.015

NOTE: The values indicated in this table are recommended Manning's n design values. Actual field values for older existing pipelines may vary depending on the effects of abrasion, corrosion, deflection, and joint conditions. Concrete pipe with poor joints and deteriorated walls may have n values of 0.014 to 0.018. Corrugated metal pipe with joint and wall problems may also have higher n values, and in addition, may experience shape changes which could adversely effect the general hydraulic characteristics of the pipeline.