CHAPTER

7

STORMWATER DRAINAGE DESIGN

7.1 Stormwater Drainage System Design

Stormwater drainage design is an integral component of both site and overall stormwater management design. Drainage design for new developments must: strive to maintain compatibility and minimize interference with existing drainage patterns; control flooding of property, structures and roadways for design flood events; and minimize potential environmental impacts on stormwater runoff. Stormwater collection systems must be designed to provide adequate surface drainage while at the same time meeting Knox County stormwater management goals such as water quality, streambank channel protection, habitat protection and groundwater recharge.

7.1.1 Drainage System Components

In every location there are two stormwater drainage systems that must be considered: the minor system and the major system. Three factors influence the design of these systems: flooding; public safety; and, water quality.

The purpose of the minor drainage system, which is designed for the 25-year storm event, is to remove stormwater from areas such as streets and sidewalks for public safety reasons. This system consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, and small underground pipe systems which collect stormwater runoff and transport it to structural BMP facilities, pervious areas and/or the major drainage system (i.e., natural waterways, large man-made conduits, and large water impoundments). If the minor system is exceeded during a storm event, the major system is then utilized.

The major system is defined by flow paths for runoff from less frequent storms, up to the 100-yr frequency. It consists of natural waterways, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainageways such as overload relief swales and infrequent temporary ponding areas. The major system includes not only the trunk line system that receives the water from the minor system, but also the natural backup system which functions in case of overflow from or failure of the minor system. Overland relief must not flood or damage houses, buildings or other property.

The major/minor concept may be described as a 'system within a system' for it comprises two distinct, but conjunctive, drainage networks. The major and minor systems are closely interrelated, and the design of components for each must be done in conjunction with the design of structural BMPs and the overall stormwater management standards.

This chapter is intended to provide design criteria and guidance on common drainage system components, including: street and roadway gutters; inlets and storm drain pipe systems; culverts; vegetated and lined open channels; and energy dissipation devices for outlet protection. This chapter also provides important considerations for the planning and design of stormwater drainage facilities.

7.1.2 Checklist for Drainage Planning and Design

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

(1) Analyze topography

a) Check the off-site drainage pattern. Where is water coming onto the site? Where is water leaving the site?

b) Check the on-site topography for surface runoff storage and infiltration.

1. Determine runoff pattern; high points, ridges, valleys, streams, and swales. Where is the water going?

2. Overlay the grading plan and indicate watershed areas, calculate square footage (acreage), points of concentration, low points, etc.

c) Check the potential drainage outlets and discharge methods for the site.

1. On-site (structural BMP, receiving water)

2. Off-site (highway, storm drain, receiving water, regional control)

3. Natural drainage system (swales)

4. Existing drainage system (drain pipe)

(2) Consider other site conditions, such as:

a) land use and physical obstructions such as walks, drives, parking, patios, landscape edging, fencing, grassed area, landscaped area, tree roots, etc.;

b) soil types, which determine the infiltration capacity of the soil;

c) vegetative cover, which will determine the amount of site slope possible without erosion.

(3) Determine the probable location(s) of drainage structures and BMP facilities.

(4) Identify the type and size of drainage system components that are required. Design the drainage system and integrate with the overall stormwater management system and plan.

7.1.3 General Drainage Design Standards

The traditional design of stormwater drainage systems has been to collect and convey stormwater runoff as rapidly as possible to a suitable location where it can be discharged. Knox County desires to take a different approach wherein the design methodologies and concepts of drainage design are integrated with minimum standards for stormwater quantity and quality presented in Volume 2, Chapter 1 of this manual. This means that:

- stormwater conveyance systems must be designed to remove water efficiently enough to meet flood protection criteria and level of service requirements; and,
- stormwater conveyance systems must complement the ability of the site design and structural BMPs to mitigate the major impacts of urban development.

Minimum design criteria (i.e., storm event frequency) for drainage system components are stated in the Knox County Stormwater Management Ordinance and in Volume 2, Chapter 2 of this manual. This chapter contains additional design standards and guidance for individual components of the stormwater drainage system. Standards that are specific to each type of component are included in the discussion of each component that is presented in this chapter. General design standards and guidelines, relevant to all stormwater system (major and minor) controls, are listed below.

- All stormwater system components shall be designed in accordance with the criteria stated in the Knox County Stormwater Management Ordinance and in this manual (Volume 2, Chapter 2).
- Stormwater systems shall be designed to conform to natural drainage patterns and discharge to natural drainage paths within a drainage basin where practicable. These natural drainage paths should be modified as necessary to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or stormwater systems. The Ten Percent Rule, discussed in Chapters 2 and 3, implements this requirement.
- In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points, it must be demonstrated that the change will not have any adverse impacts on downstream properties or stormwater systems and may require an off-site drainage easement.
- It is important to ensure that the combined minor and major system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor systems and/or major structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of stormwater networks, it is essential to ensure that flows will not discharge onto private property during times when stormwater flows are equal to or exceed the major system design capacity.

7.2 Storm Drain Pipe Systems

Storm drain pipe systems, sometimes referred to as *storm sewers* or *lateral closed systems*, are pipe conveyances used in the minor stormwater drainage system for transporting runoff from the roadway and other inlets to outfalls at structural stormwater BMPs and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

7.2.1 Design Standards and Considerations

All storm drain pipe systems designed and installed in Knox County shall conform to the standards listed below. Additional standards and policies are included in sections pertaining to the design of storm drain pipe systems that follow.

- For ordinary conditions, storm drain pipes shall be sized on the assumption that they will flow full or practically full under the design discharge, but will not be placed under pressure head. The Manning Formula (presented later in this section) shall be used for capacity calculations.
- The maximum hydraulic gradient shall not produce a velocity that exceeds 15 ft/s.
- The minimum desirable physical slope shall be 0.5%, or the slope that will produce a velocity of 2.5 feet per second when the storm sewer is flowing full, whichever is greater.

The list below presents additional considerations for the design of storm drain pipe systems.

- The use of better site design practices (and corresponding site design credits) should be considered to reduce the overall length of a piped stormwater conveyance system.
- Shorter and smaller conveyances can be designed to carry runoff to nearby holding areas, natural preservation areas, or filter strips (with spreaders at the end of the pipe).

- Ensure that storms in excess of pipe design flows can be safely conveyed through a development without damaging structures or flooding major roadways. This is often done through design of both a major and minor drainage system. The minor (piped) system carries the mid-frequency design flows while larger runoff events may flow across lots and along streets as long as it will not cause property damage or impact public safety.

7.2.2 General Design Procedure

The following procedure can be utilized when designing a storm drainage pipe system.

(Step 1) Determine inlet location and spacing.

(Step 2) Prepare a tentative plan layout of the storm drainage system, including:

a. location of storm drains;

b. direction of flow;

c. location of access points (maximum separation is 400 feet); and,

d. location of existing facilities such as water, gas, or underground cables.

(Step 3) Determine drainage areas and compute runoff using the methods stated in Chapter 3.

(Step 4) After the tentative locations of inlets, drain pipes, and outfalls (including tailwaters) have been determined and the inlets sized, compute the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from upstream of a line to the downstream point at which the line connects with other lines or discharges through the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream. The storm drain system design computation form, presented in Figure 7-1 (using the Rational Method), can be used to summarize hydrologic, hydraulic and design computations.

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

It should be recognized that the rate of discharge to be carried by any particular section of storm drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. As well, it is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

7.2.3 Capacity Calculations

Equations to be used for storm drain pipe system capacity calculations are presented in Equations 7-1 through 7-6.

Formulas for Gravity and Pressure Flow

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning's Formula, expressed by Equation 7-1.

Equation 7-1

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

where:

V = mean velocity of flow, ft/s

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

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

n = Manning's roughness coefficient

Figure 7-1. Storm Drain System Computation Form

(Source: AASHTO Model Drainage Manual, 1991)

STORM DRAIN COMPUTATION SHEET

ROUTE ______
SECTION ______
COUNTY ______

COMPUTED ______ DATE ______
CHECKED ______ DATE ______

STATION		LENGTH FEET	DRAINAGE AREA "A" (ACRES)		RUNOFF COEFFICIENT "C"	"A" • "C"		FLOW TIME MIN.		RAINFALL INTENSITY "I" in/hour	TOTAL RUNOFF C.I.A. = Q (c.f.s.)	DIAMETER PIPE inches	CAPACITY FULL c.f.s.	VELOCITY f.p.s.		INVERT ELEV		MANHOLE INVERT DROP	SLOPE OF DRAIN ft/ft
FROM	TO		INCREMENT	TOTAL		INCREMENT	TOTAL	TO UPPER END	IN SECTION					FLOWING FULL	DESIGN FLOW	UPPER END	LOWER END		

In terms of discharge, the above equation can be written as shown in Equation 7-2.

Equation 7-2

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

where:

Q = rate of flow, cfs
A = cross sectional area of flow, ft^2

For circular pipes flowing full, the Manning Formula can be written as shown in Equations 7-3 and 7-4.

Equation 7-3

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

Equation 7-4

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

where:

D = diameter of pipe, ft

Equations 7-5 and 7-6 present the Manning's Equation reformulated to determine friction losses for storm drain pipes.

Equation 7-5

$$H_f = \frac{2.87 n^2 V^2 L}{S^{4/3}}$$

Equation 7-6

$$H_f = \frac{29 n^2 V^2 L}{R^{4/3}(2g)}$$

where:

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

The nomograph solution of Manning's formula for full flow in circular storm drain pipes is shown in Figures 7-2, 7-3, and 7-4. Figure 7-5 has been provided to solve the Manning's Equation for partially full flow in storm drains.

7.2.4 Hydraulic Grade Lines

All head losses in a storm sewer system must be considered in computing the hydraulic grade line to determine the water surface elevations, under design conditions for the various inlets, catch basins, manholes, junction boxes, etc.

Figure 7-2. Nomograph for Solution of Manning's Formula for Flow in Storm Sewers

(Source: AASHTO Model Drainage Manual, 1991)

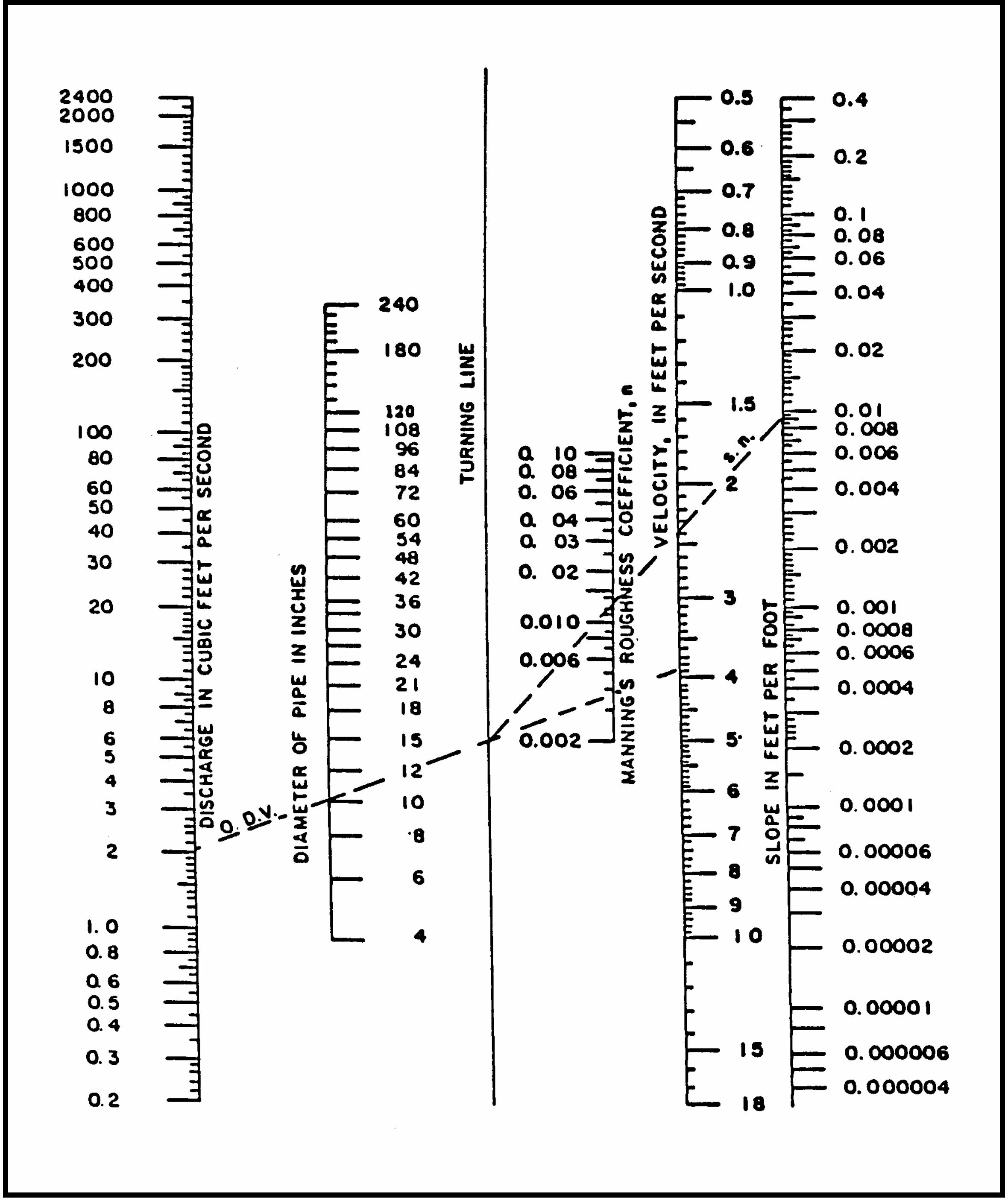


Figure 7-3. Nomograph for Computing Required Size of Circular Drain, Flowing Full n = 0.013 or 0.015

(Source: AASHTO Model Drainage Manual, 1991)

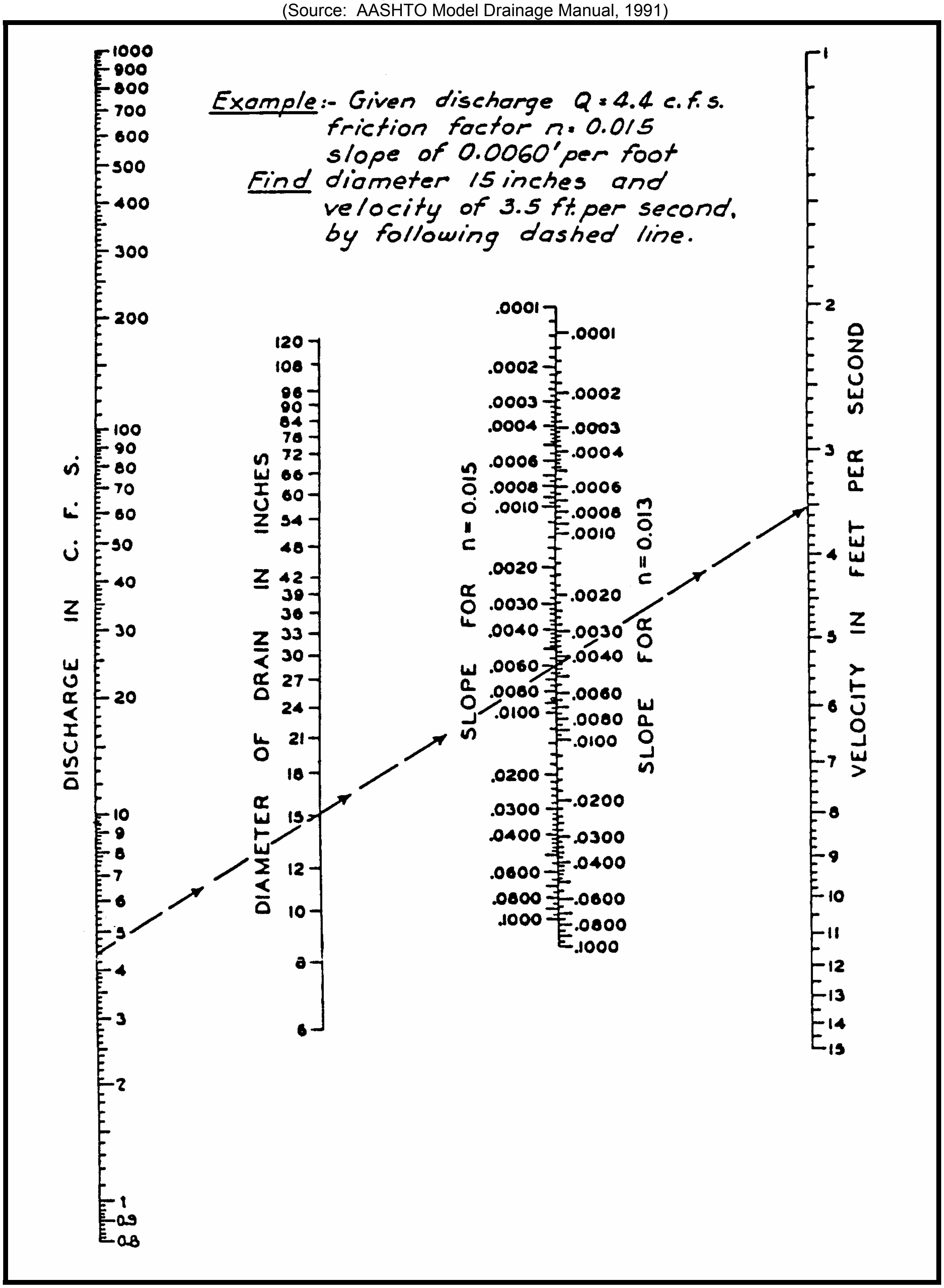


Figure 7-4. Concrete Pipe Flow Nomograph

(Source: AASHTO Model Drainage Manual, 1991)

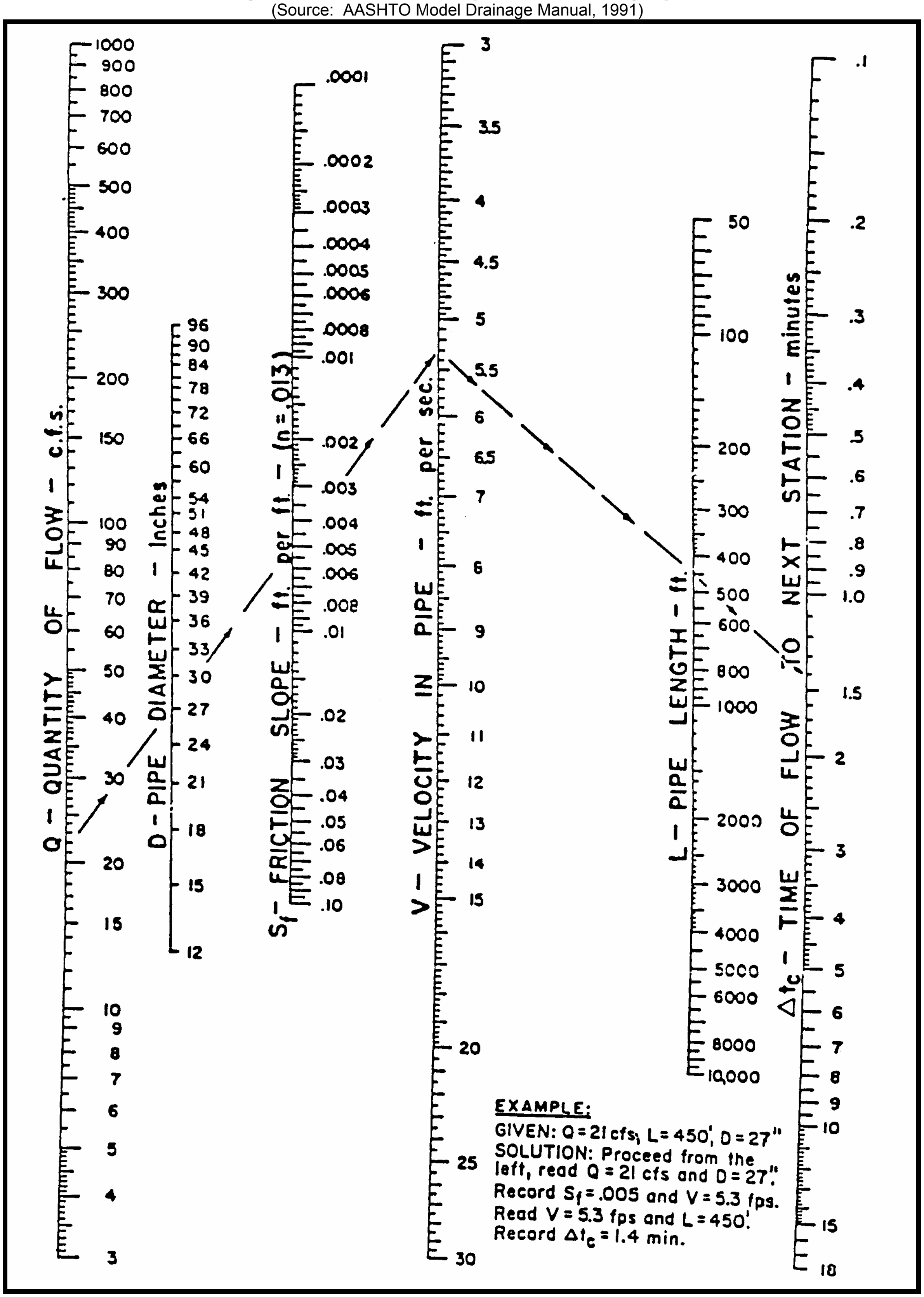
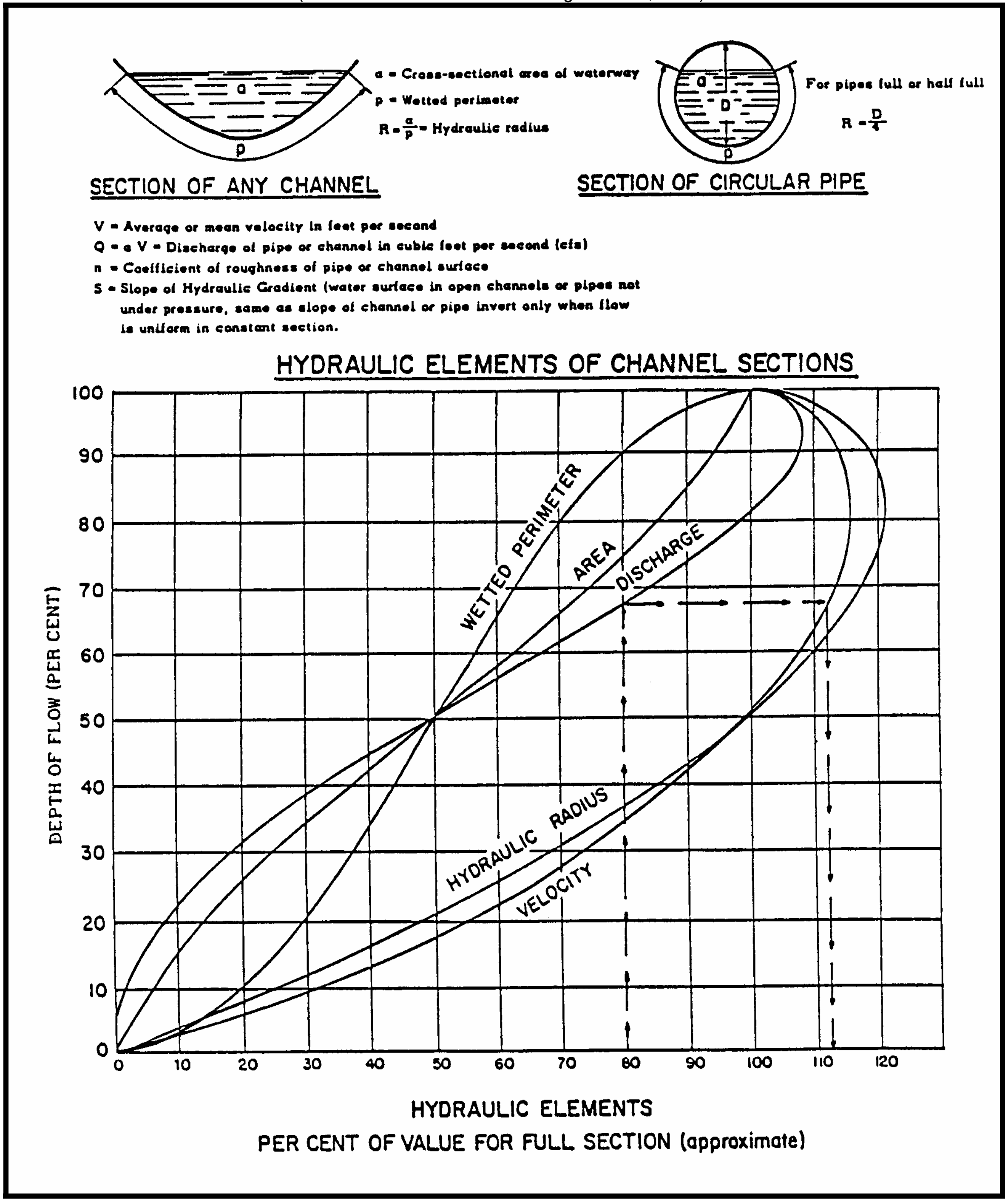


Figure 7-5. Values of Various Elements of Circular Section for Various Depths of Flow

(Source: AASHTO Model Drainage Manual, 1991)



V = Average of mean velocity in feet per second
Q = Discharge of pipe or channel in cubic feet per second
S = Slope of hydraulic grade line

Hydraulic control is a set water surface elevation from which the hydraulic calculations are begun. All hydraulic controls along the alignment are established. If the control is at a main line upstream inlet (inlet control), the hydraulic grade line is the water surface elevation minus the entrance loss minus the difference in velocity head. If the control is at the outlet, the water surface is the outlet pipe hydraulic grade line.

Design Procedure - Outlet Control

The head losses are calculated beginning from the control point upstream to the first junction and the procedure is repeated for the next junction. The computation for outlet control may be tabulated on Figure 7-6 using the following procedure:

(Step 1) Enter in Column 1 the station for the junction immediately upstream of the outflow pipe. Hydraulic grade line computations begin at the outfall and are worked upstream taking each junction into consideration.

(Step 2) Enter in Column 2 the outlet water surface elevation if the outlet will be submerged during the design storm. If the outlet is not submerged, enter the critical depth + D/2.

(Step 3) Enter in Column 3 the diameter (D_o) of the outflow pipe.

(Step 4) Enter in Column 4 the design discharge (Q_o) for the outflow pipe.

(Step 5) Enter in Column 5 the length (L_o) of the outflow pipe.

(Step 6) Enter in Column 6 the friction slope (S_f) in ft/ft of the outflow pipe. This can be determined by using Equation 7-7.

Equation 7-7

$$S_f = \frac{Q^2}{K^2}$$

where:

S_f = friction slope
K = $[1.49\ AR^{2/3}]/n$
A = cross sectional area of flow, ft^2
R = hydraulic radius, ft
n = Manning's ("n") roughness coefficient

(Step 7) Multiply the friction slope (S_f) in Column 6 by the length (L_o) in Column 5 and enter the friction loss (H_f) in Column 7. On curved alignments, calculate curve losses by using the formula $H_c = 0.002\ (\Delta)(V_o^2/2g)$, where Δ = angle of curvature in degrees and add to the friction loss.

(Step 8) Enter in Column 8 the velocity of the flow (V_o) of the outflow pipe.

(Step 9) Enter in Column 9 the contraction loss (H_o) by using the formula:

$H_o = [0.25\ V_o^2)]/2g$, where g = 32.2 ft/s^2

(Step 10) Enter in Column 10 the design discharge (Q_i) for each pipe flowing into the junction. Neglect lateral pipes with inflows of less than 10% of the mainline outflow. Inflow must be adjusted to the mainline outflow duration time before a comparison is made.

(Step 11) Enter in Column 11 the velocity of flow (V_i) for each pipe flowing into the junction (for exception see Step 10).

Figure 7-6. Hydraulic Grade Line Computation Form

(Source: AASHTO Model Drainage Manual, 1991)

HYDRAULIC GRADE LINE

PROJECT

STATION	Outlet Water Surface Elev.	D_o	Q_o	L_o	S_{f_o} %	H_f	JUNCTION LOSS V_o	H_o	Q_i	V_i	Q_iV_i	$\frac{V_i^2}{2g}$	H_i	ANGLE	H_Δ	H_t	1.3 H_t	0.5 H_t	FINAL H	Inlet Water Surface Elev.	Rim Elev.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)		(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

$H_i = 0.35 \frac{V_i^2}{2g}$ $H_o = 0.25 \frac{V_o^2}{2g}$ $H_\Delta = K \frac{V_i^2}{2g}$

FINAL $H = H_i + H_t$

$H_t = H_o + H_i + H_\Delta$

90° K = 0.70	50° K = 0.47	20° K = 0.16	
80° K = 0.66	40° K = 0.38	15° K = 0.10	
70° K = 0.61	30° K = 0.28		
60° K = 0.55	25° K = 0.22		

(Step 12) Enter in Column 12 the product of Q_i x V_i for each inflowing pipe. When several pipes inflow into a junction, the line producing the greatest Q_i x V_i product is the one that should be used for expansion loss calculations.

(Step 13) Enter in Column 13 the controlling expansion loss (H_i) using the formula:

Equation 7-8

$$H_i = \frac{0.35V_1^2}{2g}$$

(Step 14) Enter in Column 14 the angle of skew of each inflowing pipe to the outflow pipe (for exception, see Step 10).

(Step 15) Enter in Column 15 the greatest bend loss (H) calculated by using the formula H = $[KV_i^2)]/2g$ where K = the bend loss coefficient corresponding to the various angles of skew of the inflowing pipes.

(Step 16) Enter in Column 16 the total head loss (H_t) by summing the values in Column 9 (H_o), Column 13 (H_i), and Column 15 (H_Δ).

(Step 17) If the junction incorporates adjusted surface inflow of 10% or more of the mainline outflow, i.e., drop inlet, increase H_t by 30% and enter the adjusted H_t in Column 17.

(Step 18) If the junction incorporates full diameter inlet shaping, such as standard manholes, reduce the value of H_t by 50% and enter the adjusted value in Column 18.

(Step 19) Enter in Column 19 the FINAL H, the sum of H_f and H_t, where H_t is the final adjusted value of the H_t.

(Step 20) Enter in Column 20 the sum of the elevation in Column 2 and the final H in Column 19. This elevation is the potential water surface elevation for the junction under design conditions.

(Step 21) Enter in Column 21 the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Column 20. If the potential water surface elevation exceeds 1 foot below ground elevation for the design flow, the top of the pipe or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the Hydraulic Grade Line (H.G.L.).

(Step 22) Repeat the procedure starting with Step 1 for the next junction upstream.

(Step 23) At last upstream entrance, add $V_1^2/2g$ to get upstream water surface elevation.

7.2.5 Minimum Grade

Knox County requires that storm drains be designed such that velocities of flow will not be less than 2.5 feet per second at design flow, with a minimum slope of 0.5%. For very flat flow lines, the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. Upper reaches of a storm drain system should have flatter slopes than slopes of lower reaches. Progressively increasing slopes keep solids moving toward the outlet and deter settling of particles due to steadily increasing flow streams.

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

Equation 7-9

$$S = \frac{(nV)^2}{2.22R^{4/3}}$$

where:

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

7.3 Culvert Design

A *culvert* is a short, closed (covered) conduit that conveys stormwater runoff under an embankment, usually a roadway. The primary purpose of a culvert is to convey surface water, but properly designed it may also be used to restrict flow and reduce downstream peak flows. In addition to the hydraulic function, a culvert must also support the embankment and/or roadway, and protect traffic and adjacent property owners from flood hazards to the extent practicable.

Most culvert design is empirical and relies on nomographs and standard procedures. The purpose of this section is to provide an overview of culvert design standards and procedures.

7.3.1 Symbols and Definitions

To provide consistency within this section the symbols listed in Table 7-1 will be used. These symbols were selected because of their wide use.

Table 7-1. Culvert Design Symbols and Definitions

Symbol	Definition	Units
A	Area of cross section of flow	ft^2
B	Barrel width	ft
C_d	Overtopping discharge coefficient	-
D	Culvert diameter or barrel width	in or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_u	Uniform depth of flow	ft
g	Acceleration due to gravity	ft/s^2
H_f	Depth of pool or head, above the face section of invert	ft
h_o	Height of hydraulic grade line above outlet invert	ft
HW	Headwater depth above invert of culvert (depth from inlet to upstream total energy grade line)	ft
K_e	Inlet loss coefficient	-
L	Length of culvert	ft
N	Number of barrels	-
Q	Rate of discharge	cfs
S	Slope of culvert	ft/ft
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
V_c	Critical velocity	ft/s

7.3.2 Design Standards and Considerations

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The list below presents the key considerations for the design of culverts.

- Culverts can serve double duty as flow retarding structures in grass channel design. Care should be taken to design them as storage control structures if flow depths exceed several feet, and to ensure public safety.
- Improved inlet designs can absorb considerable energy for steeper sloped and skewed inlet condition designs, thus helping to protect channels.

All culverts designed and installed in Knox County shall conform to the design standards listed in the following sections.

7.3.2.1 Frequency Flood

The 25-year frequency storm shall be routed through all culverts and the 100-year storm shall be used as a check, to verify structures (e.g., houses, commercial buildings) are not flooded or increased damage does not occur to the highway or adjacent property for this design event.

7.3.2.2 Velocity Limitations

Both minimum and maximum velocities shall be considered when designing a culvert. The maximum velocity shall be consistent with channel stability requirements at the culvert outlet. The maximum allowable velocity is 15 feet per second. Outlet protection shall be provided where discharge velocities will cause erosion problems. To ensure self-cleaning during partial depth flow, culverts shall have a minimum velocity of 2.5 feet per second at design flow or lower, with a minimum slope of 0.5%.

7.3.2.3 Buoyancy Protection

Buoyancy protection shall be provided for all flexible culverts. This can be provided through the use of headwalls, endwalls, slope paving or other means of anchoring.

7.3.2.4 Length and Slope

The culvert length and slope shall be chosen to approximate existing topography. To the degree practicable, the culvert invert should be aligned with the channel bottom and the skew angle of the stream, and the culvert entrance should match the geometry of the roadway embankment.

7.3.2.5 Debris Control

Debris control shall be performed in a manner consistent with *Hydraulic Engineering Circular No. 9* entitled <u>Debris Control Structures</u> (FHWA, 1971), which contains criteria pertaining to the design of debris control structures.

7.3.2.6 Headwater Limitations

Headwater is water above the culvert invert at the entrance end of the culvert. The allowable headwater elevation is that elevation above which damage may be caused to adjacent property and/or the roadway. The headwater elevation is determined from an evaluation of land use upstream of the culvert and the proposed or existing roadway elevation. It is this allowable headwater depth that is the primary basis for sizing a culvert.

The following criteria related to headwater should be considered when designing a culvert for the 25-year design storm event.

- The *allowable headwater* is the depth of water that can be ponded at the upstream end of the culvert during the 100 yr event with clogged conditions, which will be limited by one or more of the following constraints or conditions.
  (1) The allowable headwater must not damage upstream property.
  (2) The ponding depth is to be no greater than the low point in the road grade.
  (3) The ponding depth is to be no greater than the elevation where flow diverts around the culvert.
  (4) Headwater elevations shall be established to delineate potential flood zones.
- In general, the constraint that gives the lowest allowable headwater elevation (HW) establishes the criteria for the hydraulic calculations.
- For drainage facilities with cross-sectional area equal to or less than 30 ft^2, HW/D should be equal to or less than 1.5.

- For drainage facilities with cross-sectional area greater than 30 ft^2, HW/D should be equal to or less than 1.2.
- The headwater should be checked using the peak discharge for the 100-year frequency event (Qp_{100}) to ensure compliance with storm system design criteria. As well, the culvert should be sized to maintain flood-free conditions on classified roadways.
- The maximum acceptable outlet velocity shall be identified (see Section 7.4.3) in drainage calculations included with the Stormwater Management Plan.
- Acceptable flow velocities shall be achieved by one of two ways: the headwater shall be set to an appropriate elevation; or, stabilization or energy dissipation shall be provided where acceptable velocities are exceeded.
- Other site-specific design considerations shall be addressed as required by the Director.

7.3.2.7 Tailwater Considerations

The hydraulic conditions downstream of the culvert site must be evaluated to determine a tailwater depth for a range of discharge. At times, there may be a need for calculating backwater curves to establish the tailwater conditions. The following conditions must be considered when establishing tailwater conditions.

- If the culvert outlet is operating with a free outfall, the critical depth and equivalent hydraulic grade line should be determined.
- For culverts that discharge to an open channel, the stage-discharge curve for the channel must be determined (see Section 7.4).
- If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.
- If the culvert discharges to a lake, pond, or other major water body, the expected high water elevation of the particular water body can be used to establish the culvert tailwater.

7.3.2.8 Storage

If storage is being assumed or will occur upstream of the culvert, storage routing must be performed in accordance with the information provided in Section 7.3.4.6.

7.3.2.9 Culvert Inlets

Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient K_e, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 7-2.

Table 7-2. Inlet Coefficients

(Source: USDOT, 1985)

Type of Structure & Design of Entrance	Coefficient K_e
Pipe Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12(D)]	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope*	0.5

Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal[1]	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
End-Section conforming to fill slope*	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of [1/12(D)] or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of [1/12(D)] or beveled top edge	0.2
Wingwalls at 10° or 25° to barrel	
Square-edged at crown	0.5
Side- or slope-tapered inlet	0.2

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

* Note: End Section conforming to fill slope, made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control.

7.3.2.10 Inlets with Headwalls

Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, providing embankment protection against erosion, providing protection from buoyancy, and shortening the length of the required structure. Headwalls are required for all metal culverts and where buoyancy protection is necessary. If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, a short channel apron shall be provided at the toe of the headwall. This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

7.3.2.11 Wingwalls

Wingwalls must be used where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.

7.3.2.12 Improved Inlets

Where inlet conditions control the amount of flow that can pass through the culvert, improved inlets can greatly increase the hydraulic performance of the culvert.

7.3.2.13 Material Selection

Reinforced concrete pipe (RCP) is recommended for use (1) under a roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams. RCP must be used for culverts designed for a 100-year storm, if the culverts lie in public lands or easements.

RCP and fully coated corrugated metal pipe can be used in all other cases. High-density polyethylene (HDPE) pipe may also be used where permitted by the Director. Table 7-3 gives recommended Manning's "n" values for different materials.

Table 7-3. Manning's "n" Values
(Source: USDOT, 1985)

Type of Conduit	Wall & Joint Description	Manning's "n"
Concrete Pipe	Good joints, smooth walls	0.012
	Good joints, rough walls	0.016
	Poor joints, rough walls	0.017
Concrete Box	Good joints, smooth finished walls	0.012
	Poor joints, rough, unfinished walls	0.018
Corrugated Metal Pipes and Boxes Annular Corregations	2-2/3- by ½-inch corrugations	0.024
	6- by 1-inch corrugations	0.025
	5- by 1-inch corrugations	0.026
	3- by 1-inch corrugations	0.028
	6-by 2-inch structural plate	0.035
	9-by 2-1/2 inch structural plate	0.035
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	2-2/3-by ½-inch corrugated 24-inch plate width	0.012
Spiral Rib Metal Pipe	3/4 by 3/4 in recesses at 12 inch spacing, good joints	0.013
High Density Polyethylene (HDPE)	Corrugated Smooth Liner	0.015
	Corrugated	0.020
Polyvinyl Chloride (PVC)		0.011
Note: For further information concerning Manning "n" values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5, page 163		

7.3.2.14 Culvert Skews

Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval.

7.3.2.15 Culvert Sizes

The minimum allowable pipe diameter shall be 15 inches.

7.3.2.16 Weep Holes

Weep holes are sometimes used to relieve uplift pressure. Filter materials shall be used in conjunction with the weep holes in order to intercept the flow and prevent the formation of piping channels. The filter materials shall be designed as an underdrain filter so as not to become clogged and so that piping cannot occur through the pervious material and the weep hole.

7.3.2.17 Outlet Protection

Water shall not discharge from a culvert in an erosive manner. Outlet protection shall be provided for all design storms. See Section 7.5 for information on the design of outlet protection.

7.3.2.18 Environmental Considerations

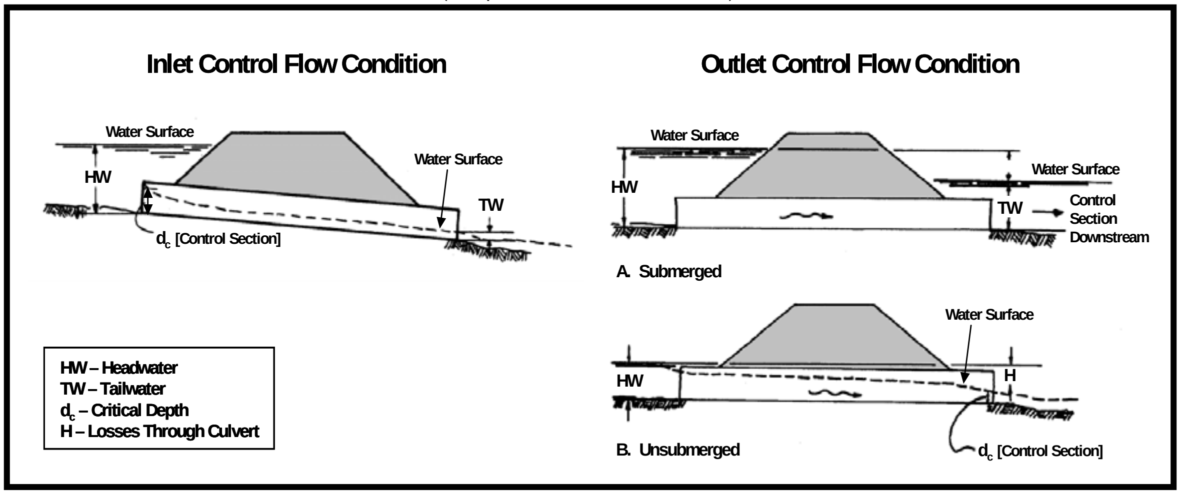
Where compatible with good hydraulic engineering, a site should be selected that will permit the culvert to be constructed to cause the least impact on the stream, wetlands and other environmentally sensitive features that may be located on the site. This selection must consider the entire site, including any necessary lead channels.

7.3.3 Design Procedures

7.3.3.1 Types of Flow Control

There are two types of flow conditions for culverts that are based upon the location of the control section and the critical flow depth. These flow conditions are presented in Figure 7-7 and are described briefly below.

Figure 7-7. Culvert Flow Conditions
(Adapted from: USDOT, 1985)



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

Outlet Control – Outlet control flow 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 flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control, see Hydraulic Design of Highway Culverts (USDOT, 2001).

7.3.3.2 Procedures

There are two procedures for designing culverts: manual use of inlet and outlet control nomographs; and the use computer programs such as HY8 or Haestad Methods programs such as CulvertMaster or FlowMaster. It is recommended that a computer model be used for culvert design. The computer software packages available use the theoretical basis from the nomographs to size culverts. In addition, these software packages can evaluate improved inlets, route hydrographs, consider road overtopping, and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scour.

7.3.3.3 Nomographs

The use of culvert design nomographs requires a trial and error solution. Nomograph solutions provide reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional, separate computations beyond what can be obtained from the nomographs. Figures 7-8(a) and (b) show examples of an inlet control and outlet control nomograph for the design of concrete pipe culverts. For other culvert designs, refer to the complete set of nomographs in Appendix A.

7.3.3.4 Design Procedure

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

(Step 1) List design data:

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

(Step 2) Determine trial culvert size by assuming a trial velocity 5 to 8 ft/s and computing the culvert area, A = Q/V. Determine the culvert diameter.

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

- For inlet control, enter inlet control nomograph with D and Q and find HW/D for the proper entrance type.
- Compute HW and, if too large or too small, try another culvert size before computing HW for outlet control.
- For outlet control enter the outlet control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter.
- To compute HW, connect the length scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from Equation 7-10.

Equation 7-10

$$HW = H + h_o - LS$$

where:

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

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Figure 7-8(a). Headwater Depth for Concrete Pipe Culvert with Inlet Control
(Source: USDOT, 1985)

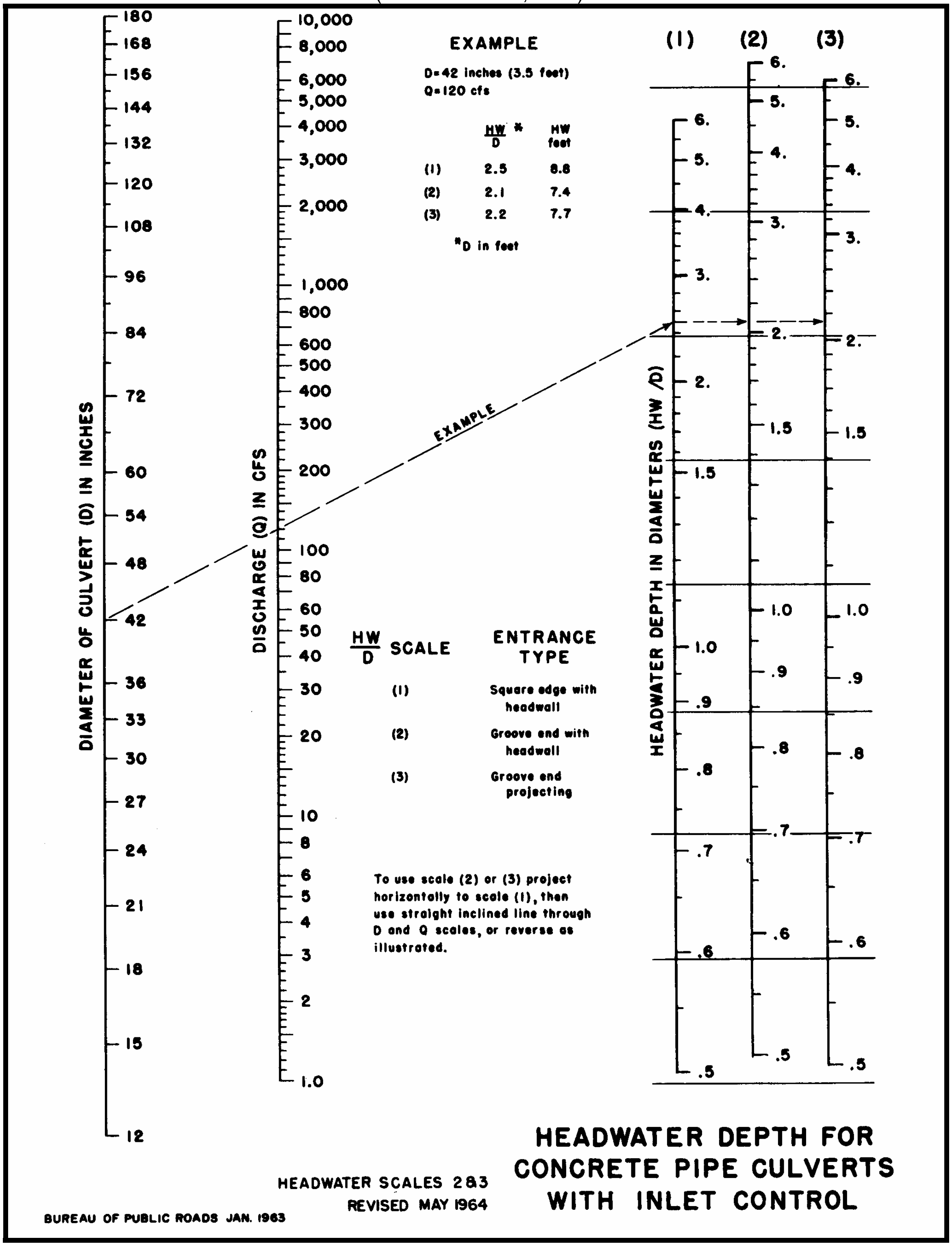
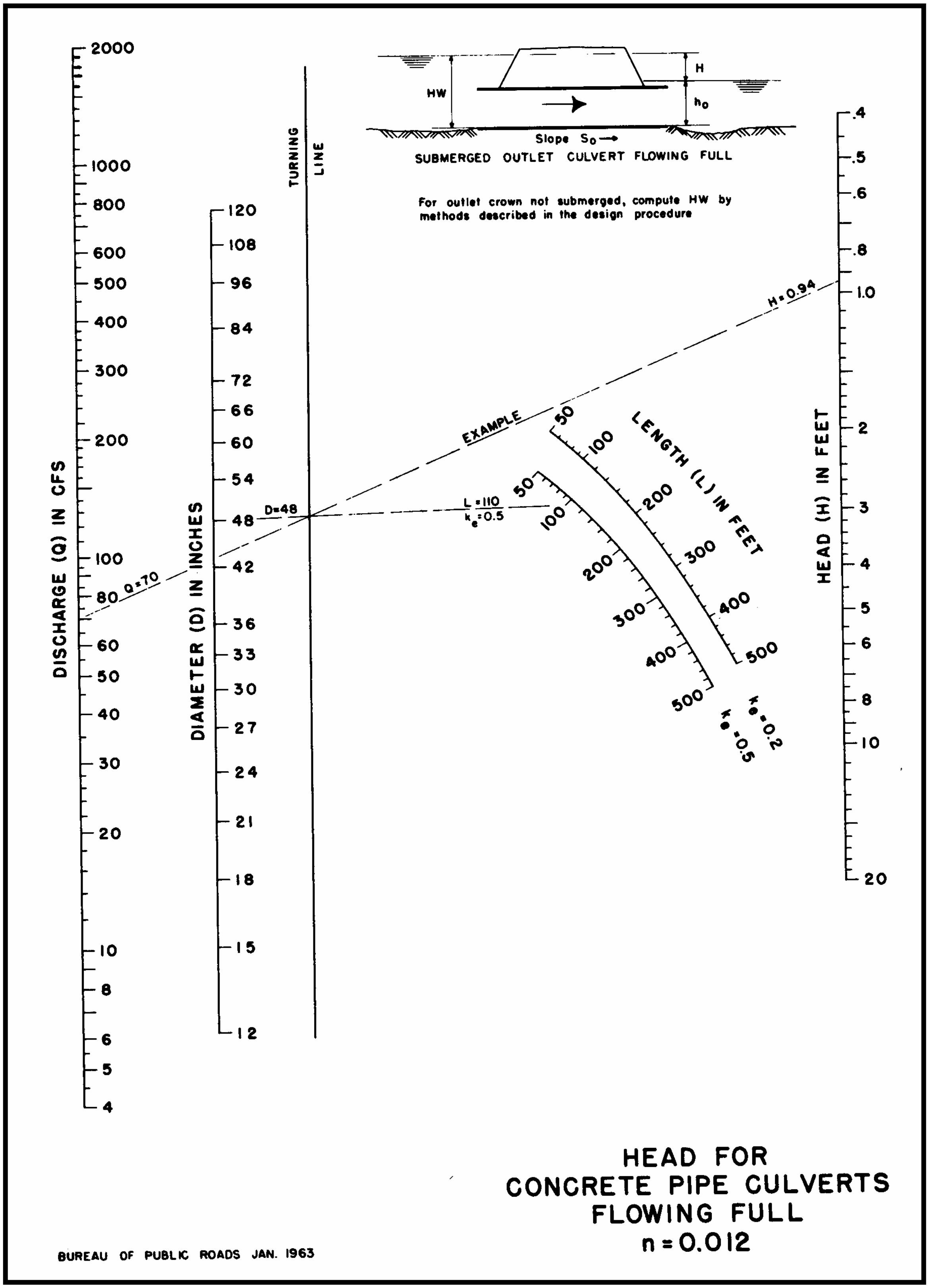


Figure 7-8(b). Head for Concrete Pipe Culverts Flowing Full

(Source: USDOT, 1985)



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

- If inlet control governs, then the design is complete and no further analysis is required.
- If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

(Step 5) Calculate exit velocity and if erosion problems might be expected, refer to Section 7.5 for appropriate energy dissipation designs.

7.3.3.5 Performance Curves - Roadway Overtopping

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

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

The overall performance curve can be determined using the following general procedure.

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

(Step 2) Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

(Step 3) When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation 7-11 to calculate flow rates across the roadway.

Equation 7-11

$$Q = C_d L (HW)^{1.5}$$

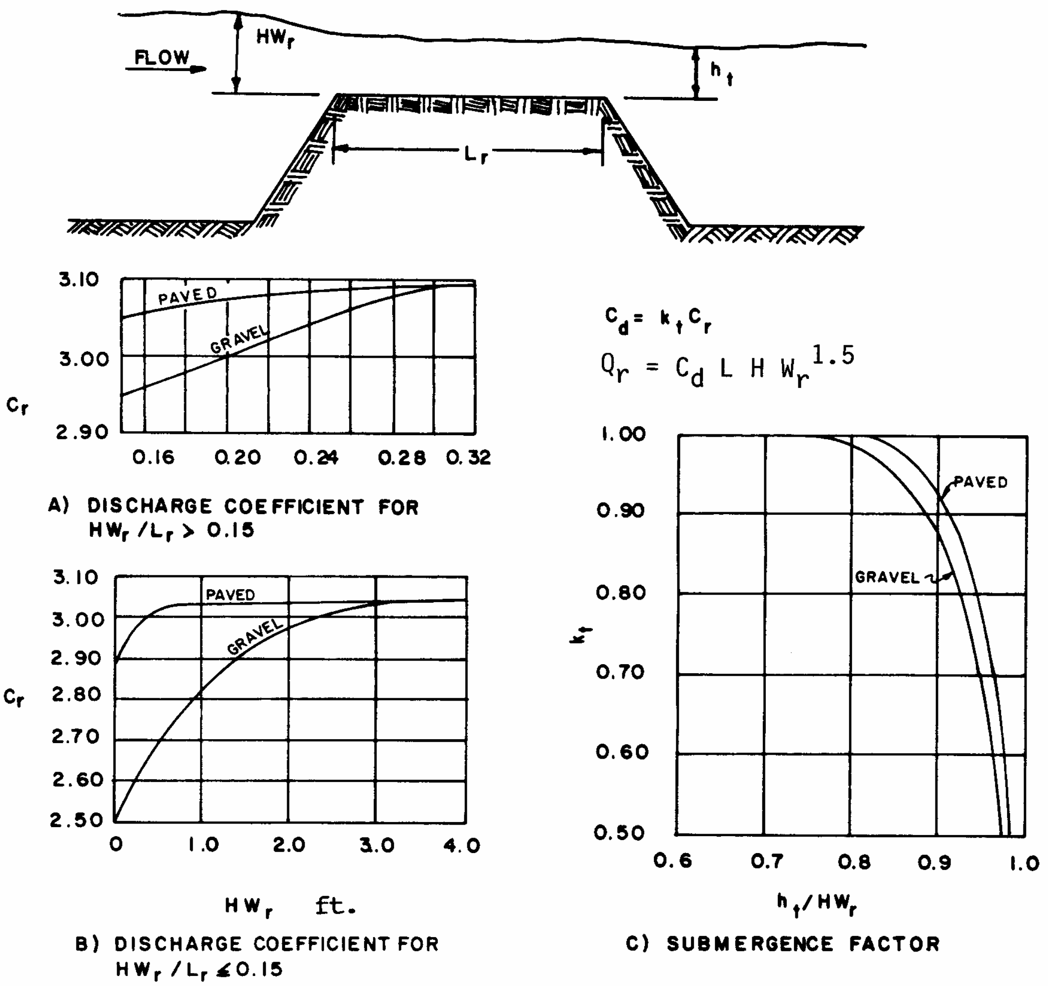
where:

Q = overtopping flow rate (cfs)
C_d = overtopping discharge coefficient
L = length of roadway (ft)
HW = upstream depth, measured from the roadway crest to the water surface up-stream of the weir drawdown (ft)

Note: See Figure 7-9 for guidance in determining a value for C_d. For more information on calculating overtopping flow rates see pages 38 - 40 in the Hydraulic Design of Highway Culverts (USDOT, 2001).

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

Figure 7-9. Discharge Coefficients for Roadway Overtopping
(Source: USDOT, 1985)



7.3.3.6 Storage Routing

A significant storage capacity behind a highway embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced considerably. If significant storage is anticipated behind a culvert, the design should be checked by routing the design hydrographs through the culvert to determine the discharge and stage behind the culvert. See Section 7.3.7 and Volume 2, Chapter 3, Section 3.2 for more information on routing. Additional routing procedures are outlined in Hydraulic Design of Highway Culverts (USDOT, 2001).

Note: Storage should be taken into consideration only if the storage area will remain available for the life of the culvert as a result of purchase of ownership or right-of-way or an easement has been acquired.

7.3.4 Culvert Design Example

The following example problem illustrates the procedures to be used in designing culverts using the nomographs.

Example 7-1. Culvert Sizing Example

Size a concrete pipe culvert given the following example data, which were determined by physical limitations at the culvert site and hydraulic procedures described elsewhere in this manual.

Input Data

Discharge for 2-yr flood = 35 cfs
Discharge for 25-yr flood = 70 cfs
Allowable headwater (HW) for 25-yr discharge = 5.25 ft
Length of culvert = 100 ft
Natural channel invert elevations - inlet = 15.50 ft, outlet = 14.30 ft
Culvert slope = 0.012 ft/ft
Tailwater depth for 25-yr discharge = 3.5 ft
Tailwater depth is the normal depth in downstream channel (assumed)
Entrance type = Groove end with headwall

Computations

Step 1. Assume a culvert velocity of 5 ft/s. Required flow area = 70 cfs/(5 ft/s) = 14 ft^2 (for the 25-yr recurrence flood).

Step 2. The corresponding culvert diameter is about 48 in (4 ft). This can be calculated by using the formula for area of a circle: Area = $(3.14D^2)/4$ or D = $(\text{Area times } 4/3.14)^{0.5}$. Therefore,

$D = ((14)(4)/3.14)^{0.5}(12\text{in}/1\text{ft})$
D = 50.7 in

Step 3. A grooved end culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 7-8(a)), with a pipe diameter of 48 inches and a discharge of 70 cfs; read a HW/D value of 0.93.

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

Step 5. Using the same procedures outlined in Steps 3 and 4 the following results were obtained.

42-inch culvert – HW = 4.13 ft
36-inch culvert – HW = 4.98 ft
Select a 36-inch culvert to check for outlet control.

Step 6. The culvert is checked for outlet control by using Figure 7-8(b).

With an entrance loss coefficient K_e of 0.20, a culvert length of 100 ft, and a pipe diameter of 36 in., an H value of 2.8 ft is determined. The headwater for outlet control is computed by the equation: $HW = H + h_o - LS$

Compute h_o
h_o = TW or ½ (critical depth in culvert + D), whichever is greater.
$h_o = 3.5$ ft or $h_o = 0.5(2.7+3.0) = 2.85$ ft
Note: critical depth is obtained from Chart 4, located in Appendix A.

Therefore: h_o = 3.5 ft
The headwater depth for outlet control is:
$HW = H + h_o - LS = 2.8 + 3.5 - (100)(0.012) = 5.10$ ft

Step 7. Since HW for outlet (5.10 ft) is greater than the HW for inlet control (4.98 ft), outlet control governs the culvert design. Thus, the maximum headwater expected for a 25-year recurrence flood is 5.10 ft, which is less than the allowable headwater of 5.25 ft.

Step 8. Estimate outlet exit velocity. Since this culvert is on outlet control and discharges into an open channel downstream with tailwater above culvert, the culvert will be flowing full at the flow depth in the channel. Using the design peak discharge of 70 cfs and the area of a 36-inch or 3.0-foot diameter culvert the exit velocity will be:

$Q = VA$
Therefore:
$V = 70/((3.14)(3.0)^2/4) = 9.9$ ft/s

With this high velocity, consideration should be given to provide an energy dissipator at the culvert outlet. See Section 7.5 (Energy Dissipation Design).

Step 9. Check for minimum velocity using the 2-year flow of 35 cfs.

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

Step 10. The 100-year flow should be routed through the culvert to determine if any flooding problems will be associated with this event.

7.3.5 Design Procedures for Beveled-Edged Inlets

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side-tapered, and slope-tapered inlets. Those designers interested in using side- and slope-tapered inlets should consult the detailed design criteria and example designs outlined in the Hydraulic Design of Highway Culverts (USDOT, 1985).

7.3.5.1 Design Figures

Four inlet control figures for culverts with beveled edges are included in Appendix A.

Chart	Use for:
3	circular pipe culverts with beveled rings
9	wingwalls with flare angles of 18 to 45 degrees
10	90° headwalls (same for 90 ° wingwalls)
11	skewed headwalls

The following symbols are used in Figure 7-10:

B - width of culvert barrel or diameter of pipe culvert;
D - height of box culvert or diameter of pipe culvert;
H - depth of pool or head, above the face section of invert;
N - number of barrels; and,
Q - design discharge.

Figure 7-10. Culvert Design Calculation Form

(Source: USDOT, 1985)

PROJECT: ______________________

STATION: ______________________

SHEET ________ OF ________

CULVERT DESIGN FORM

DESIGNER / DATE: ________ OF ________

REVIEWER / DATE: ________ OF ________

HYDROLOGICAL DATA

- ☐ METHOD: ______________
- ☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________
- ☐ CHANNEL SHAPE: ______________
- ☐ ROUTING: ________ ☐ OTHER: ________

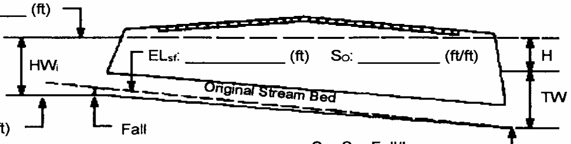
SEE ADD'L SHTS.

DESIGN FLOWS/TAILWATER

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

ROADWAY ELEVATION: ____________ (ft)

EL_{hi}: ________ (ft)



EL_i: ________ (ft)

Fall

$S = S_o$ - Fall/L_o

S = ________ (ft/ft)

L_o = ________ (ft)

EL_o: ________ (ft)

CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (cfs)	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)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

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)

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

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

SUSCRIPT DEFINITIONS:

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

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ______________

SHAPE: ______________

MATERIAL: ______________ n ________

ENTRANCE: ______________

7.3.5.2 Design Procedure

The figures (Appendix A) for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier.

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

Top Bevel = d = 6 ft x 0.5 inch/ft = 3 inches; and,
Side Bevel = b = 8 ft x 0.5 inch/ft = 4 inches.

For a 1.5:1 bevel, the dimension computations would result in d = 6 and b = 8 inches.

7.3.5.3 Design Figure Limits

The improved inlet design figures (Appendix A) are based on research results from culvert models with barrel width, B, to depth, D, ratios of 0.5:1 to 2:1. For box culverts with more than one barrel, the figures are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size. For example, in a double 8 ft by 8 ft box culvert:

The top bevel is proportioned based on the height of 8 feet which results in a bevel of 4 in. for the 1:1 bevel and 8 in. for the 1.5:1 bevel.

The side bevel is proportioned based on the clear width of 16 feet, which results in a bevel of 8 in. for the 1:1 bevel and 16 in. for the 1.5:1 bevel.

7.3.5.4 Multi-barrel Installations

For multi-barrel culvert installations exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width, B, or three times the height, whichever is smaller.

The top bevel dimension should always be based on the culvert height.

The shape of the upstream edge of the intermediate walls of multi-barrel installations is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert.

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

7.3.5.5 Skewed Inlets

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

7.3.6 Flood Routing and Culvert Design

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. If flood routing is not considered, it is possible that the findings from culvert analyses will be conservative. If the selected allowable headwater is accepted without flood routing, then costly over design of both the culvert and outlet protection may result, depending on the amount of temporary storage involved. However, if storage is used in the design of culverts, consideration should be given to:

- the total area of flooding,
- the average time that bankfull stage is exceeded for the design flood up to 48 hours in rural areas or 6 hours in urban areas, and
- ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement.

7.3.6.1 Design Procedure

The design procedure for flood routing through a culvert is the same as for reservoir routing. The site data and roadway geometry are obtained and the hydrology analysis completed to include estimating a hydrograph. Once this essential information is available, the culvert can be designed. Flood routing through a culvert can be time consuming. It is recommended that a computer program be used to perform routing calculations; however, an engineer should be familiar with the culvert flood routing design process. Modified Puls routing is required for culvert flood routing.

7.4 Open Channel Design

Open channel systems and their design are an integral part of stormwater drainage design, particularly for development sites utilizing better site design practices and open channel structural BMPs. Open channels include drainage ditches, grass channels, dry and wet enhanced swales, riprap channels and concrete-lined channels.

The purpose of this section is to provide an overview of open channel design standards and methods, including the use of channel design nomographs.

7.4.1 Open Channel Types

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

<u>Vegetative Linings</u> – Vegetation, where practical, is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, provides habitat and provides water quality benefits (see Chapters 2, 3 and Chapter 5 for more details on using enhanced swales and grass channels for water quality purposes).

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

- high velocities;
- continuously flowing water;
- lack of regular maintenance necessary to prevent growth of taller or woody vegetation;
- lack of nutrients and inadequate topsoil; and/or,
- excessive shade.

Proper seeding, mulching and soil preparation are required during construction to ensure establishment of healthy vegetation.

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

Rigid Linings – Rigid linings are generally constructed of concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel headcutting.

7.4.2 Symbols and Definitions

The symbols listed in Table 7-4 will be used to provide consistency within this section. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 7-4. Open Channel Design Symbols and Definitions

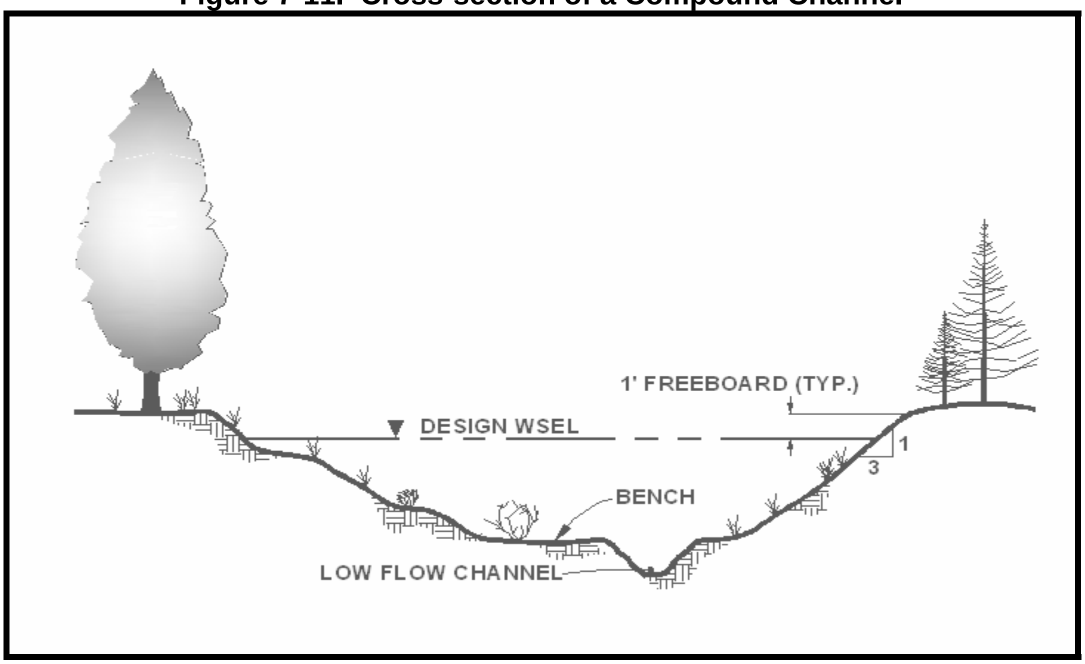
Symbol	Definition	Units
α	Energy coefficient	-
A	Cross-sectional area	ft^2
b	Bottom width	ft
C_g	Specific weight correction factor	-
D or d	Depth of flow	ft
d	Stone diameter	ft
delta d	Superelevation of the water surface profile	ft
d_x	Diameter of stone for which x percent, by weight, of the gradation is finer	ft
E	Specific energy	ft
Fr	Froude Number	-
g	Acceleration due to gravity	32.2 ft/s^2
h_{loss}	Head loss	ft
K	Channel conveyance	-
k_e	Eddy head loss coefficient	-
K_T	Trapezoidal open channel conveyance factor	-
L	Length of channel	ft
L_p	Length of downstream protection	ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge rate	cfs
R	Hydraulic radius of flow	ft
R_c	Mean radius of the bend	ft
S	Slope	ft/ft
SW_s	Specific weight of stone	lbs/ft^3
T	Top width of water surface	ft
V	Velocity of flow	ft/s
w	Stone weight	lbs
y_c	Critical depth	ft
y_n	Normal depth	ft
z	Critical flow section factor	-

7.4.3 Design Standards and Considerations

The list below presents the key considerations for the design of open channels.

- Open channels provide opportunities for reduction of flow peaks and pollution loads. They may be designed as wet or dry enhanced swales or grass channels.
- Channels can be designed with natural meanders improving both aesthetics and pollution removal through increased contact time.
- Grass channels generally provide better habitat than hardened channel sections, though studies have shown that riprap interstices provide significant habitat as well. Velocities should be carefully checked at design flows and the outer banks at bends should be specifically designed for increased shear stress.
- Compound channels (characterized by a cross-section that includes a low-flow channel and natural or man-made benches on one or both sides of the channel) can be developed that carry the channel protection volume (CPv) in the lower section and higher flows above them. A compound section is shown in Figure 7-11. This reduces channel erosion at lower flows, and meandering, self-forming low flow channels that attack banks. The bench in the compound section should have a minimum 1:12 (V:H) slope to ensure drainage.
- Flow control structures can be placed in the channels to increase residence time. Channel slope stability can also be ensured through the use of grade control structures that can serve as pollution reduction enhancements if they are set above the channel bottom. Regular maintenance is necessary to remove sediment and prevent aggradation and the loss of channel capacity.

Figure 7-11. Cross-section of a Compound Channel



All open channels designed and constructed in Knox County shall conform to the design standards listed below.

- Open channels shall be designed to follow natural drainage alignments whenever possible.

- Vegetated channels that are eligible to gain water quality volume (WQv) credits for stormwater treatment shall be granted to Knox County as water quality easements. All easements shall include an unobstructed drive path width of twelve (12) feet.
- Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1, or with compound cross sections.
- Channel side slopes shall be stable throughout the entire length and the side slope shall depend on the channel material. A maximum of 2:1 shall be used for channel side slopes, unless otherwise justified by calculations. Roadside ditches shall have a maximum side slope of 3:1.
- The design of artificial channels shall consider the frequency and type of maintenance required, and shall make allowances for access of maintenance equipment.
- Trapezoidal cross sections are preferred over triangular shapes for artificial channel designs.
- For vegetative channels, design stability shall be determined using low vegetative retardance conditions (Class D). For design capacity determination, higher vegetative retardance conditions (Class C) shall be used.
- For vegetative channels, flow velocities within the channel should not exceed the maximum permissible velocities given in Tables 7-5 and 7-6.
- The final design of artificial open channels shall be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for selected lining categories are presented in Table 7-5. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 7-6. Vegetative lining calculations are presented in Section 7.4.7 and riprap procedures are presented in Section 7.4.8.

Table 7-5. Maximum Velocities for Comparing Lining Materials

(Source: AASHTO, 1991)

Material	Maximum Velocity (ft/s)
Sand	2.0
Silt	3.5
Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay	5.0
Graded Loam or Silt to Cobbles	5.0
Course Gravel	6.0
Shales and Hard Pans	6.0

- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated. Unless proper authorization is obtained from Knox County and the adjacent property owner(s), open channels must enter and exit a site where the channel historically flows. Knox County is not responsible for obtaining any State and/or Federal permits that may be applicable to channel relocation on a development or redevelopment site.
- Streambank stabilization shall be provided, when appropriate, as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site. No actively eroding, bare or unstable vertical banks shall remain unless the Director has determined there is no better alternative.

Table 7-6. Maximum Velocities for Vegetative Channel Linings
(Source: Atlanta Regional Council, 2001)

Vegetation Type	Slope Range (%)[1]	Maximum Velocity[2] (ft/s)
Bermuda Grass	0 - 10	5.0
Bahia	0 - 10	4.0
Tall fescue grass mixtures[3]	0 - 10	4.0
Kentucky Bluegrass	0 - 5	6.0
Buffalo Grass	0 - 10	5.0
	> 10	4.0
Grass Mixtures	0 - 5	4.0
	5 -10	3.0
Sericea lespedeza, Weeping lovegrass, Alfalfa	0 - 5[4]	3.0
Annuals[5]	0 - 5	3.0
Sod	Any	4.0
Lapped Sod	Any	5.0

[1] Do not use on slopes steeper than 10% except for side-slope in combination channel.
[2] Use velocities exceeding 5 ft/s only where good stands can be maintained.
[3] Mixtures of Tall Fescue, Bahia, and/or Bermuda
[4] Do not use on slopes steeper than 5% except for side-slope in combination channel.
[5] Annuals - used on mild slopes or as temporary protection until permanent covers are established

- Open channel drainage systems shall be sized to handle the minimum design storm stated in the Knox County Stormwater Management Ordinance plus one (1) foot of freeboard to determine if applicable building elevations are exceeded.

7.4.4 Manning’s “n” Values

The Manning’s “n” value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning’s “n” values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process.

Recommended Manning’s “n” values for artificial channels with rigid, unlined, temporary, and riprap linings are given in Table 7-7. Recommended values for vegetative linings should be determined using Figure 7-12, which provides a graphical relationship between Manning’s “n” values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see Table 7-8). Figure 7-12 is used iteratively as described in Section 7.4.6.

Recommended Manning's values for natural channels that are either excavated or dredged and natural are given in Table 7-9. For natural channels, Manning’s “n” values should be estimated using experienced judgment and information presented in publications such as the Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains (USDOT, 1984).

7.4.5 Uniform Flow Calculations

7.4.5.1 Design Charts

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

Table 7-7. Manning's Roughness Coefficients (n) for Artificial Channels

(Source: USDOT, 1986)

Category	Lining Type	Depth Ranges		
		0 – 0.5 ft	0.5 – 2.0 ft	> 2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D_{50}	0.044	0.033	0.030
	2-inch D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}		0.078	0.040

Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.
*Some "temporary" linings become permanent when buried.

7.4.5.2 Manning's Equation

Manning's Equation, presented in three forms in Equations 7-12, 7-13 and 7-14 below, is recommended for evaluating uniform flow conditions in open channels.

Equation 7-12

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

Equation 7-13

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

Equation 7-14

$$S = \left[\frac{Qn}{\left(1.49 A R^{2/3}\right)} \right]^2$$

Figure 7-12. Manning's "n" Values for Vegetated Channels
(Source: USDA, 1947)

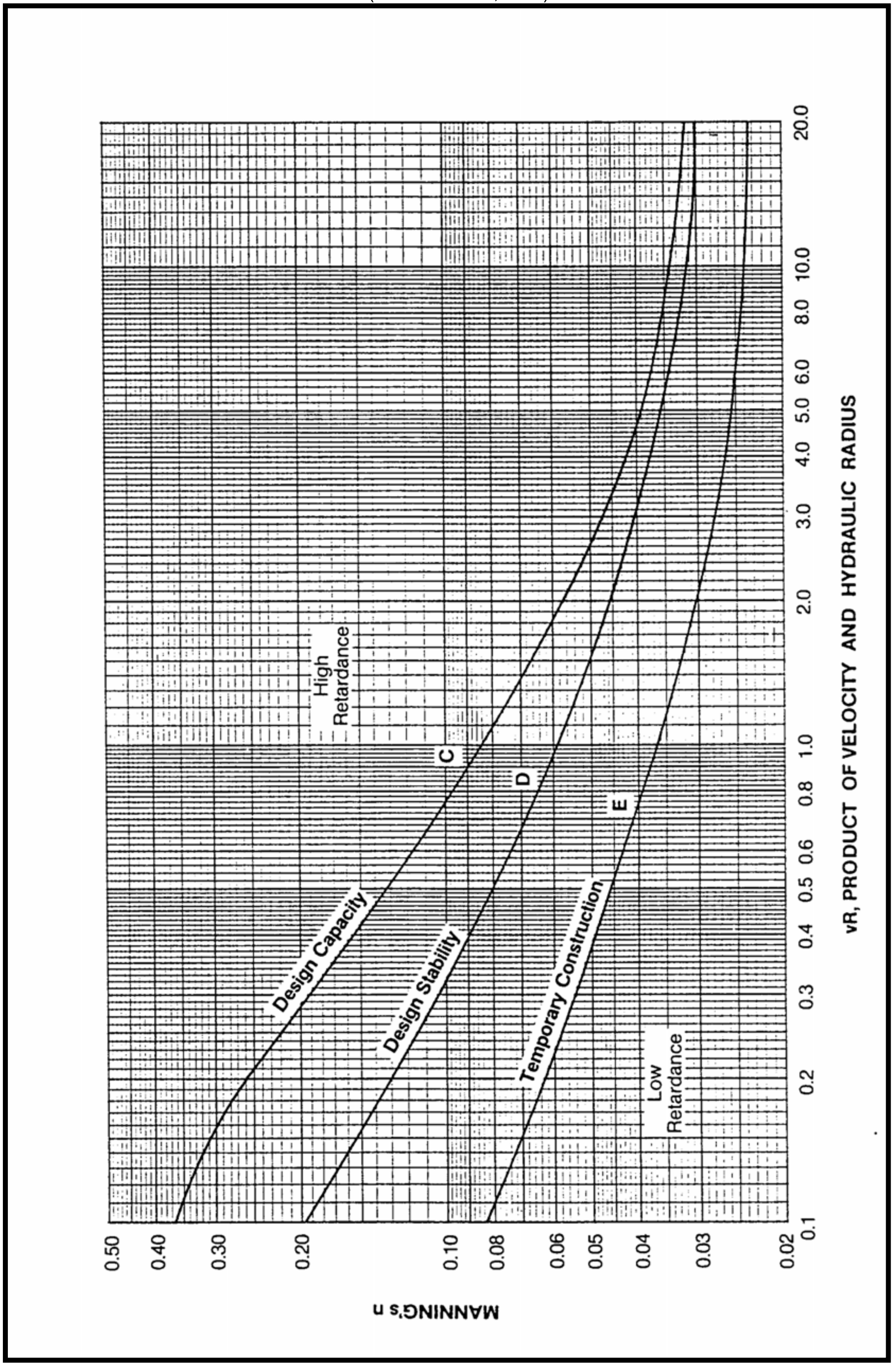


Table 7-8. Classification of Vegetal Covers as to Degrees of Retardance
(Source: USDOT, 1986)

Retardance	Cover	Condition
A	Weeping Lovegrass	Excellent stand, tall (average 30")
	Yellow Bluestem Ischaemum	Excellent stand, tall (average 36")
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12")
	Native grass mixture little bluestem, bluestem, blue gamma other short and long stem Midwest lovegrass	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 24")
	Laspedeza sericea	Good stand, not woody, tall (average 19")
	Alfalfa	Good stand, uncut (average 11")
	Weeping lovegrass	Good stand, unmowed (average 13")
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 13”)
C	Crabgrass	Fair stand, uncut (10 - 48")
	Bermuda grass	Good stand, mowed (average 6")
	Common lespedeza	Good stand, uncut (average 11")
	Grass-legume mixture: summer (orchard grass redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 - 8")
	Centipede grass	Very dense cover (average 6")
	Kentucky bluegrass	Good stand, headed (6 - 12")
D	Bermuda grass	Good stand, cut to 2.5"
	Common lespedeza	Excellent stand, uncut (average 4.5")
	Buffalo grass	Good stand, uncut (3 - 6")
	Grass-legume mixture: fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza	Good stand, uncut (4 - 5")
	Lespedeza serices	After cutting to 2" (very good before cutting)
E	Bermuda grass	Good stand, cut to 1.5"
	Bermuda grass	Burned stubble

where:

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

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are assumed to be equal.

Table 7-9. Uniform Flow Values of Roughness Coefficient n
(Source: USDOT, 1986)

Type of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
a. Earth, straight and uniform	0.016	0.018	0.02
1. Clean, recently completed	0.018	0.022	0.025
2. Clean, after weathering	0.022	0.025	0.03
3. Gravel, uniform section, clean	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.03
2. Grass, some weeds	0.025	0.03	0.033
3. Dense weeds/plants in deep channels	0.03	0.035	0.04
4. Earth bottom and rubble sides	0.025	0.03	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.03	0.04	0.05
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.05	0.06
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.04
2. Jagged and irregular	0.035	0.04	0.05
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.05	0.08	0.12
2. Clean bottom, brush on sides	0.04	0.05	0.08
3. Same, highest stage of flow	0.045	0.07	0.11
4. Dense brush, high stage	0.08	0.1	0.14
NATURAL STREAMS			
Minor streams (top width at flood stage < 100 ft)			
a. Streams on Plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.03	0.033
2. Same as above, but more stones and weeds	0.03	0.035	0.04
3. Clean, winding, some pools and shoals	0.033	0.04	0.045
4. Same as above, but some weeds and some stones	0.035	0.045	0.05
5. Same as above, lower stages, more ineffective slopes and sections	0.04	0.048	0.055
6. Same as 4, but more stones	0.045	0.05	0.06
7. Sluggish reaches, weedy, deep pools	0.05	0.07	0.08
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.1	0.15
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			

Type of Channel and Description	Minimum	Normal	Maximum
1. Bottom: gravels, cobbles, few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land, tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
Major Streams (top width at flood stage >100 ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025		0.060
b. Irregular and rough section	0.035		0.100

7.4.5.3 Geometric Relationships

Area, wetted perimeter, hydraulic radius, and channel top width for standard channel cross sections can be calculated from geometric dimensions. Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

7.4.5.4 Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from Equation 7-13. The slope can be calculated using Equation 7-14 when the discharge, roughness coefficient, area, and hydraulic radius are known.

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 7-13 and 7-14. Figure 7-13 provides a general solution for the velocity form of Manning's Equation, while Figure 7-14 provides a solution of Manning's Equation for trapezoidal channels.

A direct solution of Manning's Equation is provided in Example 7-2 below.

General Solution Nomograph

The following steps are used for the general solution nomograph in Figure 7-13.

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

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

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

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

Example 7-2. Uniform Flow Example Problem – Direct Solution of Manning's Equation

Use Manning's Equation to find the velocity, v, for an open channel with a hydraulic radius value of 0.6 ft, an n value of 0.020, and slope of 0.003 ft/ft. Solve using Figure 7-13:

Step 1. Connect a line between the slope scale at 0.003 and the roughness scale at 0.020 and note the intersection point on the turning line.

Step 2. Connect a line between that intersection point and the hydraulic radius scale at 0.6 ft and read the velocity of 2.9 ft/s from the velocity scale.

Trapezoidal Solution Nomograph

The trapezoidal channel nomograph solution to Manning's Equation in Figure 7-14 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

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

(1) Given Q, find d.

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

b. Connect a line from the turning point from Step 2a to the b scale and find the intersection with the z = 0 scale.

c. Project horizontally from the point located in Step 2b to the appropriate z value and find the value of d/b.

d. Multiply the value of d/b obtained in Step 2c by the bottom width b to find the depth of uniform flow, d.

Figure 7-13. Nomograph for the General Solution of Manning's Equation

(Source: USDOT, 1961)

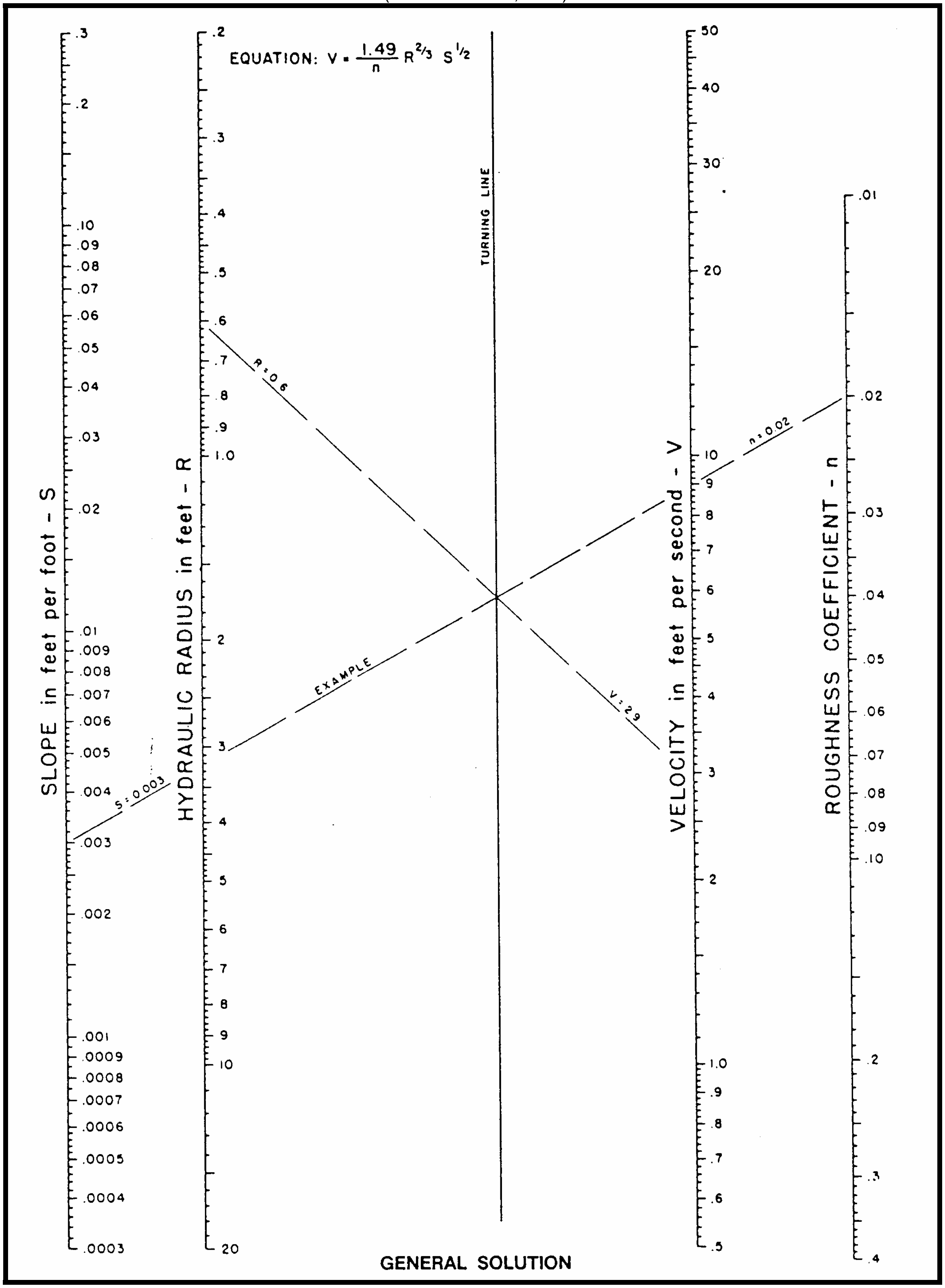
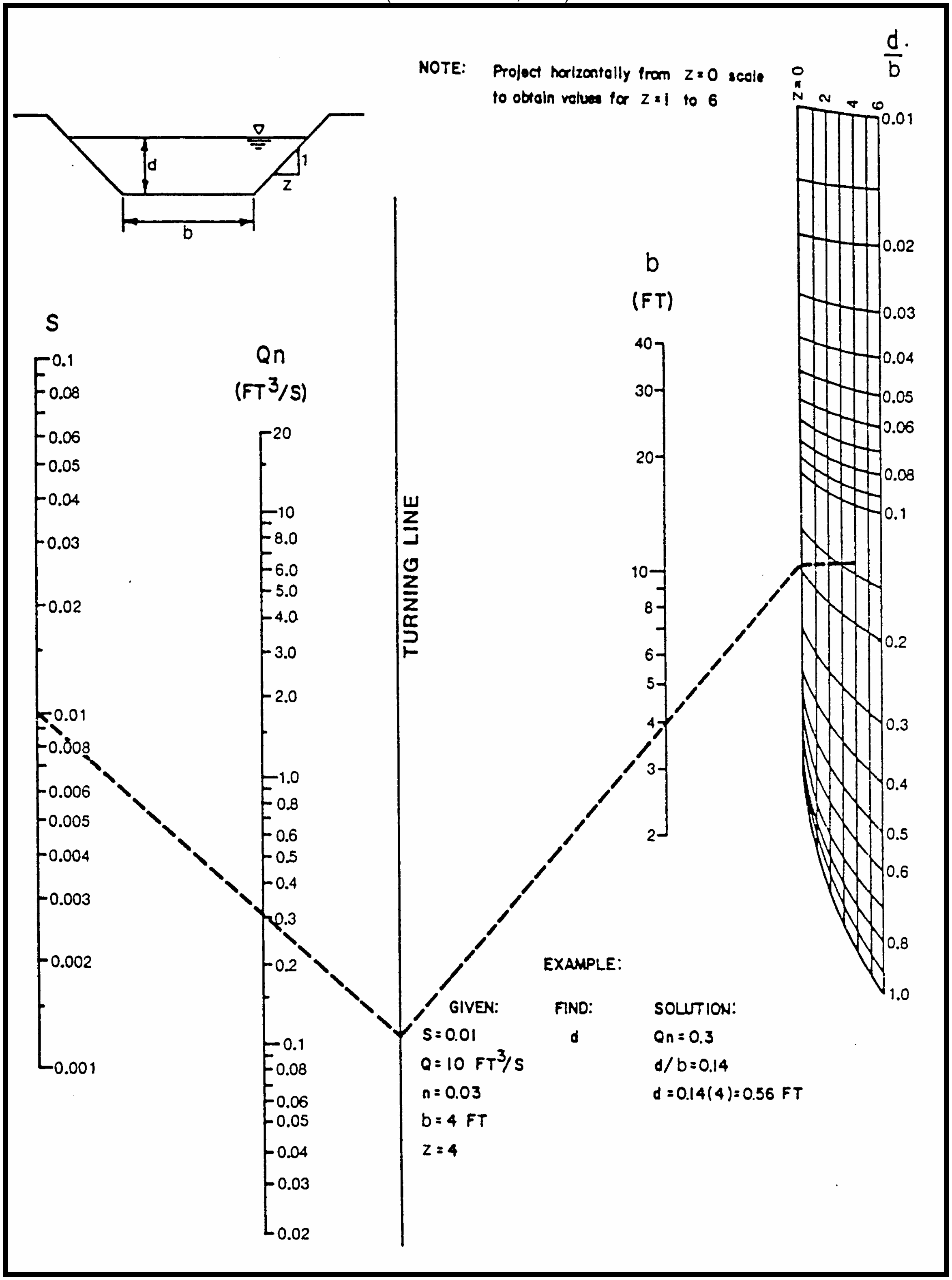


Figure 7-14. Solution of Manning's Equation for Trapezoidal Channels

(Source: USDOT, 1986)



(2) Given d, find Q

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

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

c. Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Q_n scale.

d. Divide the value of Qn obtained in Step 3c by the n value to find the design discharge, Q.

7.4.5.5 Trial and Error Solutions

A trial and error procedure for solving Manning's Equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process, Manning's Equation can be arranged as shown in Equation 7-15.

Equation 7-15

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

where:

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

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of Equation 7-15 is satisfied such that the design flow is conveyed for the slope and selected channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in Figure 7-15 for trapezoidal channels.

(Step 1) Determine input data, including design discharge (Q), Manning's "n" value, channel bottom width (b), channel slope (S), and channel side slope (z).

(Step 2) Calculate the trapezoidal conveyance factor using Equation 7-16.

Equation 7-16

$$K_T = \frac{(Qn)}{b^{8/3}S^{1/2}}$$

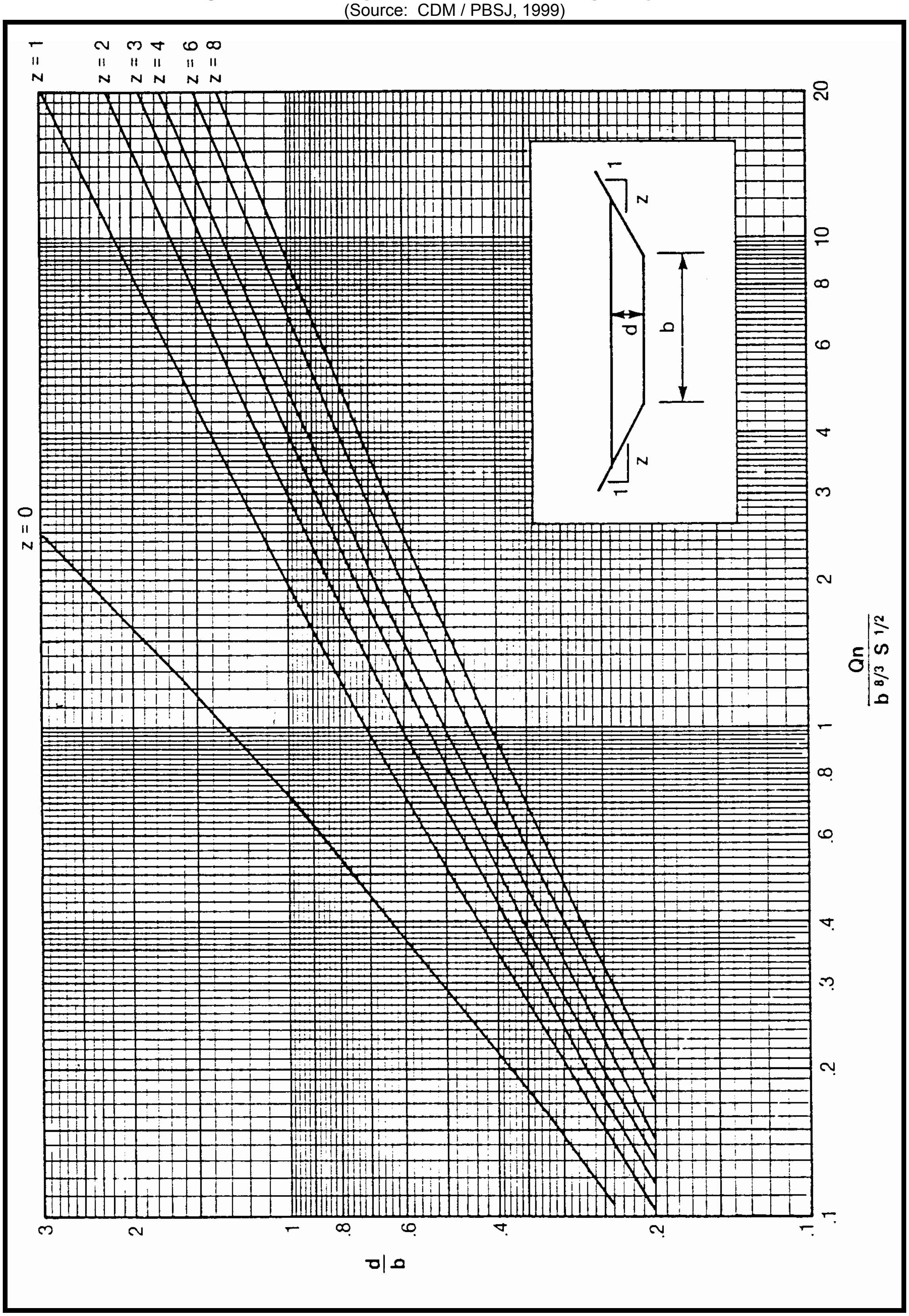
where:

K_T = trapezoidal open channel conveyance factor
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
b = bottom width (ft)
S = slope of the energy grade line (ft/ft)

(Step 3) Enter the x-axis of Figure 7-15 with the value of K_T calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate z value from Step 1.

Figure 7-15. Trapezoidal Channel Capacity Chart

(Source: CDM / PBSJ, 1999)



(Step 4) From the point of intersection obtained in Step 3, draw a horizontal line to the y-axis and read the value of the normal depth of flow over the bottom width, d/b.

(Step 5) Multiply the d/b value from Step 4 by b to obtain the normal depth of flow.

Note: If bends are considered, refer to Equation 7-22.

7.4.6 Critical Flow Calculations

7.4.6.1 Background

In the design of open channels, it is important to calculate the critical depth in order to determine if the flow in the channel will be subcritical or supercritical. If the flow is subcritical, it is relatively easy to handle the flow through channel transitions because the flows are tranquil and wave action is minimal. In subcritical flow, the depth at any point is influenced by a downstream control, which may be either the critical depth or the water surface elevation in a pond or larger downstream channel. In supercritical flow, the depth of flow at any point is influenced by a control upstream, usually critical depth. In addition, the flows have relatively shallow depths and high velocities.

Critical depth depends only on the discharge rate and channel geometry. Equation 7-17 presents the general equation for determining critical depth.

Equation 7-17

$$\frac{Q^2}{g} = \frac{A^3}{T}$$

where:

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

Note: A trial and error procedure is needed to solve Equation 7-17.

7.4.6.2 Semi-Empirical Equations

Semi-empirical equations (as presented in Table 7-10) or section factors (as presented in Figure 7-16) can be used to simplify trial and error critical depth calculations. Equation 7-18 is used to determine critical depth with the critical flow section factor, Z:

Equation 7-18

$$Z = \frac{Q}{g^{0.5}}$$

where:

Z = critical flow section factor
Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.2 ft/sec^2)

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

Equation 7-19

$$Fr = \frac{v}{\left(\frac{gA}{T}\right)^{0.5}}$$

where:

Fr = Froude number (dimensionless)
v = velocity of flow (ft/s)
g = acceleration of gravity (32.2 ft/sec^2)
A = cross-sectional area of flow (ft^2)
T = top width of flow (ft)

Table 7-10. Critical Depth Equations for Uniform Flow in Selected Channel Cross Sections

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

where:
d_c = critical depth (ft)
Q = design discharge (cfs)
g = acceleration due to gravity (32.2 ft/ sec^2)
b = bottom width of channel (ft)
z = side slopes of a channel (horizontal to vertical)
D = diameter of circular conduit (ft)
A = cross-sectional area of flow (ft^2)
T = top width of water surface (ft)

[1]See Figure 7-16 for channel sketches
[2]Assumes uniform flow with the kinetic energy coefficient equal to 1.0
[3]Reference: French, 1985
[4]Reference: USDOT, 1965
[5]Reference: Brater and King, 1976

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

7.4.7 Vegetative Design

7.4.7.1 Introductory Guidance

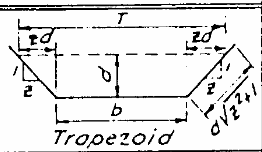
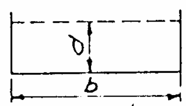
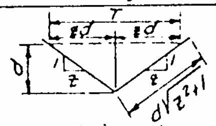
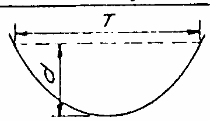
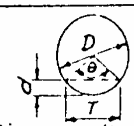
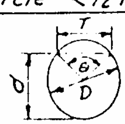
A two-part procedure is recommended for final design of temporary and vegetative channel linings. Part 1, the design stability component, involves determining channel dimensions for low vegetative retardance conditions, using Class D as defined in Table 7-9. Part 2, the design capacity component, involves determining the depth increase necessary to maintain capacity for higher vegetative retardance conditions, using Class C as defined in Table 7-9. If a temporary lining is to be used during construction, vegetative retardance Class E should be used for the design stability calculations.

If the channel slope exceeds 10%, or a combination of channel linings will be used, additional procedures may be found in reference documents (USDOT, 1986 and USDOT, 1983).

The following guidelines are given for evaluating critical flow conditions of open channel flow:

(1) A normal depth of uniform flow within about 10% of critical depth is unstable and should be avoided in design, if possible.

Figure 7-16. Open Channel Geometric Relationships for Various Cross Sections
(Source: USDA, 1956)

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

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

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

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

(2) If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.

(3) If the velocity head is equal to one-half the mean depth of flow, the flow is critical.

(4) If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

Note: The head is the height of water above any point, plane or datum of reference. The velocity head in flowing water is calculated as the velocity squared divided by 2 times the gravitational constant ($V^2/2g$).

7.4.7.2 Design Stability

The steps listed below should be used for design stability calculations. Example design stability calculations are presented in Example 7-3.

(Step 1) Determine appropriate design variables, including discharge, Q, bottom slope, S, cross section parameters, and vegetation type.

(Step 2) Use Table 7-6 to assign a maximum velocity (v_m), based on vegetation type and slope range.

(Step 3) Assume a value of n and determine the corresponding value of vR from the n versus vR curves in Figure 7-12. Use retardance Class D for permanent vegetation and Class E for temporary construction.

(Step 4) Calculate the hydraulic radius using the equation:

Equation 7-20

$$R = \frac{vR}{v_m}$$

where:

R = hydraulic radius of flow (ft)
vR = value obtained from Figure 7-12 in Step 3
v_m = maximum velocity from Step 2 (ft/s)

(Step 5) Use the following form of Manning's Equation to calculate the value of vR:

Equation 7-21

$$vR = \frac{\left(1.49 R^{5/3} S^{1/2}\right)}{n}$$

where:

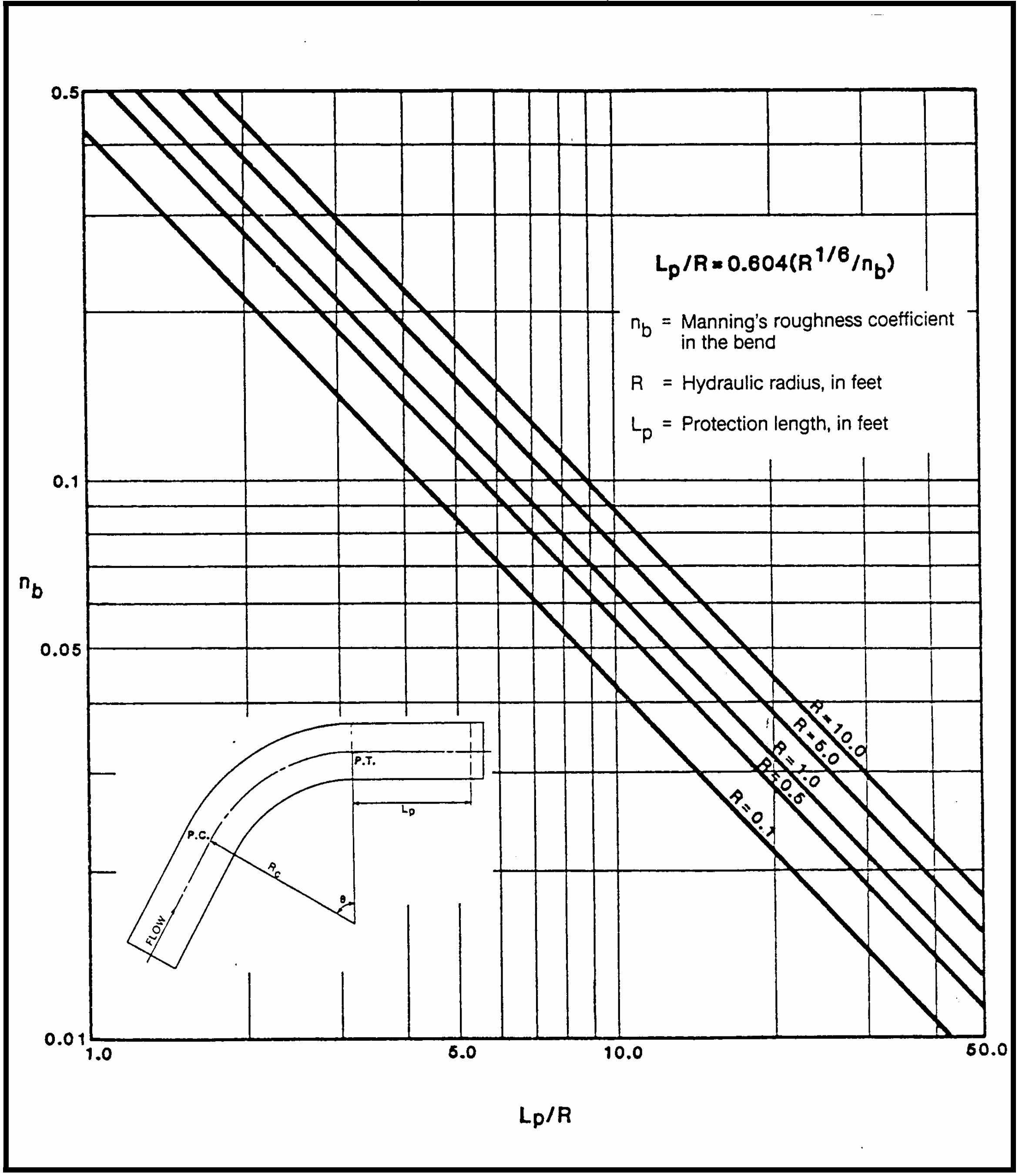
vR = calculated value of vR product
R = hydraulic radius value from Step 4 (ft)
S = channel bottom slope (ft/ft)
n = Manning's "n" value assumed in Step 3

(Step 6) Compare the vR product value obtained in Step 5 to the value obtained from Figure 7-12 for the assumed n value in Step 3. If the values are not reasonably close, return to Step 3 and repeat the calculations using a new assumed n value.

(Step 7) For trapezoidal channels, find the flow depth using Figures 7-14 or 7-15, as described in Section 7.4.6. The depth of flow for other channel shapes can be evaluated using the trial and error procedure described in Section 7.4.5.5.

(Step 8) If bends are considered, calculate the length of downstream protection, L_p, for the bend, using Figure 7-17. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

Figure 7-17. Protection Length, Lp, Downstream of Channel Bend
(Source: USDOT, 1986)



7.4.7.3 Design Capacity

The following steps should be used for design capacity calculations. Example design capacity calculations are presented in Example 7-3.

(Step 1) Assume a depth of flow greater than the value from Step 7 above and compute the waterway area and hydraulic radius (see Figure 7-16 for equations).

(Step 2) Divide the design flow rate, obtained using appropriate procedures from Chapter 2, by the waterway area from Step 1 to find the velocity.

(Step 3) Multiply the velocity from Step 2 by the hydraulic radius from Step 1 to find the value of vR.

(Step 4) Use Figure 7-12 to find a Manning's "n" value for retardance Class C based on the vR value from Step 3.

(Step 5) Use Manning's Equation (Equation 7-12) or Figure 7-13 to find the velocity using the hydraulic radius from Step 1, Manning's "n" value from Step 4, and appropriate bottom slope.

(Step 6) Compare the velocity values from Steps 2 and 5. If the values are not reasonably close, return to Step 1 and repeat the calculations.

(Step 7) Add one foot of freeboard to the final depth from Step 6.

(Step 8) If bends are considered, calculate super-elevation of the water surface profile at the bend using Equation 7-22.

Equation 7-22

$$\Delta d = \frac{v^2 T}{g R_c}$$

where:

Δd = super-elevation of the water surface profile due to the bend (ft)
v = average velocity from Step 6 (ft/s)
T = top width of flow (ft)
g = acceleration of gravity (32.2 ft/sec^2)
R_c = mean radius of the bend (ft)

Example 7-3. Uniform Flow Example Problem - Grassed Channel Design Stability

A trapezoidal channel (grass mixture with a bottom slope less than 5%) is required to carry 50 cfs at a bottom slope of 0.015 ft/ft. Find the channel dimensions required for design stability criteria (retardance Class D) for a grass mixture.

Step 1 Use design variables given: Q = 50cfs, slope 0.015 ft/ft, class D grass mixture.channel bottom.

Step 2. From Table 7-6, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5% is 4 ft/s.

Step 3. Assume an n value of 0.035 and find the value of vR from Figure 7-12, vR = 5.4

Step 4. Use Equation 7-20 to calculate the value of R: R = 5.4/4 = 1.35 ft

Step 5. Use Equation 7-21 to calculate the value of vR:

$vR = \{1.49(R)^{5/3}(S)^{1/2}\}/n = \{1.49(1.35)^{5/3}(0.015)^{1/2}\}/0.035 = 8.60\ ft^2/s$

Step 6. Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

Assumed n Value	vR (Figure 7-12)	R (Eq. 7-20)	vR (Eq. 7-21)
0.035	5.40	1.35	8.60
0.038	3.80	0.95	4.41
0.039	3.40	0.85	3.57
0.040	3.20	0.80	3.15

Select n = 0.040 for stability criteria.

Step 7. Use Figure 7-14 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

Qn = (50)(0.040) = 2.0 S = 0.015
For b = b = 8 ft, d = (8)(0.14) = 1.12 ft
try b=10 ft, d = (10)(0.098) = 0.98 ft
Select: b = 10 ft, such that R is approximately 0.80 ft
z = 3
d = 1 ft
v = 3.9 ft/s (Equation 7-12) = $[1.49(0.80)^{0.67}(0.015)^{0.5}]/0.40 = 3.9$
Fr = 0.76 (Equation 7-19) = $3.9/[(32.2)(13)/16]^{0.5} = 0.76$
Flow is subcritical

Design capacity calculations for this channel are presented in Example 7-4 below.

Example 7-4. Uniform Flow Example Problem - Grassed Channel Design Capacity

Use a 10-ft bottom width and 3:1 side-slopes for the trapezoidal channel sized in Example 7-3 and find the depth of flow for retardance Class C.

Step 1. Assume a depth of 1.0 ft and calculate the following (see Figure 7-16):

$A = \{b + zd\}d = \{10 + (3)(1)\}(1) = 13\ ft^2$
$R = \{b + zd\}d/b + \{2d(1 + z^2)^{0.5}\} = \{10 + (3)(1)(1)/10 + \{(2)(1)(1 + 3^2)^{0.5}\} = 0.796\ ft$

Step 2. Find the velocity:

v = Q/A = 3.85 ft/s

Step 3. Find the value of vR:

vR = (3.85)(0.796) = 3.06 ft^2/s

Step 4. Using the vR product from Step 3, find Manning's "n" from Figure 7-12 for retardance Class C (n = 0.047)

Step 5. Use Figure 7-13 or Equation 7-12 to find the velocity for S = 0.015, R = 0.796, and n = 0.047:

v = 3.34 ft/s

Step 6. Since 3.34 ft/s is less than 3.85 ft/s, a higher depth is required and calculations are repeated. Results from each trial of calculations are presented below:

Assumed Depth, d (ft)	Area (ft^2)	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's "n" (Fig. 7-15)	Velocity (Eq. 7-22)
1.00	13.00	0.796	3.85	3.06	0.047	3.34
1.05	13.81	0.830	3.62	3.00	0.047	3.39
1.10	14.63	0.863	3.42	2.95	0.048	3.45
1.20	16.32	0.928	3.06	2.84	0.049	3.54

Step 7. Select a depth of 1.1 with an n value of 0.048 for design capacity requirements. Add 1.0 ft for freeboard to give a design depth of 2.1 ft. Design data for the trapezoidal channel are summarized as follows:

Vegetation lining = grass mixture, v_m = 4 ft/s
Q = 50 cfs
b = 10 ft, d = 2.1 ft, z = 3, S = 0.015

Top Width = 10 +(2)(3)(2.1) = 22.6 ft

n (stability) = 0.040, d = 1.0 ft, v = 3.9 ft/s, Froude number = 0.76 (Equation 7-19)
n (capacity) = 0.048, d = 2.1 ft, v = 3.45 ft/s, Froude number = 0.60 (Equation 7-19)

7.4.8 Riprap Design for Lining Open Channels

7.4.8.1 Assumptions

The following procedure is based on results and analysis of laboratory and field data (Maynord, 1987; Reese, 1984; Reese, 1988). This procedure applies to riprap placement in both natural and prismatic channels and has the following assumptions and limitations:

- Minimum riprap thickness equal to d_{100}
- The value of d_{85}/d_{15} less than 4.6
- Froude number less than 1.2
- Side slopes up to 2:1
- A safety factor of 1.2
- Maximum velocity less than 18 feet per second

This procedure is not applicable if significant turbulence is caused by boundary irregularities, such as installations near obstructions or structures.

7.4.8.2 Procedure

Following are the steps in the procedure for riprap design:

(Step 1) Determine the average velocity in the main channel for the design condition in accordance with procedures given previously. Manning's "n" values for riprap can be calculated from the equation:

Equation 7-23

$$n = 0.0395(d_{50})^{1/6}$$

where:

n = Manning's "n" (roughness coefficient) for stone riprap
d_{50} = diameter of stone for which 50%, by weight, of the gradation is finer (ft)

(Step 2) If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bend correction coefficient, C_b, given in Figure 7-18 for either a natural or prismatic channel. This requires determining the channel top width, T, just upstream from the bend and the centerline bend radius, R_b.

(Step 3) If the specific weight of the stone varies significantly from 165 pounds per cubic foot, multiply the velocity from Step 1 or 2 (as appropriate) by the specific weight correction coefficient, C_g, from Figure 7-19.

(Step 4) Determine the required minimum d_{30} value from Figure 7-20, or from the equation:

Equation 7-24

$$\frac{d_{30}}{D} = 0.193 Fr^{2.5}$$

where:

d_{30} = diameter of stone for which 30%, by weight, of the gradation is finer (ft)
D = depth of flow above stone (ft)
Fr = Froude number (see Equation 7-19), dimensionless

(Step 5) Determine available riprap gradations. A well graded riprap is preferable to uniform size or gap graded. The diameter of the largest stone, d_{100}, should not be more than 1.5 times the d_{50} size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines (below d_{15}) should be available to fill the voids in the larger rock sizes. The stone weight for a selected stone size can be calculated from the equation:

Equation 7-25

$$W = 0.5236 SW_S d^3$$

where:

W = stone weight (lbs)
SW_s = specific weight of stone (lbs/ft^3)
d = selected stone diameter (ft)

Filter fabric or a filter stone layer should be used to prevent turbulence or groundwater seepage from removing bank material through the stone or to serve as a foundation for unconsolidated material. Layer thickness should be increased by 50% for underwater placement or riprap.

(Step 6) If d_{85}/d_{15} is between 2.0 and 2.3 and a smaller d_{30} size is desired, a thickness greater than d_{100} can be used to offset the smaller d_{30} size. Figure 7-21 can be used to make an approximate adjustment using the ratio of d_{30} sizes. Enter the y-axis with the ratio of the desired d_{30} size to the standard d_{30} size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.

(Step 7) Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and provide toe protection to avoid riprap undermining.

A riprap design calculation for purposes of channel lining is provided in Example 7-5.

Figure 7-18. Riprap Lining Bend Correction Coefficient

(Source: Maynord, 1987)

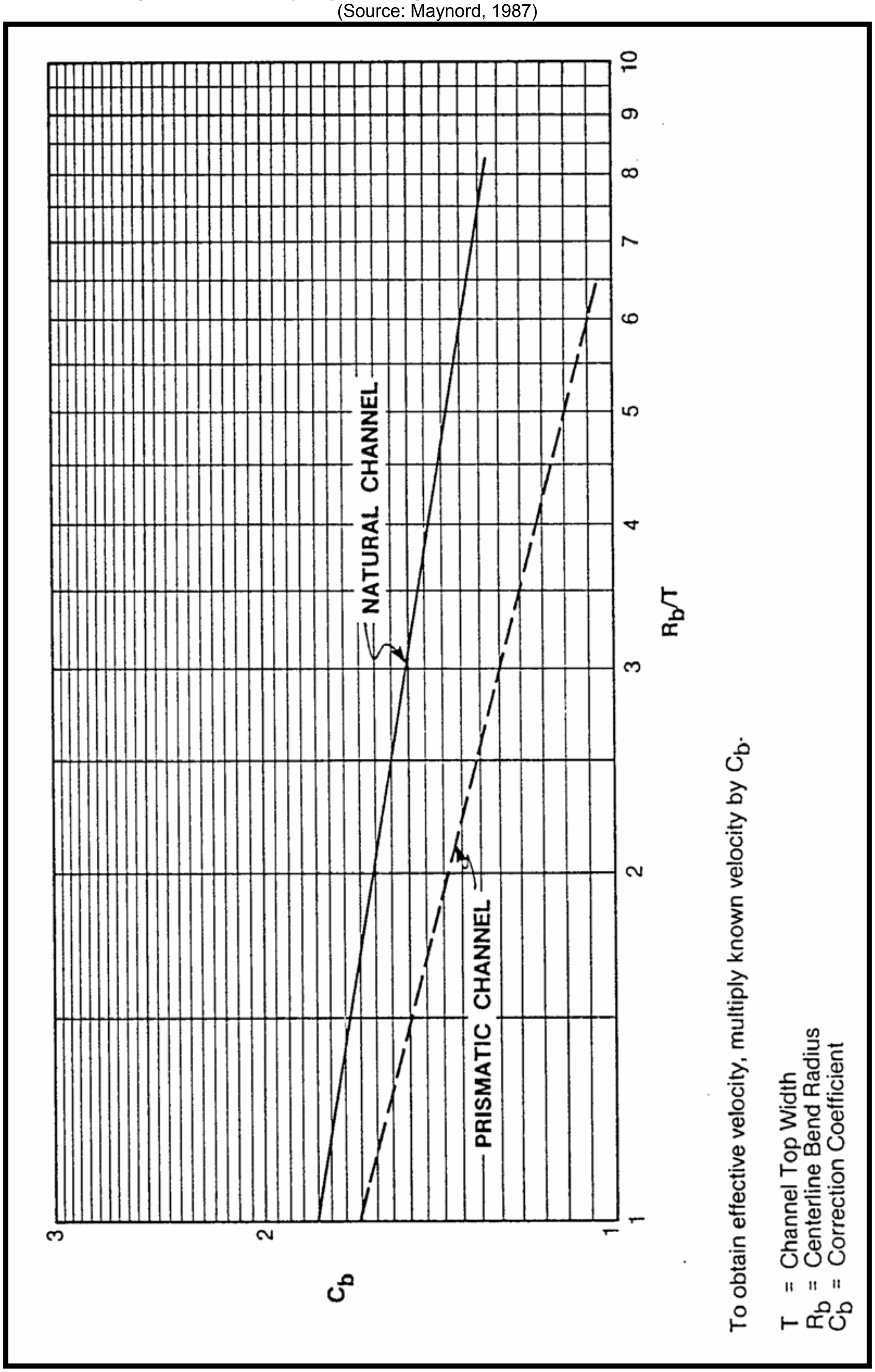


Figure 7-19. Riprap Lining Specific Weight Correction Coefficient

(Source: CDM / PBS&J, 1999)

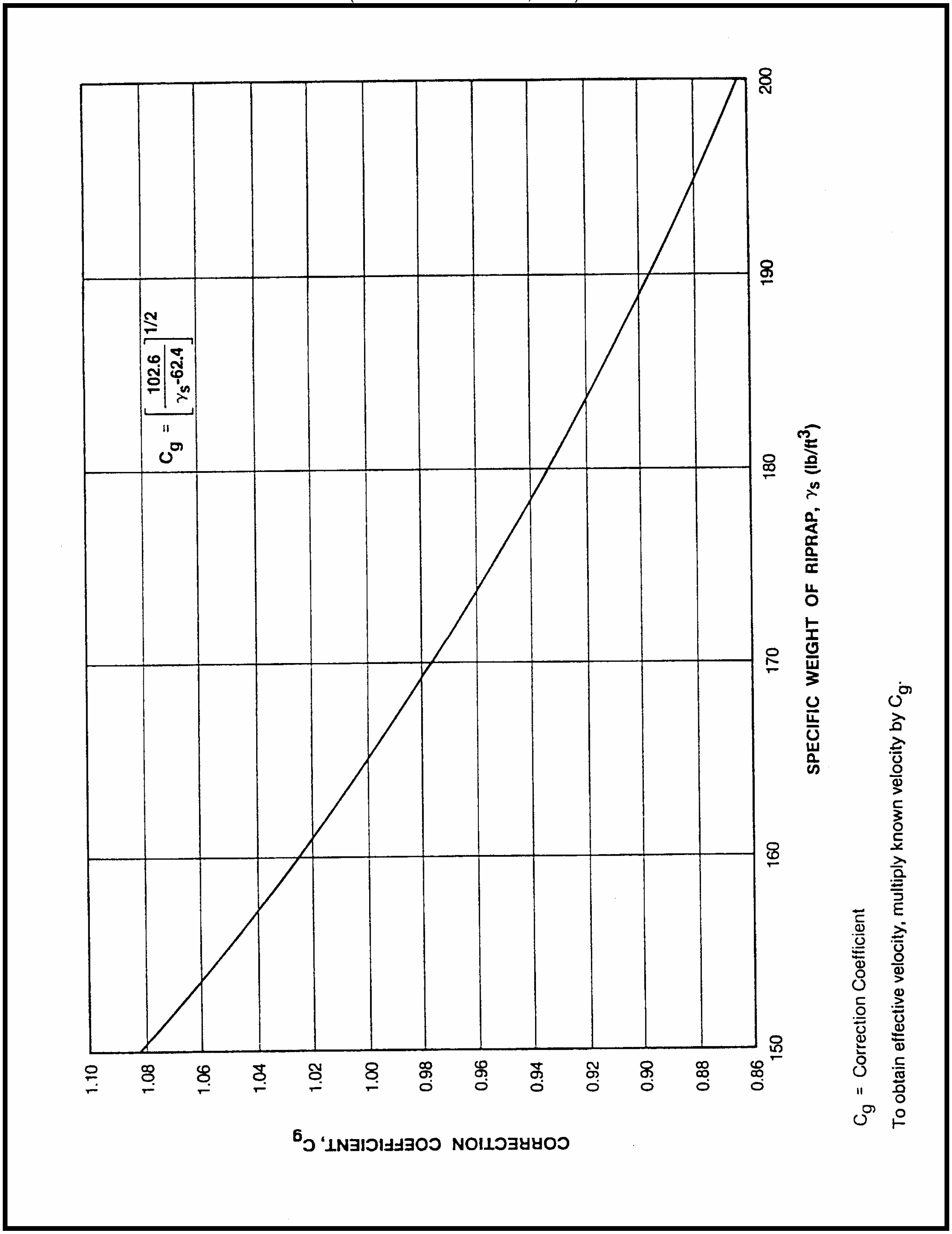
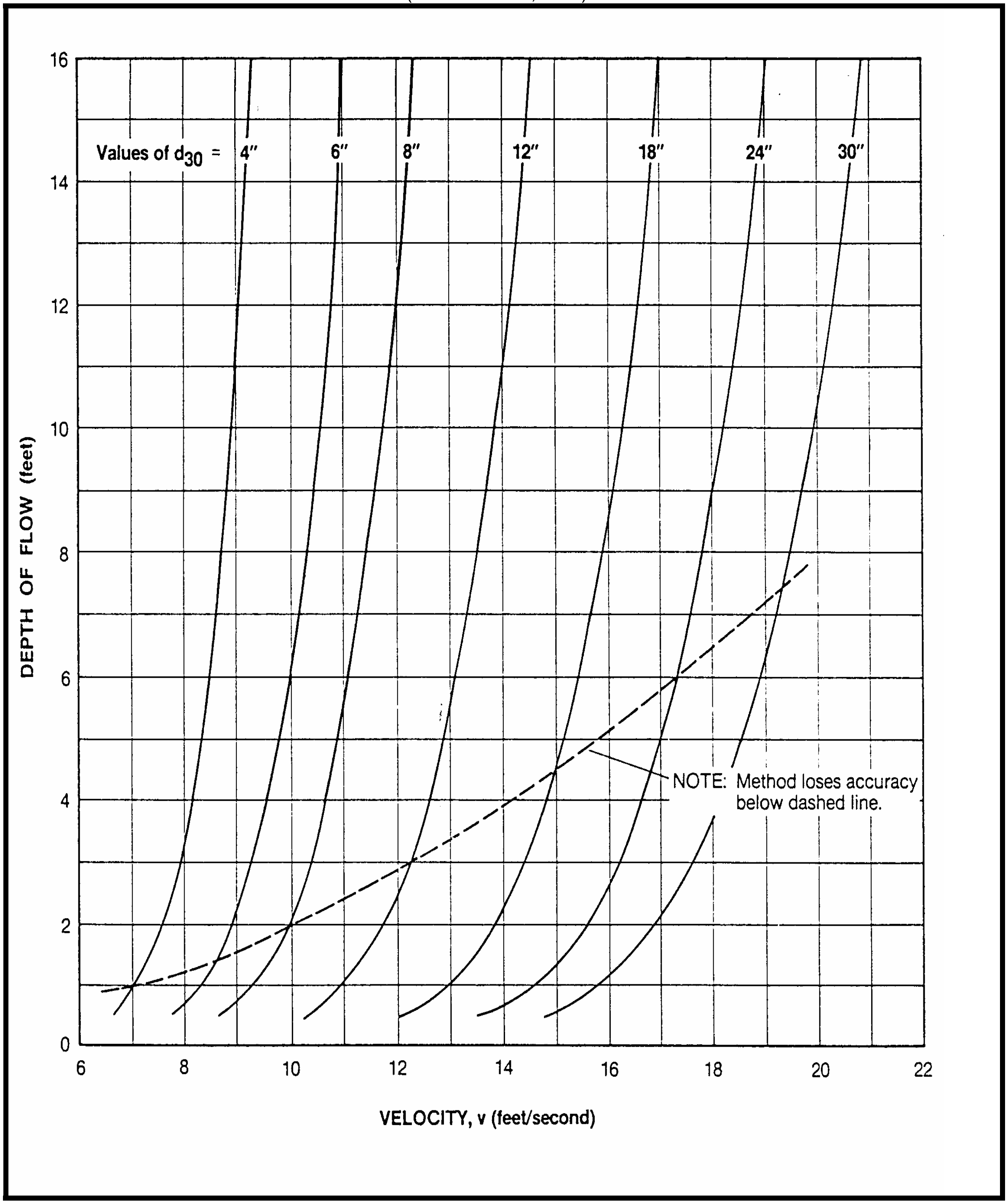


Figure 7-20. Riprap Lining d_{30} Stone Size – Function of Mean Velocity and Depth

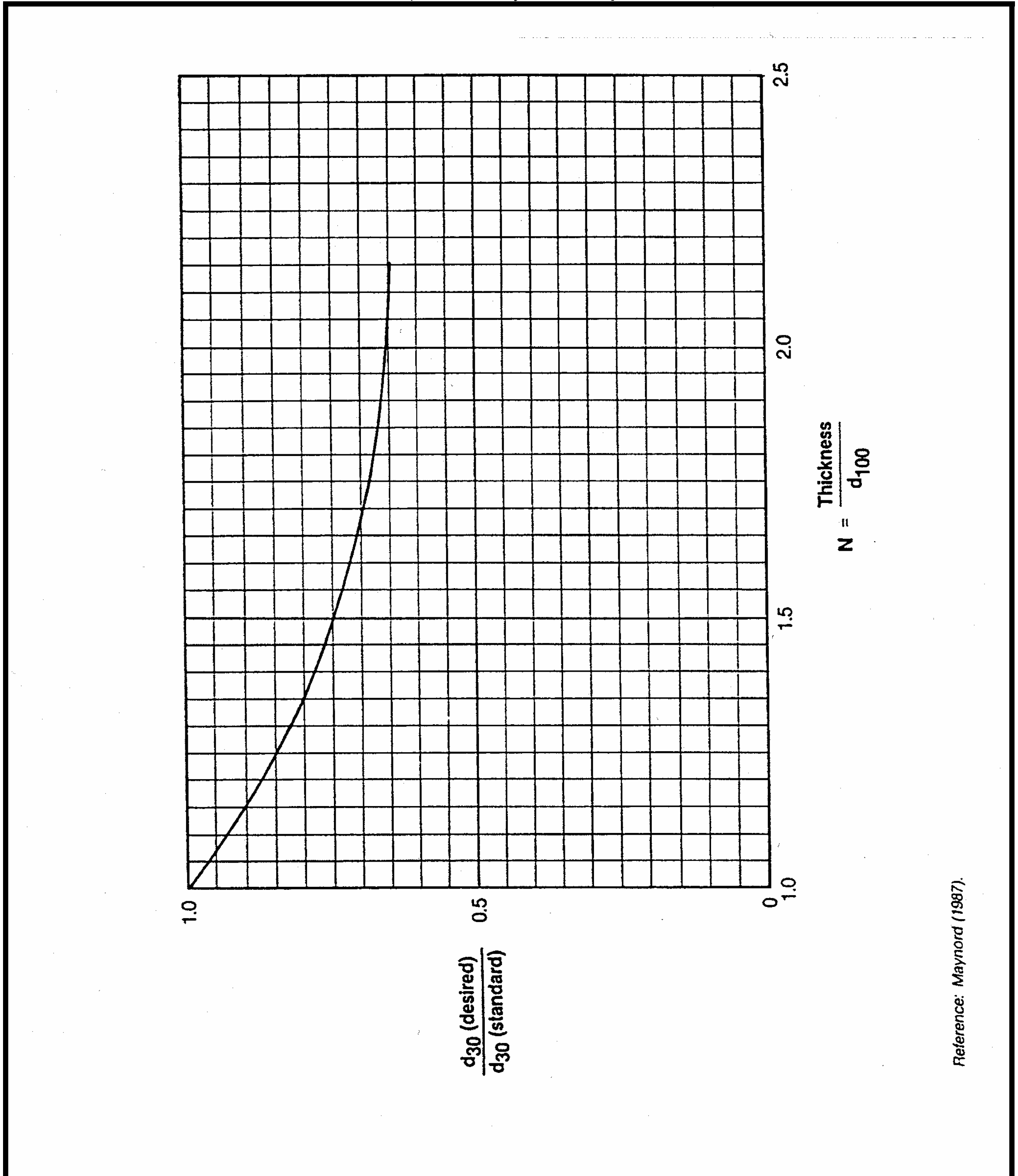
(Source: Reese, 1988)



Example 7-5. Riprap Design for Channel Lining

A natural channel has an average bankfull channel velocity of 8 ft per second with a top width of 20 ft and a bend radius of 50 ft. The depth over the toe of the outer bank is 5 ft. Available stone weight is 170 lbs/ft^3. Stone placement is on a side slope of 2:1 (horizontal:vertical). Determine riprap size at the outside of the bend.

Figure 7-21. Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 1.0 to 2.3
(Source: Maynord, 1987)



Step 1. Use 8 ft/s as the design velocity, because the reach is short and the bend is not protected.

Step 2. Determine the bend correction coefficient for the ratio

Rb/T = 50/20 = 2.5

From Figure 7-18, C_b = 1.55. The adjusted effective velocity is (8) (1.55) = 12.4 ft/s.

Step 3. Determine the correction coefficient for the specific weight of 170 lbs from Figure 7-19 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 ft/s.

Step 4. *(Optional)* Another gradation is available with a d_{30} of 8 inches. The ratio of desired to standard stone size is 8/10 or 0.8. From Figure 7-21, this gradation would be acceptable if the blanket thickness was increased from the original d_{100} (diameter of the largest stone) thickness by 35% (a ratio of 1.35 on the horizontal axis).

Step 5. Perform preliminary design. Make sure that the stone is carried up and downstream far enough to ensure stability of the channel and that the toe will not be undermined. The downstream length of protection for channel bends can be determined using Figure 7-17.

7.4.9 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm drain system inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used programs are, HEC-RAS, developed by the U.S. Army Corps of Engineers (USACE) and Bridge Waterways Analysis Model (WSPRO) developed for FHWA. These programs can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the direct step method. For an irregular non-uniform channel, the standard step method is recommended, although it is a more tedious and iterative process. The use of HEC-RAS is recommended for standard step calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity of the stream and floodplain. In general, a cross section should be obtained at each location where there are significant changes in stream width, shape, or vegetal patterns. Sections should usually be no more than 4 to 5 channel widths apart or 100 feet apart for ditches or streams and 500 feet apart for floodplains, unless the channel is very regular.

7.4.10 Rectangular, Triangular, and Trapezoidal Open Channel Design

FHWA has prepared numerous design figures and nomographs (herein referred to as the FHWA nomographs) to aid in the design of open channels. The FHWA nomographs are not included in this manual, but can be purchased from FHWA's website, http://www.fhwa.dot.gov. The FHWA nomographs are used for the direct solution of the Manning's Equation for various sized open channels with rectangular, triangular, and trapezoidal cross sections. Each nomograph (except for the triangular cross section) is prepared for a channel of given bottom width and a particular value of Manning's "n". Example nomographs, provided for purposes of example problems in this manual, are provided in Figures 7-22, 7-23 and 7-24. An example nomograph for a triangular channel is presented in Figure 7-25.

For design conditions not covered by the FHWA nomographs, a trial and error solution of Manning's Equation must be used.

7.4.10.1 Instructions for Rectangular and Trapezoidal Figures

The FHWA nomographs provide a solution of the Manning Equation for flow in open channels of uniform slope, cross section, and roughness, provided the flow is not affected by backwater and the channel has a length sufficient to establish uniform flow.

Figure 7-22. Example Nomograph #1
(Source: http://www.fhwa.dot.gov)

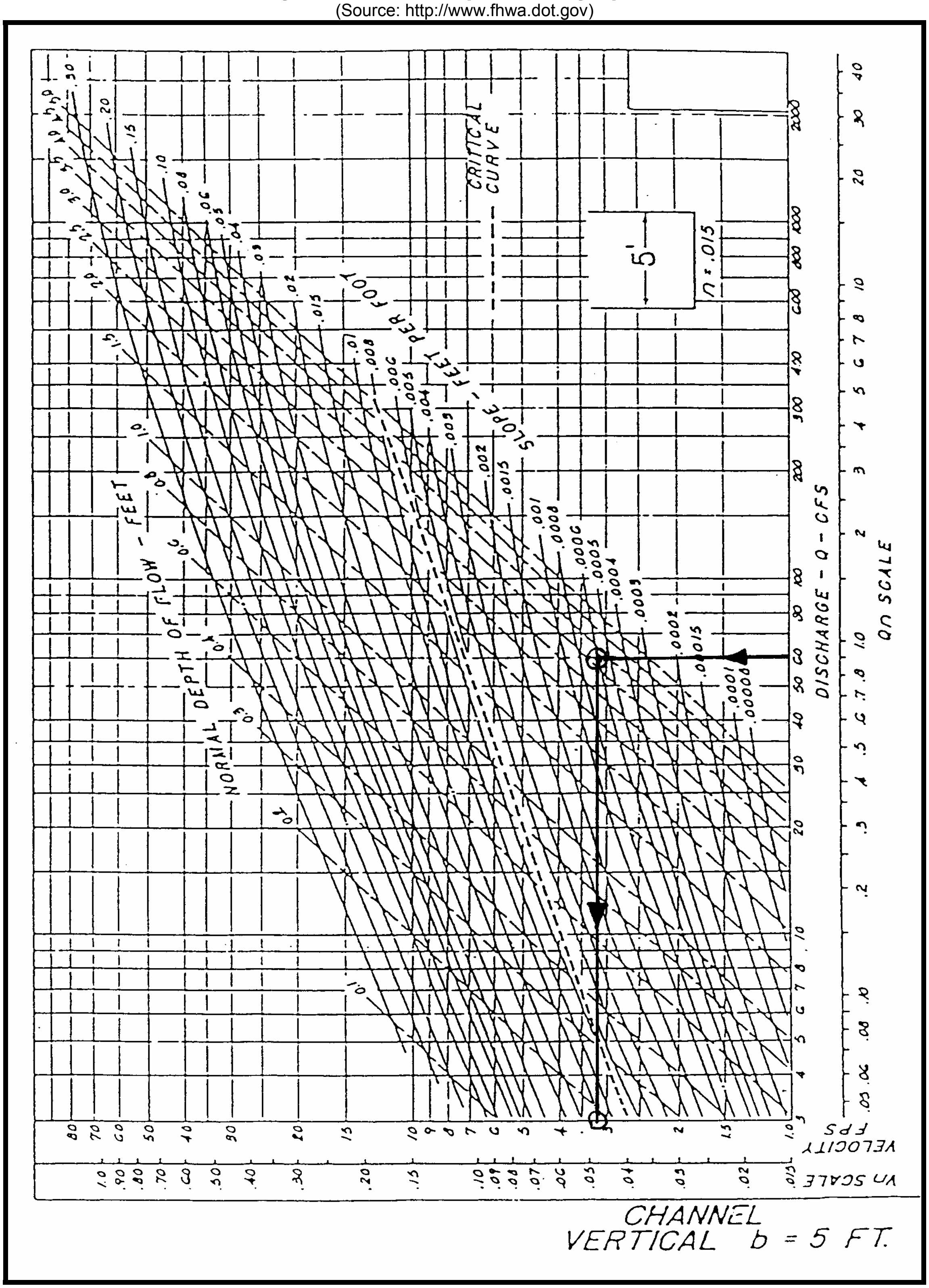


Figure 7-23. Example Nomograph #2

(Source: http://www.fhwa.dot.gov)

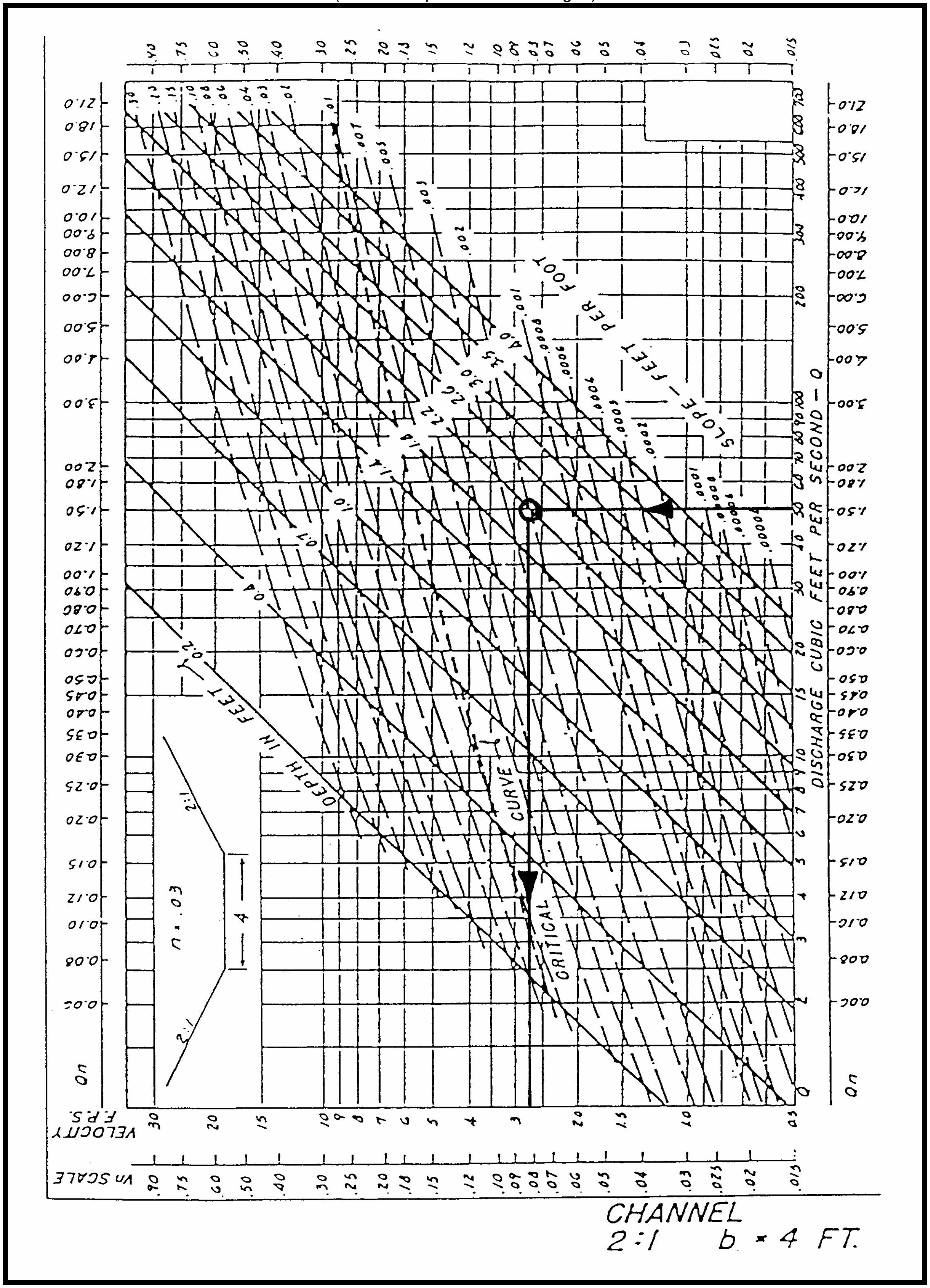


Figure 7-24. Example Nomograph #3

(Source: http://www.fhwa.dot.gov)

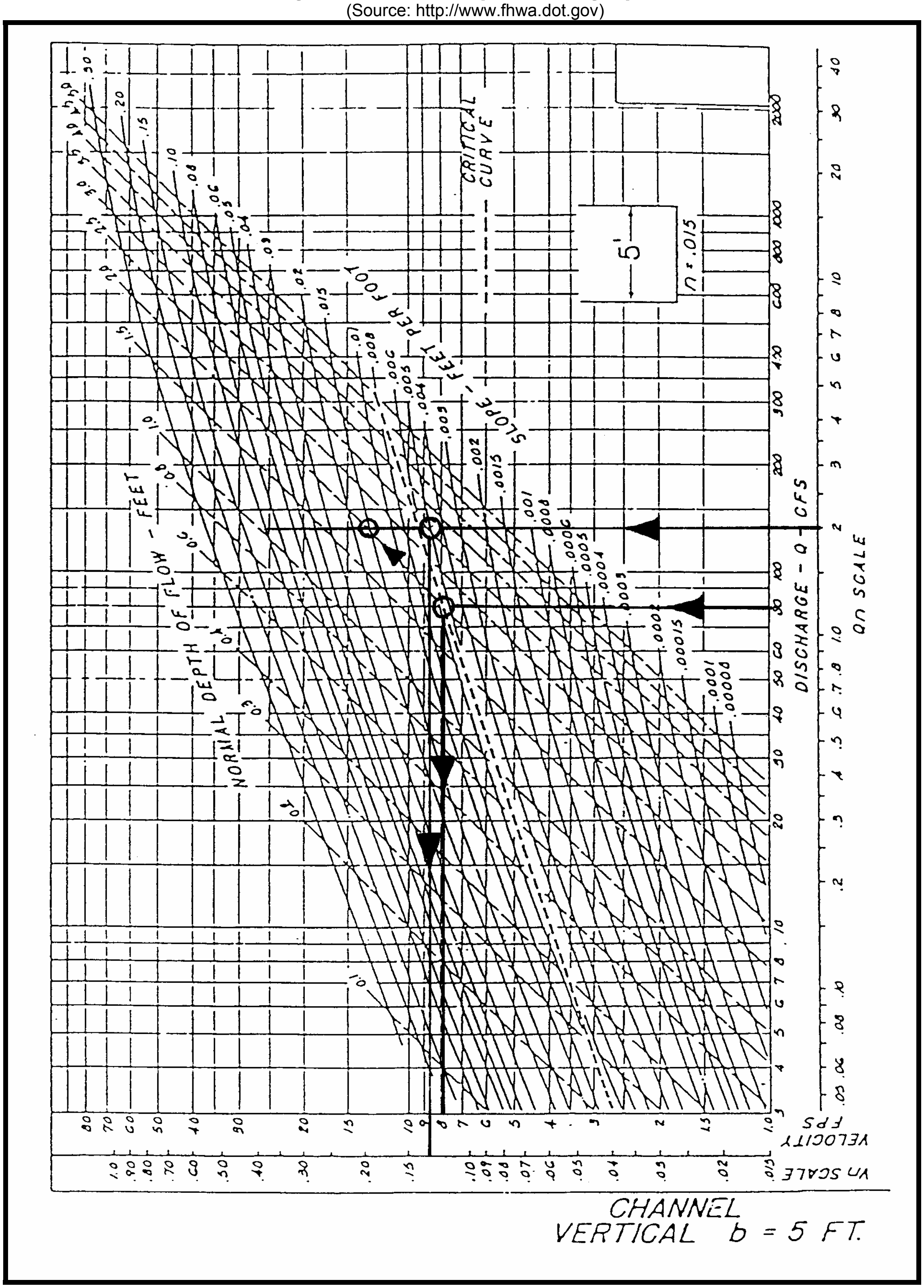
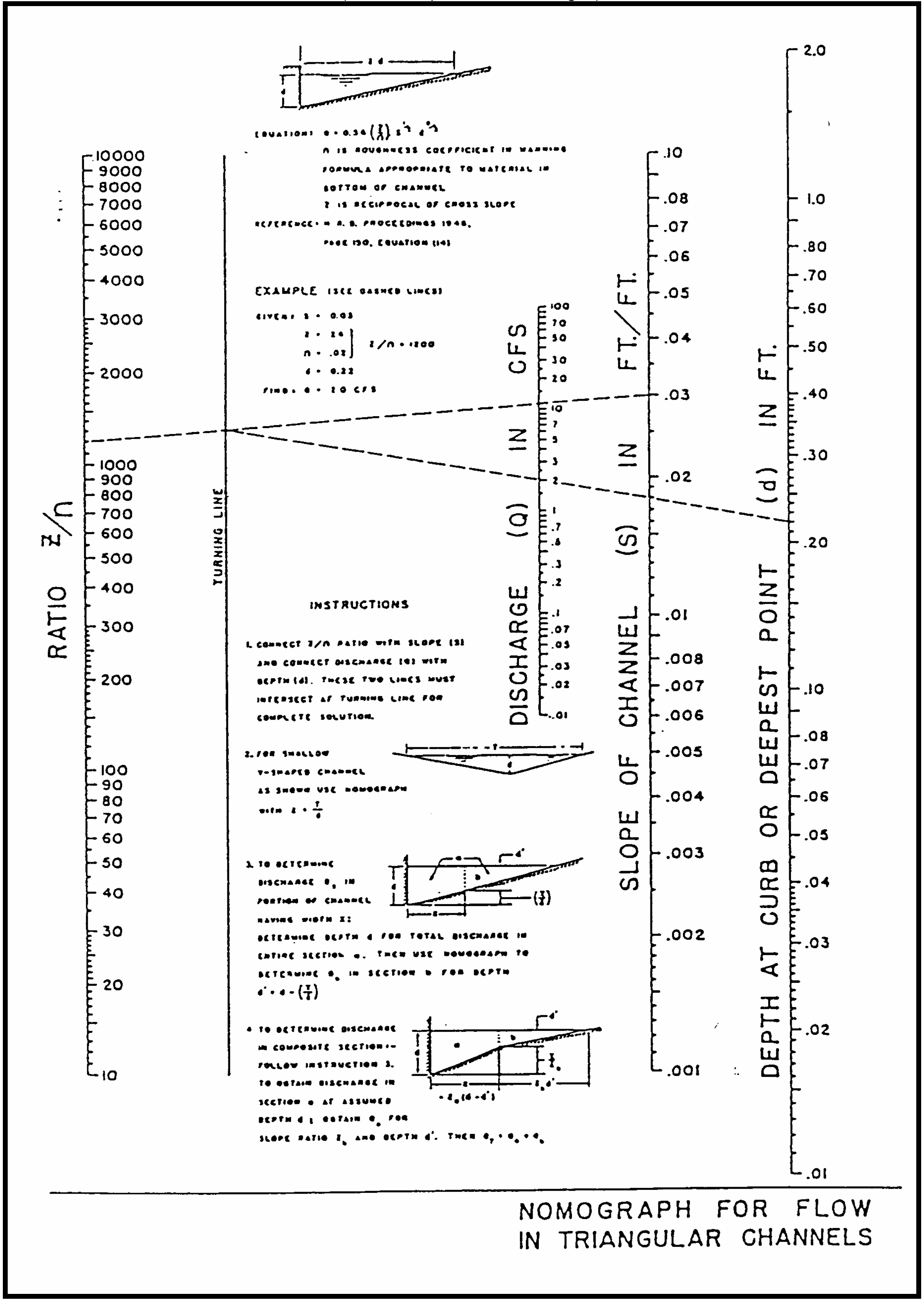


Figure 7-25. Triangular Channel Nomograph
(Source: http://www.fhwa.dot.gov)



For a given slope and channel cross section, when n is 0.015 for rectangular channels or 0.03 for trapezoidal channels, the depth and velocity of uniform flow may be read directly from the nomograph for that size channel. The initial step is to locate the intersection of a vertical line through the discharge (abscissa) and the appropriate slope line. At this intersection, the depth of flow is read from the depth lines, and the mean velocity is read on the ordinate scale.

The procedure is reversed to determine the discharge at a given depth of flow. Critical depth, slope, and velocity for a given discharge can be read on the appropriate scale at the intersection of the critical curve and a vertical line through the discharge.

Auxiliary scales, labeled Qn (abscissa) and Vn (ordinate), are provided so the FHWA nomographs can be used for values of n other than those for which the charts were basically prepared. To use these scales, multiply the discharge by the value of n and use the Qn and Vn scales instead of the Q and V scales, except for computation of critical depth or critical velocity. To obtain normal velocity, V, from a value on the Vn scale, divide the value by n. The following examples will illustrate these points.

Example 7-6. Rectangular Channel Design.

Given: A rectangular concrete channel 5 ft wide with n = 0.015, 0.06 percent slope (S = .0006), discharging 60 cfs.

Find: Depth, velocity, and type of flow

Procedure:

Step 1. From the appropriate FHWA nomograph (Figure 7-22), select the rectangular figure for a 5-ft width.

Step 2. From 60 cfs on the Q scale, move vertically to intersect the slope line S = 0.0006, and from the depth lines read d_n = 3.7 ft.

Step 3. Move horizontally from the same intersection and read the normal velocity, V = 3.2 ft/s, on the ordinate scale.

Step 4. The intersection lies below the critical curve, and the flow is therefore in the subcritical range.

Example 7-7. Trapezoidal Channel Design

Given: A trapezoidal channel with 2:1 side slopes and a 4 ft bottom width, with n = 0.030, 0.2% slope (S = 0.002), discharging 50 cfs.

Find: Depth, velocity, type flow.

Procedure:

Step 1. From the appropriate FHWA nomograph (Figure 7-23), select the trapezoidal figure for b = 4 ft.

Step 2. From 50 cfs on the Q scale, move vertically to intersect the slope line S = 0.002 and from the depth lines read d_n = 2.2 ft.

Step 3. Move horizontally from the same intersection and read the normal velocity, V = 2.75 ft/s, on the ordinate scale. The intersection lies below the critical curve; the flow is therefore subcritical.

Example 7-8. Rectangular Channel Design

Example 7-8 presents a different calculation approach for rectangular channel design than that presented in Example 7-6.

Given: A rectangular cement rubble masonry channel 5 ft wide, with n = 0.025, 0.5% slope (S = 0.005), discharging 80 cfs.

Find: Depth velocity and type of flow

Procedure:

Step 1. From the appropriate FHWA nomograph (Figure 7-24) select the rectangular figure for a 5 ft width.

Step 2. Multiply Q by n to obtain Qn:

$Qn = (80)(0.025) = 2.0\ ft^3/s$

Step 3. From 2.0 on the Qn scale, move vertically to intersect the slope line, S = 0.005, and at the intersection read d_n = 3.1 ft.

Step 4. Move horizontally from the intersection and read Vn = 0.13, then

$V = Vn/n = 0.13/0.025\ ft/s = 5.2\ ft/s$

Step 5. Critical depth and critical velocity are independent of the value of n so their values can be read at the intersection of the critical curve with a vertical line through the discharge. For 80 cfs, on Figure 7-24, d_c = 2.0 ft and V_c = 7.9 ft/s. The normal velocity, 5.2 ft/s (from Step 4), is less than the critical velocity, and the flow is therefore subcritical. It will also be noted that the normal depth, 3.0 ft, is greater than the critical depth, 2.0 ft, which also indicates subcritical flow.

Step 6. To determine the critical slope for Q = 80 cfs and n = 0.025, start at the intersection of the critical curve and a vertical line through the discharge, Q = 80 cfs, finding d_c (2.0 ft) at this point. Follow along this d_c line to its intersection with a vertical line through Qn = 2.0 (Step 2), at this intersection read the slope value S_c = 0.015.

7.4.10.2 Grassed Channel Figures

The Manning Equation can be used to determine the capacity of a grass-lined channel, but the Mannings “n” value varies with the type of grass, development of the grass cover, depth, and velocity of flow. The variability of Manning’s “n” complicates the solution of the Manning Equation. The depth and velocity of flow must be estimated and the Manning Equation solved using the “n” value that corresponds to the estimated depth and velocity. The trial solution provides better estimates of the depth and velocity for a new value of “n” and the equation is again solved. The procedure is repeated until a depth is found that carries the design discharge.

To prevent excessive erosion, the velocity of flow in a grass-lined channel must be kept below some maximum value (referred to as permissible velocity). The permissible velocity in a grass-lined channel depends upon the type of grass, condition of the grass cover, texture of the soil comprising the channel bed, channel slope, and to some extent the size and shape of the drainage channel. To guard against overtopping, the channel capacity should be computed for taller grass than is expected to be maintained, while the velocity used to check the adequacy of the protection should be computed assuming a lower grass height than will likely be maintained.

FHWA nomographs have also been prepared to aid in the design of grass channels. These can be found at the FHWA website http://www.fhwa.dot.gov. Following is a brief description of general design standards, instructions on how to use the figures, and several example design problems. For design conditions not covered by the figures, a trial-and-error solution of the Manning Equation must be used.

Example FHWA nomographs for grassed channels are presented in Figures 7-26 and 7-27. The FHWA nomographs for grass channels are designed for use in the direct solution of the Manning Equation for various channel sections lined with grass. The nomographs are similar in appearance and use to the nomographs for trapezoidal cross sections described earlier. However, their construction is much more difficult because Mannings "n" changes as higher velocities and/or greater depths change the condition of the grass. The effect of velocity and depth of flow on "n" is evaluated by the product of velocity and hydraulic radius (v times R). The variation of Manning's "n" with the retardance (Table 7-9) and the product (v times R) is shown in Figure 7-12. As indicated in Table 7-9, retardance varies with the height of the grass and the condition of the stand. Both of these factors depend upon the type of grass, planting conditions, and maintenance practices. Table 7-9 is used to determine retardance classification.

The grassed channel nomographs shown in Figures 7-26 and 7-27 each have two graphs, the upper graph for retardance Class D and the lower graph for retardance Class C. The figures are plotted with discharge in cubic feet per second on the abscissa and slope in feet per foot on the ordinate. Both scales are logarithmic.

Superimposed on the logarithmic grid are lines for velocity in feet per second and lines for depth in feet. A dashed line shows the position of critical flow.

7.4.10.3 Instructions for Grassed Channel FHWA Nomographs

The FHWA grassed channel nomographs provide a solution of the Manning Equation for flow in open grassed channels of uniform slope and cross section. The flow should not be affected by backwater and the channel should have a length sufficient to establish uniform flow. The FHWA nomographs are sufficiently accurate for design of drainage channels of fairly uniform cross section and slope, but are not appropriate for irregular natural channels.

The design of grassed channels requires two operations: (1) selecting a section that has the capacity to carry the design discharge on the available slope and (2) checking the velocity in the channel to ensure that the grass lining will not be eroded. Because the retardance of the channel is largely beyond the control of the designer, it is good practice to compute the channel capacity using retardance Class C and the velocity using retardance Class D. The calculated velocity should then be checked against the permissible velocities listed in Tables 7-5 and 7-6. The use of the FHWA nomographs is explained in the following steps:

(Step 1) Select the channel cross section to be used and find the appropriate nomograph.

(Step 2) Enter the lower graph (for retardance Class C) on the nomograph with the design discharge value on the abscissa and move vertically to the value of the slope on the ordinate scale. As this intersection, read the normal velocity and normal depth and note the position of the critical curve. If the intersection point is below the critical curve, the flow is subcritical; if it is above, the flow is supercritical.

Figure 7-26. Example Nomograph #4
(Source: http://www.fhwa.dot.gov)

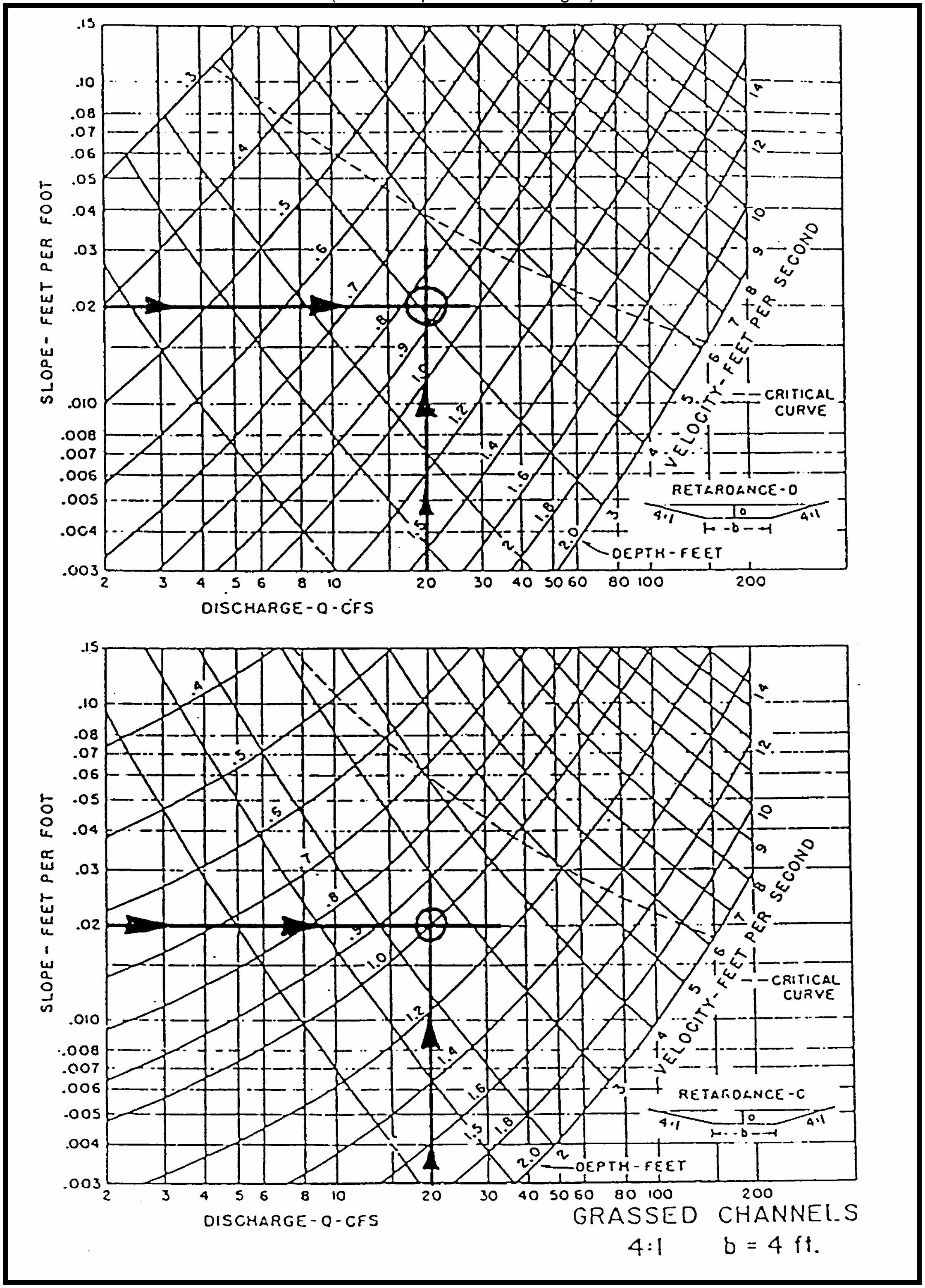
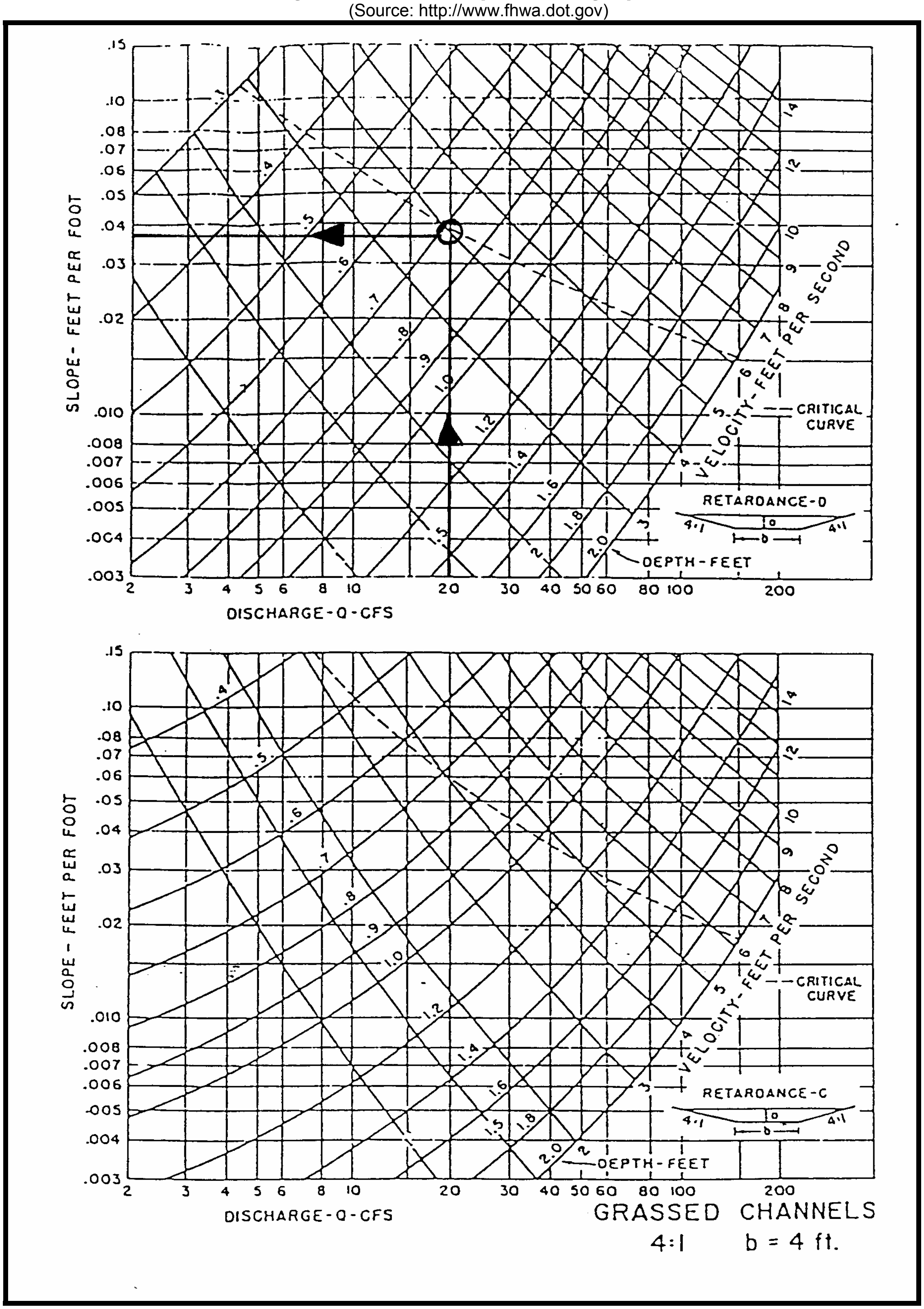


Figure 7-27. Example Nomograph #5
(Source: http://www.fhwa.dot.gov)



Step 3) To check the velocity developed against the permissible velocities (Tables 7-5 and 7-6), enter the upper graph on the same figure and repeat Step 2. Then compare the computed velocity with the velocity permissible for the type of grass, channel slope, and erosion resistance of the soil. If the computed velocity is less, the design is acceptable. If not, a different channel section must be selected and the process repeated.

Example 7-9. Vegetated Trapezoidal Channel Design

Given: A trapezoidal channel in easily eroded soil, lined with a grass mixture with 4:1 side slopes, and a 4 ft bottom width on slope of 0.02 ft per foot (S = 0.02), discharging 20 cfs.

Find: Depth, velocity, type of flow, and adequacy of grass to prevent erosion

Procedure:

Step 1. From the appropriate FHWA nomograph (Figure 7-26), select for 4:1 side slopes.

Step 2. Enter the lower graph with Q = 20 cfs, and move vertically to the line for S=0.02. At this intersection read d_n = 1.0 ft, and normal velocity V_n 2.6 ft/s.

Step 3. The velocity for checking the adequacy of the grass cover should be obtained from the upper graph, for retardance Class D. Using the same procedure as in step 2, the velocity is found to be 3.0 ft/s. This is about three-quarters of that listed as permissible, 4.0 ft/s in Table 7-6. Therefore the design is acceptable.

Example 7-10. Maximum Slope of Vegetated Trapezoidal Channel Design

Given: The channel and discharge of Example 7-9.

Find: The maximum grade on which the 20 cfs could safely be carried

Procedure:

With an increase in slope (but still less than 5%), the allowable velocity is estimated to be 4 ft/s (see Table 7-6). On the upper graph of Figure 7-27 for short grass, the intersection of the 20 cfs line and the 4 ft/s line indicates a slope of 3.7% and a depth of 0.73 ft.

7.5 Energy Dissipation Design

7.5.1 Overview

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

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

7.5.1.1 General Design Standards and Considerations

- Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.
- Energy dissipators shall be employed whenever the velocity of flows leaving a stormwater management facility exceeds the erosion velocity of the downstream area channel system.
- Energy dissipator designs will vary based on discharge specifics and tailwater conditions.
- Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

Key considerations for the design of energy dissipators are as follows.

- Energy dissipaters should be designed to return flows to non-erosive velocities to protect downstream channels.
- Care must be taken during construction that design criteria are followed exactly. The designs presented in this manual have been carefully developed through model and full-scale tests. Each part of the criteria is important to the proper function.

7.5.1.2 Recommended Energy Dissipators

For many designs, the following outlet protection devices and energy dissipators provide sufficient protection at a reasonable cost:

- level spreaders;
- riprap aprons;
- riprap outlet basins; and,
- baffled outlets.

This section focuses on the design of these measures. Refer to the reference Hydraulic Design of Energy Dissipators for Culverts and Channels (USDOT, 1983) for the design procedures of other energy dissipators.

7.5.2 Symbols and Definitions

To provide consistency within this section as well as throughout this manual, the symbols listed in Table 7-11 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

7.5.3 Design Guidelines

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

   a. Level spreaders are used mainly to disperse concentrated runoff uniformly over the ground as sheet flow, upstream of a vegetated buffer or filter strip. Knox County Engineering must approve the use of a level spreader for any other application.

Table 7-11. Energy Dissipation Design Symbols and Definitions

Symbol	Definition	Units
A	Cross-sectional area	ft^2
D	Height of box culvert	ft
d_{50}	Size of riprap	ft
d_w	Culvert width	ft
Fr	Froude Number	-
g	Acceleration of gravity	ft/s^2
h_s	Depth of dissipator pool	ft
L	Length	ft
L_a	Riprap apron length	ft
L_B	Overall length of basin	ft
L_s	Length of dissipator pool	ft
PI	Plasticity index	-
Q	Rate of discharge	cfs
Sv	Saturated shear strength	lbs/in^2
t	Time of scour	min.
tc	Critical tractive shear stress	lbs/in^2
TW	Tailwater depth	ft
V_L	Velocity L feet from brink	ft/s
V_o	Normal velocity at brink	ft/s
V_o	Outlet mean velocity	ft/s
V_s	Volume of dissipator pool	ft^3
W_o	Diameter or width of culvert	ft
W_s	Width of dissipator pool	ft
y_e	Hydraulic depth at brink	ft
y_o	Normal flow depth at brink	ft

b. Riprap outlet basins may also be used when the outlet Fr is less than or equal to 2.5. They are generally used for transitions from culverts to stable channels. Since riprap outlet basins function by creating a hydraulic jump to dissipate energy, performance is impacted by tailwater conditions.

c. Baffled outlets have been used with outlet velocities up to 50 feet per second. Practical application typically requires an outlet Fr between 1 and 9. Baffled outlets may be used at both terminal outlet and channel outlet transitions. They function by dissipating energy through impact and turbulence and are not significantly affected by tailwater conditions.

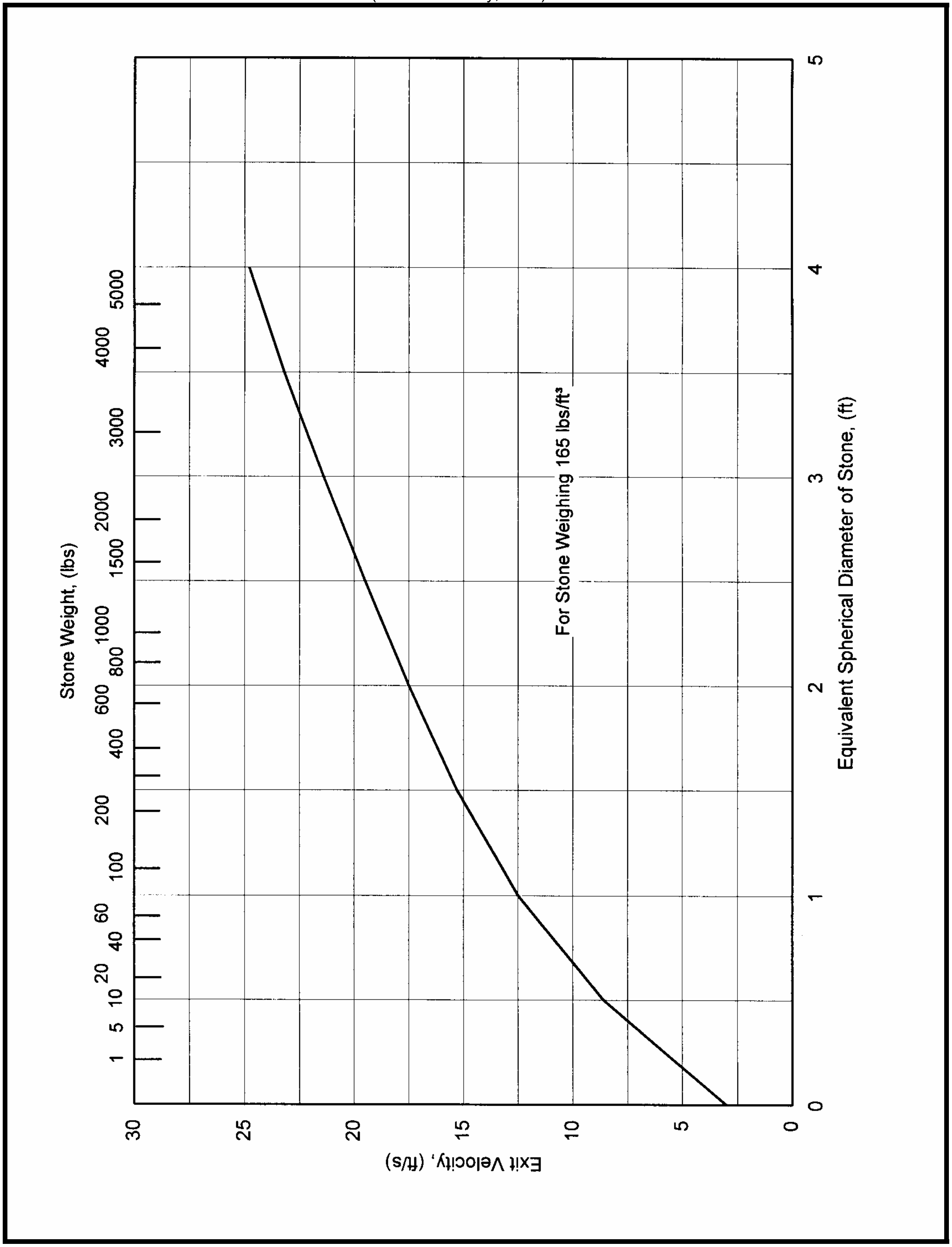
(2) When outlet protection facilities are selected, appropriate design flow conditions and site-specific factors affecting erosion and scour potential, construction cost, and long-term durability should be considered.

(3) If outlet protection is not provided, energy dissipation will occur through formation of a local scour hole. A cutoff wall will be needed at the discharge outlet to prevent structural undermining. The wall depth should be slightly greater than the computed scour hole depth (h_s). The scour hole should then be stabilized. If the scour hole is of such size that it will present maintenance, safety, or aesthetic problems, other outlet protection will be needed.

(4) Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur. Figure 7-28 provides the riprap size recommended for use downstream of energy dissipators.

(5) Energy dissipation in the form of a level spreader is required when a transition is needed between shallow concentrated or channel flow to sheet flow to protect a downstream structural BMP, such as a filter strip.

Figure 7-28. Riprap Size for Use Downstream of Energy Dissipator
(Source: Searcy, 1967)



7.5.4 Riprap Aprons

7.5.4.1 Description

A riprap-lined apron is a commonly used practice for energy dissipation because of its relatively low cost and ease of installation. A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. Riprap aprons are appropriate when the culvert outlet Froude number is less than or equal to 2.5.

7.5.4.2 Design Procedure

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

(Step 1) If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 7-29 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 7-30 should be used.

(Step 2) Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figures 7-29 and 7-30. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 7-31.

a. *For pipes flowing full:*
Use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, L_a, and median riprap diameter, d_{50}, from the appropriate curves.

b. *For pipes flowing partially full:*

Use the depth of flow, d, in feet, and velocity, v, in ft/s. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, L_a, from the scale on the left.

c. *For box culverts:*

Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth, d. Find the minimum apron length, L_a, using the scale on the left.

(Step 3) If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in Figure 7-31. This will provide protection under either of the tailwater conditions.

Figure 7-29. Design of Riprap Apron under Minimum Tailwater Conditions

(Source: USDA, 1975)

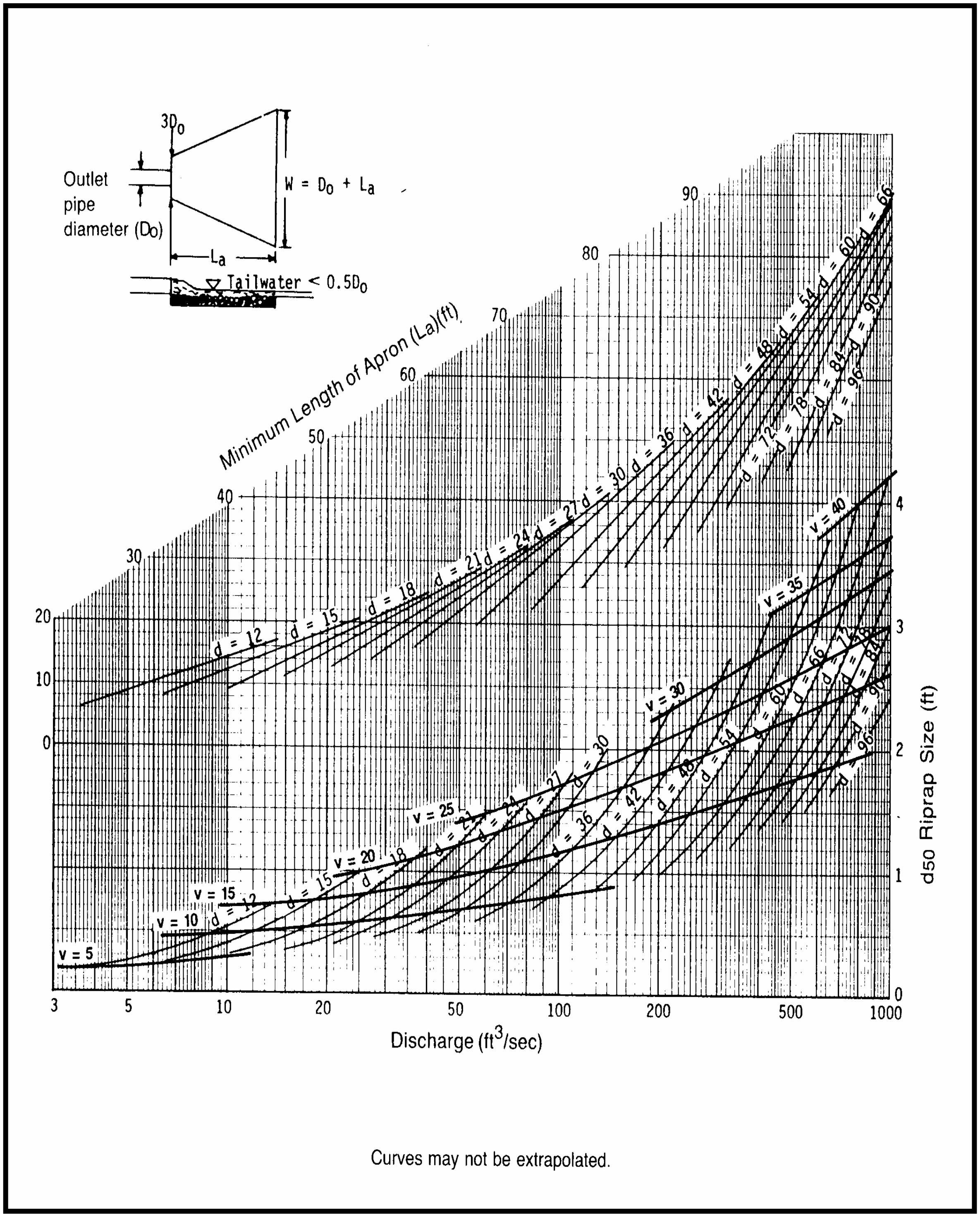


Figure 7-30. Design of Riprap Apron under Maximum Tailwater Conditions
(Source: USDA, 1975)

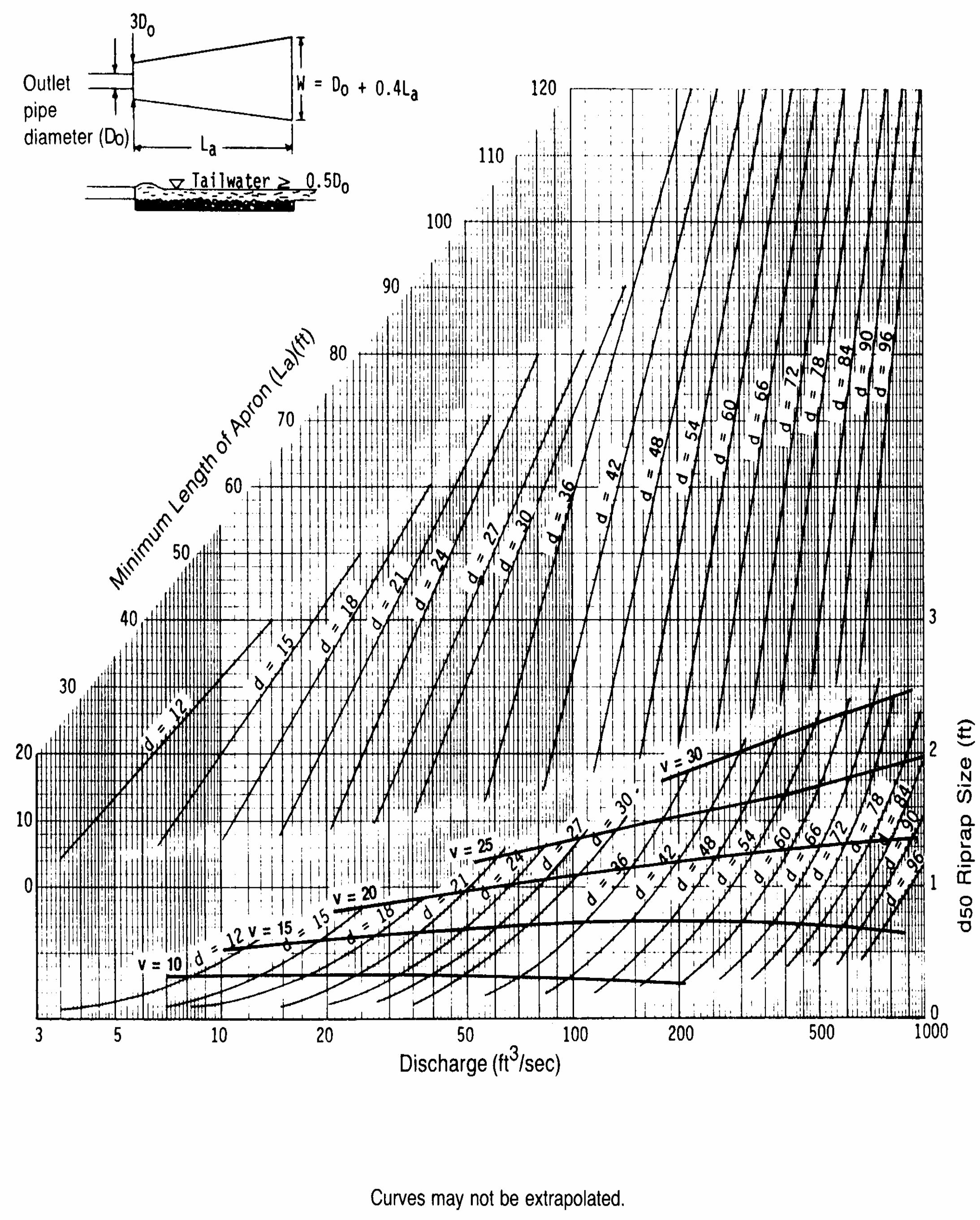
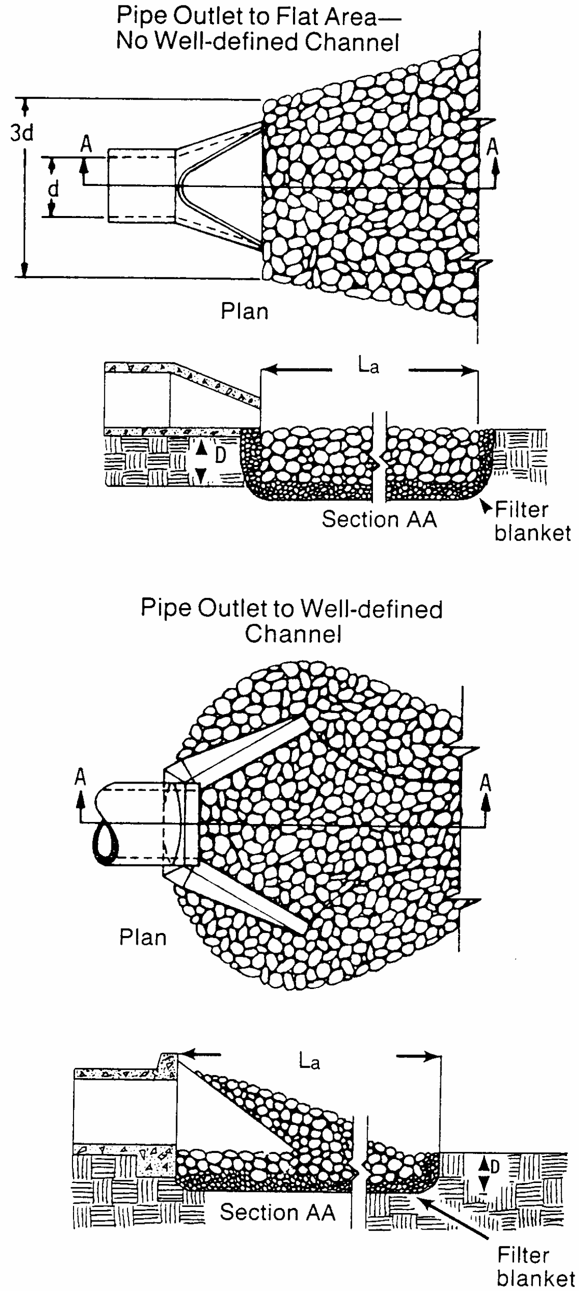


Figure 7-31. Riprap Apron
(Source: Atlanta Regional Council, 2001)



Notes

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

7.5.4.3 Design Considerations

The following items should be considered during riprap apron design:

- The maximum stone diameter (d_{max}) should be 1.5 times the median riprap diameter: d_{max} = 1.5 x d_{50}, where d_{50} = the median stone size in a well-graded riprap apron.

- The riprap thickness should be 1.5 times d_{max} or 6 inches, whichever is greater: apron thickness = 1.5 x d_{max}. Note: the apron thickness may be reduced to 1.5 x d_{50} when an appropriate filter fabric is used under the apron.
- The apron width at the discharge outlet should be at least equal to the pipe diameter or culvert width, d_w. Riprap should extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 2:1 and a height not less than the pipe diameter or culvert height, and should taper to the flat surface at the end of the apron.
- If there is a well-defined channel, the apron length should be extended as necessary so that the downstream apron width is equal to the channel width. The sidewalls of the channel should not be steeper than 2:1.
- If the ground slope downstream of the apron is steep, channel erosion may occur. The apron should be extended as necessary until the slope is gentle enough to prevent further erosion.
- The potential for vandalism should be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased or the rocks held in place using concrete or grout.

Riprap apron design calculations are presented in Examples 7-11 and 7-12.

Example 7-11. Riprap Apron Design for Minimum Tailwater Conditions

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

Step 1. Minimum tailwater conditions = 0.5 d_o, d_o = 66 in = 5.5 ft; therefore, 0.5 d_o = 2.75 ft.

Step 2. Since TW = 2 ft, use Figure 7-29 for minimum tailwater conditions.

Step 3. By Figure 7-29, the apron length, L_a, and median stone size, d_{50}, are 38 ft and 1.2 ft, respectively.

Step 4. The downstream apron width equals the apron length plus the pipe diameter:
$W = d + L_a = 5.5 + 38 = 43.5$ ft

Step 5. Maximum riprap diameter is 1.5 times the median stone size:
$1.5(d_{50}) = 1.5(1.2) = 1.8$ ft

Step 6. Riprap depth:
$1.5(d_{max}) = 1.5(1.8) = 2.7$ ft

Example 7-12. Riprap Apron Design for Maximum Tailwater Conditions

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

Step 1. Compute 0.5 d_o = 0.5 (5.0) = 2.5 ft.

Step 2. Since TW = 5.0 ft is greater than 2.5 ft, use Figure 7-30 for maximum tailwater conditions.
$v = Q/A = [600/(5)(10)] = 12$ ft/s

<u>Step 3.</u> On Figure 7-30 at the intersection of the curve, d_0 = 60 in and v = 12 ft/s, d_{50} = 0.4 ft. Reading up to the intersection with d = 60 in, find L_a = 40 ft.

<u>Step 4.</u> Apron width downstream
Apron width downstream = $d_w = 0.4L_a = 10 + 0.4(40) = 26$ ft

<u>Step 5.</u> Maximum stone diameter
Maximum stone diameter = $1.5(d_{50}) = 1.5(0.4) = 0.6$ ft

<u>Step 6.</u> Riprap depth
Riprap depth = $1.5(d_{max}) = 1.5(0.6) = 0.9$ ft

Table 7-12 has been developed from Figures 7-29 and 7-30 for quick reference.

Table 7-12. Selecting Riprap Apron Length (L_A) and Riprap Size (D_{50})
(for circular culverts flowing full)

Riprap Aprons for Low Tailwater
(downstream flow depth < 0.5 x pipe diameter)

Culvert Diameter (inches)	Lowest Value			Intermediate values to interpolate from:									Highest Value		
	Q cfs	L_A ft	d_{50} in	Q cfs	L_A ft	d_{50} in	Q cfs	L_A ft	d_{50} in	Q cfs	L_A ft	d_{50} in	Q cfs	L_A ft	d_{50} in
12	4	7	2.5	6	10	3.5	9	13	3.5	12	16	7	14	17	8.5
15	6.5	8	3	10	12	5	15	16	5	20	18	10	25	20	12
18	10	9	3.5	15	14	5.5	20	17	5.5	30	22	11	40	25	14
21	15	11	4	25	18	7	35	22	7	45	26	13	60	29	18
24	21	13	5	35	20	8.5	50	26	8.5	65	30	16	80	33	19
27	27	14	5.5	50	24	9.5	70	29	9.5	90	34	18	110	37	22
30	36	16	6	60	25	9.5	90	33	9.5	120	38	20	140	41	24
36	56	20	7	100	32	13	140	40	13	180	45	23	220	50	28
42	82	22	8.5	120	32	12	160	39	12	200	45	20	260	52	26
48	120	26	10	170	37	14	220	46	14	270	54	23	320	64	37

Riprap Aprons for High Tailwater
(downstream flow depth > 0.5 x pipe diameter)

Culvert Diameter (inches)	Lowest Value			Intermediate values to interpolate from:									Highest Value		
	Q cfs	L_A ft	d_{50} in	Q cfs	L_A ft	d_{50} in	Q cfs	L_A ft	d_{50} in	Q cfs	L_A ft	d_{50} in	Q cfs	L_A ft	d_{50} In
12	4	8	2	6	18	2.5	9	28	4.5	12	36	7	14	40	8
15	7	8	2	10	20	2.5	15	34	5	20	42	7.5	25	50	10
18	10	8	2	15	22	3	20	34	5	30	50	9	40	60	11
21	15	8	2	25	32	4.5	35	48	7	45	58	11	60	72	14
24	20	8	2	35	36	5	50	55	8.5	65	68	12	80	80	15
27	27	10	2	50	41	6	70	58	10	90	70	14	110	82	17
30	36	11	2	60	42	6	90	64	11	120	80	15	140	90	18
36	56	13	2.5	100	60	7	140	85	13	180	104	18	220	120	23
42	82	15	2.5	120	50	6	160	75	10	200	96	14	260	120	19
48	120	20	2.5	170	58	7	220	85	12	270	105	16	320	120	20

This table is intended to select two parameters for the design of riprap outlet protection, based upon outlet velocities that correspond with circular culverts flowing full. Flow values less than the lowest value for the culvert size usually indicate a full-flow velocity less than 5 feet per second, for which riprap is usually not necessary. Flow values more than the highest value for the culvert size usually indicate that a concrete stilling basin or energy dissipater structure is necessary. Adjust values upward if the circular culvert is not flowing full based upon outlet conditions. For noncircular pipe, convert into an equivalent cross-sectional area of circular culvert to continue design.

7.5.5 Riprap Basins

7.5.5.1 Description

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

7.5.5.2 Basin Features

General details of the basin recommended in this section are shown in Figure 7-32. Principal features of the basin are:

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

7.5.5.3 Design Procedure

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

(Step 1) Estimate the flow properties at the brink (outlet) of the culvert. Establish the outlet invert elevation such that $TW/y_o \leq 0.75$ for the design discharge.

(Step 2) For subcritical flow conditions (culvert set on mild or horizontal slope) use Figure 7-33 or Figure 7-34 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning Equation for appropriate slope, section, and discharge.

(Step 3) For channel protection, compute the Froude number for brink (outlet) conditions with $y_e = (A/2)^{1.5}$ for non-rectangular sections. Select d_{50}/y_e as appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25 < d_{50}/y_e < 0.45$). Obtain h_s/ye from Figure 7-35, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if h_s/d_{50} falls out of this range.

(Step 4) Size basin as shown in Figure 7-33.

(Step 5) Where allowable dissipator exit velocity is specified:

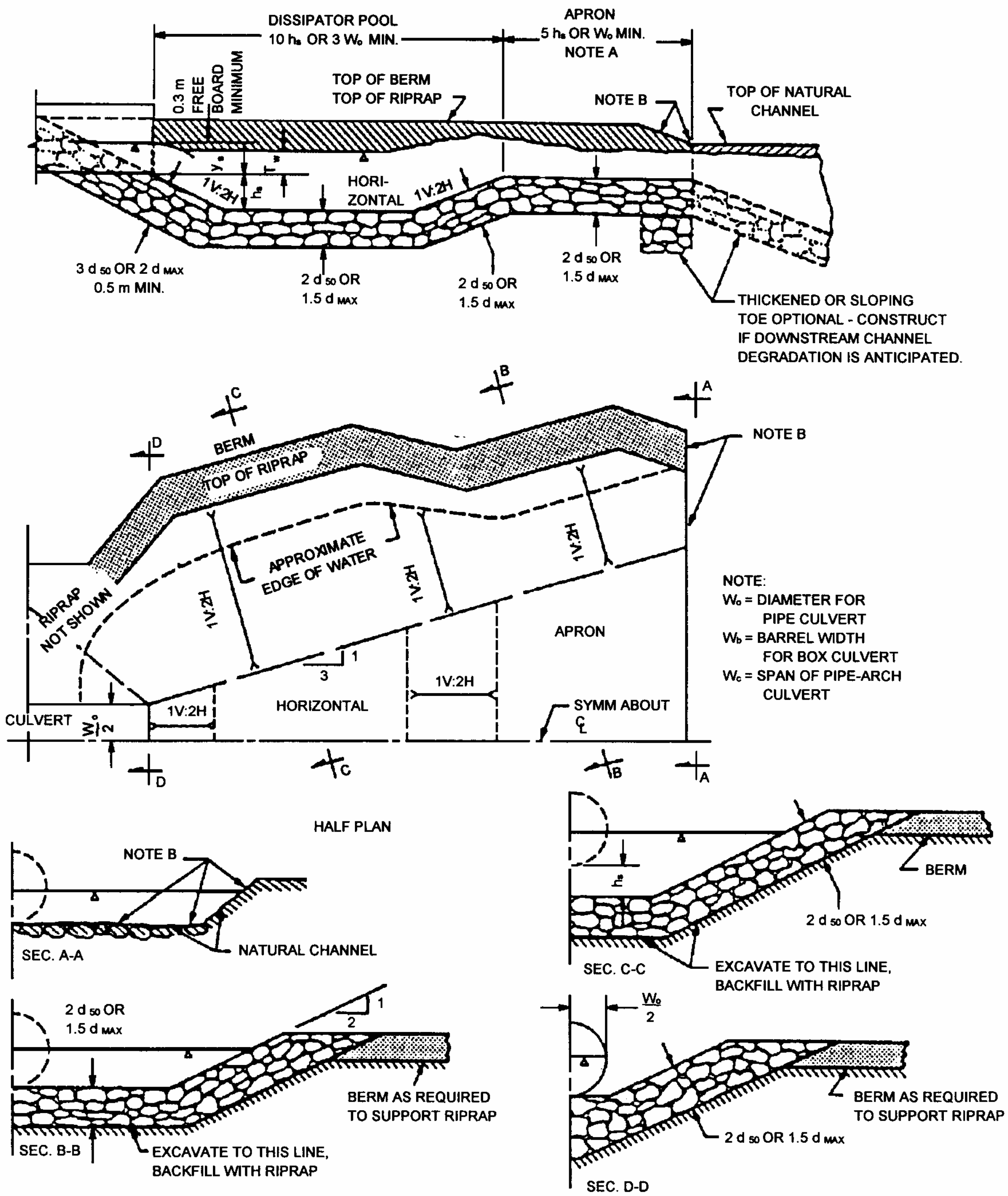
a. Determine the average normal flow depth in the natural channel for the design discharge.

b. Extend the length of the energy basin (if necessary) so that the width of the energy basin at section A-A, Figure 7-32, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

(Step 6) In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so that the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

Figure 7-32. Details of Riprap Outlet Basin

(Source: USDOT, 1983)



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

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

Figure 7-33. Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes

(Source: USDOT, 1983)

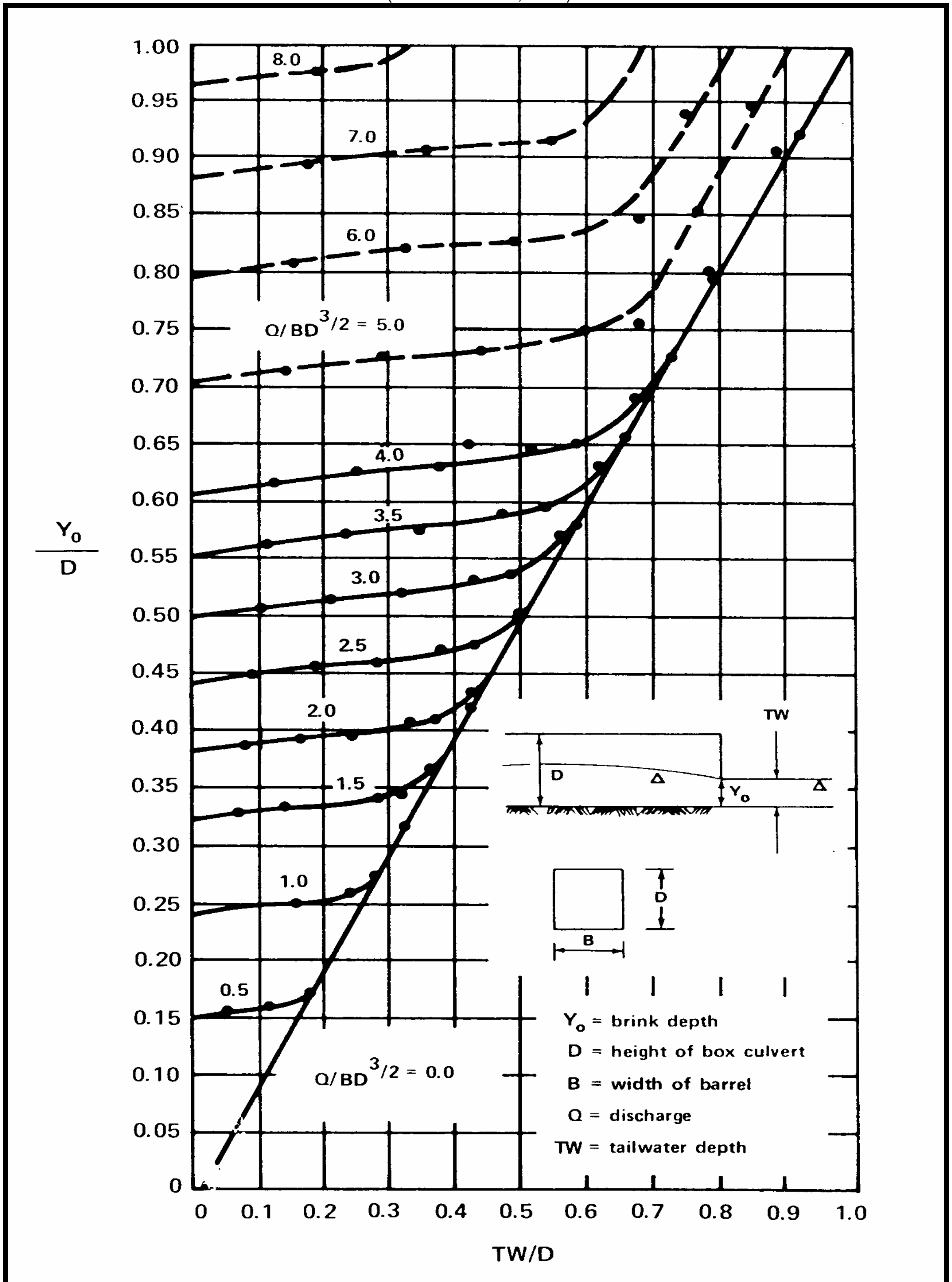


Figure 7-34. Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes

(Source: USDOT, 1983)

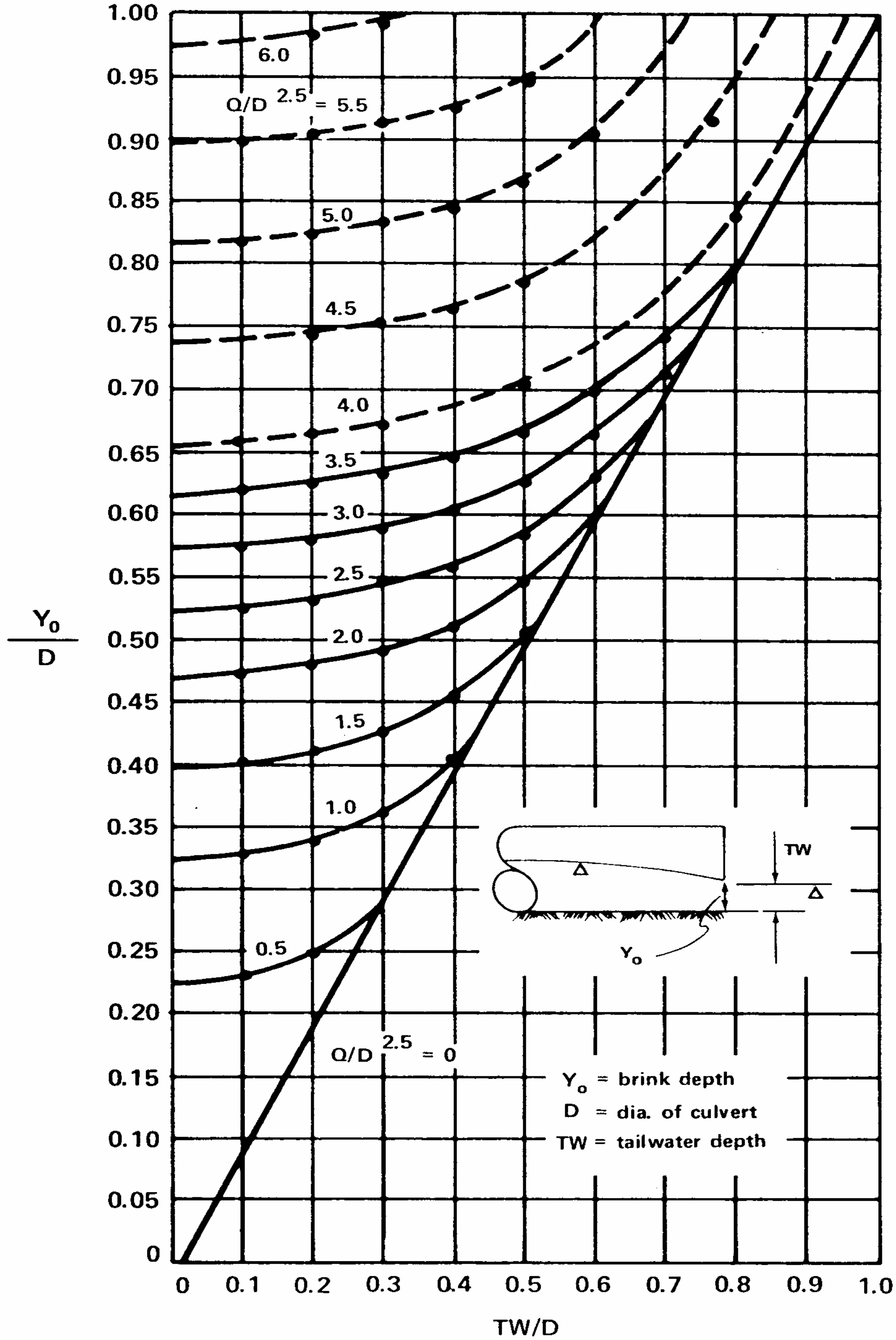
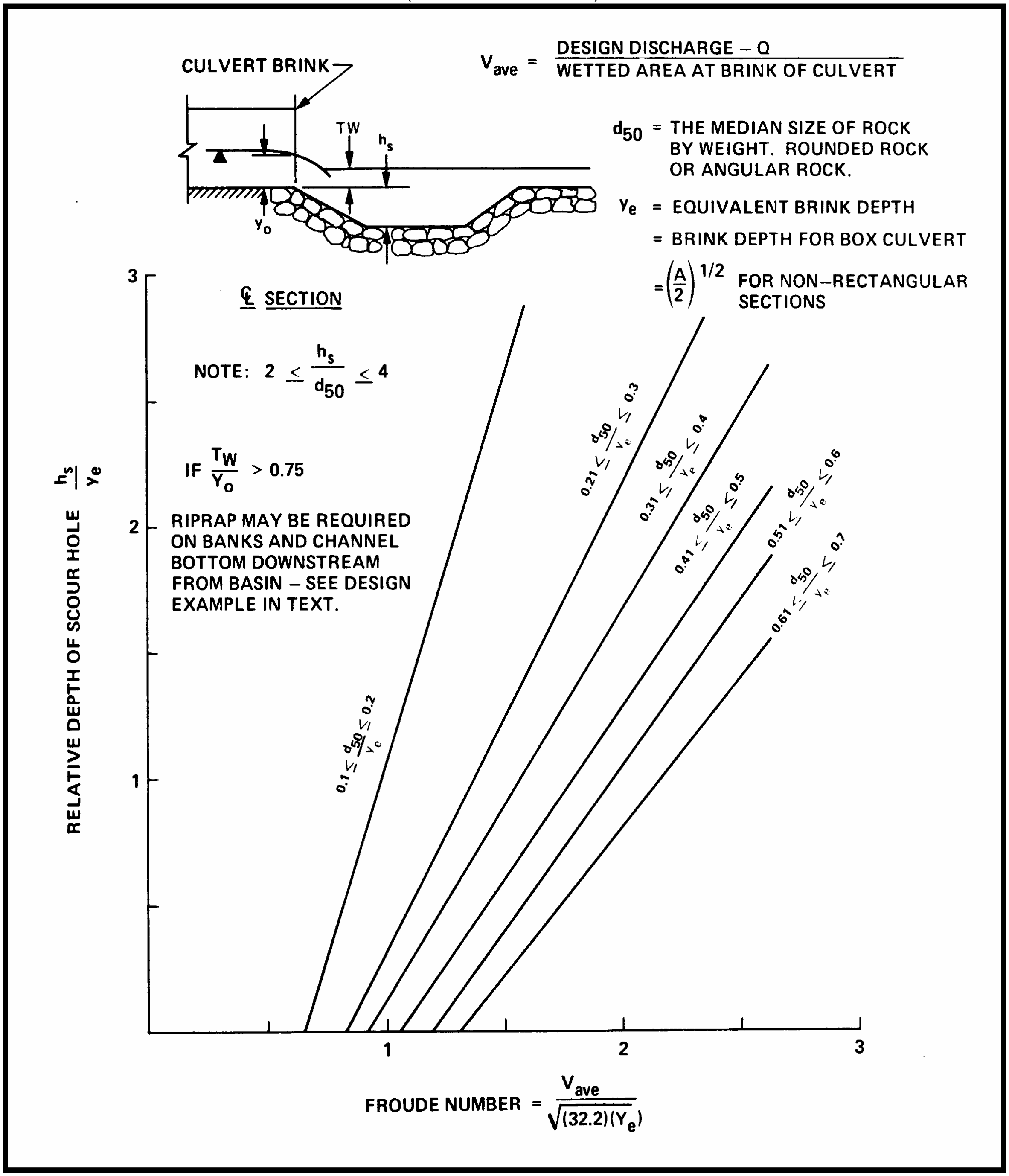


Figure 7-35. Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable

(Source: USDOT, 1983)



(Step 7) If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

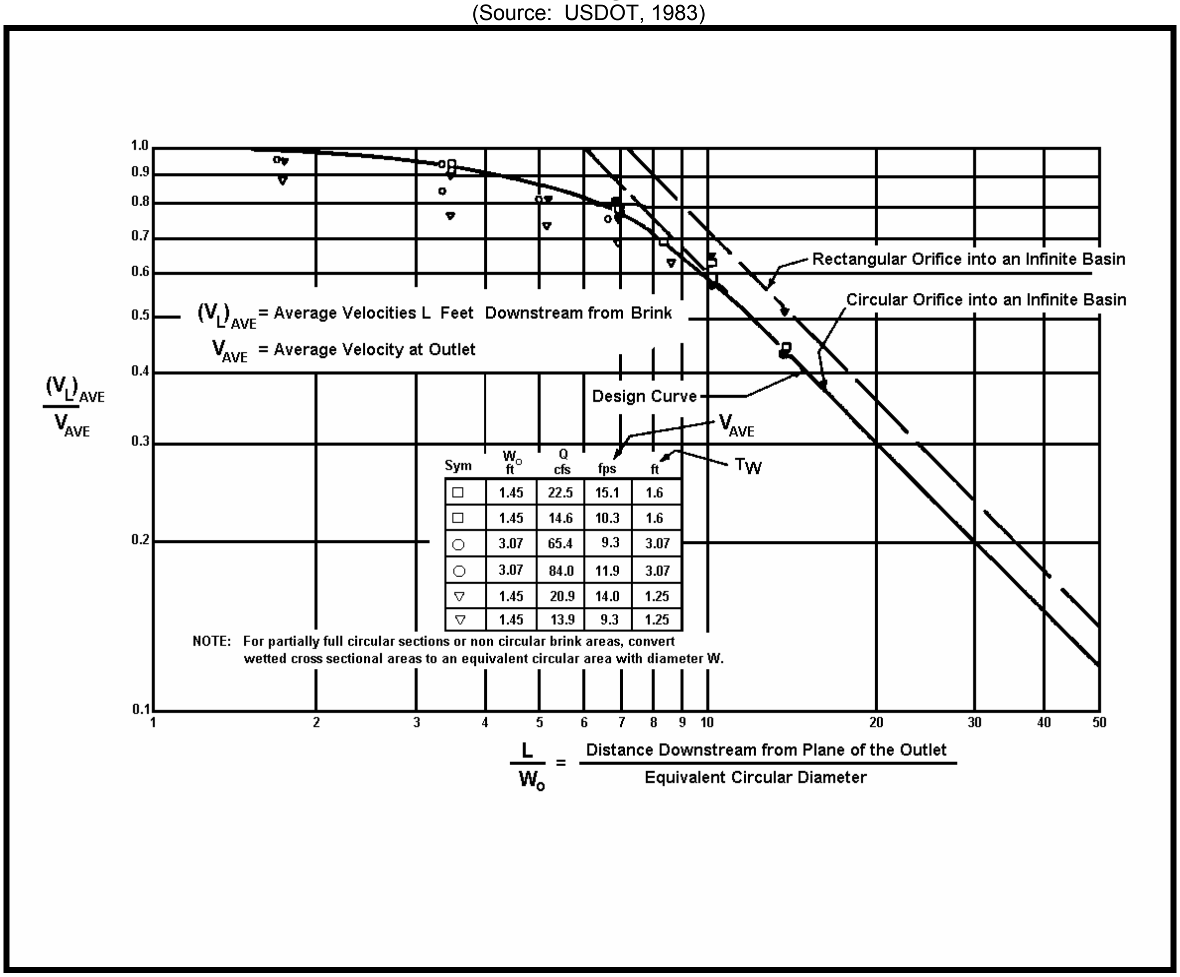
- Design a conventional basin for low tailwater conditions in accordance with the instructions above.
- Estimate centerline velocity at a series of downstream cross sections using the information shown in Figure 7-36.

- Shape downstream channel and size riprap using Figure 7-28 and the stream velocities obtained above.

Material, construction techniques, and design details for riprap should be in accordance with specifications in the document entitled Use of Riprap for Bank Protection (Searcy, 1967).

Figure 7-36. Distribution of Centerline Velocity for Flow from Submerged Outlets to be Used for Predicting Channel Velocities Downstream from Culvert Outlet Where High Tailwater Prevails

(Source: USDOT, 1983)



Sym	W_o ft	Q cfs	fps	ft
□	1.45	22.5	15.1	1.6
□	1.45	14.6	10.3	1.6
○	3.07	65.4	9.3	3.07
○	3.07	84.0	11.9	3.07
▽	1.45	20.9	14.0	1.25
▽	1.45	13.9	9.3	1.25

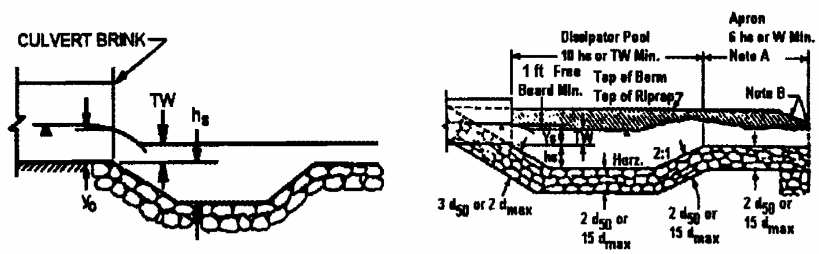
Other basin dimensions should be designed in accordance with details shown on Figure 7-32. Figure 7-37 is provided as a convenient form to organize and present the results of riprap basin designs.

Note: When using the design procedure outlined in this section, it is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the 3 x d_{50} thickness of riprap on the approach and the 2 x d_{50} thickness of riprap on the basin floor and the apron in the downstream portion of the basin, the damage should be superficial.

Figure 7-37. Riprap Basin Design Form
(Source: USDOT, 1983)

RIPRAP BASIN

Project No. ______________________________
Designer ______________________________ Date ________
Reviewer ______________________________ Date ________



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

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

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

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

7.5.5.4 Design Considerations

Riprap basin design should include consideration of the following:

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

- When the ratio of tailwater depth to brink (outlet) depth, TW/y_o, is less than 0.75 and the ratio of scour depth to size of riprap, h_s/d_{50}, is greater than 2.0, the scourhole should function very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunges into the hole, a jump forms against the downstream extremity of the scourhole, and flow is generally well dispersed leaving the basin.
- The mound of material formed on the bed downstream of the scourhole contributes to the dissipation of energy and reduces the size of the scourhole; that is, if the mound from a stable scoured basin is removed and the basin is again subjected to design flow, the scourhole will enlarge.
- For high tailwater basins (TW/y_o greater than 0.75), the high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin and diffuses similarly to a concentrated jet diffusing in a large body of water. As a result, the scourhole is much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.
- It should be recognized that there is a potential for limited degradation to the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2(d_{50})$ thickness of riprap, the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, such damage should be superficial.
- See standards in the document entitled Use of Riprap for Bank Protection (Searcy, 1967) for details on riprap materials and use of filter fabric.
- Stability of the surface at the outlet of a basin should be considered using the methods for open channel flow as outlined in Section 7.4.

Example 7-13. Riprap Basin Design Calculation 1

Given: Box culvert - 8 ft by 6 ft
Design Discharge Q = 800 cfs
Supercritical flow in culvert
Flow depth = Brink depth
$Y_o = 4$ ft
Tailwater depth TW = 2.8 ft

Find: Riprap basin dimensions for these conditions

Calculations: Definition of terms in Steps 1 through 5 can be found in Figures 7-32 and 7-35.

Step 1. $y_o = y_e$ for rectangular section; therefore, with y_o given as 4 ft, $y_e = 4$ ft.

Step 2. Calculate V_o
$V_o = Q/A = 800/(4)(8) = 25$ ft / s

Step 3. Froude Number $= Fr = V/[(g)(y_e)]^{0.5}$ ($g = 32.2$ ft/s²)
$Fr = V/[(g)(y_e)]^{0.5} = 25/[(32.2)(4)]^{0.5} = 2.20 < 3.0$, therefore OK

Step 4. $TW/y_e = 2.8/4.0 = 0.7$ Therefore, $TW/y_e < 0.75$ OK

Step 5. Try $d_{50}/y_e = 0.45$,
$d_{50} = (0.45)(4) = 1.80$ ft
From Figure 7-35, $h_s/y_e = 1.6$,
$h_s = (4)(1.6) = 6.4$ ft
$h_s/d_{50} = 6.4/1.8 = 3.6$ ft, $2 < h_s/d_{50} < 4$, therefore OK

Step 6. Calculate L_s (length of energy dissipator pool) and L_B (overall length of riprap basin)
$L_s = 10(h_s) = 10\ (6.4) = 64$ ft
$L_{s\ min} = 3(W_o) = 3\ (8) = 24$ ft
Therefore, use $L_s = 64$ ft
$L_B = 15(h_s) = 15\ (6.4) = 96$ ft
$L_{B\ min} = 4(W_o) = 4\ (8) = 32$ ft
Therefore, use $L_B = 96$ ft

Step 7. Calculate thickness of riprap on the approach and for remainder
Approach riprap thickness = $3(d_{50}) = 3\ (1.8) = 5.4$ ft
Remainder riprap thickness = $2(d_{50}) = 2\ (1.8) = 3.6$ ft
Other basin dimensions designed according to details shown in Figure 7-32.

Example 7-14. Riprap Basin Design Calculation 2

Given: Same design data as Example 7-13 except:
Tailwater depth TW = 4.2 ft
Downstream channel can tolerate only 7 ft/s discharge

Find: Riprap basin dimensions for these conditions

Calculations: Note -- High tailwater depth, $TW/y_o = 4.2/4 = 1.05 > 0.75$
Step 1. From Example 7-13: $d_{50} = 1.8$ ft, $h_s = 6.4$ ft, $L_s = 64$ ft, $L_B = 96$ ft.

Step 2. Design riprap for downstream channel. Use Figure 7-36 for estimating average velocity along the channel. Compute equivalent circular diameter D_e for brink area from:
$A = 3.14D_e^2/4 = (y_e)(W_o) = 32\ ft^2$
$D_e = [(32)(4)/3.14]0.5 = 6.4$ ft
$V_o = 25$ ft/s (From Example 7-13)

Step 3. Set up the following table:

L/D_e (Assume $D_e = W_o$)	L (ft) (Compute)	V_L/V_o (Fig. 7-36)	v_1 (ft/s) (Fig. 7-29)	Rock Size d_{50} (ft)
10	64	0.59	14.7	1.4
15*	96	0.37	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

*L/W_o is on a logarithmic scale so interpolations must be done logarithmically.

Riprap should be at least the size shown but can be larger. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream from the culvert brink. Channel should be shaped and riprap should be installed in accordance with details shown in the HEC No. 11 publication (USDOT, 1967).

Example 7-15. Calculation of d_{50}

Given: 6-ft diameter corrugated metal culvert
Design discharge Q = 135 cfs
Slope channel $S_o = 0.004$
Manning's "n" = 0.024
Normal depth in pipe for Q = 135 cfs is 4.5 ft
Normal velocity is 5.9 ft/s
Flow is subcritical
Tailwater depth TW = 2.0 ft

Find: Riprap basin dimensions for these conditions.

Solution:

Step 1. Determine y_o and V_o
From Figure 7-34, $y_o/D = 0.45$
$Q/D^{2.5} = 135/6^{2.5} = 1.53$
$TW/D = 2.0/6 = 0.33$ ft
$y_o = 0.45(6) = 2.7$ ft
$TW/y_o = 2.0/2.7 = 0.74$
$TW/y_o < 0.75$ O.K.
Determine Brink Area (A) for $y_o/D = 0.45$
From Uniform Flow in Circular Sections Table (from Section 7.3)
For $y_o/D = d/D = 0.45$
$A/D^2 = 0.3428$;
$A = 0.3428(D)^2 = 0.3428(6)^2 = 12.3\ ft^2$
$V_o = Q/A = 135/12.3 = 11.0$ ft/s

Step 2. For Froude number calculations at brink conditions,
$y_o = (A/2)^{1/2} = (12.3/2)^{1/2} = 2.48$ ft

Step 3. Check Froude number:
$Fr = Vo/[(g)(y_o)]^{1/2} = 11/[(32.2)(2.48)]^{1/.2} = 1.23 < 2.5$ therefore, OK

Step 4. For most satisfactory results - $0.25 < d_{50}/y_o < 0.45$
Try $d_{50}/y_o = 0.25$
$d_{50} = 0.25(y_o) = 0.25(2.48) = 0.62$ ft
From Figure 7-35, $h_s/y_o = 0.75$
$h_s = 0.75(y_o) = 0.75(2.48) = 1.86$ ft
Uniform Flow in Circular Sections Flowing Partly Full (From Section 7.3)
Check: $h_s/d_{50} = 1.86/0.62 = 3$, $2 < h_s/d_{50} < 4$ OK

Step 5. Calculate L_s
$L_s = 10(h_s) = 10(1.86) = 18.6$ ft or $L_s = 3(W_o) = 3(6) = 18$ ft
therefore, use $L_s = 18.6$ ft
$L_B = 15(h_s) = 15(1.86) = 27.9$ ft or $L_B = 4(W_o) = 4(6) = 24$ ft
therefore, use $L_B = 27.9$ ft
$d_{50} = L_s/L_B = 18.6/27.9 = 0.66$ ft or use $d_{50} = 8$ in

7.5.6 Baffled Outlets

7.5.6.1 Description

The baffled outlet (also known as the Impact Basin - USBR Type VI) is a boxlike structure with a vertical hanging baffle and an end sill, as shown in Figure 7-38. Energy is dissipated primarily through the impact of the water striking the baffle and, to a lesser extent, through the resulting turbulence. This type of outlet protection has been used with outlet velocities up to 50 feet per second and with Froude numbers from 1 to 9. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

7.5.6.2 Design Procedure

The following design procedure is based on physical modeling studies summarized in Design of Small Canal Structures (US Department of Interior, 1978). The dimensions of a baffled outlet as shown in Figure 7-38 should be calculated as follows:

(Step 1) Determine input parameters, including:

h = Energy head to be dissipated, in ft (can be approximated as the difference between channel invert elevations at the inlet and outlet)

Q = Design discharge (cfs)

v = Theoretical velocity (ft/s = $\sqrt{(2gh)}$)

A = Q/v = Flow area (ft^2)

d = $A^{0.5}$ = Representative flow depth entering the basin (ft) *assumes square jet*

Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

(Step 2) Calculate the minimum basin width, W, in ft, using Equation 7-26.

Equation 7-26 $$W = 2.88dFr^{0.566}$$

where:

W = minimum basin width (ft)
d = depth of incoming flow (ft)
Fr = Froude number, dimensionless

The limits of the W/d ratio are from 3 to 10, which corresponds to Froude numbers 1 and 9. If the basin is much wider than W, flow will pass under the baffle and energy dissipation will not be effective.

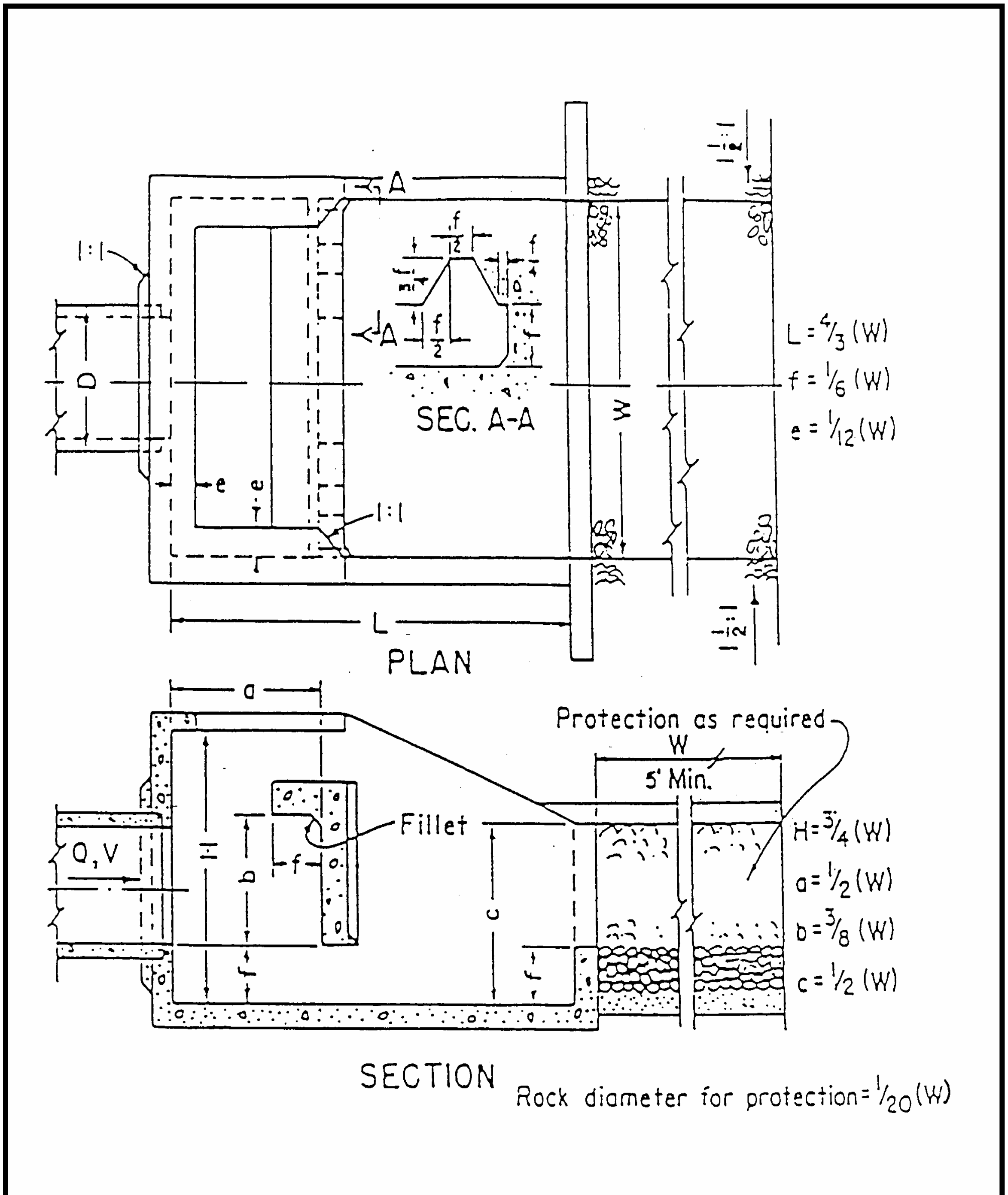
(Step 3) Calculate the other basin dimensions as shown in Figure 7-38, as a function of W. Construction drawings for selected widths are available from the United States Department of the Interior (1978).

(Step 4) Calculate required protection for the transition from the baffled outlet to the natural channel based on the outlet width. A riprap apron should be added of width (W), length (L) (or a 5-foot minimum), and depth (f) (W/6). The side slopes should be 1.5:1, and median rock diameter should be at least W/20.

(Step 5) Calculate the baffled outlet invert elevation based on expected tailwater. The maximum distance between expected tailwater elevation and the invert should be b + f or some flow will go over the baffle with no energy dissipation. If the tailwater is known and fairly controlled, the baffled outlet invert should be a distance, b/2 + f, below the calculated tailwater elevation. If tailwater is uncontrolled, the baffled outlet invert should be a distance, f, below the downstream channel invert.

(Step 6) Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 ft/s flowing full.

Figure 7-38. Schematic of a Baffled Outlet
(Source: U.S. Department of the Interior, 1978)



(Step 7) If the entrance pipe slopes steeply downward, the outlet pipe should be turned horizontal for at least 3 ft before entering the baffled outlet.

(Step 8) If it is possible that both the upstream and downstream ends of the pipe will be submerged, provide an air vent approximately 1/6 the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

Example 7-16. Baffled Outlet Basin Calculations

A cross-drainage pipe structure has a design flow rate of 150 cfs, a head, h, of 15 ft from invert of pipe, and a tailwater depth, TW, of 3 ft above ground surface. Find the baffled outlet basin dimensions and inlet pipe requirements.

Step 1. Compute the theoretical velocity from
$v = (2gh)^{0.5} = [(2)(32.2)(15)]^{0.5} = 31.1$ ft/s
This is less than 50 ft/s, so a baffled outlet is suitable.

Step 2. Determine the flow area using the theoretical velocity as follows:
$A = Q/v = 150/31.1 = 4.8$ ft^2

Step 3. Compute the flow depth using the area from Step 2.
$d = A^{0.5} = (4.8)^{0.5} = 2.19$ ft

Step 4. Compute the Froude number using the results from Steps 1 and 3.
$Fr = v/(gd)^{0.5} = 31.1/[(32.2)(2.19)]^{0.5} = 3.7$

Step 5. Determine the basin width using Equation 7-26 with the Froude number from Step 4.
$W = 2.88dFr^{0.566} = 2.88(2.19)(3.7)^{0.566} = 13.2$ ft (minimum)
Use 13.2 ft as the design width.

Step 6. Compute the remaining basin dimensions (as shown in Figure 7-38):
$L = 4/3(W) = 4/3(13.2) = 17.6$ ft
use L = 17 ft, 7 in
$f = {}^1/_6(W) = {}^1/_6(13.2) = 2.2$ ft
use f = 2 ft, 2 in
$e = {}^1/_{12}(W) = {}^1/_{12}(13.2) = 1.1$ ft
use e = 1 ft, 1 in
$H = \frac{3}{4}(W) = \frac{3}{4}(13.2) = 9.9$ ft
use H = 9 ft, 11 in
$a = \frac{1}{2}(W) = \frac{1}{2}(13.2) = 6.6$ ft
use a = 6 ft, 7 in
$b = {}^3/_8(W) = {}^3/_8(13.2) = 4.95$ ft
use b = 4 ft, 11 in
$c = \frac{1}{2}(W) = \frac{1}{2}(13.2) = 6.6$ ft
use c = 6 ft, 7 in
Baffle opening dimensions would be calculated as shown in Figure 7-38.

Step 7. Basin invert should be at b/2 + f below tailwater, or
(4 ft, 11 in)/2 + 2 ft, 2 in = 4.73 ft
Use 4 ft 8 in; therefore, invert should be 1 ft, 8 in below ground surface.

Step 8. The riprap transition from the baffled outlet to the natural channel should be 13.2 ft long by 13.2 ft wide by 2 ft, 2 in deep (W x W x f). Median rock diameter should be of diameter W/20(12) = 7.92 in., or about 8 in.

Step 9. Inlet pipe diameter should be sized for an inlet velocity of about 12 ft/s.
$3.14d^2/4 = Q/v$
$d = [(4Q)/3.14v]^{0.5} = [4(150)/3.14(12)]^{0.5} = 3.99$ ft
Use 48-in pipe. If a vent is required, it should be about 1/6 of the pipe diameter or 8 in.

7.5.7 Level Spreaders

7.5.7.1 Description

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

7.5.7.2 Spreader Features

General details of level spreaders recommended in this section are shown in Figure 7-39.

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

7.5.7.3 Design Standards

All level spreaders designed and constructed in Knox County shall conform to the design standards listed below.

<u>For impervious surface runoff applications</u>:

- The capacity for the level spreader is determined in the design of the filter strip or buffer area to which it discharges. For filter strip design guidance, refer to Volume 2, Chapter 4. Buffer area guidance is provided in Volume 2, Chapter 6.
- The spreader shall run linearly along the entire length of the filter strip (or buffer area) to which it discharges. In most cases, the spreader will be the same length as the contributing impervious surface. The ends of the spreader shall be tied into higher ground to prevent flow around the spreader.

<u>For diversion outlet applications</u>:

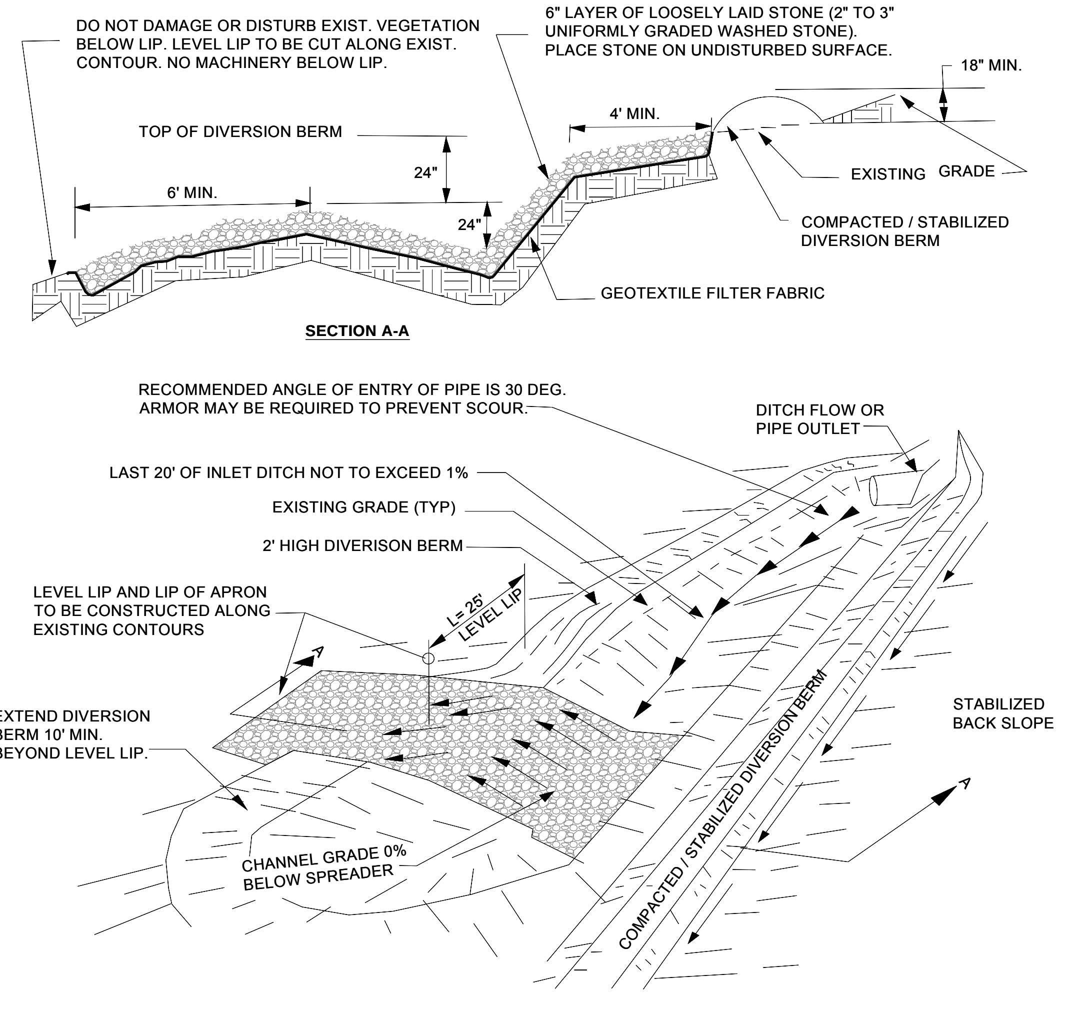
The capacity of the spreader shall be determined using the peak flow from the 10-year, 24-hour storm. The drainage area shall be restricted so that maximum 10-year design flows into the spreader will not exceed thirty (30) cfs.

<u>For all level spreader applications</u>:

- The minimum depth shall be 6 inches and the minimum width shall be 6 feet for the lower side slope. Side slopes shall be 2:1 (horizontal to vertical) or flatter.
- The grade of the spreader shall be 0%.
- The appropriate length, width, and depth of spreader should be selected from Table 7-13.
- It will be necessary to construct a 20 foot transition section in the diversion channel so the width of the channel will smoothly meet the width of the spreader to ensure uniform outflow.
- The last 20 feet of the diversion channel shall provide a smooth transition from the channel grade to the level spreader and where possible, shall be less than or equal to 1%.

Figure 7-39. Level Spreader

(Source: Maine Erosion and Sediment Control BMP Manual, 2003)



CONSTRUCTION SPECIFICATIONS:

1. SPREADERS SHALL BE INSTALLED WITH A LEVEL INSTRUMENT. CONSTRUCT LEVEL LIP TO 0% GRADE TO ENSURE UNIFORM SHEET FLOW. LEVEL SPREADER SHALL BE CONSTRUCTED ON UNDISTURBED SOIL (NOT FILL)
2. SELECT GEOTEXTILE FABRIC BASED ON UNDISTURBED SOILS (SANDS, SILTS, CLAYS, ECT.)
3. PLACE 6" LAYER OF UNIFORMLY GRADED STONE (2" TO 3" IN DIA.) RAKE TO FORM SMOOTH UNIFORM SURFACE. DO NOT FILL VOIDS IN STONE.
4. THE INLET DITCH SHALL NOT EXCEED A 1% GRADE FOR AT LEAST 20 FEET BEFORE ENTERING THE SPREADER.
5. STORM RUNOFF CONVERTED TO SHEET FLOW ACROSS OUTLET APRON SHALL FLOW ONTO STABILIZED AREAS. RUNOFF SHALL NOT BE RECONCENTRATED IMMEDIATELY BELOW THE POINT OF DISCHARGE.
6. PERIODIC INSPECTION AND REQUIRED MAINTENANCE SHALL BE PROVIDED.
7. CONSTRUCTION OF LEVEL LIP SPREADER SHALL BE FROM UPHILL SIDE ONLY. LEVEL LIP & AREA BELOW SPREADER SHALL BE AT EXISTING GRADES & UNDISTURBED BY EARTHWORK OR EQUIPMENT
8. CONSTRUCT SPREADER WITH LIP AT EXISTING ELEVATION AS SPECIFIED.
9. DOWNGRADIENT RECEIVING AREA MUST BE NATURALLY WELL VEGETATED.

- The receiving area below the level spreader shall be protected from harm during construction. Minor disturbed areas shall be stabilized with vegetative measures. A temporary stormwater diversion may be necessary until the level spreader has fully stabilized.
- Level spreaders must blend smoothly into the downstream receiving area without any sharp drops or irregularities, to avoid channelization, turbulence and hydraulic "jumps."

Table 7-13. Level Flow Spreader Dimensions
(Source: City of Raleigh, 2002)

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

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

The level spreader lip shall be protected with erosion resistant material to prevent erosion and allow vegetation to be established.

7.6 MINOR DRAINAGE SYSTEM DESIGN

7.6.1 Introduction and General Criteria

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

This section is intended to provide standards and guidance for the design of minor drainage system components including:

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

7.6.1.1 General Considerations for Street and Roadway Gutters

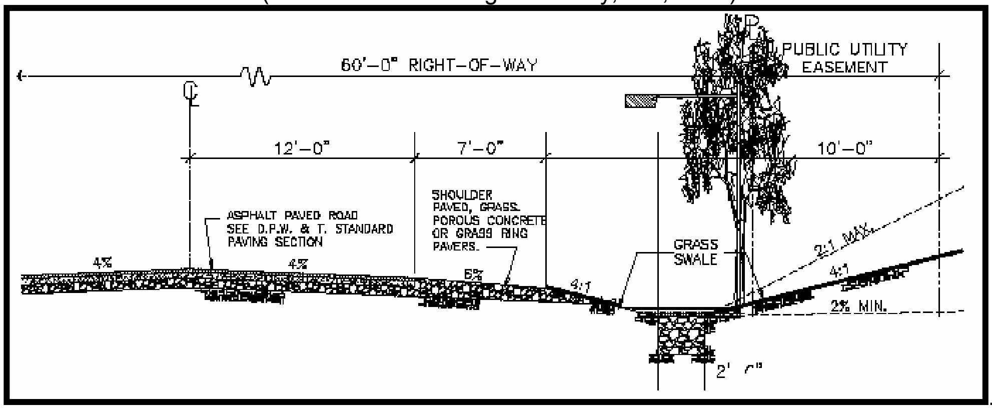
The list below presents the key considerations for the design of street and roadway gutters.

- Gutters are efficient flow conveyance structures. This is not always an advantage if removal of pollutants and reduction of runoff is an objective. Therefore, impervious surfaces should be disconnected hydrologically where possible and runoff should be allowed to flow across pervious surfaces or through grass channels. Gutters should be used only after other options have been investigated and only after runoff has had as much chance as possible to infiltrate and filter through vegetated areas.
- It may not be possible to use gutters at all, or modify them to carry runoff to off-road pervious areas or open channels. For example, curb opening type designs take roadway runoff to smaller feeder grass channels. Care should be taken not to create erosion problems in off-road areas. Protection during construction, establishment of strong stands of grass, and active maintenance may be necessary in some areas.

- Use road cross sections that include grass channels or swales instead of gutters to provide for pollution reduction and reduce the impervious area required. Figure 7-40 is an alternate roadway section which illustrates a roadway cross section that eliminates gutters for residential neighborhoods. Flow can also be directed to center median strips in divided roadway designs. To protect the edge of pavement, ribbons of concrete can be used along the outer edges of asphalt roads.

Figure 7-40. Alternate Roadway Section without Gutters[1]

(Source: Prince George's County, MD, 1999)



1- Figure 7-40 is not to scale

- Catch basins shall be spaced so that the spread in the street for the 25-year design flow shall not exceed the following, as measured from the face of the curb:
  - ✓ 8 feet if the street is classified as a collector or arterial street (for 2-lane streets spread may extend to one-half of the travel lane; for 4-lane streets spread may extend across one travel lane);
  - ✓ 16 feet at any given section, but in no case greater than 10 feet on one side of the street, if the street is classified as a local street.

7.6.1.2 General Considerations for Inlets and Drains

The list below presents the key considerations for the design of inlets and drains.

- Inlets should be located to maximize overland flow path, take advantage of pervious areas, and seek to maximize vegetative filtering and infiltration. For example, it might be possible to design a parking lot so that water flows into vegetated areas prior to entering the nearest inlet.
- Inlet location should not compromise safety or aesthetics. In most cases, it should not allow for standing water in areas of vehicular or pedestrian traffic, but should take advantage of natural depression storage where possible.
- Inlets should be located to serve as overflows for structural BMPs. For example, a bioretention device in a commercial area could be designed to overflow to a catch basin for larger storm events.
- The choice of inlet type should match its intended use. An inlet in a sump may be more effective supporting water quality objectives.
- Use several smaller inlets instead of one large inlet in order to:
  - (1) Prevent erosion on steep landscapes by intercepting water before it accumulates too much volume and velocity.
  - (2) Provide a safety factor. If a drain inlet clogs, the other surface drains may pick up the water.

(3) Improve aesthetics. Several smaller drain inlets will be less obvious than one large drain inlet.

(4) Surface runoff will have better access and opportunity to discharge into the drain, as well as shorter travel distances, by utilizing smaller drain inlets with shorter spacing intervals.

7.6.2 Symbols and Definitions

The symbols listed in Table 7-14 will be used to provide consistency within this section. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

7.6.3 Street and Roadway Gutters

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

This section presents design guidance for gutter flow hydraulics originally published by FHWA (HEC-12, 1984) and the American Association of State Highway and Transportation Officials (AASHTO) (1998).

7.6.3.1 Basic Design Guidance

Equation 7-27 presents a form of Manning's Equation that should be used to evaluate gutter flow hydraulics:

Equation 7-27

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

where:

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

Figure 7-41 is a nomograph for solving Equation 7-27. Manning’s “n” values for various pavement surfaces are presented in Table 7-15.

7.6.3.2 Uniform Cross Slope

The nomograph in Figure 7-41 is used with the following procedures to find gutter capacity for uniform cross slopes:

<u>Condition 1</u>: Find spread, given gutter flow.

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

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

Table 7-14. Minor Drainage System Design Symbols and Definitions

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

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

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

Condition 2: Find gutter flow, given spread.

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

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

Table 7-15. Manning's "n" Values for Street and Pavement Gutters

(Source: USDOT, 1961)

Type of Gutter or Pavement	Range of Manning's "n"
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth texture	0.013
Rough texture	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
Brick or pavers	0.016
For gutters with small slopes, where sediment may accumulate, increase above values of n by:	0.002

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

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

7.6.3.3 Composite Gutter Sections

Figure 7-42 in combination with Figure 7-41 can be used to find the flow in a gutter with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

Figure 7-42 provides a direct solution of gutter flow in a composite gutter section. The flow rate at a given spread or the spread at a known flow rate can be found from this figure. Figure 7-42 involves a complex graphical solution of the equation for flow in a composite gutter section. Figure 7-43 presents a useful graph of flow in composite gutter sections. Typical of graphical solutions, extreme care in using the figure is necessary to obtain accurate results.

Condition 1: Find spread, given gutter flow.

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

(Step 2) Calculate the gutter flow in W (Q_w), using Equation 7-28.

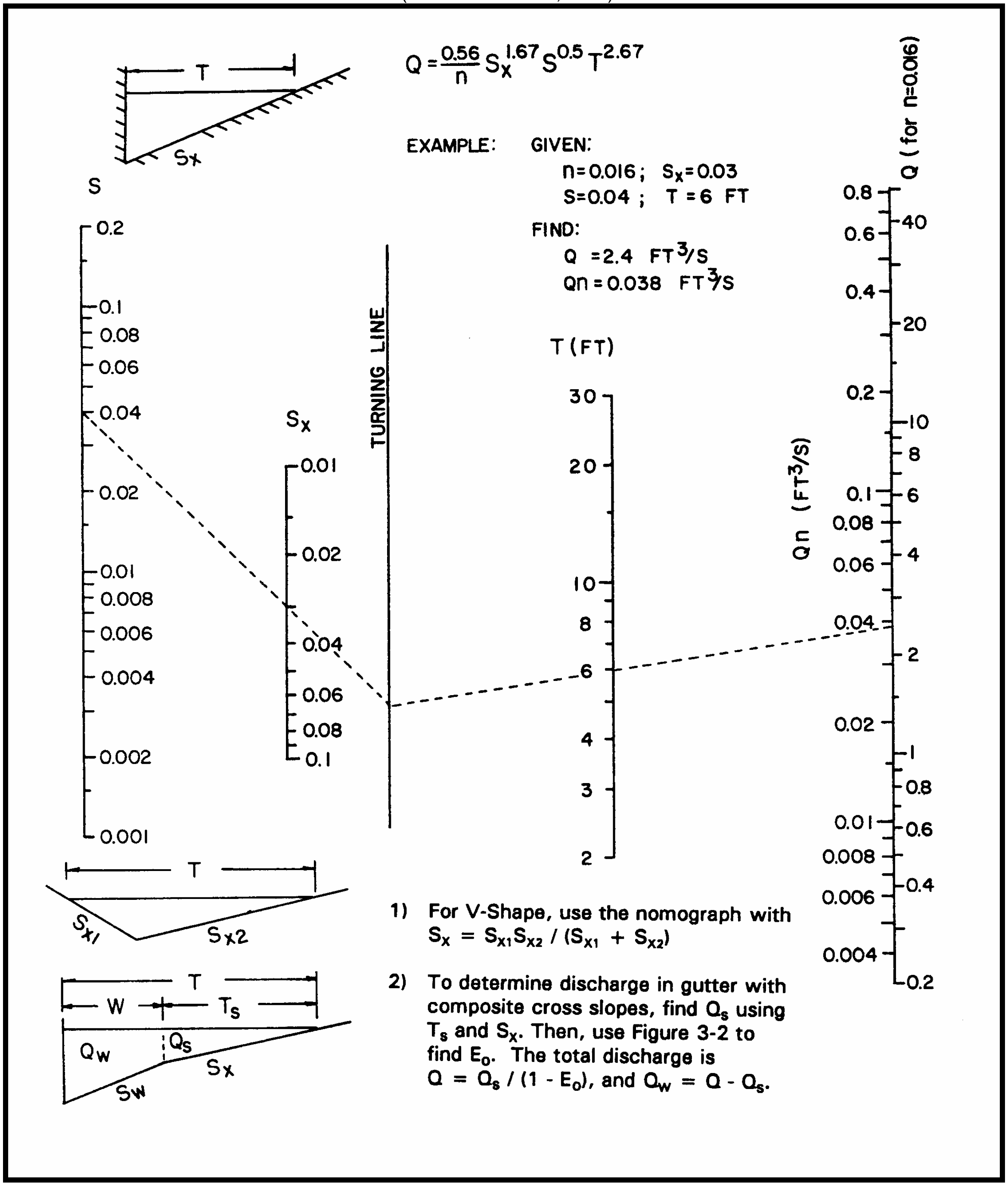
Equation 7-28

$$Q_w = Q - Q_s$$

where:

Q_w = flow in width of curb depression, cfs
Q = gutter flow rate, cfs
Q_s = flow capacity of the gutter section above the depressed section, cfs

Figure 7-41. Flow in Triangular Gutter Sections
(Source: AASHTO, 1998)

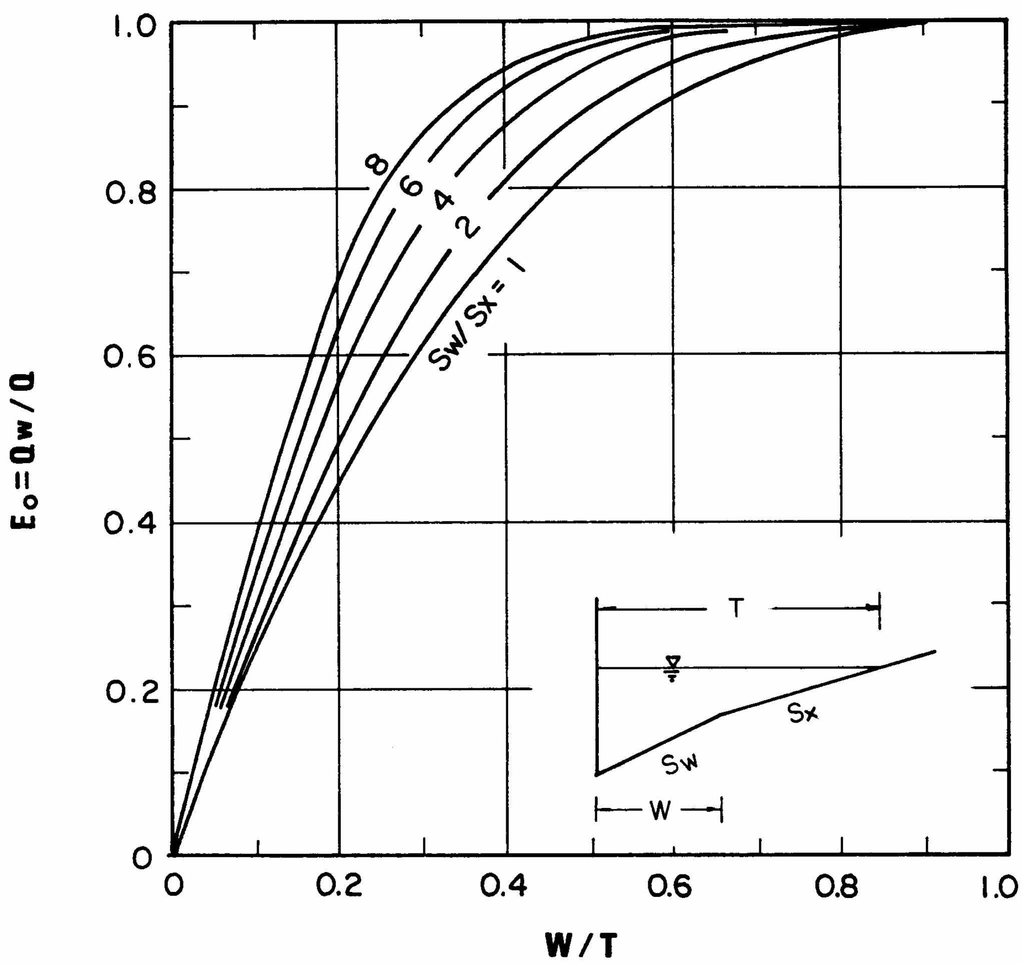


(Step 3) Calculate the ratios Q_w/Q or E_o and S_w / S_x and use Figure 7-42 to find an appropriate value of W/T.

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

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

Figure 7-42. Ratio of Frontal Flow to Total Gutter Flow
(Source: AASHTO, 1998)



(Step 6) Use the value of T_s from Step 5 along with Manning's "n", S, and S_x to find the actual value of Q_s from Figure 7-41.

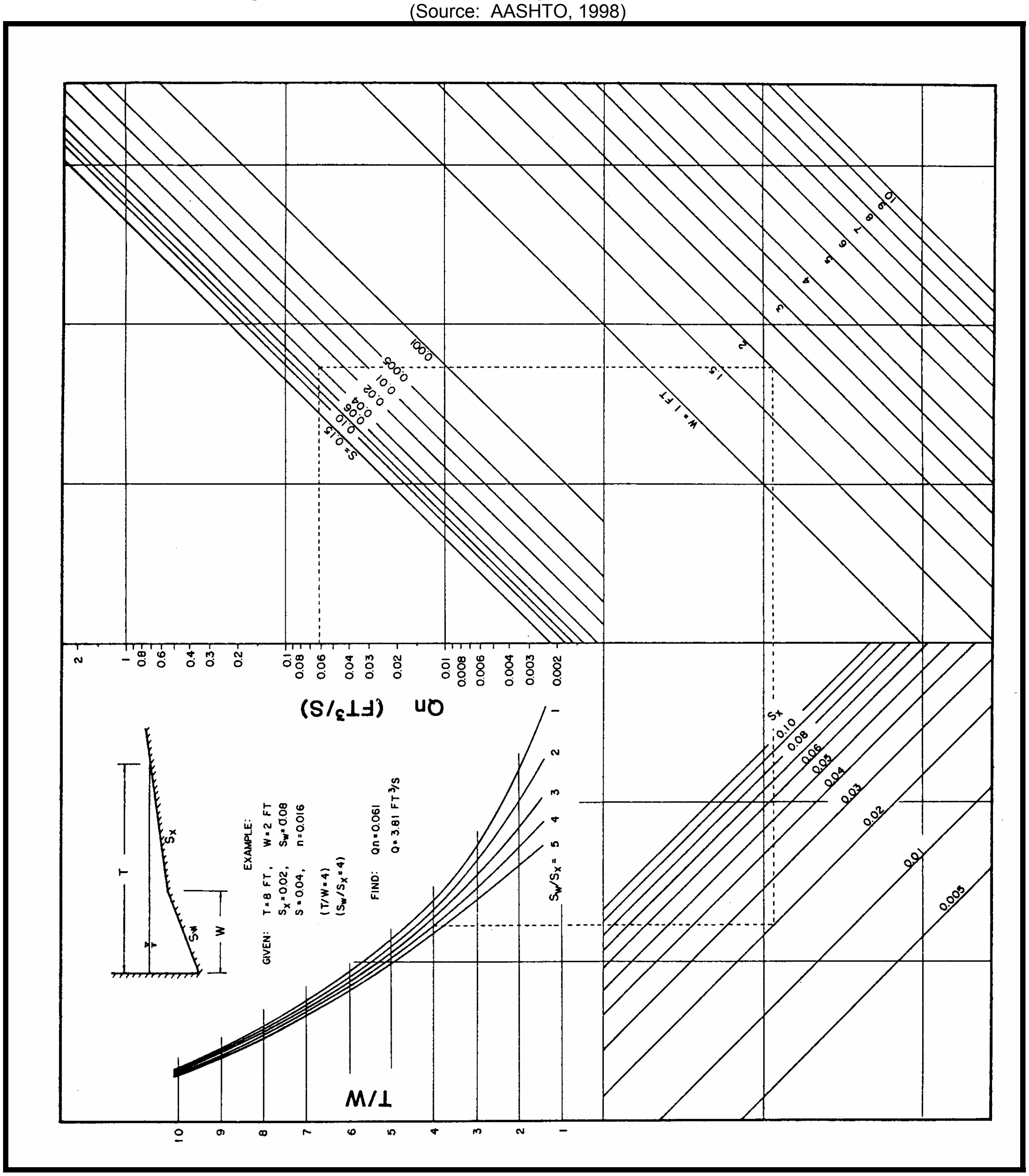
(Step 7) Compare the value of Q_s from Step 6 to the trial value from Step 1. If the values are not comparable, select a new value of Q_s and return to Step 1.

Condition 2: Find gutter flow, given spread.

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

(Step 2) Use Figure 7-41 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes, substituting T_s for T.

Figure 7-43. Flow in Composite Gutter Sections
(Source: AASHTO, 1998)



(Step 3) Calculate the ratios W/T and S_w / S_x, and, from Figure 7-42, find the appropriate value of E_o (the ratio of Q_w/Q).

(Step 4) Calculate the total gutter flow using Equation 7-29.

Equation 7-29

$$Q = \frac{Q_s}{(1 - E_o)}$$

where:

Q = gutter flow rate, cfs
Q_s = flow capacity of the gutter section above the depressed section, cfs
E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

(Step 5) Calculate the gutter flow in width (W), using Equation 7-28.

Example 7-17. Calculate Gutter Flow and Width

Given: $T = 8$ ft $S_x = 0.025$ ft/ft
$n = 0.015$ $S = 0.01$ ft/ft

Find: (a) Flow in gutter at design spread
(b) Flow in width (W = 2 ft) adjacent to the curb

Solution:

Step 1. From Figure 7-41, $Qn = 0.03$
$Q = Qn/n = 0.03/0.015 = 2.0$ cfs

Step 2. $T = 8 - 2 = 6$ ft
$(Qn)_2 = 0.014$ (Figure 7-41) (flow in 6-foot width outside of width (W))
$Q_s = (Qn)_2/n = 0.014/0.015 = 0.9$ cfs
$Q_w = Q - Q_s = 2.0 – 0.9 = 1.1$ cfs

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

Example 7-18. Composite Gutter Flow Calculation

Given: $T = 6$ ft $S_W = 0.0833$ ft/ft
$T_S = 6 - 1.5 = 4.5$ ft $W = 1.5$ ft
$S_X = 0.03$ ft/ft $n = 0.014$
$S = 0.04$ ft/ft

Find: Flow in the composite gutter

Solution:

Step 1. Use Figure 7-41 to find the gutter section capacity above the depressed section.
$Q_S n = 0.038$
$Qs = Q_s n/n = 0.038/0.014 = 2.7$ cfs

Step 2. Calculate W/T_s
$W/ T_s = 1.5/6 = 0.25$
$S_w/S_x = 0.0833/0.03 = 2.78$
Use Figure 7-42 to find $E_o = 0.64$

Step 3. Calculate the gutter flow using Equation 7-29
$Q = Q_s/1\text{-}E_o = 2.7/(1-0.64) = 7.5$ cfs

Step 4. Calculate the gutter flow in width, W, using Equation 7-28
$Q_w = Q – Q_s = 7.5 – 2.7 = 4.8$ cfs

7.6.4 Stormwater Inlets

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

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

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

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

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

The design of grate, curb and combination inlets are discussed in later sections.

7.6.5 Grate Inlet Design

7.6.5.1 Grate Inlets on a Grade

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

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. They also handle debris better than other grate inlets but the vanes of the grate must be turned in the proper direction. Where debris is a problem, consideration should be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was made to qualitatively simulate field conditions. Table 7-16 presents the results of debris handling efficiencies of several grates.

The ratio of frontal flow to total gutter flow, E_o, for straight cross slope is expressed by Equation 7-30.

Equation 7-30

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

where:

- E_o = ratio of frontal flow to total gutter flow
- Q_w = flow in width W, cfs
- Q = total gutter flow, cfs
- W = width of depressed gutter or grate, ft
- T = total spread of water in the gutter, ft

Table 7-16. Grate Debris Handling Efficiencies
(Source: USDOT, 1984)

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

Figure 7-42 provides a graphical solution of E_o for either depressed gutter sections or straight cross slopes. The ratio of side flow, Q_s, to total gutter flow can be calculated using Equation 7-31.

Equation 7-31

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

where:

- Q_s = flow capacity of the gutter section above the depressed section, cfs
- Q = total gutter flow, cfs
- Q_w = flow in width W, cfs
- E_o = ratio of frontal flow to total gutter flow

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by Equation 7-32.

Equation 7-32

$$R_f = 1 - 0.09(V - V_o)$$

where:

- R_f = ratio of frontal flow intercepted to total frontal flow
- V = velocity of flow in the gutter, ft/s (using Q from Figure 7-41)
- V_o = gutter velocity where splash-over first occurs, ft/s (from Figure 7-44)

This ratio is equivalent to frontal flow interception efficiency. Figure 7-44 provides a solution of Equation 7-32, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 7-44 is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by Equation 7-33.

Equation 7-33

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

where:

R_s = ratio of side flow intercepted to total side flow
V = velocity of flow in the gutter, ft/s
L = length of the grate, ft

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Figure 7-44. Grate Inlet Frontal Flow Interception Efficiency
(Source: USDOT, 1984)

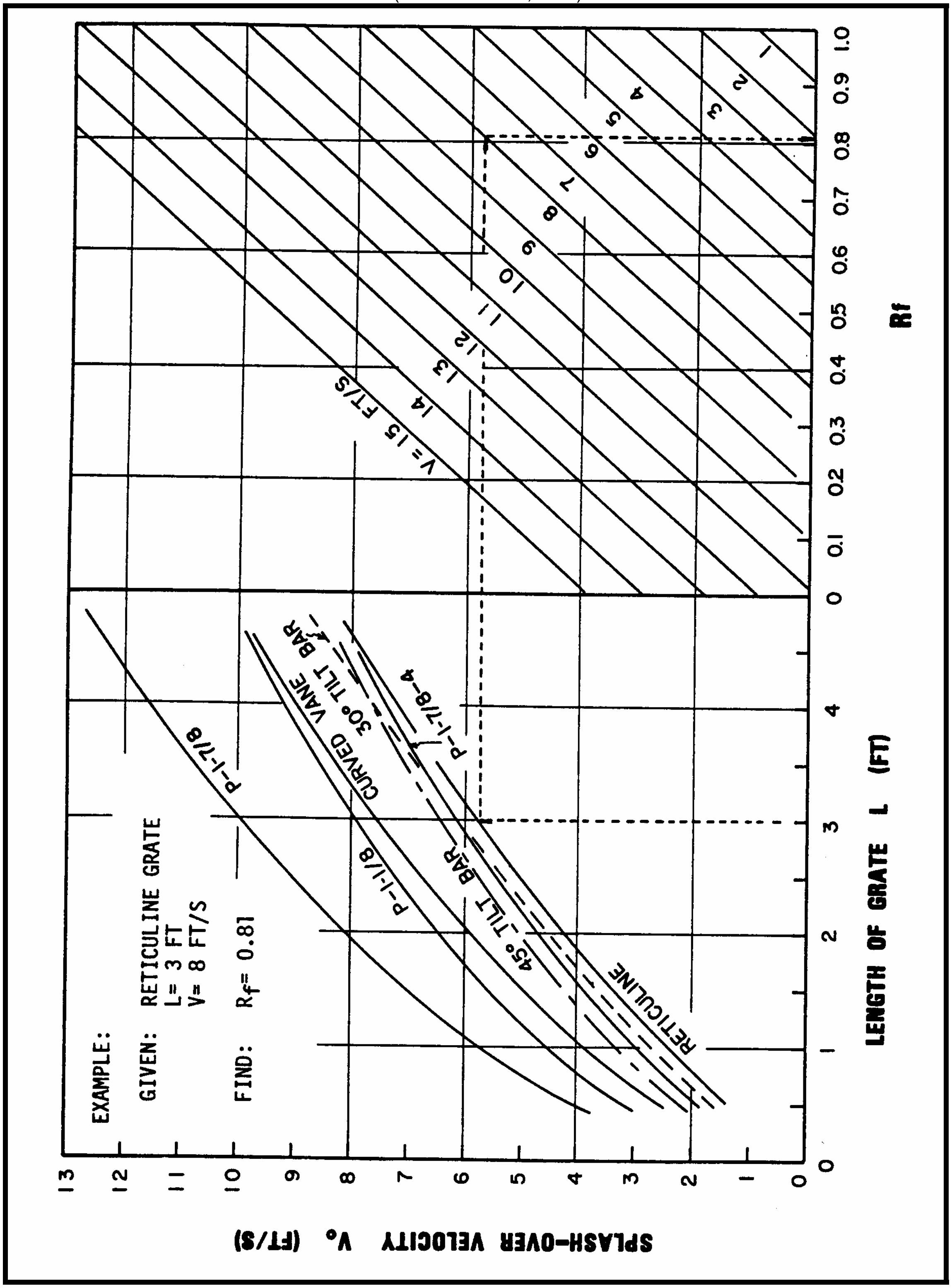
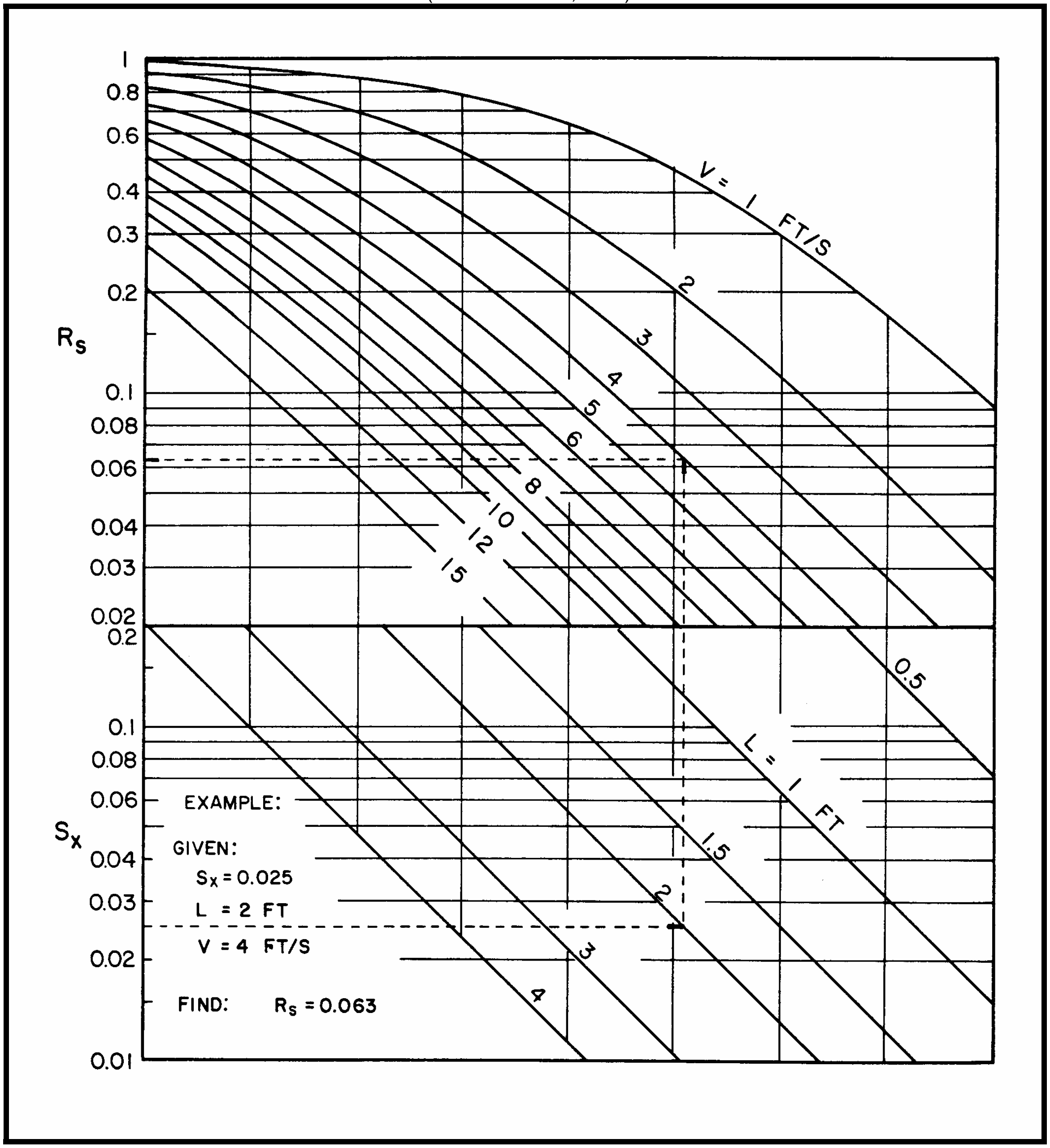


Figure 7-45 provides a solution to Equation 7-33. The efficiency, E, of a grate can be calculated using Equation 7-34.

Figure 7-45. Grate Inlet Side Flow Interception Efficiency
(Source: USDOT, 1984)



Equation 7-34

$$E = R_f E_o + R_s(1 - E_o)$$

where:

E = efficiency of grate opening
R_f = ratio of frontal flow intercepted to total frontal flow
E_o = ratio of frontal flow to total gutter flow
R_s = ratio of side flow intercepted to total side flow

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow, shown in Equation 7-35.

Equation 7-35 $$Q_i = EQ = Q\left[R_f E_o + R_s\left(1 - E_o\right)\right]$$

Example 7-19. Composite Gutter Flow Calculation

Given: W = 2 ft
T = 8 ft
S_X = 0.025 ft/ft   S = 0.01 ft/ft
E_O = 0.69   Q = 3.0 cfs
V = 3.1 ft/s   Gutter depression = 2 in

Find: Interception capacity of: (1) a curved vane grate; and, (2) a 2ft by 2ft reticuline grate

Solution: From Figure 7-44 for Curved Vane Grate, R_f = 1.0
From Figure 7-44 for Curved Vane Grate, R_f = 1.0
From Figure 7-44 for Reticuline Grate, R_f = 1.0
From Figure 7-45 R_s = 0.1 for both grates
From Equation 7-35:
$Q_i = EQ = Q\ [R_fE_o + R_s(1 - E_o)] = 3.0[(1.0)(0.69) + 0.1(1 - 0.69)] = 2.2$ cfs

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

7.6.5.2 Grate Inlets in a Sag

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

The capacity of grate inlets operating as a weir is calculated using Equation 7-36.

Equation 7-36 $$Q_i = 3.0Pd^{1.5}$$

where:

Q_i = the capacity of the grate inlet
P = perimeter of grate excluding bar widths and the side against the curb, ft
d = depth of water above grate, ft

The capacity of grate inlets operating as an orifice is calculated using Equation 7-37.

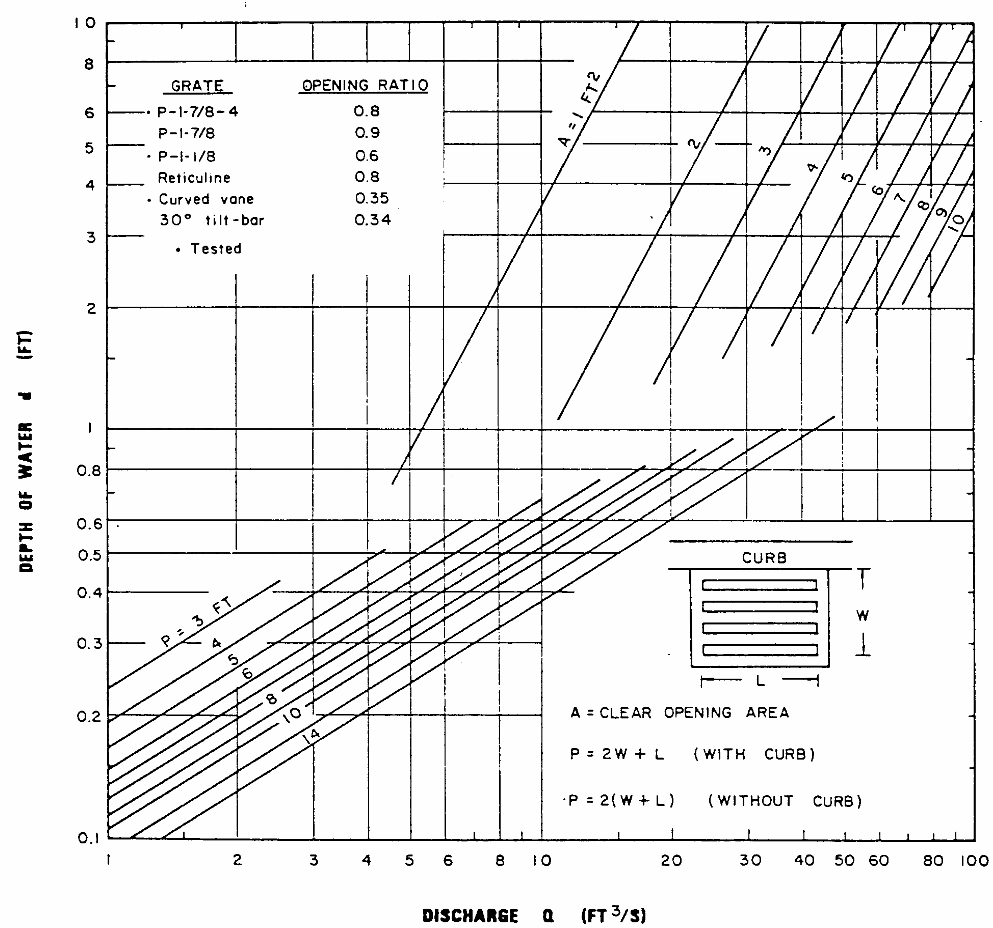
Equation 7-37 $$Q_i = CA\left(2gd\right)^{0.5}$$

where:

Q_i = the capacity of the grate inlet
C = 0.67 orifice coefficient
A = clear opening area of the grate, ft^2
g = 32.2 ft/s^2
d = depth of water above grate, ft

Figure 7-46 is a plot of Equations 7-36 and 7-37 for various grate sizes. The effects of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used. The following example illustrates the use of this figure.

Figure 7-46. Grate Inlet Capacity in Sag Conditions
(Source: USDOT, 1984)



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

Example 7-20. Find Q_i for Curb Opening Inlets

Given: A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate.
Q_b = 3.6 cfs   Q = 8 cfs, 10-year storm
T = 10 ft, design S_x = 0.05 ft/ft   d = TS_x = 0.5 ft

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

Solution: From Figure 7-46, a grate must have a perimeter of 8 ft to intercept 8 cfs at a depth of 0.5 ft.

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

Some assumptions must be made regarding the nature of the clogging in order to compute the capacity of a partially clogged grate. If the area of a grate is 50% covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50%. For example if a 2-ft x 4-ft grate is clogged so that the effective width is 1 ft, then the perimeter, P = 1 + 4 + 1 = 6 ft, rather than 12 ft, the total perimeter, or 4 ft, half of the total perimeter.

The area of the opening would be reduced by 50% and the perimeter by 25%. Therefore, assuming 50% clogging along the length of the grate, a 4 x 4, a 2 x 6, or a 3 x 5 grate would meet requirements of an 8-ft perimeter 50% clogged.

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

For 25-year flow: d = 0.5 ft (from Figure 7-46)

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

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

Q_b = 3.6 cfs, T = 8.2 ft (25-year storm) (from Figure 7-41)

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

7.6.6 Curb Inlet Design

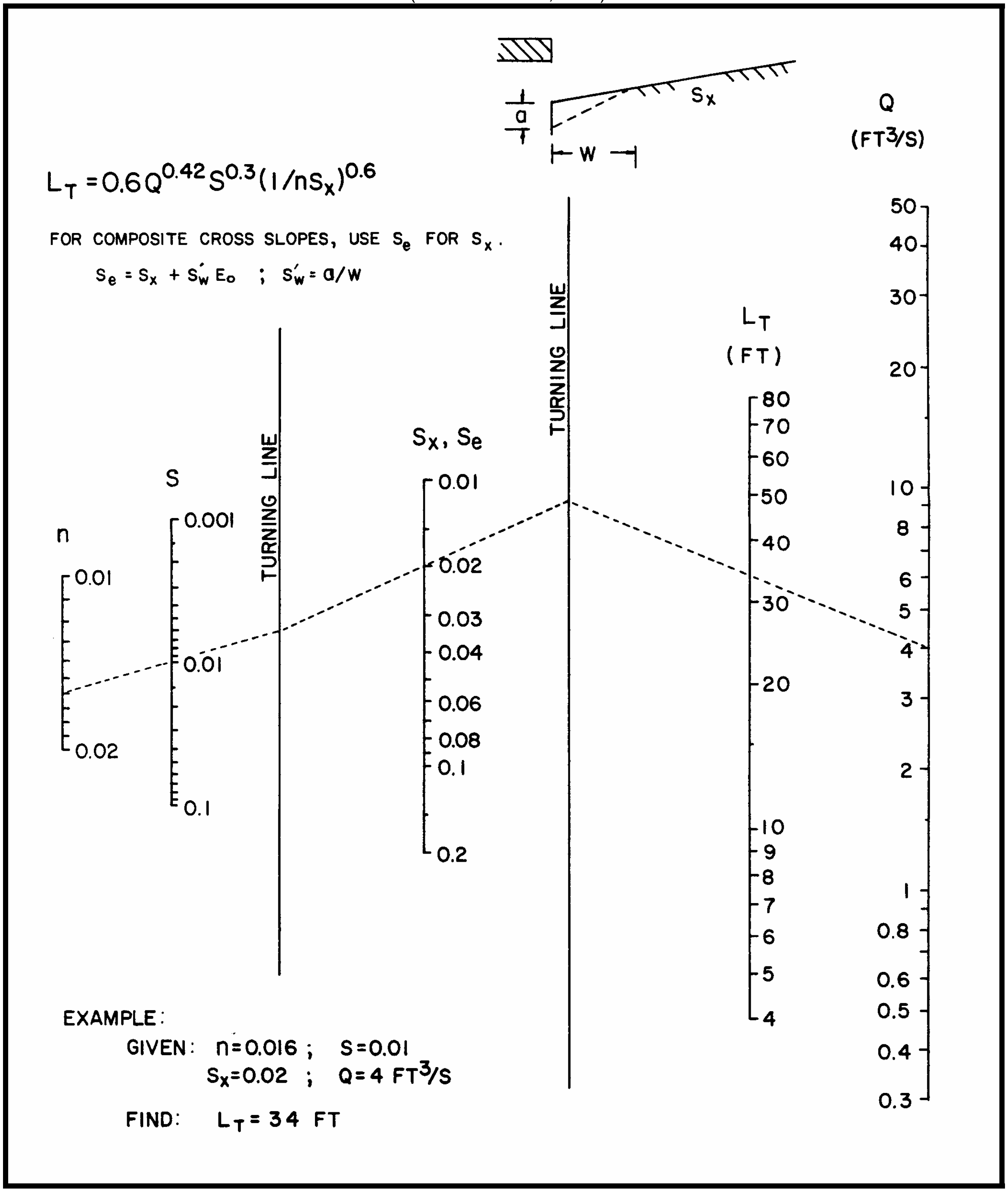
7.6.6.1 Curb Inlets on a Grade

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

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is determined using Figure 7-47. The efficiency of curb-opening inlets shorter than the length required for total interception is determined using Figure 7-48.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in Equation 7-38.

Figure 7-47. Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: USDOT, 1984)

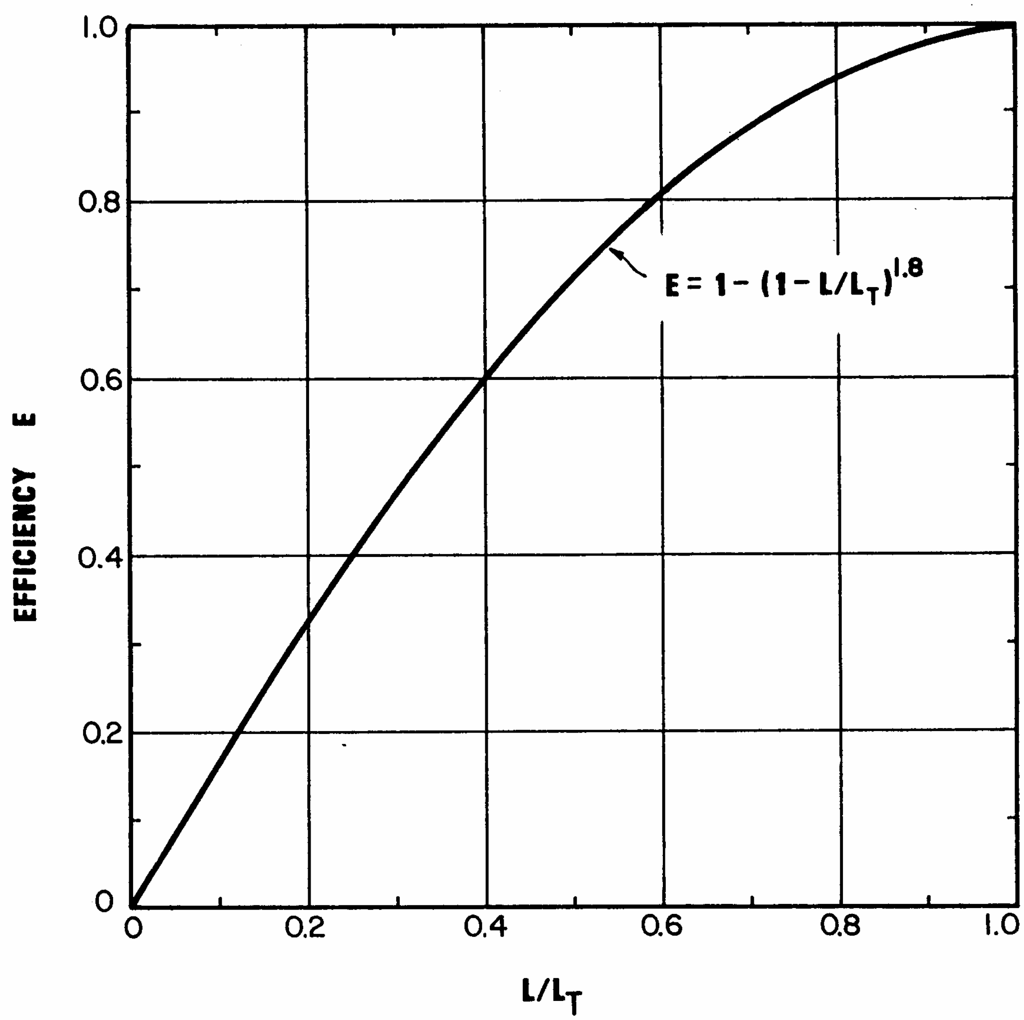


Equation 7-38 $$S_e = S_x + S'_w E_o$$

where:

- S_e = the equivalent cross slope, ft/ft
- S_x = pavement cross slope, ft/ft
- S'_w = cross slope of gutter measured from the cross slope of the pavement, S_x
- S'_w = (a/12W), where: a = gutter depression (inches); W = width of depressed gutter (feet)
- E_o = ratio of flow in the depressed section to total gutter flow

Figure 7-48. Curb-Opening and Slotted Drain Inlet Interception Efficiency
(Source: USDOT, 1984)



It is apparent from examination of Figure 7-47 that the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

Design Steps
Steps for using Figures 7-46 and 7-47 in the design of curb inlets on grade are given below.

(Step 1) Determine the following input parameters:
Cross slope = S_x(ft/ft)
Longitudinal slope = S (ft/ft)
Gutter flow rate = Q (cfs)
Manning's "n" = n
Spread of water on pavement = T (ft) from Figure 7-41

(Step 2) Enter Figure 7-47 using the two vertical lines on the left side labeled n and S. Locate the value for Manning's "n" and longitudinal slope and draw a line connecting these points and extend this line to the first turning line.

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

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

(Step 5) If the curb-opening inlet is shorter than the value obtained in Step 4, Figure 7-48 can be used to calculate the efficiency. Enter the x-axis with the L/L_T ratio and draw a vertical line upward to the E curve. From the point of intersection, draw a line horizontally to the intersection with the y-axis and read the efficiency value.

Example 7-21. Find Q_i for Curb Opening Inlets

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

Find:
(1) Q_i for a 10-ft curb-opening inlet
(2) Q_i for a depressed 10-ft curb-opening inlet with a = 2 in, W = 2 ft, T = 8 ft (Figure 7-41)

Solution:

Step 1.
From Figure 7-47, L_T = 41 ft, $L/L_T = 10/41 = 0.24$
From Figure 7-48, E = 0.39
$Q_i = EQ = (0.39)(5) = 2$ cfs

Step 2.
Find Qn
$Qn = (5.0)(0.016) = 0.08$ cfs
$S_w/S_x = (0.03 + 0.083)/0.03 = 3.77$
T/W = 3.5 (from Figure 7-43)
T = (3.5)(2) = 7 ft
W/T = 2/7 = 0.29 ft
E_o = 0.72 (from Figure 7-42)
$S_e = S_x + S'_w E_o = 0.03 + (0.083)(0.72) = 0.09$
From Figure 7-47, L_T = 23 ft,
$L/L_T = 10/23 = 0.4$
From Figure 7-48, E = 0.64,
$Q_i = EQ = (0.64)(5) = 3.2$ cfs

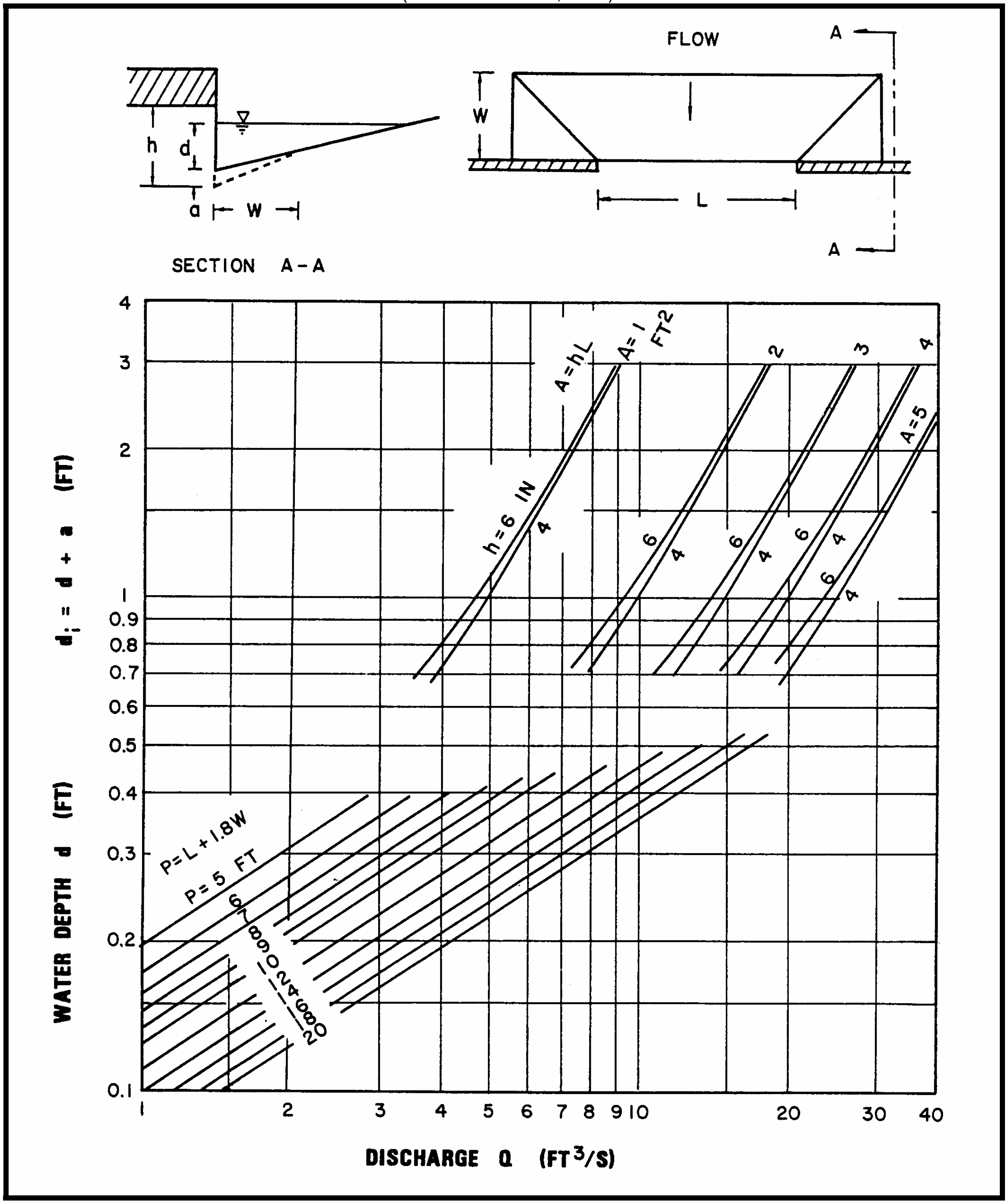
The depressed curb-opening inlet will intercept 1.6 times the flow intercepted by the undepressed curb opening and over 60% of the total flow.

7.6.6.2 Curb Inlets in a Sump

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

The capacity of curb-opening inlets in a sump location can be determined from Figure 7-49, which accounts for the operation of the inlet as a weir and as an orifice at depths greater than 1.4h. This figure is applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter depression. The height (h) in the figure assumes a vertical orifice opening (see sketch on Figure 7-49). The weir portion of Figure 7-49 is valid for a depressed curb-opening inlet when d $\leq$ (h + a/12).

Figure 7-49. Depressed Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO, 1998)



The capacity of curb-opening inlets in a sump location with a vertical orifice opening but without any depression can be determined from Figure 7-50. The capacity of curb-opening inlets in a sump location with other than vertical orifice openings can be determined by using Figure 7-51.

Figure 7-50. Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO, 1998)

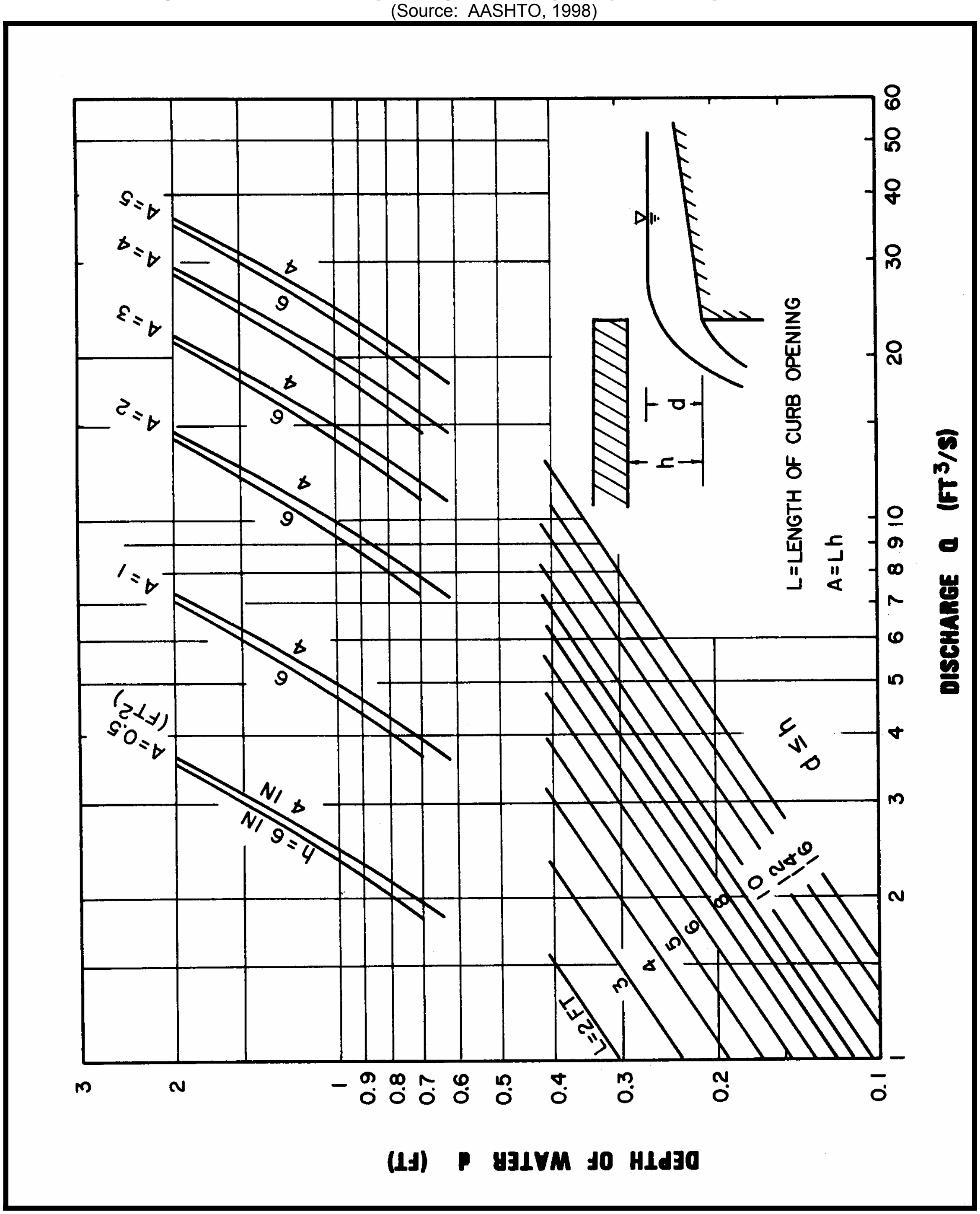
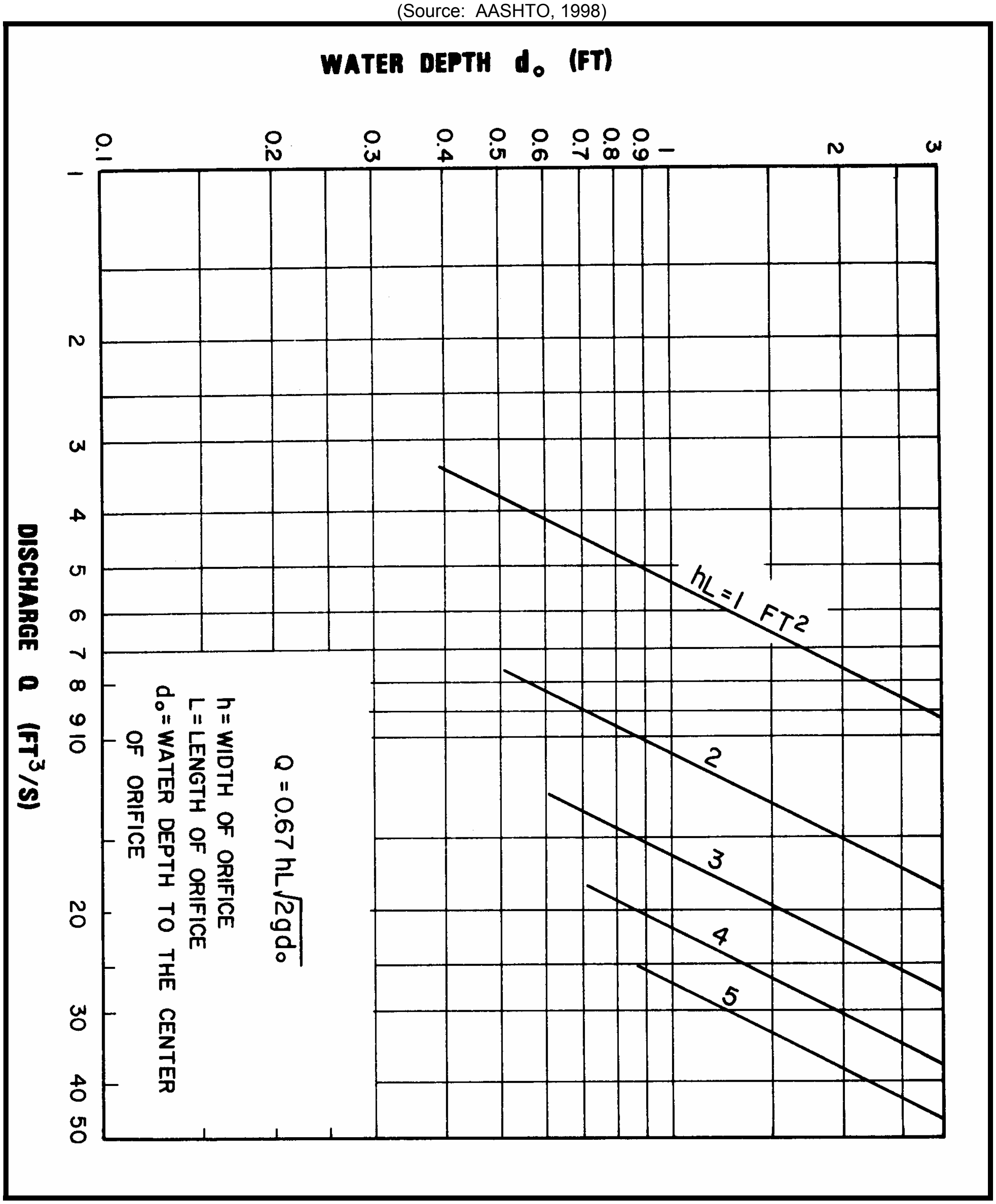


Figure 7-51. Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats

(Source: AASHTO, 1998)



Design Steps

Steps for using Figures 7-48, 7-49, and 7-50 in the design of curb-opening inlets in sump locations are given below.

(Step 1) Determine the following input parameters:
Cross slope = S_x (ft/ft)
Spread of water on pavement = T (ft) from Figure 7-41

Gutter flow rate = Q (cfs) or dimensions of curb-opening inlet [L (ft) and H (in)]
Dimensions of depression if any [a (in) and W (ft)]

(Step 2) To determine discharge given the other input parameters, select the appropriate figure (7-48, 7-49, or 7-50 depending on whether the inlet is in a depression and if the orifice opening is vertical).

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

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

Example 7-23. Calculation of d and P with Curb Inlets

Given: Curb-opening inlet in a sump location
$L = 5$ ft
$h = 5$ in

(1) Undepressed curb opening
$S_x = 0.05$ ft/ft
$T = 8$ ft

(2) Depressed curb opening
$S_x = 0.05$ ft/ft
$a = 2$ in
$W = 2$ ft
$T = 8$ ft

Find: Discharge Q_i

Step 1. Find d
$D = TS_x = (8)(0.05) = 0.4$ ft
$d < h$
From Figure 7-50, $Q_i = 3.8$ cfs

Step 2. $d = 0.4$ ft
$(h + a)/12 = (5 + 2)/12 = 0.58$ ft
since $d < 0.58$ the weir portion of Figure 7-49 is applicable (lower portion of the figure).
$P = L + 1.8W = 5 + 1.8(2) = 8.6$ ft
From Figure 7-49, $Q_i = 5$ cfs

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

7.6.7 Combination Inlets

7.6.7.1 Combination Inlets on a Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of Figures 7-43, 7-44 and 7-45.

7.6.7.2 Combination Inlets in a Sump

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity. Assuming complete clogging of the grate, Figures 7-48, 7-49, and 7-50 for curb-opening inlets should be used for design.

7.7 Bridge Hydraulics

The hydraulics of bridges is an essential part of determining water surface characteristics. Accurate water surface profiles are very important in several applications including flood reduction studies, channel design, stream stability, and scour evaluations. The Tennessee Department of Transportation (TDOT) has published guidance (TDOT, 2001), which is available on the TDOT website at www.tdot.state.tn.us.

7.7.1 Bridge Hydraulics Concepts

There are several unique concepts in hydraulics that occur during water flow through a bridge opening. If the bridge opening causes the floodplain to 'narrow' through the opening, energy will be lost upstream of the bridge and water surface profiles will rise. The end result is that the water surface will be higher upstream of the bridge than it would be if the bridge did not exist. This phenomenon is often described as backwater.

As water approaches a bridge opening during flood conditions, the effective flow width will constrict from its full valley flow (that is unobstructed flow) to a minimum width through the bridge opening. The maximum backwater water surface elevation increase will occur at the point where the water starts to constrict. From this point downstream through the bridge opening, water velocity and friction losses will increase and water surfaces will decrease. Additionally, energy will be lost in this reach as a result of turbulent exchange of momentum seen in contracting flows.

In the bridge opening itself, the flow velocity will reach a maximum and energy losses will be high. In addition to the losses caused by the high velocities, there are also energy losses associated with the interaction of the water with piers and abutments. After flowing through the bridge opening, the water starts to expand again until it reaches the full-width flow conditions. This reach is referred to as the expansion reach. All energy losses that are higher than normal that are associated with the flow through a bridge opening occur between the point of maximum backwater effects to the point of full expansion downstream of the bridge.

7.7.2 Computer Applications

In most applications associated with hydraulic modeling of streams and rivers, an accurate water surface profile is important in the application. As a result, an accurate water surface profile at and around bridges is a requirement. This is often done with computer applications. The most popular are HEC-RAS and HEC-2 developed by USACE and WSPRO developed by FHWA. While two-dimensional models are available, most practical applications require only a one-dimensional model. HEC-RAS, HEC-2 and WSPRO are one-dimensional models.

7.7.2.1 HEC-2

The cross section requirements for HEC-2 are the same as for HEC-RAS mentioned above. HEC-2 utilizes two different methods for modeling water surface profiles through bridges; the normal bridge routine and the special bridge routine.

Normal Bridge
The normal bridge routine treats the bridge section as a natural channel with the area of the bridge below the water surface being subtracted from the total area. Additionally, the wetted perimeter will increase where the water is in contact with the bridge structure. The user must input two cross sections inside the bridge and the bridge deck.

Special Bridge
The special bridge method can be used to calculate profiles for low flow, weir flow, pressure flow, or a combination of these. Various hydraulic formulas are utilized to determine energy changes as

well as the water surface elevations in the bridge structure. Class A, B, and C low flows as well as pressure/weir flow calculations are computed in a similar manner as described in the HEC-RAS section.

7.7.2.2 HEC-RAS

For HEC-RAS one-dimensional modeling of bridge structures, four cross sections are required for accurate analysis. Section 4 is located upstream of the bridge at the point where the water starts contracting toward the bridge opening. Section 3 is located just upstream of the bridge. Section 2 is located just downstream of the bridge. Section 1 is located at the point where the water is fully expanded downstream of the bridge. The following sections will describe the two computer applications in greater detail.

HEC-RAS allows for several different methods of modeling the flow through the bridge. The options include low flow (Class A, B, and C), low flow and weir flow, pressure flow, pressure and weir flow, and highly submerged flows.

LOW FLOWS

Class A Low Flow is defined when all the flow is passing through the bridge opening and is lower than the low chord of the bridge deck. HEC-RAS starts by using the momentum equation to determine the classification of flow. Class A flow occurs when the water surface profile is completely subcritical through the bridge opening. For this type of flow, friction and energy losses must be computed within HEC-RAS to determine the water surface profile. There are three options for computing these losses through the bridge in HEC-RAS:

1) Energy Equation
2) Momentum Balance
3) Yarnell Equation

Class B Low Flow occurs when the flow passes through critical depth in the bridge structure. This can occur either from subcritical flow or from supercritical flow. For subcritical profiles, either the energy or momentum equations can be used to calculate the water surface profiles. For supercritical profiles, the bridge will control the water surface profile upstream of the bridge and either the momentum or energy equation could be used to calculate both the upstream and downstream profiles.

Class C Low Flow occurs when the flow through the bridge structure is entirely supercritical. For this type of flow, either the energy equation or the momentum equation can be used.

HIGH FLOWS

High flows occur when the water surface comes in contact with the low chord of the bridge deck. Once the flow comes into contact with the bridge structure, backwater effects begin (or increase) and orifice flow occurs. There are two methodologies within HEC-RAS to model this type of contact. The energy equation or the Pressure and Weir Flow Method can be used. There are two types of conditions for the pressure flow situation. The first case occurs when only flow at the upstream end of the bridge comes into contact with the bridge structure. The second case occurs when the bridge opening is completely full. HEC-RAS will select the appropriate equation to use. Weir flow occurs when water is flowing over the bridge and roadway. This type of flow is calculated using the standard weir equation.

7.7.2.3 FHWA Method

The WSPRO hydraulic computations rely on 4 cross sections. Section 1 is the approach section, Section 2 is the upstream bridge opening, Section 3 is a duplicate section of Section 2 to represent the downstream bridge opening, Section 3F is the full valley section just downstream of the bridge opening, and Section 4 is the exit section. Sections 1, 3F, and 4 are all unconstricted sections.

WSPRO first calculates the water surface profile for the natural conditions using Sections 4, 3F, and 1. Then it calculates the water surface profile with the bridge in place. This is done using the energy equation using sections 4, 3, 2, and 1. The basis for the energy equation is that the energy at a downstream section must equal the energy of the nearest upstream section minus any losses that occur during that stretch. The program calculates friction losses and expansion losses based on user defined parameters.

Additionally, pressure flow and weir flow are calculated as they are calculated by HEC-RAS (see section 7.7.2.2).

7.7.2.4 Parameters for Bridge Hydraulic Modeling

There are several important parameters when developing hydraulic parameters at bridges including the following:

1) Cross Section Data and Locations: It is important to select the proper location for each cross section per the selected aforementioned model as well as providing accurate ground surfaces at these cross sections. This can be done either from survey data or from topographical data.
2) Manning's "n" Values: Manning's "n" values are important in determining losses and should be applied using aerial photography or field reconnaissance.
3) Channel Bank Stations and Reach Lengths: Channel bank stations refer to applying the correct definition to the channel banks so that Manning's "n" values are applied at the correct horizontal position. Reach lengths refer to assigning the proper linear distance between cross sections.
4) Ineffective flow areas: For sections immediately upstream and downstream of the bridge structure some of the flow is 'dead water'. This refers to water that is not effective flowing through the structure because of the interaction with the bridge structure.
5) Contraction and Expansion Coefficients: These coefficients are used to calculate losses around the bridge structure that occur during the contraction into the bridge and the expansion out of the bridge.

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