Hydraulics:

1.0 Streets and Closed Conduits
2.0 Storage Design
3.0 Open Channels, Culverts, and Bridges
4.0 Energy Dissipation

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1.0 Streets and Closed Conduits

1.1. Stormwater Street and Closed Conduit Design Overview

1.1.1 Stormwater System Design

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

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

1.1.2 System Components

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

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

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

This section is intended to provide design criteria and guidance on several on-site flood control system components; including street and roadway gutters, inlets, and storm drain pipe systems. *Section 2.0* covers storage design and outlet structures. *Section 3.0* covers the design of culverts, vegetated and lined open channels, and bridges. *Section 4.0* covers energy dissipation devices for outlet protection. The rest of this section covers important considerations to keep in mind in the planning and design of stormwater drainage facilities.

1.1.3 Checklist for Planning and Design

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

A. Analyze topography, including:
   1. Check off-site drainage pattern. Where is water coming onto the site? Where is water leaving the site?

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

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

C. Check potential drainage outlets and methods, including:

   On-site (structural control, receiving water)

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

   Natural drainage system (swales)

   Existing drainage system (drain pipe)

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

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

1.1.4 Key Issues in Stormwater System Design

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

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

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

General Design Considerations

- Stormwater systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage pathways within a drainage basin. These natural drainage pathways should only be modified as a last resort to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or stormwater systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or stormwater (minor) systems.
- It is important to ensure that the combined on-site flood control system and major stormwater 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 stormwater systems and/or major stormwater structures occurs during these periods, the risk to life and property could be significantly increased.

- In establishing the layout of stormwater systems, it is essential to ensure that flows are not diverted onto private property during flows up to the major stormwater system design capacity.

Street and Roadway Gutters

- Gutters are efficient flow conveyance structures. This is not always an advantage if removal of pollutants and reduction of runoff is an objective. Therefore, impervious surfaces should be disconnected hydrologically where possible, and runoff should be allowed to flow across pervious surfaces or through vegetated channels. Gutters should be used only after other options have been investigated and only after runoff has had as much chance as possible to infiltrate and filter through vegetated areas.
- It may be possible not to use gutters at all, or to modify them to channel runoff to off-road pervious areas or open channels. For example, curb opening type designs take roadway runoff to smaller feeder grass channels. Care should be taken not to create erosion problems in off-road areas. Protection during construction, establishment of strong stands of vegetation, and active maintenance may be necessary in some areas.
- Use typical road sections that use grass channels or swales instead of gutters to provide for pollution reduction and reduce the impervious area required. Figure 1.1 illustrates a roadway cross section that eliminates gutters for residential neighborhoods. Flow can also be directed to center median strips in divided roadway designs. To protect the edge of pavement, ribbons of concrete can be used along the outer edges of asphalt roads.

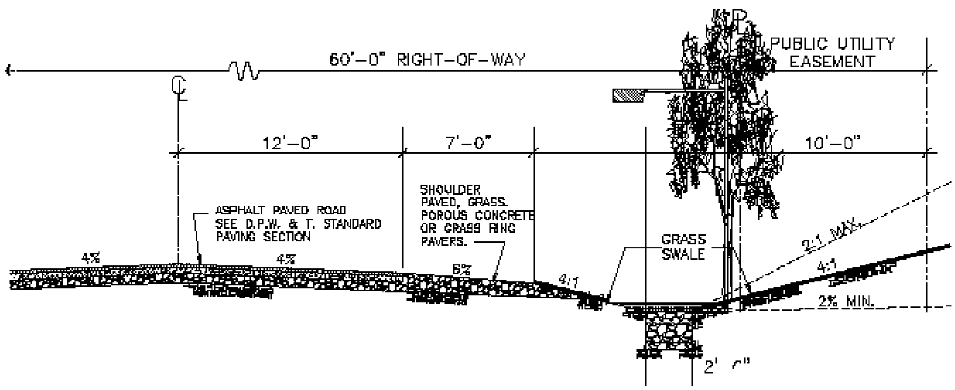


Figure 1.1 Alternate Roadway Section without Gutters
(Source: Prince George's County, MD, 1999)

Inlets and Drains

- Inlets should be located to maximize the overland flow path, take advantage of pervious areas, and seek to maximize vegetative filtering and infiltration. For example, it might be possible to design a parking lot so water flows into vegetated areas prior to entering the nearest inlet.
- Inlet location should not compromise safety or aesthetics. It should not allow for standing water in areas of vehicular or pedestrian traffic, but should take advantage of natural depression storage where possible.
- Inlets should be located to serve as overflows for structural stormwater controls. For example, a bioretention device in a commercial area could be designed to overflow to a catch basin for larger storm events.
- The choice of inlet type should match its intended use. A sumped inlet may be more effective supporting water quality objectives.

- Use several smaller inlets instead of one large inlet in order to:
  - Prevent erosion on steep landscapes by intercepting water before it accumulates too much volume and velocity.
  - Provide a safety factor. If a drain inlet clogs, the other surface drains may pick up the water.
  - Improve aesthetics. Several smaller drains will be less obvious than one large drain.
  - Spacing smaller drain inlets will give surface runoff a better chance of reaching the drain. Water will have to travel farther to reach one large drain inlet.

Closed Conduit Systems (Storm Drains/Sewers)

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

1.2 On-Site Flood Control System Design

1.2.1 Introduction

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

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

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

Ditch, channel and swale design criteria and guidance are covered in *Section 3.0.*

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

1.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 1.1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 1.1 Symbols and Definitions

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

1.2.3 Street and Roadway Gutters

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

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

Formula

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

$$Q = [0.56/n]\ S_x^{5/3}\ S^{1/2}\ T^{8/3} \qquad (1.1)$$

where:

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

Equation 1.1 was first presented by C.F. Izzard in 1946.

Nomograph

Figure 1.2 is a nomograph for solving Equation 1.1. Manning's n values for various pavement surfaces are presented in Table 1.2 below. Note: the nomograph will not work on slopes steeper than 3% - 4% for 2 and 3 lane thoroughfares. Also, the "Q" in the nomograph is only for n = 0.016.

Manning's n Table

Table 1.2 Manning's n Values for Street and Pavement Gutters

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

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

Uniform Cross Slope

The nomograph in Figure 1.2 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.

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

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

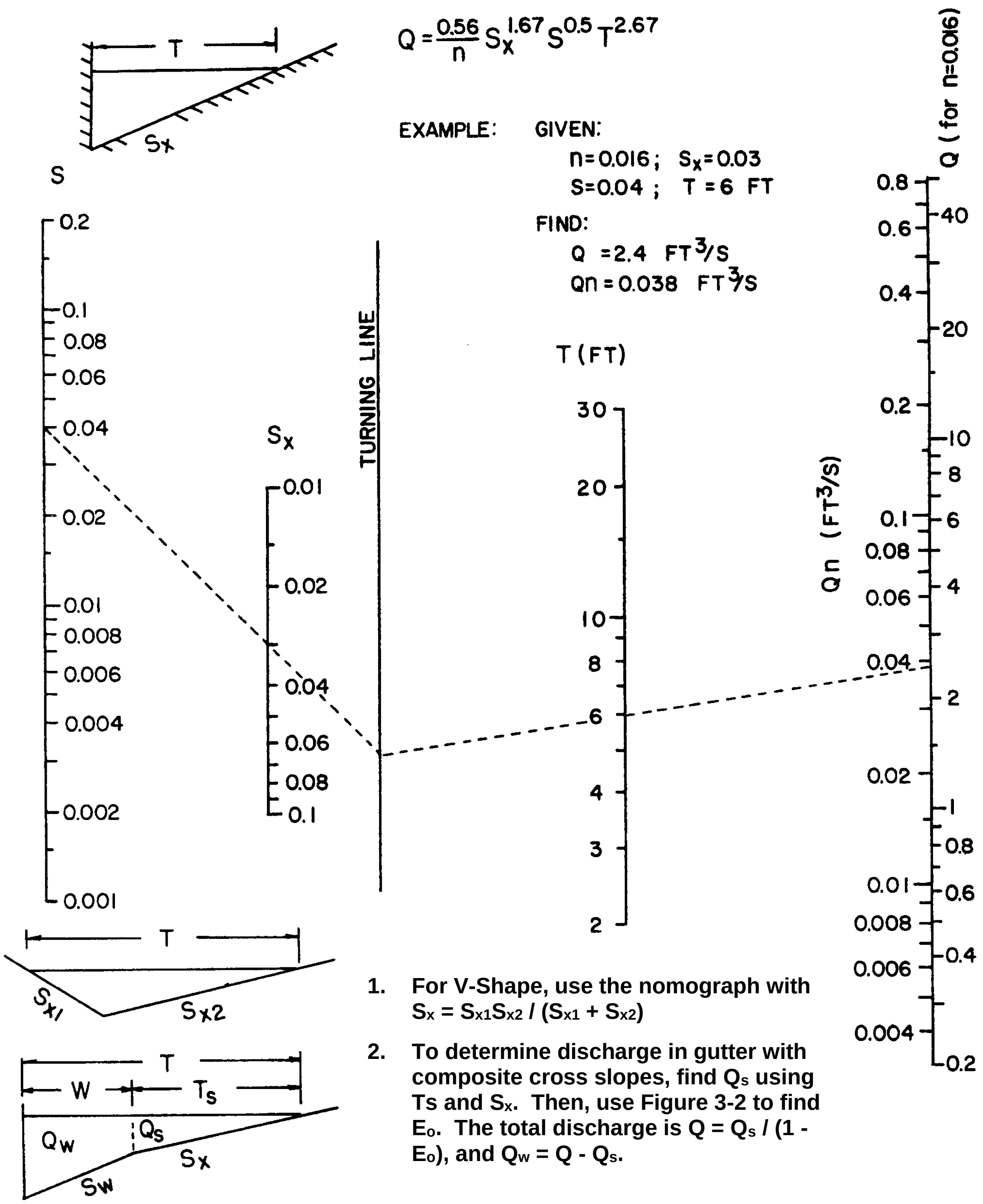
Condition 2: Find gutter flow, given spread.

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

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

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

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



1. **For V-Shape, use the nomograph with $S_x = S_{x1}S_{x2} / (S_{x1} + S_{x2})$**
2. **To determine discharge in gutter with composite cross slopes, find Q_s using Ts and S_x. Then, use Figure 3-2 to find E_o. The total discharge is $Q = Q_s / (1 - E_o)$, and $Q_w = Q - Q_s$.**

Figure 1.2 Flow in Triangular Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

Composite Gutter Sections

Figure 1.3 in combination with Figure 1.2 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 1.4 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 1.4 involves a complex graphical solution of the equation for flow in a composite gutter section. Typical of graphical solutions, extreme care in using the figure is necessary to obtain accurate results.

Condition 1: Find spread, given gutter flow.

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

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

$$\mathbf{Q_w = Q - Q_s} \qquad \textbf{(1.2)}$$

Step 3 Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 1.3 to find an appropriate value of W/T.

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

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

Step 6 Use the value of T_s from Step 5 along with Manning's n, S, and S_x to find the actual value of Q_s from Figure 1.2.

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

Condition 2: Find gutter flow, given spread.

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

Step 2 Use Figure 1.2 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes, substituting T_s for T.

Step 3 Calculate the ratios W/T and S_w/S_x, and, from Figure 1.3, find the appropriate value of E_o (the ratio of Q_w/Q).

Step 4 Calculate the total gutter flow using the equation:

$$\mathbf{Q = Q_s / (1 - E_o)} \qquad \textbf{(1.3)}$$

where:

Q = gutter flow rate, cfs

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

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

Step 5 Calculate the gutter flow in width (W), using Equation 1.2.

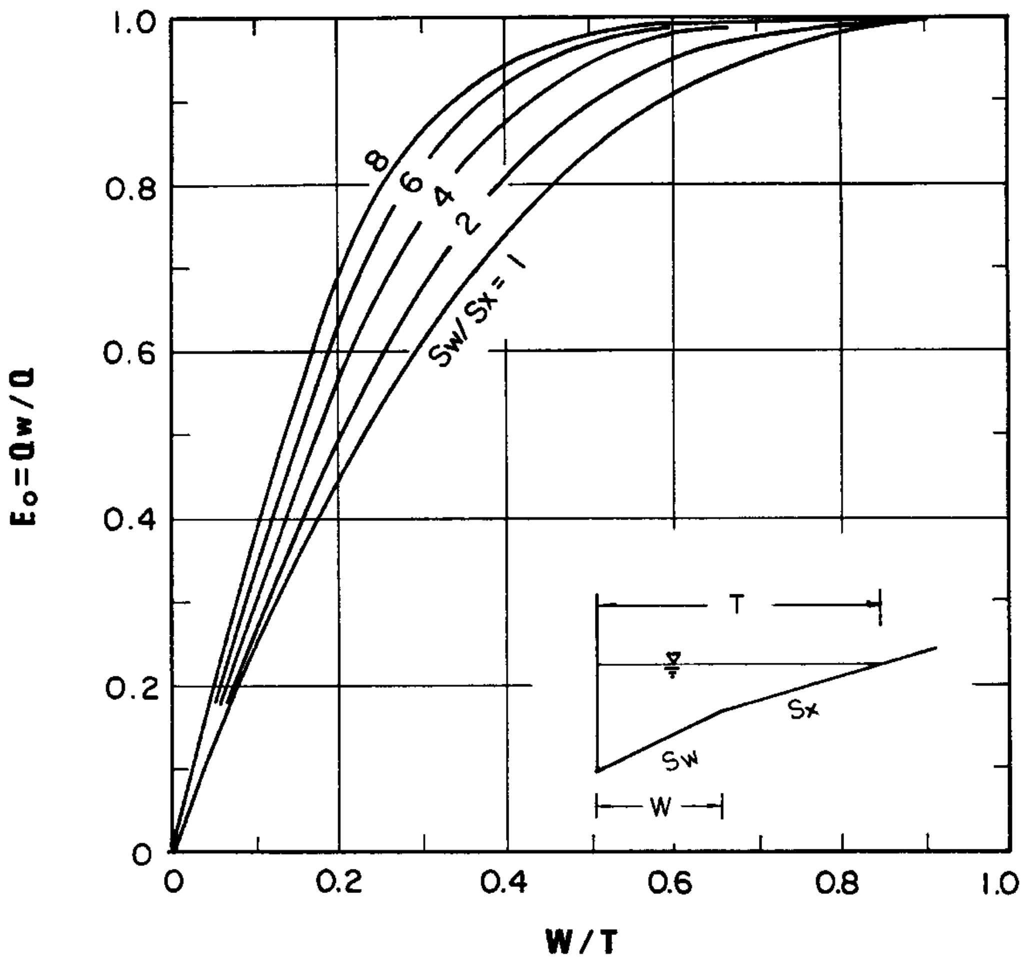


Figure 1.3 Ratio of Frontal Flow to Total Gutter Flow
(Source: AASHTO Model Drainage Manual, 1991)

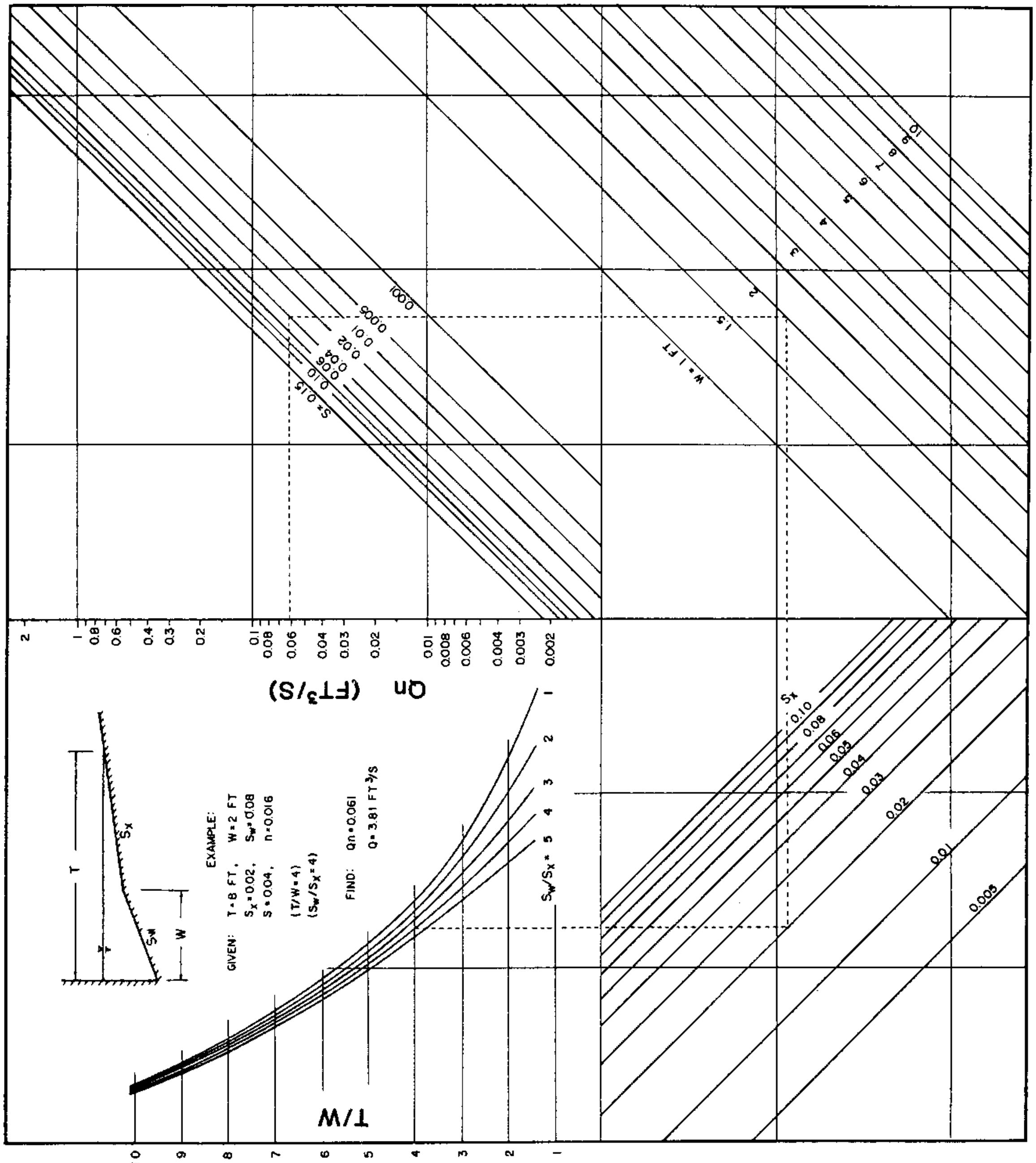


Figure 1.4 Flow in Composite Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

Examples

Example 1

Given:

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

Find:

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

Solution:

a. From Figure 1.2, $Qn = 0.03$
$Q = Qn/n = 0.03/0.015 = 2.0$ cfs

b. $T = 8 - 2 = 6$ ft
$(Qn)_2 = 0.014$ (Figure 1.2) (flow in 6-foot width outside of width (W))
$Q = 0.014/0.015 = 0.9$ cfs
$Q_w = 2.0 - 0.9 = 1.1$ cfs

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

Example 2

Given:

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

Find:

Flow in the composite gutter

Solution:

1. Use Figure 1.2 to find the gutter section capacity above the depressed section.
$Q_s n = 0.038$
$Q_s = 0.038/0.014 = 2.7$ cfs
2. Calculate $W/T = 1.5/6 = 0.25$ and
$S_w/S_x = 0.0833/0.03 = 2.78$
Use Figure 1.3 to find $E_o = 0.64$
3. Calculate the gutter flow using Equation 1.3
$Q = 2.7/(1 - 0.64) = 7.5$ cfs
4. Calculate the gutter flow in width, W, using Equation 1.2
$Q_W = 7.5 - 2.7 = 4.8$ cfs

Parabolic Cross Slope

The FHWA publication "Urban Drainage Design Manual" (HEC-22) should be consulted for parabolic and other shape roadway sections.

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

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

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

The design of grate inlets will be discussed in *Section 1.2.6*, curb inlet design in *Section 1.2.7*, and combination inlets in *Section 1.2.8*.

1.2.5 Grate Inlet Design

Grate Inlets on Grade

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

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. They also handle debris better than other grate inlets but the vanes of the grate must be turned in the proper direction. Where debris is a problem, consideration should be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was made to qualitatively simulate field conditions. Table 1.6 presents the results of debris handling efficiencies of several grates. Debris handling efficiencies were based on the total number of simulated leaves arriving at the grate and the number passed.

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

$$E_o = Q_w/Q = 1 - (1 - W/T)^{2.67} \quad (1.6)$$

where:

Q = total gutter flow, cfs

Q_W = flow in width W, cfs

W = width of depressed gutter or grate, ft

T = total spread of water in the gutter, ft

Table 1.6 Grate Debris Handling Efficiencies

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

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

Figure 1.3 provides a graphical solution of E_o for either depressed gutter sections or straight cross slopes. The ratio of side flow, Q_s, to total gutter flow is:

$$Q_s/Q = 1 - Q_w/Q = 1 - E_o \quad (1.7)$$

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

$$R_f = 1 - 0.09\,(V - V_0) \quad (1.8)$$

where:

V = velocity of flow in the gutter, ft/s (using Q from Figure 1.2)

V_o = gutter velocity where splash-over first occurs, ft/s (from Figure 1.5)

This ratio is equivalent to frontal flow interception efficiency. Figure 1.5 provides a solution of Equation 1.8, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 1.5 is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by:

$$R_s = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})] \quad (1.9)$$

where:

L = length of the grate, ft

Figure 1.5 provides a solution to Equation 1.9.

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

$$E = R_fE_o + R_s(1 - E_o) \quad (1.10)$$

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

$$Q_i = EQ = Q[R_f E_o + R_s(1 - E_o)] \qquad (1.11)$$

The following example illustrates the use of this procedure.

Given:

$W = 2$ ft

$T = 8$ ft

$S_x = 0.025$ ft/ft

$S = 0.01$ ft/ft

$E_o = 0.69$

$Q = 3.0$ cfs

$V = 3.1$ ft/s

Gutter depression = 2 in

Find:

Interception capacity of:

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

Solution:

From Figure 1.5 for Curved Vane Grate, $R_f = 1.0$

From Figure 1.5 for Reticuline Grate, $R_f = 1.0$

From Figure 1.6 $R_s = 0.1$ for both grates

From Equation 1.11:

$Q_i = 3.0[1.0 \text{ X } 0.69 + 0.1(1 - 0.69)] = 2.2$ cfs

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

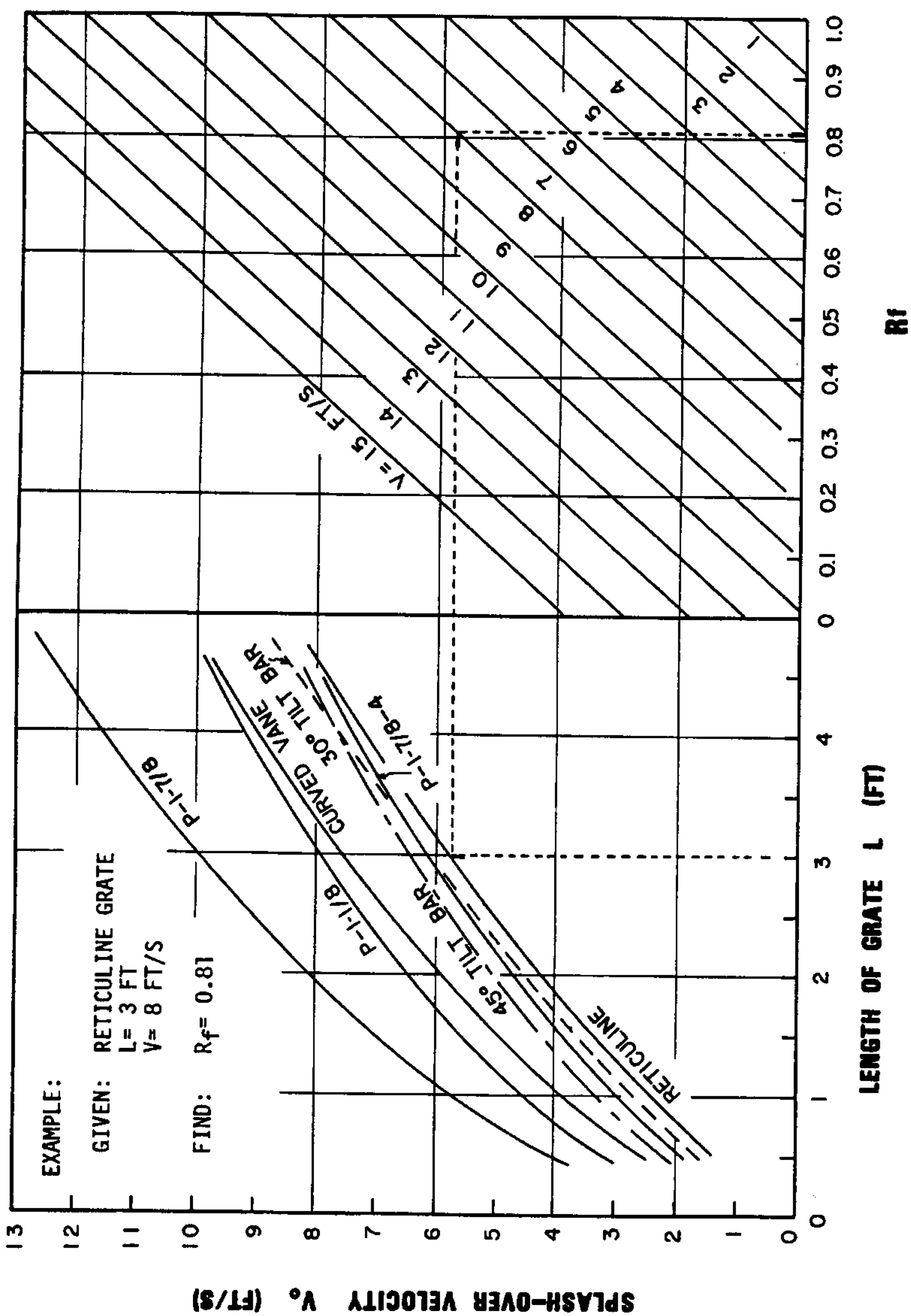


Figure 1.5 Grate Inlet Frontal Flow Interception Efficiency
(Source: HEC-12, 1984)

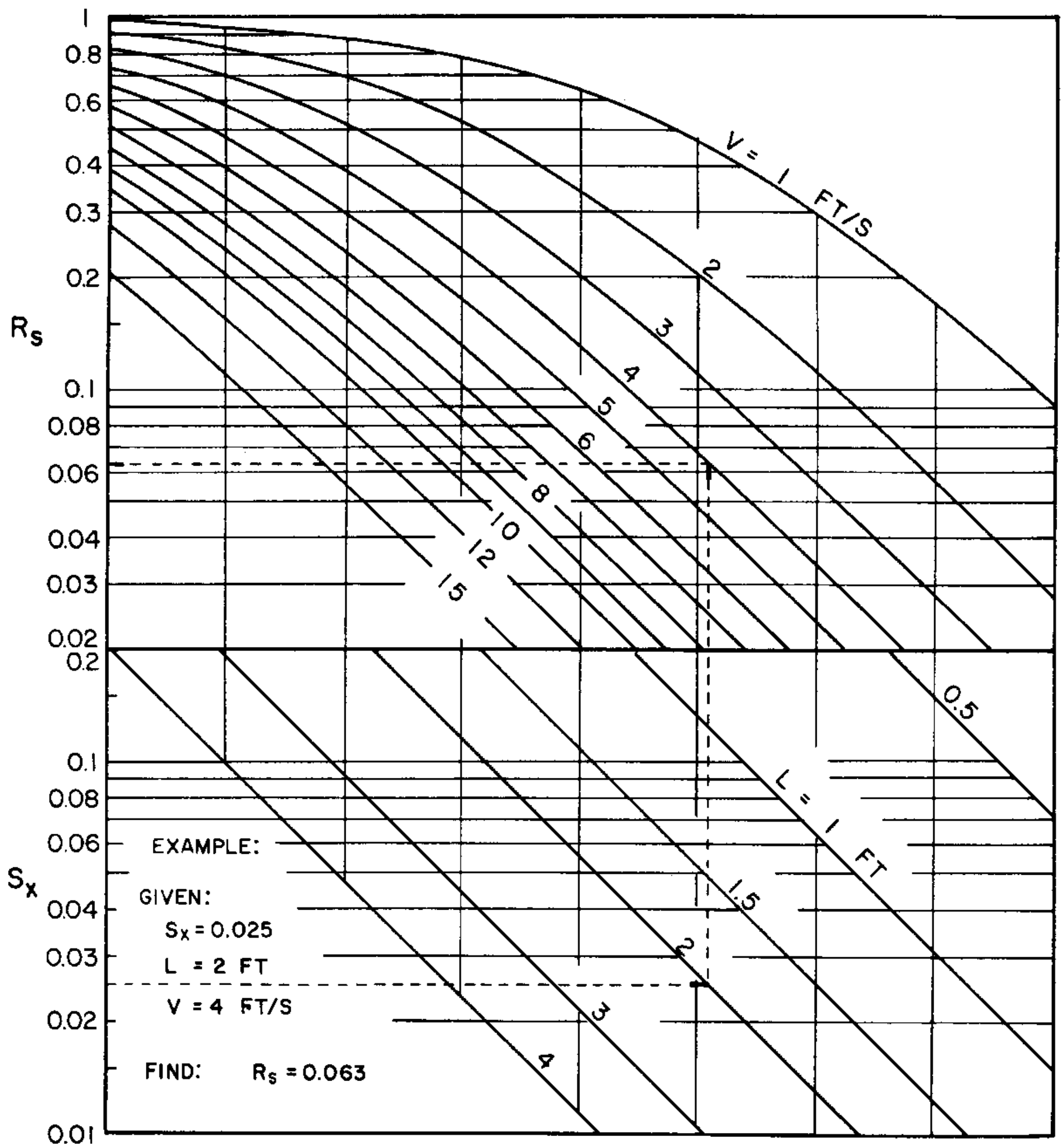


Figure 1.6 Grate Inlet Side Flow Interception Efficiency
(Source: HEC-12, 1984)

Grate Inlets in Sag

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

The capacity of grate inlets operating as a weir is:

$$\mathbf{Q_i = CPd^{1.5}} \qquad \textbf{(1.12)}$$

where:

P = perimeter of grate excluding bar widths and the side against the curb, ft

C = 3.0

d = depth of water above grate, ft

and as an orifice is:

$$\mathbf{Q_i = CA(2gd)^{0.5}} \qquad \textbf{(1.13)}$$

where:

C = 0.67 orifice coefficient

A = clear opening area of the grate, ft^2

g = 32.2 ft/s^2

Figure 1.7 is a plot of Equations 1.12 and 1.13 for various grate sizes. The effect of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used. The following example illustrates the use of this figure.

Given:

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

Q_b = 3.6 cfs

Q = 8 cfs, 25-year storm

T = 10 ft, design

S_x = 0.05 ft/ft

d = TS_x = 0.5 ft

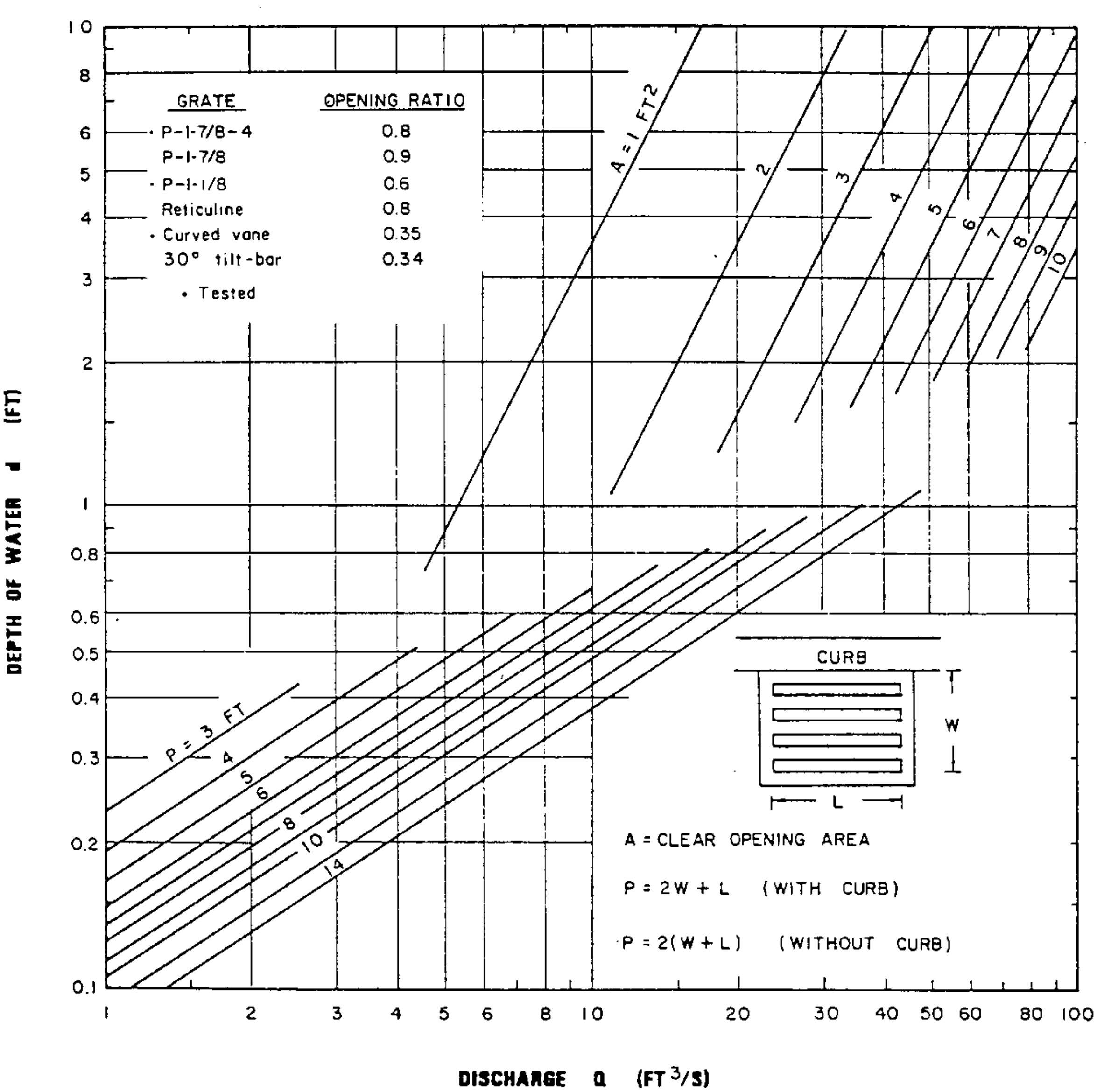
Find:

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

Solution:

From Figure 1.7, a grate must have a perimeter of 8 ft to intercept 8 cfs at a depth of 0.5 ft.

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



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

Figure 1.7 Grate Inlet Capacity in Sag Conditions
(Source: HEC-12, 1984)

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

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

For 25-year flow: d = 0.5 ft (from Figure 1.7)

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

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

Q = 3.6 cfs, T = 8.2 ft (25-year storm) (from Figure 1.2)

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

1.2.6 Curb Inlet Design

Curb Inlets on Grade

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

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is determined using Figure 1.8. The efficiency of curb-opening inlets shorter than the length required for total interception is determined using Figure 1.9.

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

$$S_e = S_x + S'_w E_o \qquad (1.14)$$

where:

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

S'_w = cross slope of gutter measured from the cross slope of the pavement, S_x (ft/ft)

S'_w = (a/12W)

where:

a = gutter depression, in

W = width of depressed gutter, ft

It is apparent from examination of Figure 1.8 that the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

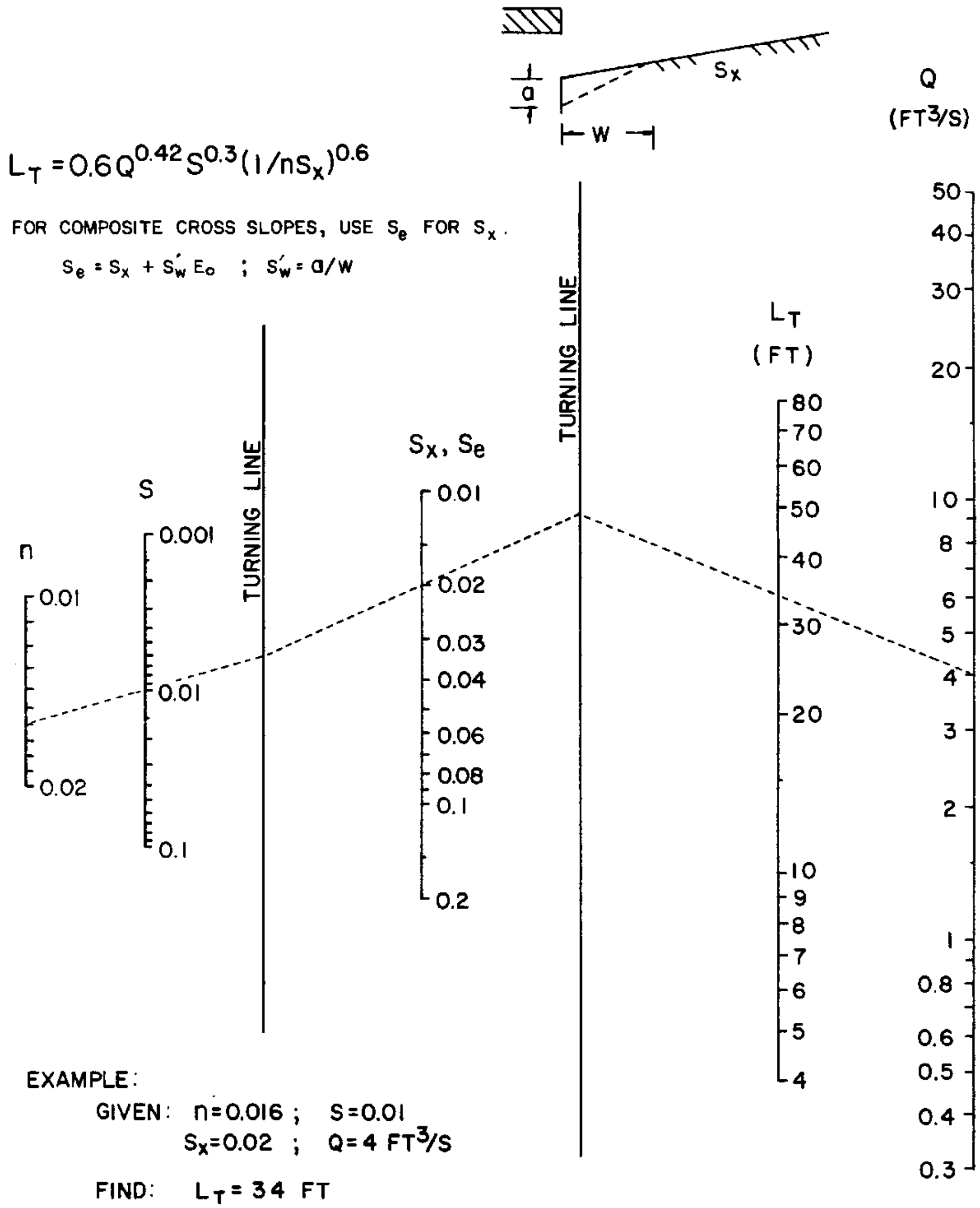


Figure 1.8 Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: HEC-12, 1984)

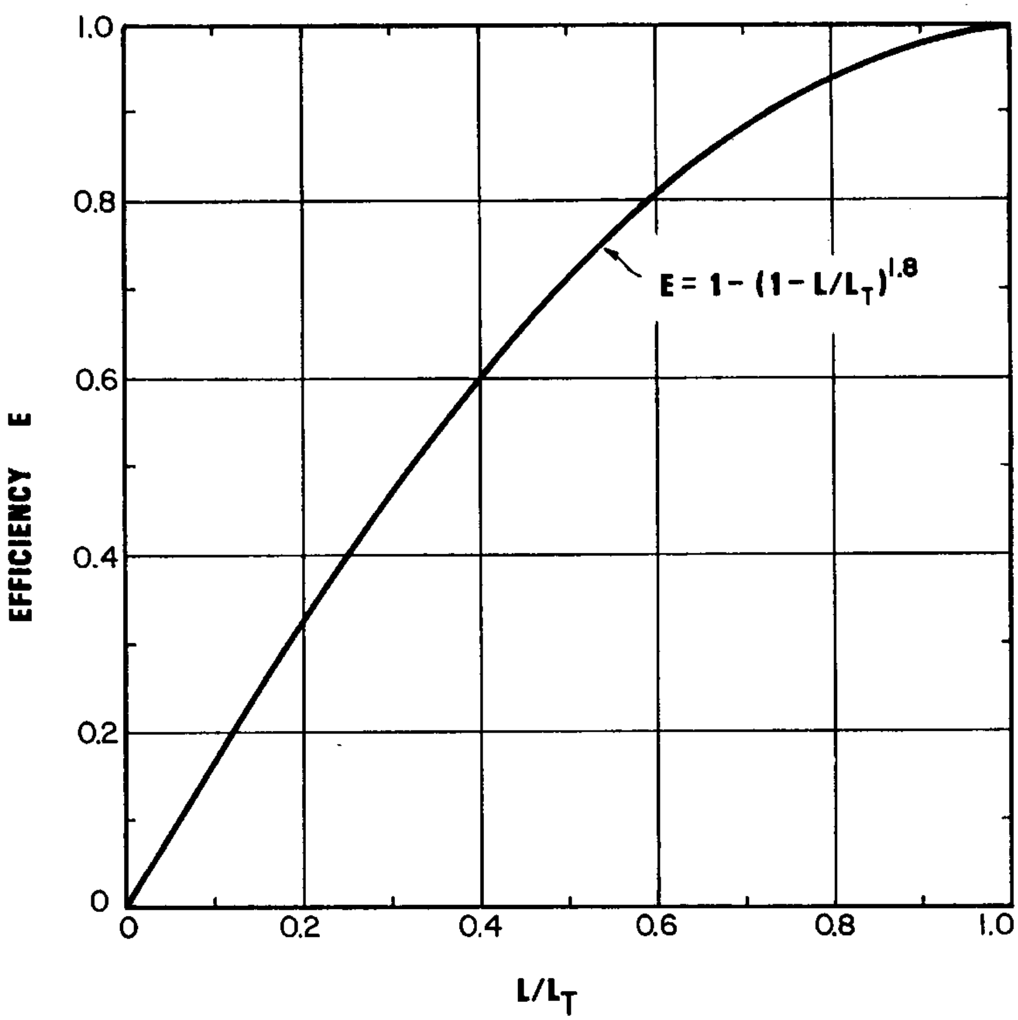


Figure 1.9 Curb-Opening and Slotted Drain Inlet Interception Efficiency
(Source: HEC-12, 1984)

<u>Design Steps</u>

Steps for using Figures 1.8 and 1.9 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 1.2

Step 2 Enter Figure 1.8 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 1.9 can be used to calculate the efficiency. Enter the x-axis with the L/L_T ratio and draw a vertical line upward to the E curve. From the point of intersection, draw a line horizontally to the intersection with the y-axis and read the efficiency value.

Example

Given:

S_x = 0.03 ft/ft

n = 0.016

S = 0.035 ft/ft

Q = 5 cfs

S'_w = 0.083 (a = 2 in, W = 2 ft)

Find:

1. Q_i for a 10-ft curb-opening inlet
2. Q_i for a depressed 10-ft curb-opening inlet with a = 2 in, W = 2 ft, T = 8 ft (Figure 1.2)

Solution:

1. From Figure 1.8, L_T = 41 ft, L/L_T = 10/41 = 0.24
   From Figure 1.9, E = 0.39, Q_i = EQ = 0.39 x 5 = 2 cfs
2. Qn = 5.0 x 0.016 = 0.08 cfs
   S_w/S_x = (0.03 + 0.083)/0.03 = 3.77
   T/W = 3.5 (from Figure 1.4)
   T = 3.5 x 2 = 7 ft
   W/T = 2/7 = 0.29 ft
   E_o = 0.72 (from Figure 1.3)
   Therefore, $S_e = S_x + S'_wE_o$ = 0.03 + 0.083(0.72) = 0.09

   From Figure 1.8, L_T = 23 ft, L/L_T = 10/23 = 0.43
   From Figure 1.9, E = 0.64, Q_i = 0.64 x 5 = 3.2 cfs

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

Curb Inlets in Sump

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

The capacity of curb-opening inlets in a sump location can be determined from Figure 1.10, 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 1.10). The weir portion of Figure 1.10 is valid for a depressed curb-opening inlet when d $\leq$ (h + a/12).

The capacity of curb-opening inlets in a sump location with a vertical orifice opening but without any depression can be determined from Figure 1.11. The capacity of curb-opening inlets in a sump location with other than vertical orifice openings can be determined by using Figure 1.12.

Design Steps

Steps for using Figures 1.10, 1.11, and 1.12 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 1.2
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 (1.10, 1.11, or 1.12 depending on whether the inlet is in a depression and if the orifice opening is vertical).

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

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

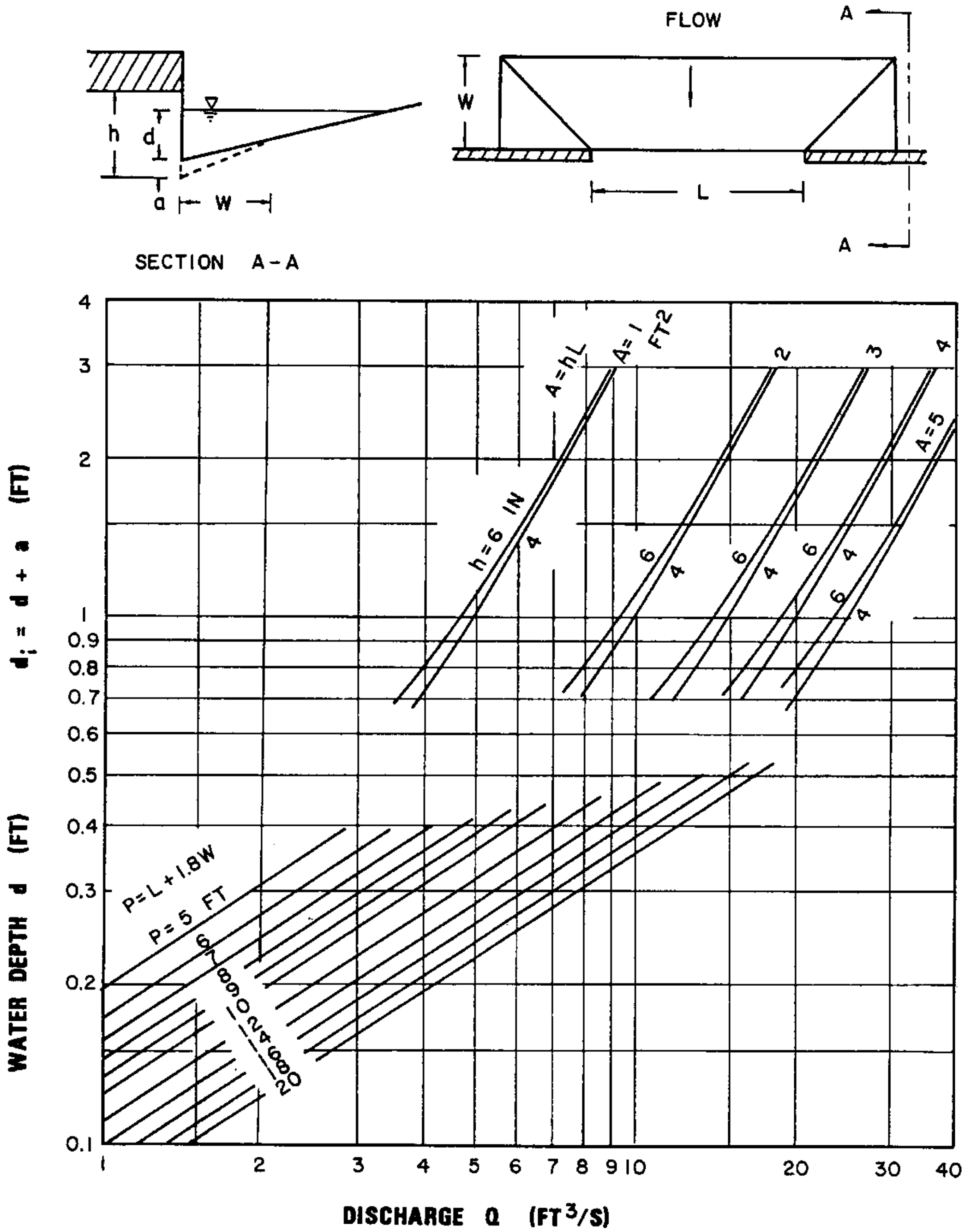


Figure 1.10 Depressed Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

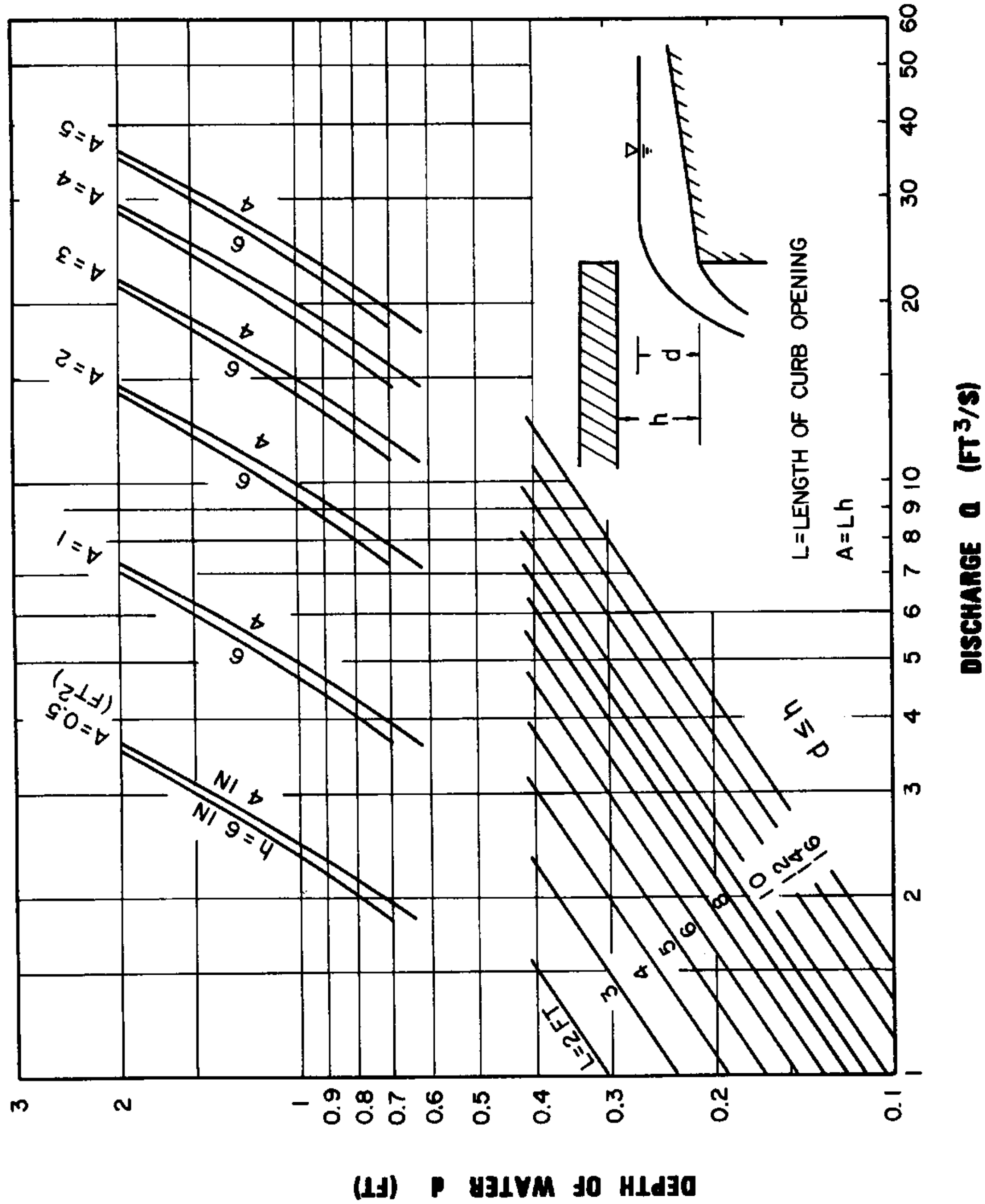


Figure 1.11 Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

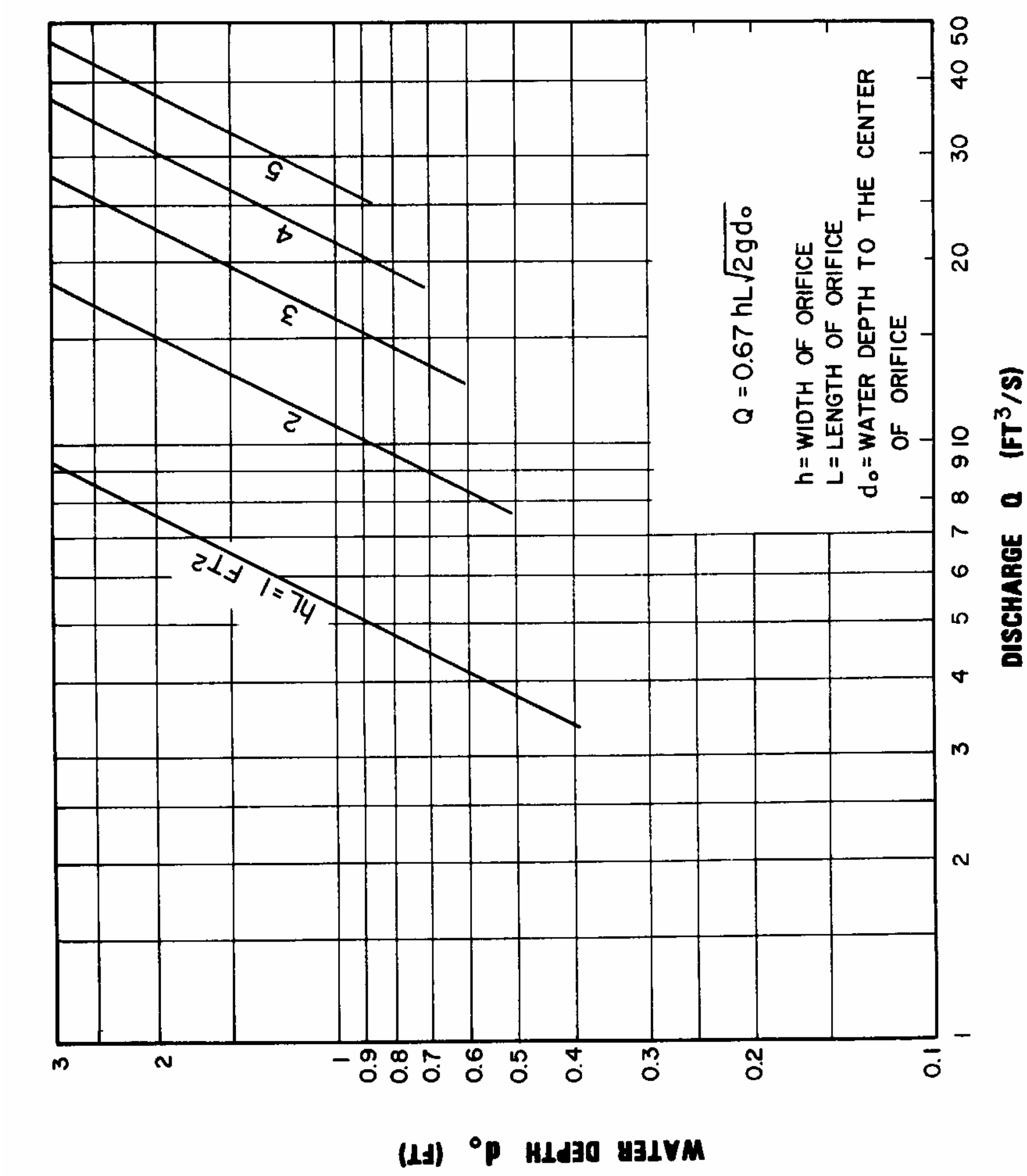


Figure 1.12 Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats
(Source: AASHTO Model Drainage Manual, 1991)

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

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

Find:
Discharge Q_i

Solution:
1. $d = TS_x = 8 \times 0.05 = 0.4$ ft
   $d < h$
   From Figure 10, $Q_i = 3.8$ cfs
2. $d = 0.4$ ft
   $h + a/12 = (5 + 2/12)/12 = 0.43$ ft

   since $d < 0.43$ the weir portion of Figure 1.10 is applicable (lower portion of the figure).

   $P = L + 1.8W = 5 + 3.6 = 8.6$ ft
   From Figure 1.9, $Q_i = 5$ cfs

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

1.2.7 Combination Inlets

Combination Inlets on Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of Figures 1.5, 1.6, and 1.7.

Combination Inlets in Sump

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, 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 1.10, 1.11, and 1.12 for curb-opening inlets should be used for design.

1.2.8 Closed Conduit Systems (Storm Drains/ Sewers)

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

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

Appurtenant structures serve many functions. Inlets, access holes, and junction chambers are presented in this section.

Inlets

The primary function is to allow surface water to enter the closed conduit system. Inlet structures may also serve as access points for cleaning and inspection. Typical inlets structures are a standard drop inlet, catch basin, curb inlet, combination inlet, and Y inlet. (See Figures 1.13 and 1.14).

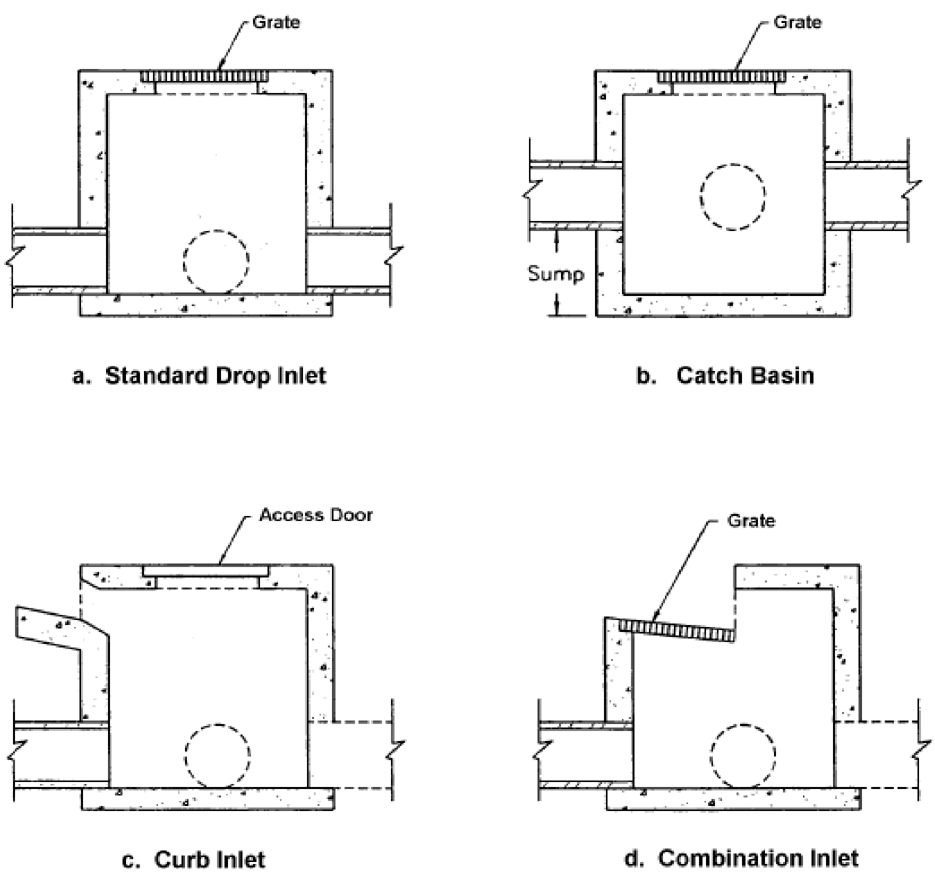


Figure 1.13 Inlet Structures
(HEC 22, 2001)

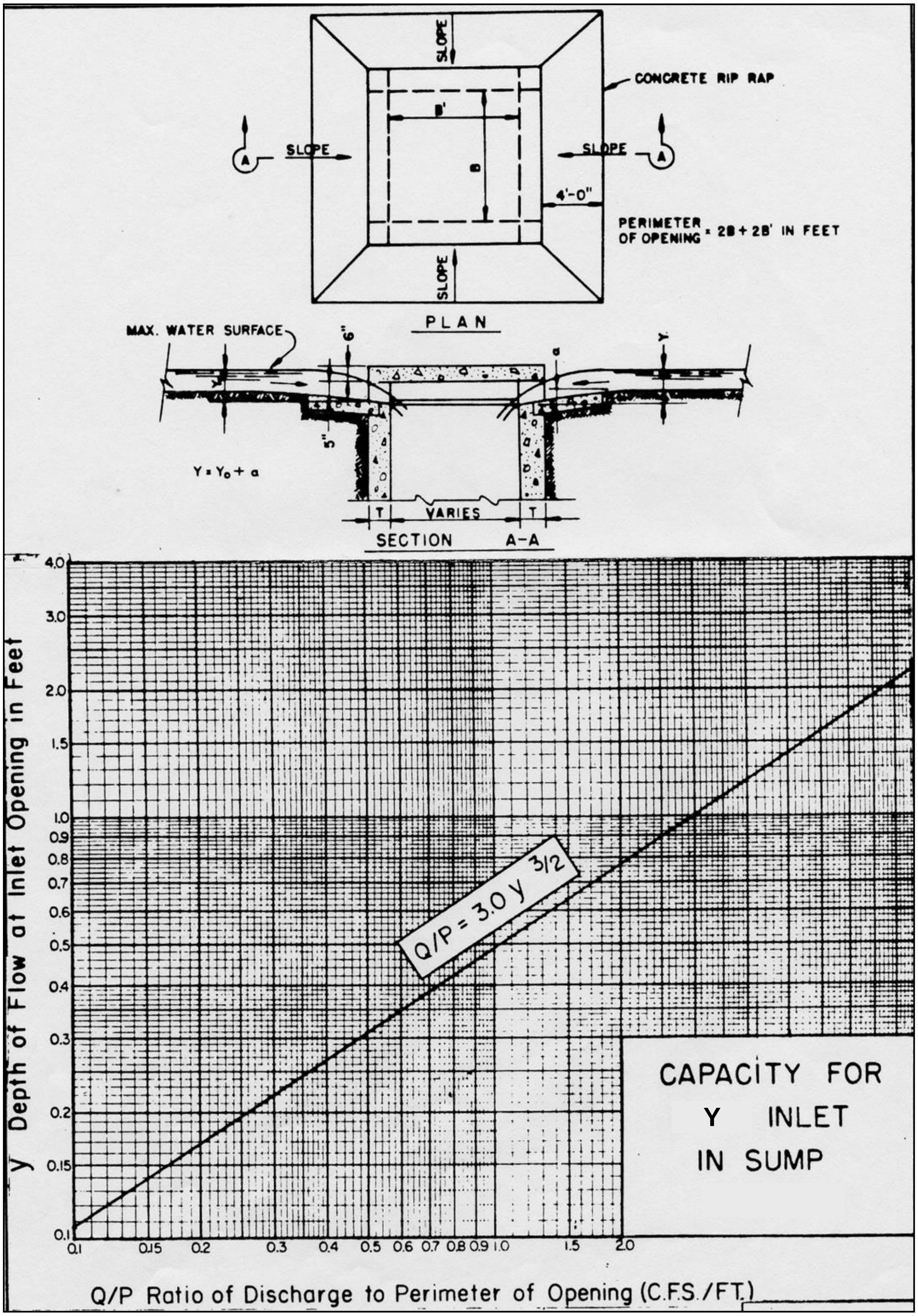


Figure 1.14 Capacity for Y Inlet in Sump
(Fort Worth, 1967)

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

Access Holes (Manholes)

The primary function of an access hole is to provide access to the closed conduit system. An access hole can also serve as a flow junction and can provide ventilation and pressure relief. Typical access holes are shown in Figures 1.15 and 1.16 (HEC 22, 2001). The materials commonly used for access hole construction are precast concrete and cast-in-place concrete.

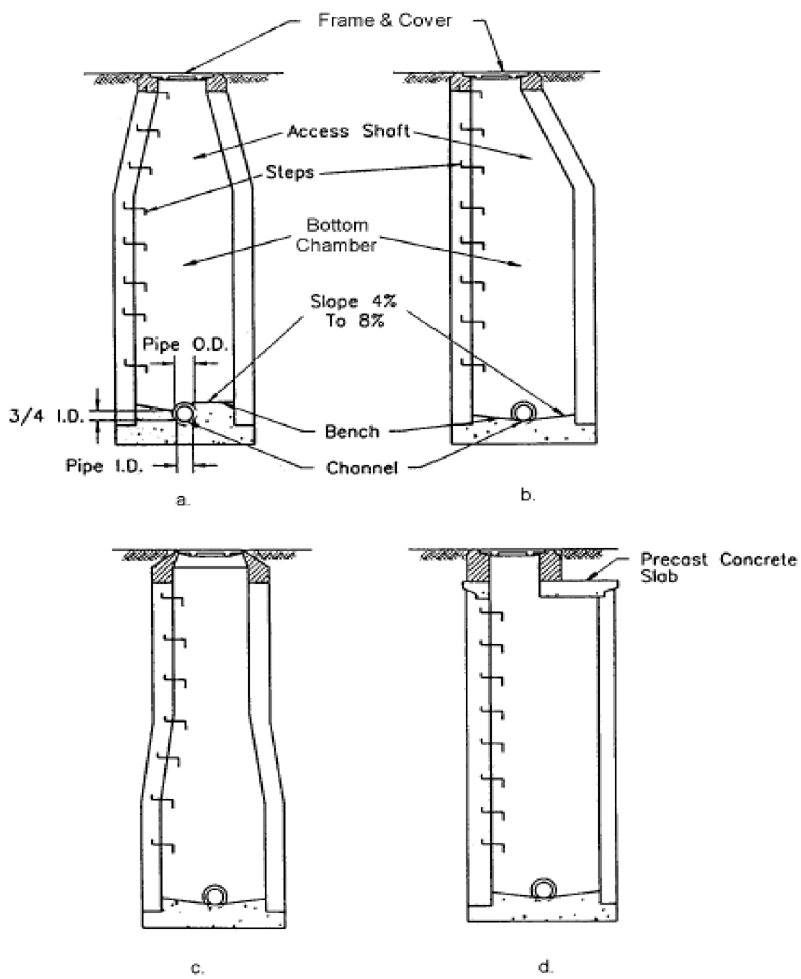


Figure 1.15 Typical Access Hole Configurations.
(HEC22, 2001)

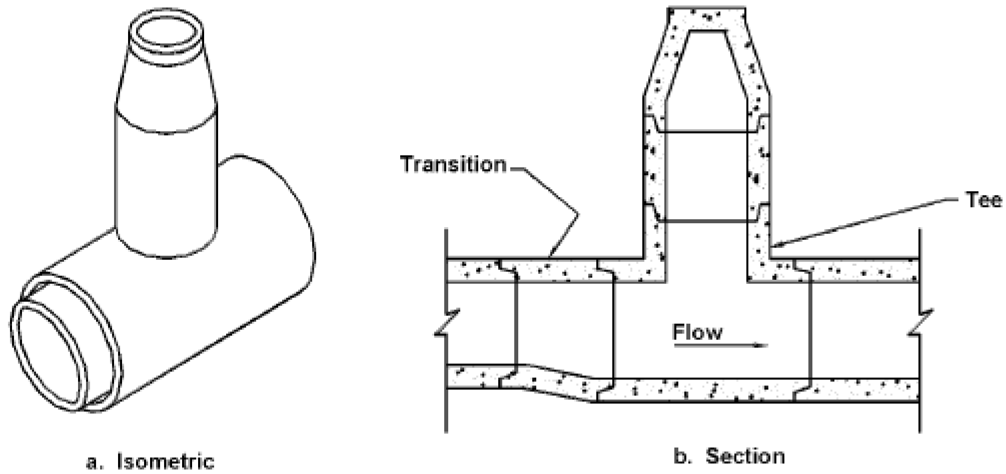


Figure 1.16 "Tee" Access Hole for Large Storm Drains
(HEC 22, 2001)

Junction Chambers

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

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

To minimize flow turbulence in junction boxes, flow channels and benches are typically built into the bottom of the chambers. Where junction chambers are used as access points for the storm drain system, their location should adhere to the spacing criteria outlined in *Table 3.9 of the Criteria Manual*.

General Design Procedure

The design of storm drain systems generally follows these steps:

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

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

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

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

Step 4 After the tentative locations of inlets, drain pipes, and outfalls (including tailwaters) have been determined and the inlets sized, compute the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from the upstream end of a line downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream. The storm drain system design computation form (Figure 1.31) can be used to summarize hydrologic, hydraulic and design computations.

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

The rate of discharge at any point in the storm drainage system is not the sum of the inlet flow rates of all inlets above the section of interest. It is generally less that this total. The Rational Method is the most common means of determining design discharges for storm drain design and is explained in *Section 1.2 of the Hydrology Technical Manual*. The designer is usually concerned with two different times of concentration: one for inlet spacing and the other for pipe sizing

The time of concentration for pipe sizing is defined as the time required for water to travel for the most hydraulically distant point in the total contributing watershed to the design point. Typically, this time consists of two components: (1) the time for overland and gutter flow to reach the first inlet, and (2) the time to flow through the storm drainage system to the point of interest. If the total time of concentration to the upstream inlet is less than five minutes, a minimum time of concentration of five minutes is used as the duration of rainfall. The time of concentration for each successive inlet should be determined independently in the same manner as was used for the first inlet.

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

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

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

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

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

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

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

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

Capacity Calculations

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

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

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

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

$$\mathbf{V = (1.486/n)\, R^{2/3}\, S^{1/2}} \qquad \mathbf{(1.15)}$$

where:

V = mean velocity of flow, ft/s

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

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

n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$\mathbf{Q = (1.486/n)\, A\, R^{2/3}\, S^{1/2}} \qquad \mathbf{(1.16)}$$

where:

Q = rate of flow, cfs

A = cross sectional area of flow, ft^2

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

$$\mathbf{V = [0.590\, D^{2/3} S^{1/2}]/n} \qquad \mathbf{(1.17)}$$

$$\mathbf{Q = [0.463\, D^{8/3} S^{1/2}]/n} \qquad \mathbf{(1.18)}$$

where:

D = diameter of pipe, ft

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

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

$$H_f = [0.453\ n^2V^2L]/[R^{4/3}] \quad (1.19)$$

$$H_f = [(2.87\ n^2V^2L]/[D^{4/3}] \quad (1.20)$$

$$H_f = [(185n^2(V^2/2g)L]/[D^{4/3}] \quad (1.21)$$

where:

H_f = total head loss due to friction, ft (S_f x L)

n = Manning's roughness coefficient

D = diameter of pipe, ft

L = length of pipe, ft

V = mean velocity, ft/s

R = hydraulic radius, ft

g = acceleration of gravity = 32.2 ft/sec^2

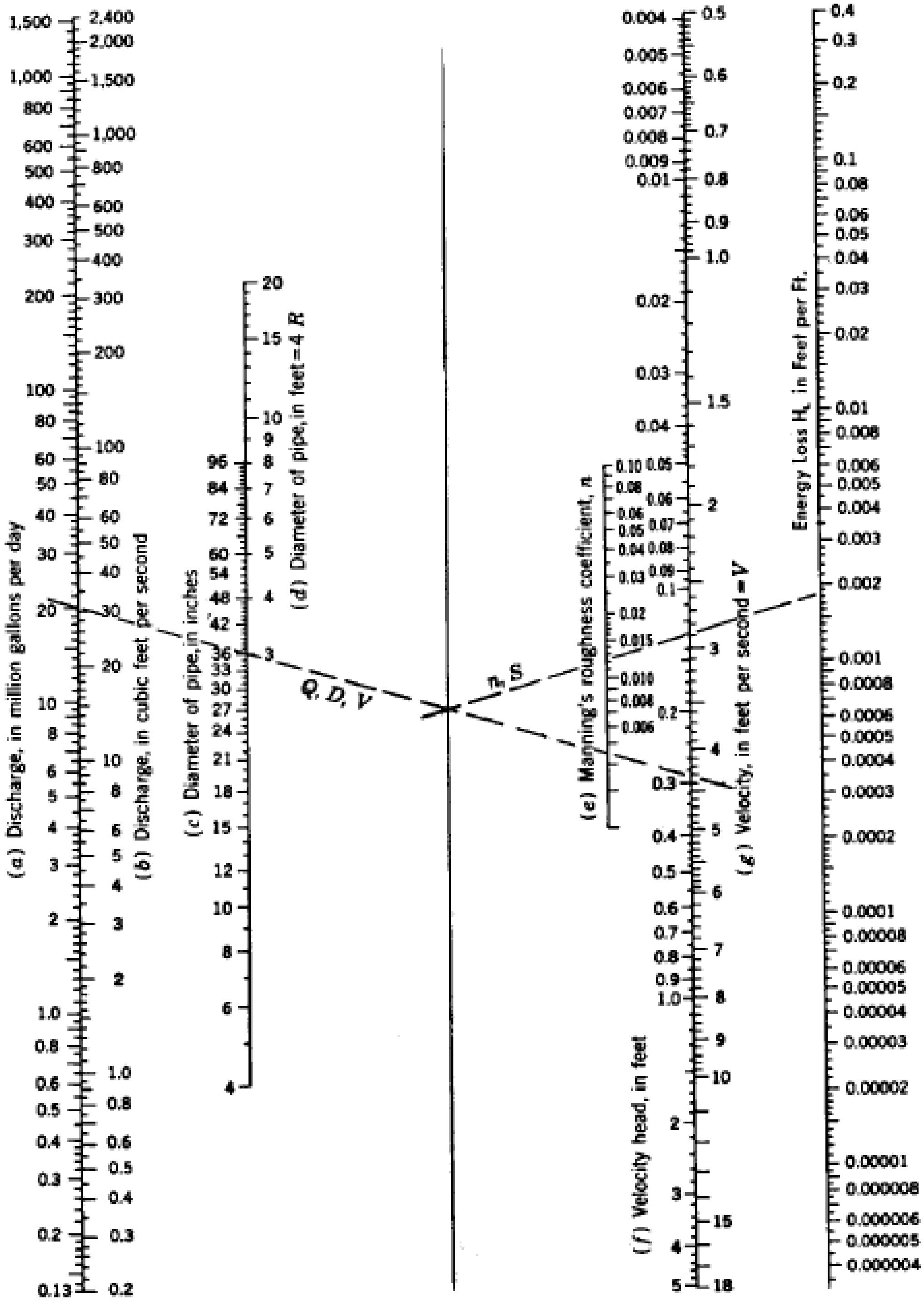
A nomograph solution of Manning's Equation for full flow in circular conduits is presented in Figure 1.17. Representative values of the Manning's coefficient for various storm drain materials are provided in Table 1.8. It should be remembered that the values in the table are for new pipe tested in a laboratory. Actual field values for conduits may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

Figure 1.18 illustrates storm drain capacity sensitivity to the parameters in the Manning's Equation. This figure can be used to study the effect changes in individual parameters will have on storm drain capacity. For example, if the diameter of a storm drain is doubled, its capacity will be increased by a factor of 6.0; if the slope is doubled, the capacity is increased by a factor of 1.4; however, if the roughness is doubled, the pipe capacity will be reduced by 50 percent.

The hydraulic elements graph in Figures 1.19a and 1.19b is provided to assist in the solution of the Manning's Equation for part full flow in storm drains. The hydraulic elements chart shows the relative flow conditions at different depths in a circular pipe and makes the following important points:

1. Peak flow occurs at 93 percent of the height of the pipe. This means that if the pipe is designed for full flow, the design will be slightly conservative.
2. The velocity in a pipe flowing half-full is the same as the velocity for full flow.
3. Flow velocities for flow depths greater than half-full are greater than velocities at full flow.
4. As the depth of flow drops below half-full, the flow velocity drops off rapidly. The shape of a storm drain conduit also influences its capacity. Although most storm drain conduits are circular, a significant increase in capacity can be realized by using an alternate shape. Table 1.7 provides a tabular listing of the increase in capacity which can be achieved using alternate conduit shapes that have the same height as the original circular shape, but have a different cross sectional area. Although these alternate shapes are generally more expensive than circular shapes, their use can be justified in some instances based on their increased capacity.

Table 1.7 Increase in Capacity of Alternate Conduit Shapes Based on a Circular Pipe with the Same Height (HEC-22, 2001)		
	Area (Percent Increase)	Conveyance (Percent Increase)
Circular	**--**	**--**
Oval	**63**	**87**
Arch	**57**	**78**
Box (B = D)	**27**	**27**



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

Figure 1.17 Solution of Manning's Equation for Flow in Storm Drains-English Units
(Taken from "Modern Sewer Design" by American Iron and Steel Institute)

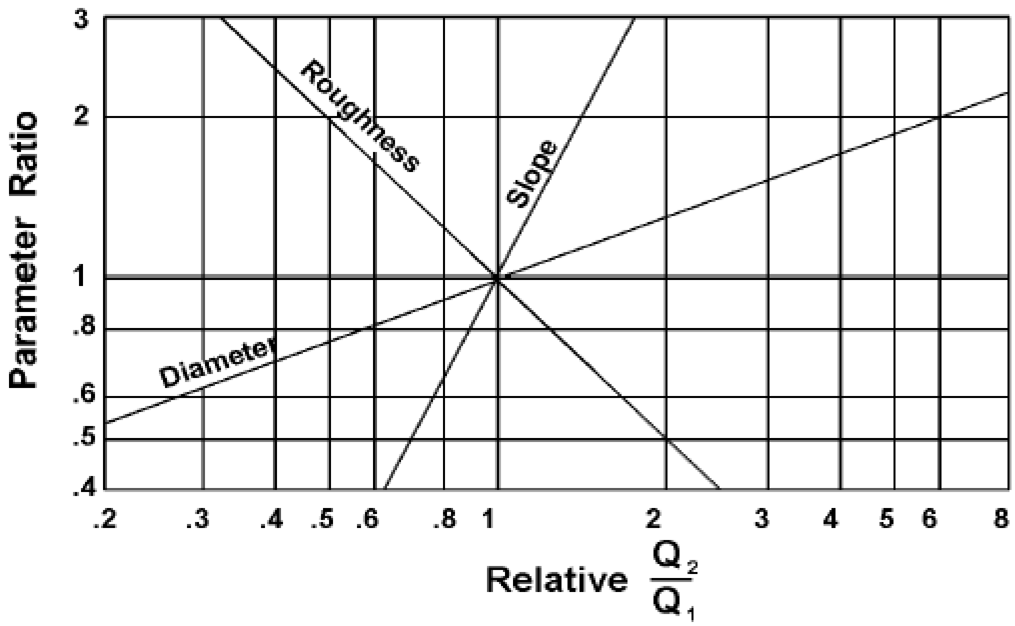


Figure 1.18 Storm Drain Capacity Sensitivity
(HEC 22, 2001)

Table 1.8 Manning's Coefficients for Storm Drain Conduits
(HEC 22, 2001)

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

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

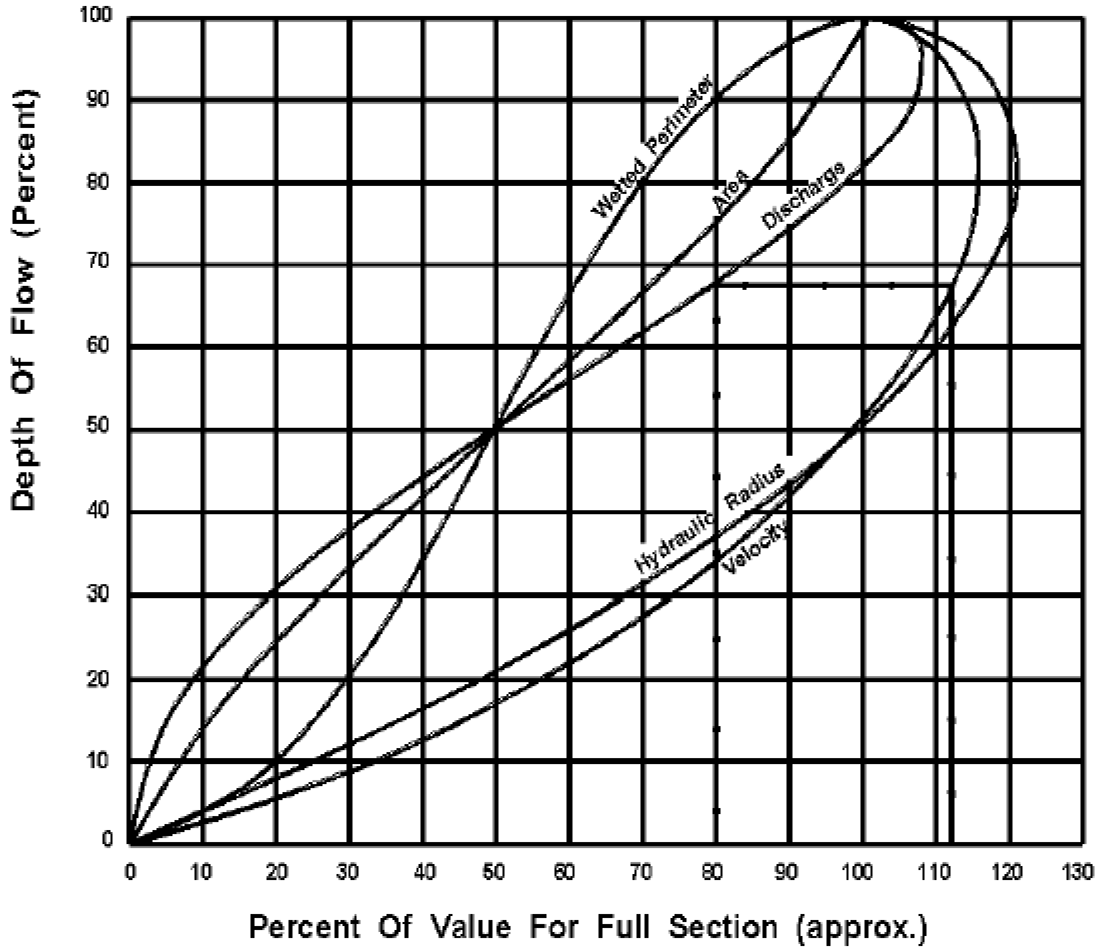


Figure 1.19a Hydraulic Elements of Circular Section
(HEC 22, 2001)

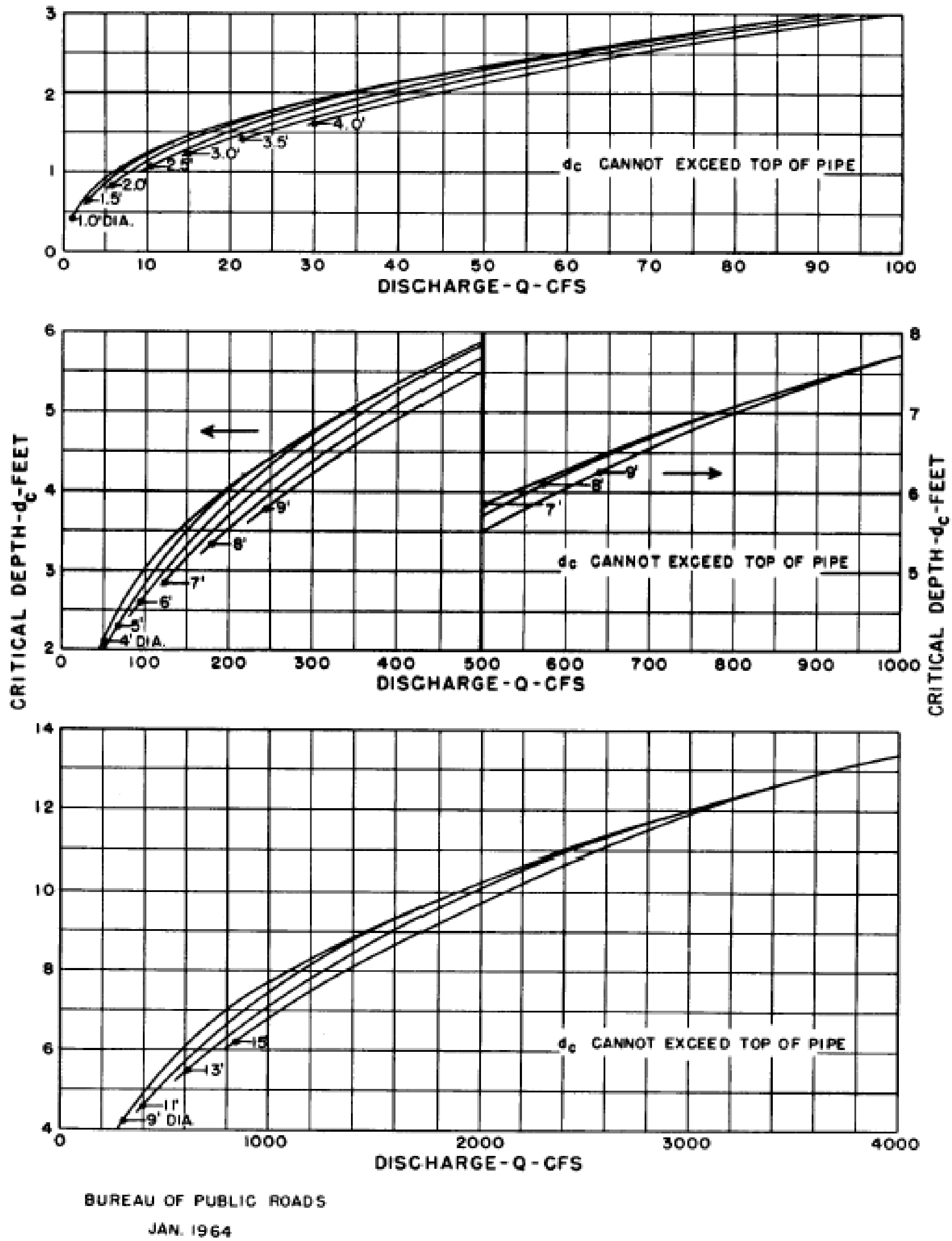


Figure 1.19b Critical Depth in Circular Pipe-English Units
(HEC 22, 2001)

Minimum Grades and Desirable Velocities

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

$$S = [(nV)^2]/[2.208R^{4/3}] \quad (1.22)$$

where:

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

For circular conduits flowing full but not under pressure, R=D/4, and the hydraulic grade line is equal to the slope of the pipe. For these conditions Equation 1.22 may be expressed as:

$$S = 2.87(nV)^2/D^{4/3} \quad (1.23)$$

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

$$S = 17.938(n^2/D^{4/3}) \quad (1.24)$$

where:

D = diameter, ft

Table 1.9 gives minimum slopes for two commonly used materials: concrete pipe with an n-value of 0.013 and corrugated metal pipe with an n-value of 0.024.

Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent excessive deposits of solid materials; otherwise objectionable clogging may result. The controlling velocity is near the bottom of the conduit and considerably less than the mean velocity of the sewer. Storm drains shall be designed to have a minimum mean velocity flowing full of 2.5 fps. Table 1.9 gives minimum slopes for two commonly used materials: concrete pipe (n = 0.013) and corrugated metal pipe (n = 0.024), flowing at 2.5 fps.

Table 1.9 Minimum Grades for Storm Drains for 2.5 fps

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

Storm Drain Storage

If downstream drainage facilities are undersized for the design flow, a structural stormwater control may be needed to reduce the possibility of flooding. The required storage volume can also be provided by using larger than needed storm drain pipe sizes and restrictors to control the release rates at manholes and/or junction boxes in the storm drain system. The same design criteria for sizing structural control storage facilities are used to determine the storage volume required in the system (see *Section 2.0* for more information).

1.2.9 Energy Grade Line/Hydraulic Grade Line

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

$$\mathbf{E = V^2/2g + p/\gamma + z} \tag{1.25}$$

where:

E = Total energy, ft

$V^2/2g$ = Velocity head, ft (kinetic energy)

p = Pressure, lbs/ft^2

γ = Unit weight of water, 62.4 lbs/ft^3

p/γ = Pressure head, ft (potential energy) z = Elevation head, ft (potential energy)

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

$$V_1^2/2g + p_1/\gamma + z_1 = V_2^2/2g + p_2/\gamma + z_2 - H_f - \Sigma H_m \quad \mathbf{(1.26)}$$

where:

H_f = Pipe friction loss, ft

ΣH_m = Sum of minor or form losses, ft

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

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

HGL, a measure of flow energy, is determined by subtracting the velocity head ($V^2/2g$) from the EGL. Energy concepts can be applied to pipe flow as well as open channel flow. Figure 1.20 illustrates the energy and hydraulic grade lines for open channel and pressure flow in pipes.

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

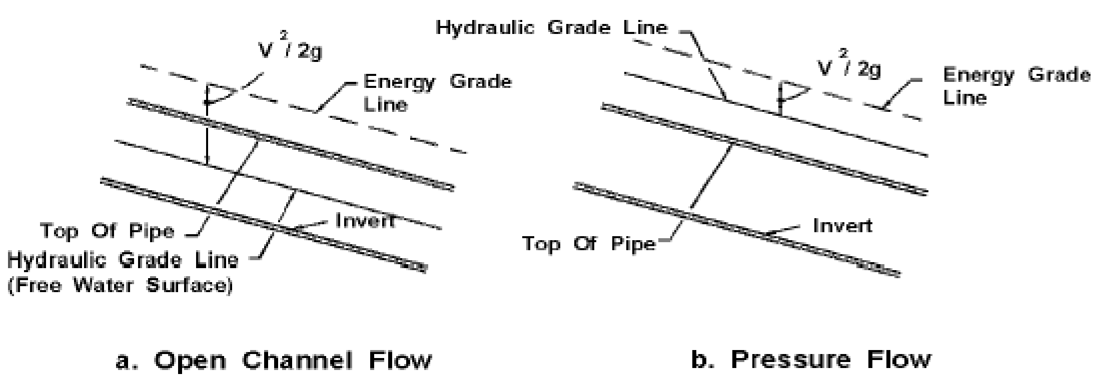


Figure 1.20 Hydraulic and Energy Grade Lines in Pipe Flow
(HEC 22, 2001)

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

A detailed procedure for evaluating the energy grade line and the hydraulic grade line for storm drainage systems is presented in *Section 1.2.11*.

Storm Drain Outfalls

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

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

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

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

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

If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by a comparison of the drainage area of the receiving stream to the area of the storm drainage system. For example, if the storm drainage system has a drainage area much smaller than that of the receiving stream, the peak discharge from the storm drainage system may be out of phase with the peak discharge from the receiving watershed. Table 1.10 provides a comparison of discharge frequencies for coincidental occurrence for the 2-, 5-, 10-, 25-, 50-, and flood mitigation design storms. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. For example, if the receiving stream has a drainage area of 200 acres and the storm drainage system has a drainage area of 2 acres, the ratio of receiving area to storm drainage area is 200 to 2 which equals 100 to 1. From Table 1.10 and considering a 10-year design storm occurring over both areas, the flow rate in the main stream will be equal to that of a five year storm when the drainage system flow rate reaches its 10-year peak flow at the outfall. Conversely, when the flow rate in the main channel reaches its 10-year peak flow rate, the flow rate from the storm drainage system will have fallen to the 5- year peak flow rate discharge. This is because the drainage areas are different sizes, and the time to peak for each drainage area is different.

Table 1.10 Frequencies for Coincidental Occurrences
(TxDOT, 2002)

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

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

Energy dissipation may be required to protect the storm drain outlet. Protection is usually required at the outlet to prevent erosion of the outfall bed and banks. Riprap aprons or energy dissipators should be provided if high velocities are expected. See *Section 4.0* for guidance on design of Energy Dissipation Structures.

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

Energy Losses

Prior to computing the hydraulic grade line, all energy losses in pipe runs and junctions must be estimated. In addition to the principal energy involved in overcoming the friction in each conduit run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access holes. The following presents relationships for estimating typical energy losses in storm drainage systems. The application of some of these relationships is included in the design example in *Section 1.2.12*.

Pipe Friction Losses

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

$$\mathbf{H_f = S_f\,L} \qquad \mathbf{(1.27)}$$

where:

H_f = friction loss, ft

S_f = friction slope, ft/ft

L = length of pipe, ft

The friction slope in Equation 1.27 is also the slope of the hydraulic gradient for a particular pipe run. As indicated by Equation 1.27, the friction loss is simply the hydraulic gradient multiplied by the length of the run. Since this design procedure assumes steady uniform flow in open channel flow, the friction slope will match the pipe slope for part full flow. Pipe friction losses for full flow can be determined by the use of Equation 1.20.

Exit Losses

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

$$\mathbf{H_o = 1.0\,[(V_o^2/2g) - (V_d^2/2g)]} \qquad \mathbf{(1.28)}$$

where:

V_o = average outlet velocity

V_d = channel velocity downstream of outlet

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

Bend Losses

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

$$h_b = 0.0033\ (\Delta)\ (V^2/2g) \qquad (1.29)$$

where:

Δ = angle of curvature in degrees

Transition Losses

A transition is a location where a conduit or channel changes size. Typically, transitions should be avoided and access holes should be used when pipe size increases. However, sometimes transitions are unavoidable. Transitions include expansions, contractions, or both. In small storm drains, transitions may be confined within access holes. However, in larger storm drains or when a specific need arises, transitions may occur within pipe runs as illustrated in Figures 1.16 and 1.21.

Energy losses in expansions or contractions in non-pressure flow can be expressed in terms of the kinetic energy at the two ends. Contraction and expansion losses can be evaluated with Equations 1.30 and 1.31 respectively.

$$H_c=K_c\ [V_1^2/(2g)-\ V_2^2/(2g)] \qquad (1.30)$$
$$H_e=K_e\ [V_1^2/(2g)-\ V_2^2/(2g)] \qquad (1.31)$$

where:

K_e = expansion coefficient

K_c = contraction coefficient (0.5 K_e)

V_1 = velocity upstream of transition

V_2 = velocity downstream of transition

g = acceleration due to gravity (32.2 ft/s_2)

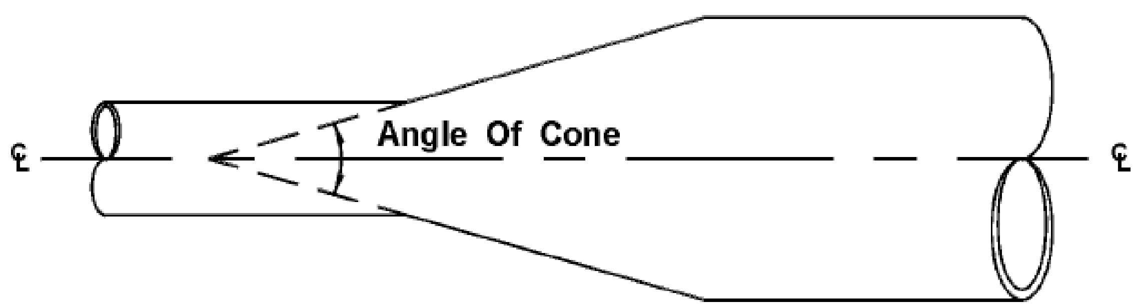


Figure 1.21 Angle of Cone for Pipe Diameter Changes

For gradual contractions, it has been observed that K_c = 0.5 K_e. Typical values of K_e for gradual enlargements are tabulated in Table 1.11a. Typical values of K_c for sudden contractions are tabulated in Table 1.11b. The angle of the cone that forms the transition is defined in Figure 1.21.

Table 1.11a Typical Values for K_e for Gradual Enlargement of Pipes in Non-Pressure Flow							
D_2/D_1	**Angle of Cone**						
	10º	20º	45º	60º	90º	120º	180º
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00
D_2/D_1= Ratio of Diameter of larger pipe to smaller pipe (ASCE, 1992)							

Table 1.11b Typical Values of K_c for Sudden Pipe Contractions	
D_2/D_1	**K_c**
0.2	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1	0
D_2/D_1= Ratio of Diameter of smaller pipe to larger pipe (ASCE, 1992)	

For storm drain pipes functioning under pressure flow, the loss coefficients listed in Tables 1.12 and 1.13 can be used with Equation 1.32 for sudden and gradual expansions respectively. For sudden contractions in pipes with pressure flow, the loss coefficients listed in Table 1.14 can be used in conjunction with Equation 1.33 (ASCE, 1992).

$$H_e=K_e\,(V_1^2/2g) \quad (1.32)$$
$$H_c=K_c\,(V_2^2/2g) \quad (1.33)$$

where:

K_e = expansion coefficient (Tables 1.13 and 1.14)

K_c = contraction coefficient (Table 1.15)

V_1 = velocity upstream of transition

V_2 = velocity downstream of transition

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

Table 1.12 Values of K_e for Determining Loss of Head due to Sudden Enlargement in Pipes

D_2/D_1	Velocity, V_1, in feet Per Second												
	2.0	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**10.0**	**12.0**	**15.0**	**20.0**	**30.0**	**40.0**
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09	0.08
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22	0.21	0.20
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33	0.32	0.32
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42	0.41	0.40
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50	0.48	0.47
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69	0.67	0.65
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76	0.74	0.72
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80	0.77	0.75
10.0	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.80
∞	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86	0.83	0.81

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

Table 1.13 Values of K_e for Determining Loss of Head due to Gradual Enlargement in Pipes

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

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

Table 1.14 Values of K_e for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity, V_2, in feet Per Second												
	2.0	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**10.0**	**12.0**	**15.0**	**20.0**	**30.0**	**40.0**
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.06
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.11	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.20
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.32	0.29	0.27
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	0.31

Table 1.14 Values of K_e for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity, V_2, in feet Per Second												
	2.0	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**10.0**	**12.0**	**15.0**	**20.0**	**30.0**	**40.0**
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
∞	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_2 = velocity in smaller pipe (downstream of transition)
(ASCE, 1992)

Junction Losses

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

$$H_j=[((Q_o V_o) - (Q_i V_i) - (Q_l V_l \cos\theta)) / (0.5g(A_o+A_i))] + h_i - h_o \qquad (1.34)$$

where:

H_j = junction loss (ft)

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

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

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

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

θ = angle between the inflow and outflow pipes (Figure 1.22)

Inlet and Access Hole Losses - Preliminary Estimate

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

The approximate method for computing losses at access holes or inlet structures involves multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 1.35. Applicable coefficients (K_{ah}) are tabulated in Table 1.15. This method can be used to estimate the initial pipe crown drop across an access hole or inlet structure to offset energy losses at the structure. The crown drop is then used to establish the appropriate pipe invert elevations. However, this method is used only in the preliminary design process and should not be used in the EGL calculations.

$$H_{ah}=K_{ah} (V_o^2/2g) \qquad (1.35)$$

Table 1.15 Head Loss Coefficients
(FHA, Revised 1993)

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

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

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

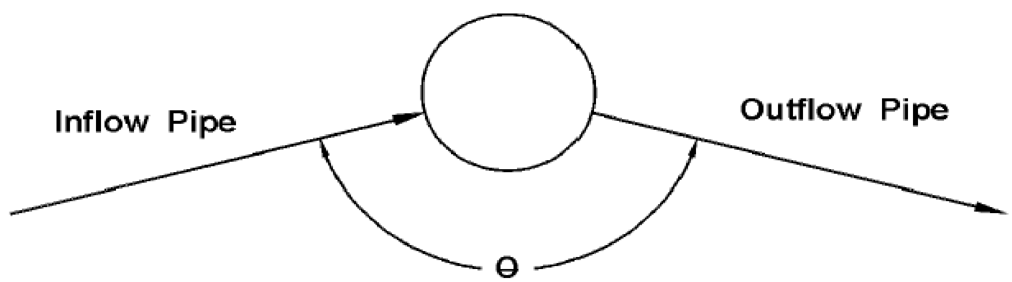


Figure 1.22 Head Loss Coefficients

The energy loss encountered going from one pipe to another through an access hole is commonly represented as being proportional to the velocity head of the outlet pipe. Using K to represent the constant of proportionality, the energy loss, H_{ah}, is approximated by Equation 1.36. Experimental studies have determined that the K value can be approximated by the relationship in Equation 1.37 when the inflow pipe invert is below the water level in the access hole.

$$H_{ah} = K (V_o^2/2g) \qquad (1.36)$$

$$K = K_O C_D C_d C_Q C_p C_B \qquad (1.37)$$

where:

- K = adjusted loss coefficient
- K_o = initial head loss coefficient based on relative access hole size
- C_D = correction factor for pipe diameter (pressure flow only)
- C_d = correction factor for flow depth
- C_Q = correction factor for relative flow
- C_p = correction factor for plunging flow
- C_B = correction factor for benching
- V_o = velocity of outlet pipe

For cases where the inflow pipe invert is above the access hole water level, the outflow pipe will function as a culvert, and the access hole loss and the access hole HGL can be computed using procedures found in *Hydraulic Design of Highway Culverts* (HDS-5, 1985). If the outflow pipe is flowing full or partially full under outlet control, the access hole loss (due to flow contraction into the outflow pipe) can be computed by setting K in Equation 1.36 to K_e as reported in Table 1.16. If the outflow pipe is flowing under inlet control, the water depth in the access hole should be computed using the inlet control nomographs in HDS- 5 (for example see Figure 3.31a).

The initial head loss coefficient, K_o in Equation 1.38, is estimated as a function of the **relative access hole** size and the angle of deflection between the inflow and outflow pipes as represented in Equation 1.6. This deflection angle is represented in Figure 1.22.

$$K_o = 0.1 (b/D_o)(1-\sin \theta) + 1.4(b/D_o)^{0.15} \sin \theta \qquad (1.38)$$

where:

- θ = angle between the inflow and outflow pipes (Figure 1.22)
- b = access hole or junction diameter
- D_o = outlet pipe diameter

A change in head loss due to differences in **pipe diameter** is only significant in pressure flow situations when the depth in the access hole to outlet pipe diameter ratio, d_{aho}/D_o, is greater than 3.2. In these cases a correction factor for pipe diameter, C_D, is computed using Equation 1.39. Otherwise C_D is set equal to 1.

$$C_D=(D_o/D_i)^3 \qquad (1.39)$$

where:

- D_o = outgoing pipe diameter
- D_i = inflowing pipe diameter

Table 1.16 Coefficients for Culverts; Outlet Control, Full, or Partly Full

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

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

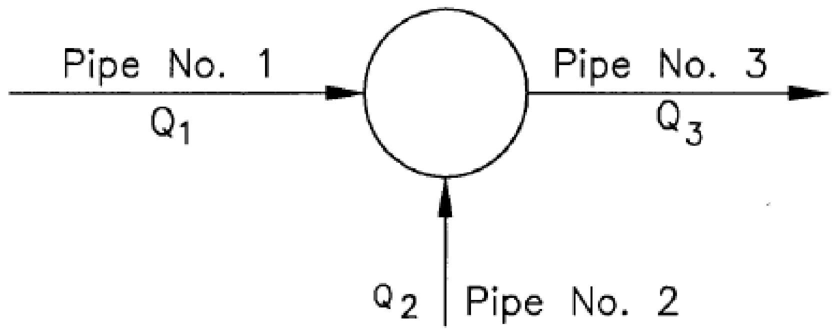


Figure 1.23 Relative flow effect

The correction factor for **flow depth**, C_d, is significant only in cases of free surface flow or low pressures, when the d_{aho}/D_o ratio is less than 3.2. In cases where this ratio is greater than 3.2, C_d is set equal to 1. To determine the applicability of this factor, the water depth in the access hole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor is calculated using Equation 1.40.

$$\mathbf{C_D = 0.5(d_{aho}/D_o)^{0.6}} \qquad \textbf{(1.40)}$$

where:

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

D_o = outlet pipe diameter

The correction factor for **relative flow**, C_Q, is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed using Equation 1.41. The correction factor is only applied to situations where there are 3 or more pipes entering the structure at approximately the same elevation. Otherwise, the value of C_Q is equal to 1.0.

$$\mathbf{C_Q = (1- 2\sin \Theta) [1-(Q_i / Q_o)]^{0.75} + 1} \qquad \textbf{(1.41)}$$

where:

C_Q = correction factor for relative flow

θ = the angle between the inflow and outflow pipes (Figure 1.22)

Q_i = flow in the inflow pipe

Q_o = flow in the outflow pipe

As can be seen from Equation 1.41, C_Q is a function of the angle of the incoming flow as well as the ratio of inflow coming through the pipe of interest and the total flow out of the structure. To illustrate this effect, consider the access hole shown in Figure 1.23 and assume the following two cases to determine the correction factor of pipe number 2 entering the access hole. For each of the two cases, the angle between the inflow pipe number 1 and the outflow pipe, θ, is 180°.

Case 1:
$Q_1 = 3\ ft^3/s$
$Q_2 = 1\ ft^3/s$
$Q_3 = 4\ ft^3/s$

Using Equation 1.41,
$C_Q = (1 - 2 \sin \theta)(1 - Q_i/Q_o)^{0.75} + 1$
$C_Q = (1 - 2 \sin 180^o)(1 - 3/4)^{0.75} + 1$
$C_Q = 1.35$

Case 2:
$Q_1 = 1.0\ ft^3/s$
$Q_2 = 3.0\ ft^3/s$
$Q_3 = 4.0\ ft^3/s$

Using Equation 1.41,
$C_Q = (1 - 2 \sin \theta)(1 - Q_i/Q_o)^{0.75} + 1$
$C_Q = (1 - 2 \sin 180^o)(1 - 1/4)^{0.75} + 1$
$C_Q = 1.81$

The correction factor for **plunging flow**, C_p, is calculated using Equation 1.42. This correction factor corresponds to the effect another inflow pipe, plunging into the access hole, has on the inflow pipe for which the head loss is being calculated. Using the notations in Figure 1.23, C_p is calculated for pipe #1 when pipe #2 discharges plunging flow. The correction factor is only applied when $h > d_{aho}$. Additionally, the correction factor is only applied when a higher elevation flow plunges into an access hole that has both an inflow line and an outflow in the bottom of the access hole. Otherwise, the value of C_p is equal to 1.0. Flows from a grate inlet or a curb opening inlet are considered to be plunging flow and the losses would be computed using Equation 1.42.

$$\mathbf{C_p = 1 + 0.2(h/D_o)\,[(h - d_{aho})/D_o]} \qquad \mathbf{(1.42)}$$

where:

C_p = correction for plunging flow

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

D_o = outlet pipe diameter

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

The correction for **benching** in the access hole, C_B, is obtained from Table 1.17. Figure 1.24 illustrates benching methods listed in Table 1.17. Benching tends to direct flow through the access hole, resulting in a reduction in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Table 1.17 Correction for Benching
(HEC 22, 2001)

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

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

**free surface flow, $d_{aho}/D_o \leq 1.0$

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

Composite Energy Loss Method

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

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

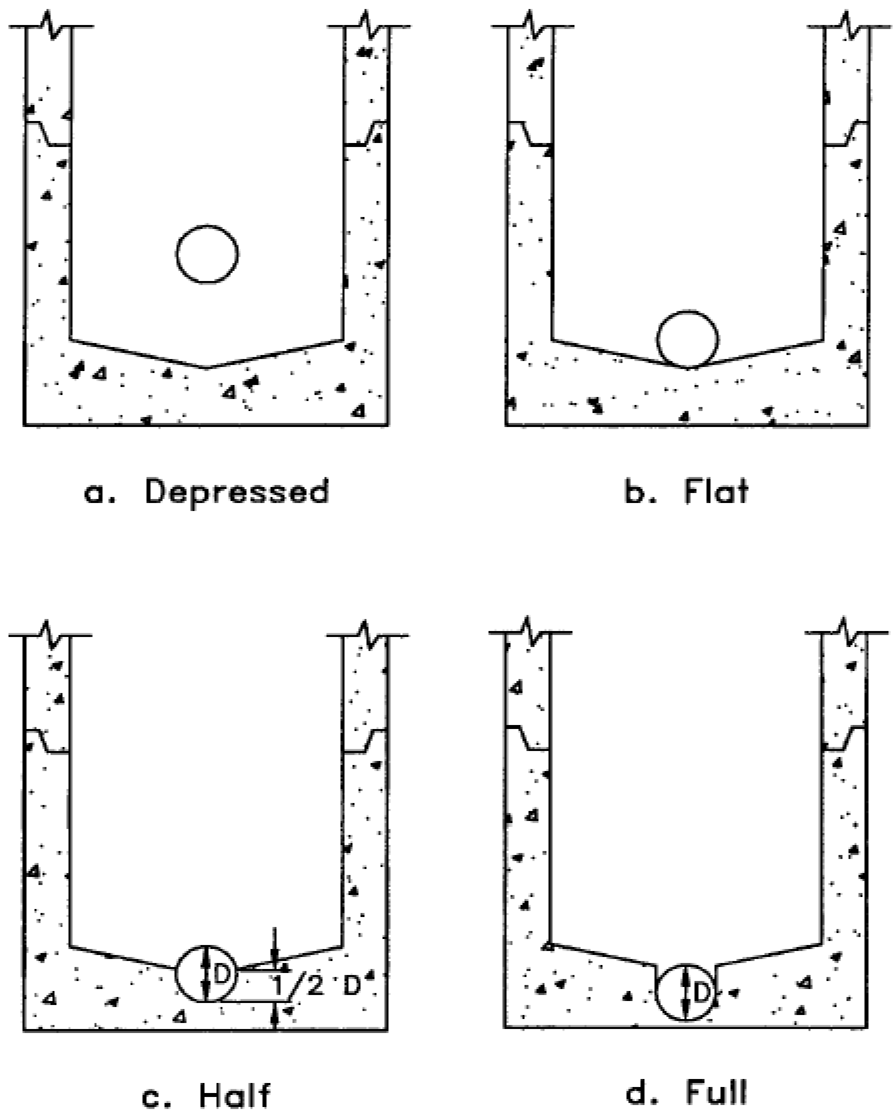


Figure 1.24 Access to Benching Methods

Preliminary Design Procedure

The preliminary design of storm drains can be accomplished by using the following steps and the storm drain computation sheet provided in Figure 1.25. This procedure assumes that each storm drain will be initially designed to flow full under gravity conditions. The designer must recognize that when the steps in this section are complete, the design is only preliminary. Final design is accomplished after the energy grade line and hydraulic grade line computations have been completed.

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

a. Location of storm drains.

b. Direction of flow.

c. Location of access holes and other structures.

d. Number or label assigned to each structure.

e. Location of all existing utilities (water, sewer, gas, underground cables, etc.).

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

a. Drainage areas.

b. Runoff coefficients.

c. Travel time

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

a. "From" and "To" stations, Columns 1 and 2b, "Length" of run, Column 3

b. "Length" of run, Column 3

c. "Inc." drainage area, Column 4

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

d. "C," Column 6

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

e. "Inlet" time of concentration, Column 9

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

f. "System" time of concentration, Column 10

The time for water to travel from the most remote point in the storm drainage system to the upstream end of the storm drain run under consideration. For the upstream most storm drain run this value will be the same as the value in Column 9. For all other pipe runs this value is computed by adding the "System" time of concentration (Column 10) and the "Section" time of concentration (Column 17) from the previous run together to get the system time of concentration at the upstream end of the section under consideration (See *Section 1.2.4 of the Hydrology Technical Manual* for a general discussion of times of concentration).

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

a. "TOTAL" area, Column 5

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

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

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

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

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

d. "I," Column 11

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

e. "TOTAL Q," Column 12

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

f. "SLOPE," Column 21

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

g. "PIPE DIA.," Column 13

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

h. "CAPACITY FULL," Column 14

Compute the full flow capacity of the selected pipe using Equation 1.18 and put this information in Column 14.

i. "VELOCITIES," Columns 15 and 16

Compute the full flow and design flow velocities (if different) in the conduit and place these values in Columns 15 and 16. If the pipe is flowing full, the velocities can be determined from V = Q/A, Equations 1.17 and 1.18. If the pipe is not flowing full, the velocity can be determined from Figure 1.19a.

j. "SECTION TIME," Column 17

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

k. "CROWN DROP," Column 20

Calculate an approximate crown drop at the structure to off-set potential structure energy losses using Equation 1.33. Place this value in Column 20.

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

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

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

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

PRELIMINARY STORM DRAIN COMPUTATION SHEET

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

ROUTE ______
SECTION ______
COUNTY ______

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

Figure 1.25 Preliminary Storm Drain Computation Sheet

1.2.10 Energy Grade Line Evaluation Procedure

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

Figure 1.26 provides a sketch illustrating use of the two grade lines in developing a storm drainage system. The following step-by-step procedure can be used to manually compute the EGL and HGL. The computation tables in Figure 1.27 and Figure 1.28 can be used to document the procedure outlined below.

Before outlining the computational steps in the procedure, a comment relative to the organization of data on the form is appropriate. In general, a line will contain the information on a specific structure and the line downstream from the structure. As the table is started, the first two lines may be unique. The first line will contain information about the outlet conditions. This may be a pool elevation or information on a known downstream system. The second line will be used to define the conditions right at the end of the last conduit. Following these first two lines the procedure becomes more general. A single line on the computation sheet is used for each junction or structure and its associated outlet pipe. For example, data for the first structure immediately upstream of the outflow pipe and the outflow pipe would be tabulated in the third full line of the computation sheet (lines may be skipped on the form for clarity). Table A (Figure 1.27) is used to calculate the HGL and EGL elevations while Table B (Figure 1.28) is used to calculate the pipe losses and structure losses. Values obtained in table B are transferred to table A for use during the design procedure. In the description of the computation procedures, a column number will be followed by a letter A or B to indicate the appropriate table to be used.

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

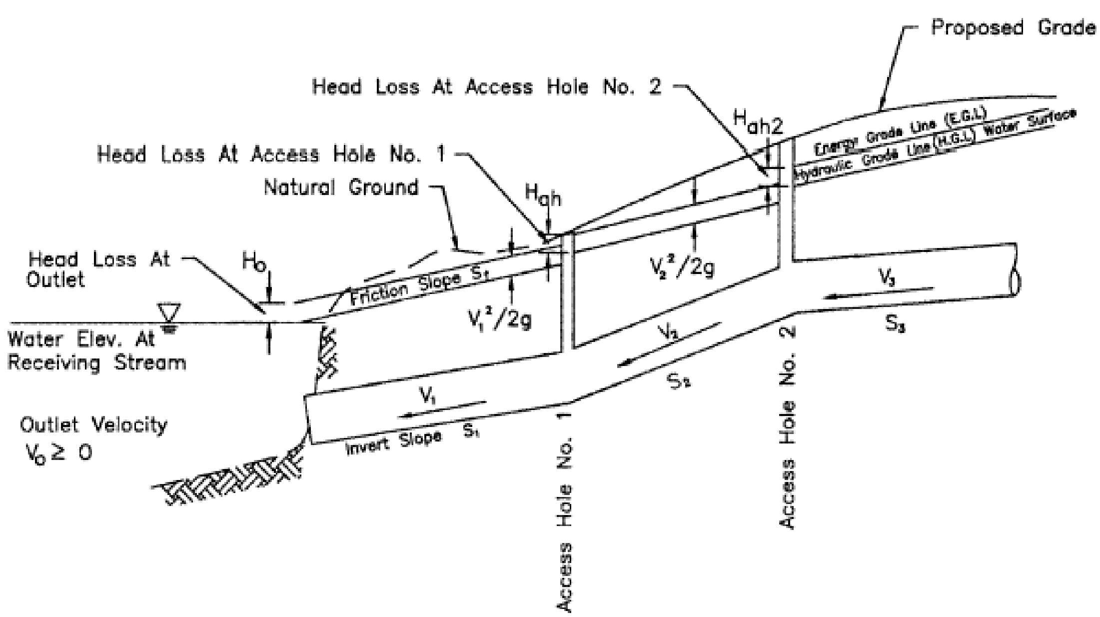


Figure 1.26 Energy and Hydraulic Grade Line Illustration

The EGL computational procedure follows:

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

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

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

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

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

ENERGY GRADE LINE COMPUTATION SHEET - TABLE A

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

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

Figure 1.27 Energy Grade Line Computation Sheet - Table A

ENERGY GRADE LINE COMPUTATION SHEET - TABLE B

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

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

Figure 1.28 Energy Grade Line Computation Sheet - Table B

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

Note: The values for d_c for circular pipes can be determined from Figure 1.19b. Charts for other conduits or other geometric shapes can be found in *Hydraulic Design of Highway Culverts*, HDS-5, and cannot be greater than the height of the conduit.

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

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

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

Step 5A Part full flow: Using the hydraulic elements graph in Figure 1.19a with the ratio of part full to full flow (values from the preliminary storm drain computation form), compute the depth and velocity of flow in the conduit. Enter these values in Column 6a and 5 respectively of Table A. Compute the velocity head ($V_2/2g$) and place in Column 7A.

Step 5B Compute critical depth for the conduit using Figure 1.19b. If the conduit is not circular, see HDS-5 for additional charts. Enter this value in Column 6b of Table A.

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

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

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

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

Step 5F Compute normal depth for the conduit using Figure 1.19a and critical depth using Figure 1.19b. If the conduit is not circular see HDS-5 for additional charts. Enter these values in Columns 6A and 6b of Table A.

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

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

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

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

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

Step 6 Compute the friction slope (S_f) for the pipe using Equation 1.19 divided by L[$S_f = H_f/L = [185\ n^2 (V^2/2g)]/D^{4/3}$] for a pipe flowing full. Enter this value in Column 8A of the current line. If full flow does not exist, set the friction slope equal to the pipe slope.

Step 7 Compute the friction loss (H_f) by multiplying the length (L) in Column 4A by the friction slope (S_f) in Column 8A and enter this value in Column 2B. Compute other losses along the pipe run such as bend losses (h_b), transition contraction (H_c) and expansion (H_e) losses, and junction losses (H_j) using Equations 1.29 through 1.34 and place the values in Columns 3B, 4B, 5B, and 6B, respectively. Add the values in 2B, 3B, 4B, 5B, and 6B and place the total in Column 7B and 9A.

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

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

Step 10 If the inflow storm drain invert is submerged by the water level in the access hole, compute access hole losses using Equations 1.36 and 1.37. Start by computing the initial structure head loss coefficient, K_o, based on relative access hole size. Enter this value in Column 9B. Continue with Step 11. If the inflow storm drain invert is not submerged by the water level in the access hole, compute the head in the access hole using culvert techniques from HDS-5 as follows:

a. If the structure outflow pipe is flowing full or partially full under outlet control, compute the access hole loss by setting K in 1.35 to K_e as reported in Table 1.15. Enter this value in Column 15B and 11A, continue with Step 17. Add a note on Table A indicating that this is a drop structure.

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

Step 11 Using Equation 1.39 compute the correction factor for pipe diameter, C_D, and enter this value in Column 10B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is greater than 3.2.

Step 12 Using Equation 1.40 compute the correction factor for flow depth, C_D, and enter this value in Column 11B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is less than 3.2.

Step 13 Using Equation 1.41, compute the correction factor for relative flow, C_Q, and enter this value in Column 12B. This factor = 1.0 if there are less than 3 pipes at the structure.

Step 14 Using Equation 1.42, compute the correction factor for plunging flow, C_p, and enter this value in Column 13B. This factor = 1.0 if there is no plunging flow. This correction factor is only applied when $h>d_{aho}$.

Step 15 Enter in Column 14B the correction factor for benching, C_B, as determined from Table 1.17. Linear interpolation between the two columns of values will most likely be necessary.

Step 16 Using Equation 1.37, compute the value of K and enter this value in Column 15B and 11A.

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

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

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

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

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

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

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

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

1.2.11 Storm Drain Design Example

The following storm drain design example illustrates the application of the design procedures outlined in *Section 1.2.10*.

Example of Preliminary Storm Drain Design

Given: The roadway plan and section illustrated in Figure 1.29, duration intensity information in Table 1.19 and inlet drainage area information in Table 1.18. All grates are type P 50 x 100, all piping is reinforced concrete pipe (RCP) with a Manning's n value of 0.013, and the minimum design pipe diameter = 18 in for maintenance purposes.

Find:

(1) Using the procedures outlined in *Section 1.2.10* determine appropriate pipe sizes and inverts for the system illustrated in Figure 1.29.

(2) Evaluate the HGL for the system configuration determined in part (1) using the procedure outlined in *Section 1.2.10*.

Solution:

(1) Preliminary Storm Drain Design

Step 1. Figure 1.29 illustrates the proposed system layout including location of storm drains, access holes, and other structures. All structures have been numbered for reference. Figure 1.30 (a) and (b) illustrate the corresponding storm drain profiles.

Step 2. Drainage areas, runoff coefficients, and times of concentration are tabulated in Figure 1.31. Example problems documenting the computation of these values are included in this section

Starting at the upstream end of a conduit run, Steps 3 and 4 from *Section 1.2.10* are completed for each storm drain pipe. A summary tabulation of the computational process is provided in Figure 1.31. The column by column computations for each section of conduit follow:

Table 1.18 Drainage Area Information for Design Example

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

Table 1.19 Intensity/Duration Data Design Example

Time (min)	5	10	15	20	30	40	50	60	120
Intensity (in/hr)	7.1	5.9	5.1	4.5	3.5	3	2.6	2.4	1.4

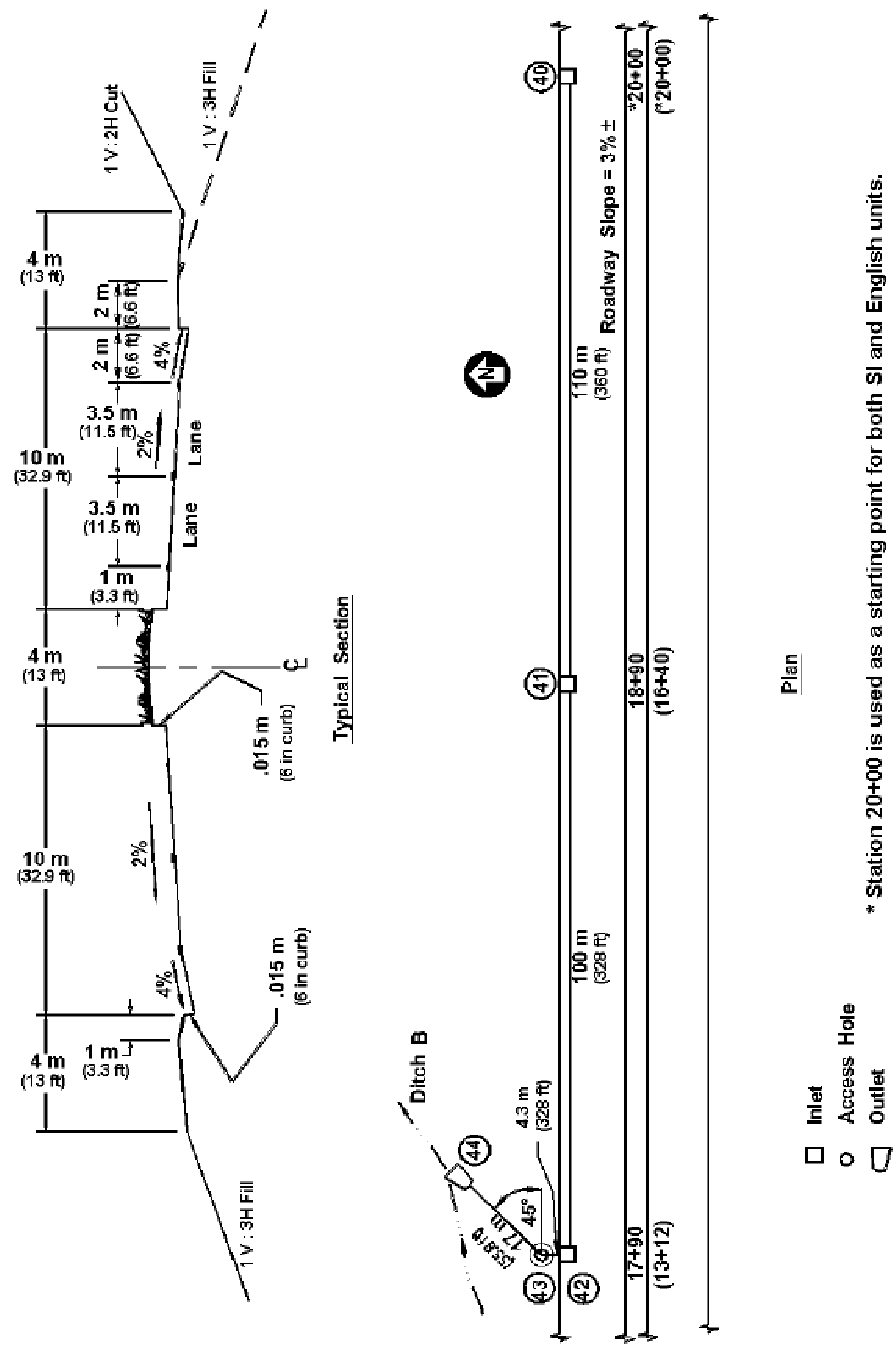


Figure 1.29 Roadway Plan and Sections for Example

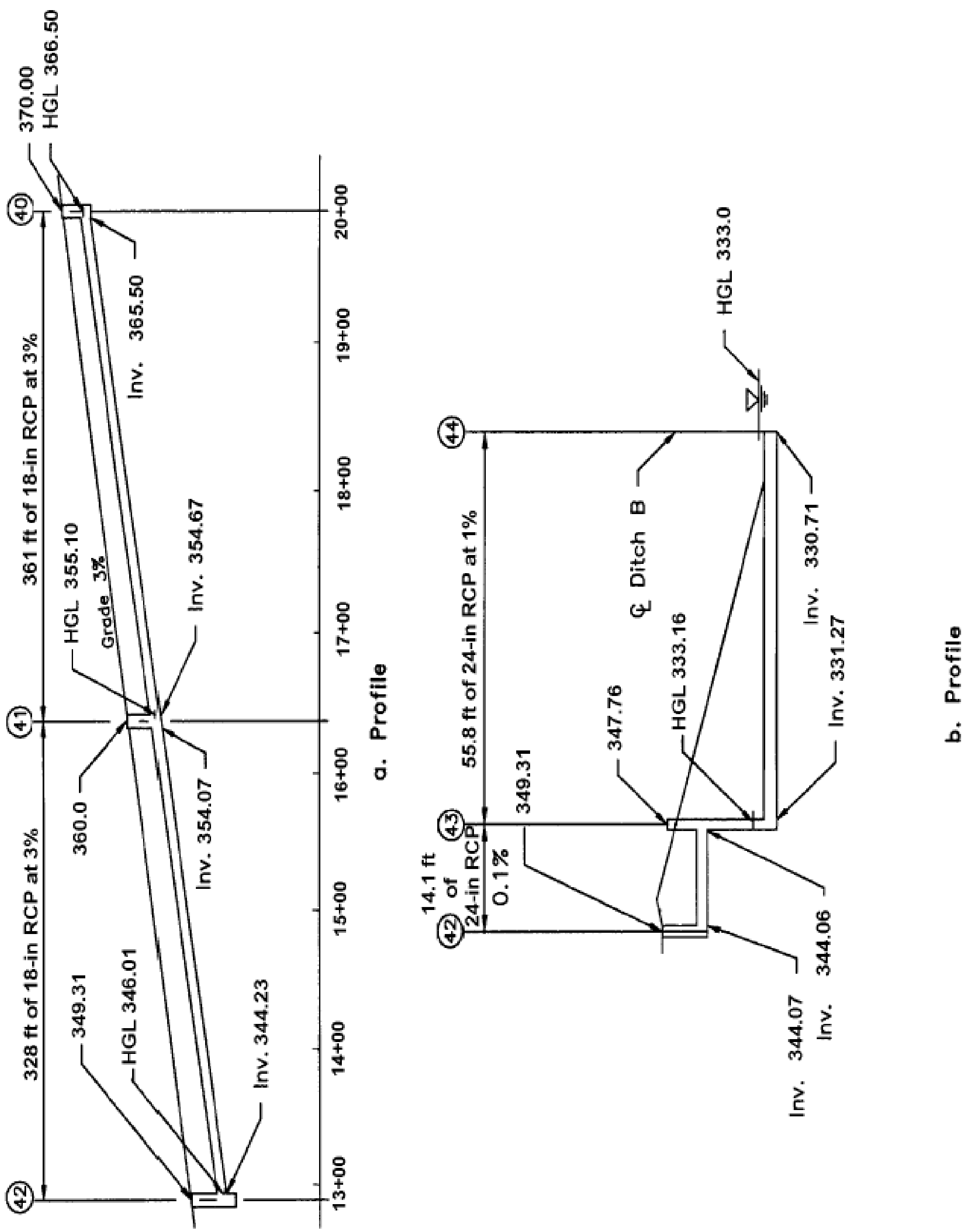


Figure 1.30 Storm Drain Profiles for Example

PRELIMINARY STORM DRAIN COMPUTATION SHEET
(English Solution)

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

STR. ID FROM (1)	STR. ID TO (2)	LENGTH (Ft) (3)	DRAINAGE AREA INC (ac) (4)	DRAINAGE AREA TOTAL (ac) (5)	RUNOFF COEFF. "C" (6)	"AREA" X "C" INC. (ac) (7)	"AREA" X "C" TOTAL (ac) (8)	TIME OF CONCENTRATION INLET (min) (9)	TIME OF CONCENTRATION SYSTEM (min) (10)	RAIN "I" (in/hr) (11)	RUNOFF "Q" (ft³ /s) (12)	PIPE DIA. (ft) (13)	Q FULL (ft³/s) (14)	VELOCITY FULL (ft /s) (15)	VELOCITY DESIGN (ft /s) (16)	SEC TIME (min) (17)	INVERT ELEV. U/S (ft) (18)	INVERT ELEV. D/S (ft) (19)	CROWN DROP (ft) (20)	SLOPE (ft/ft) (21)
40	41	361.0	0.64	0.64	0.73	0.47	0.47	3	5	7.1	3.3	1.50	18.1	10.3	7.52	1	365.50	354.67	----	0.03
41	42	328.0	0.35	0.99	0.73	0.25	0.72	2	5	7.1	5.1	1.5	18.1	10.3	8.7	1	354.07	344.23	0.6	0.03
42	43	14.1	0.32	1.31	0.73	0.23	0.95	2	5	7.1	6.75	2.0	7.12	2.28	2.6	0	344.07	344.06	0.16	0.001
43	44	55.8	0.00	1.31	-	-	0.95	-	5	7.1	6.75	2.0	22.52	7.22	6.10	0	331.27	330.71	12.79	0.01

Figure 1.31 Storm Drain Computation Sheet for Example

Structure 40 to 41

Col. 1 From structure 40		
Col. 2 To structure 41		
Col. 3 Run Length	L = 2000 ft - 1639 ft L = 361 ft	Figure 1.30
Col. 4 Inlet Area	A_i = 0.64 ac	Table 1.18
Col. 5 Total Area	A_t = 0.64 ac	Total area up to inlet 40
Col. 6 "C"	C = 0.73	Table 1.18
Col. 7 Inlet CA	CA = (0.64)(0.73) CA = 0.47 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.47 + 0 ΣCA= 0.47 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	t_i = 3 min	Table 1.18
Col. 10 Sys. Time	t_c = 3 min (use 5 min)	same as Col. 9 for upstream most section
Col. 11 Intensity	I = 7.1 in/hr	Table 1.19; System time less than 5 minutes therefore, use 5 minutes
Col. 12 Runoff	Q = C_f (CA) (I) Q = 1.0(0.47)(7.1) Q = 3.3 ft^3/sec	*Equation 1.3 of the Hydrology Technical Manual*; C_f = 1.0 (10yr) Col. 8 times Col. 11 multiplied by 1.0
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q\, S_o^{0.5})]^{0.375}$ $D = [(3.3)(0.013)/(0.46)(0.03)^{0.5})]^{0.375}$ D = 0.8 ft D_{min} = 1.5 ft	Equation 1.18 or Figure 1.17 use D_{min}
Col. 14 Full Cap	$Q_f = (K_Q/n)\, D^{2.67}\, S_o^{0.5}$ $Q_f = (0.46/0.013)\,(1.5)^{2.67}\,(0.03)^{0.5}$ Q_f = 18.1ft^3/s	Equation 1.18 or Figure 1.17
Col. 15 Vel. Full	$V_f = (K_V/n)\, D^{0.67}\, S_o^{0.5}$ $V_f = (0.59/0.013)\,(1.5)^{0.67}\,(0.03)^{0.5}$ V_f = 10.3 ft/s	Equation 1.17 or Figure 1.17
Col. 16 Vel. Design	Q/Q_f = 3.3/18.1 = 0.18 V/V_f = 0.73 V = (0.73) (10.3) V = 7.52 ft/s	Figure 1.19a
Col. 17 Sect. Time	t_s = L/V = 361 / 7.52 / 60 t_s = 0.8 min; use 1 min	Col. 3 divided by Col. 16

Col. 18 U/S Invert	= Grnd - 3.0 ft - dia = 370.0 - 3.0 - 1.5 = 365.5 ft	3 ft = min cover Ground elevation from Figure 1.30
Col. 19 D/S Invert	= (365.5) - (361.0)(0.03) = 354.67 ft	Col. 18 - (Col. 3)(Col. 21)
Col. 20 Crown Drop	= 0	Upstream most invert
Col. 21 Slope	S = 0.03	select desired pipe slope

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

	354.67 + 1.5 + 3.0 = 359.17	Invert elev. + Diam + min cover
	Ground elevation of 360.0 ft is greater than 359.17 ft so OK	

Structure 41 to 42

Col. 1 From	= 41	
Col. 2 To	= 42	
Col. 3 Run Length	L = 1639 - 1311 \ L = 328 ft	Figure 1.30
Col. 4 Inlet Area	A_i = 0.35 ac	Table 1.18
Col. 5 Total Area	A_t = 0.35 + 0.64 A_t = 0.99 ac	Col. 4 plus structure 42 total area, Table 1.18
Col. 6 "C"	C = 0.73	Table 1.18
Col. 7 Inlet CA	CA = (0.73)(0.35) CA = 0.25 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.25 + 0.47 ΣCA = 0.72 ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	t_i = 2 min	Table 1.18
Col. 10 Sys. Time	t_c = 4 min (use 5 min)	Col. 9 + Col. 17 for line 40-41
Col. 11 Intensity	I = 7.1 in/hr	Table 1.19; system time equals 5 min
Col. 12 Runoff	$Q = (C_f)(CA)(I)$ Q = 1.0(0.72) (7.1) Q = 5.1 ft^3/sec	*Equation 1.3 of the Hydrology Technical Manual*; C_f =1.0(10-yr) Col. 8 times Col. 11 times C_f
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q S_o^{0.5})]^{0.375}$ $D = [(5.1)(0.013)/(0.46)(0.03)^{0.5}]^{0.375}$ D = 0.93 ft D_{min} = 1.5 ft use D_{min}	Equation 1.18 or Figure 1.17 use D_{min}

Col. 14 Full Cap.	$Q_f = (K_Q/n)\ D^{2.67}\ S_o^{0.5}$ $Q_f = (0.46/0.013)(1.5)^{2.67}(0.03)^{0.5}$ Q_f = 18.1 ft3/s	Equation 1.18 or Figure 1.17
Col. 15 Vel. Full	$V_f = (K_V/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59/0.013)(1.5)^{0.67}\ (0.03)^{0.5}$ V_f = 10.3 ft/s	Equation 1.18 or Figure 1.17
Col. 16 Vel. Design	Q/Q_f = 5.1/18.1 = 0.28 V/V_f = 0.84 V = (0.84) (10.3) V = 8.7 ft/s	 Figure 1.19a
Col. 17 Sect. Time	T_s = L/V = 328 / 8.75 / 60 T_s = 0.6 min; use 1 min	Col. 3 divided by Col. 16
Col. 18 U/S Invert	= 354.67 - 0.6 = 354.07 ft	Downstream invert of upstream conduit minus estimated structure loss (drop)
Col. 19 D/S Invert	= (354.07) - (328)(0.03) = 344.23 ft	Col. 18 - (Col. 3)(Col. 21)
Col. 20 Crown Drop	$= H_{ah} = K_{ah}\ (V^2 / 2g)$ $= (0.5)(8.7)^2 / [(2)(32.2)]$ = 0.6 ft	Equation 1.36 with Table 1.15 K_{ah} = 0.5 for inlet - straight run
Col. 21 Slope	S = 0.03	select desired pipe slope

Structure 42 to 43

Col. 1 From structure	= 42	
Col. 2 To structure	= 43	
Col. 3 Run Length	L = 14.1 ft	Figure 1.30
Col. 4 Inlet Area	A_i = 0.32 ac	Table 1.18
Col. 5 Total Area	A_t = 0.32 + 0.99 A_t = 1.31 ac	Col. 4 plus previous Col. 5 total area, Table 1.18
Col. 6 "C"	C = 0.73	Table 1.18
Col. 7 Inlet CA	CA = (0.73)(0.32) CA = 0.23 ac	Col. 4 times Col. 6
Col. 8 Sum CA	ΣCA = 0.23 + 0.72 ΣCA = 0.95 ac	Col. 7 plus structure 43 total CA values
Col. 9 Inlet Time	t_i = 2 min	Table 1.18

Col. 10 Sys. Time	$t_c = 5$ min	Col. 9 + Col. 17 for line 40-41 plus Col.17 for line 41-42
Col. 11 Intensity	$I = 7.1$ in/hr	Table 1.19
Col. 12 Runoff	$Q = (C_f)(CA)(I)$ $Q = 1.0\ (0.95)\ (7.1)$ $Q = 6.75\ ft^3/sec$	Col. 8 times Col. 11
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q\ S_o^{0.5})]^{\ 0.375}$ $D =[(6.75)(0.013)/(0.46)(0.001)^{0.5}]^{0.375}$ $D = 1.96$ ft $D = 2.0$ ft	Equation 1.18 or Figure 1.17 Use nominal size
Col. 14 Full Cap .	$Q_f = (K_Q/n)(D^{2.67})(S_o^{0.5})$ $Q_f = (0.46/(0.013)(2.0)^{\ 2.67}\ (0.001)^{0.5}$ $Q_f = 7.12\ ft^3/s$	Equation 1.18 or Figure 1.17
Col. 15 Vel. Full	$V_f = (K_v/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59)/(0.013)(2.0)^{\ 0.67}\ (0.001)^{0.5}$ $V_f = 2.28$ ft/s	Equation 1.18 or Figure 1.17
Col. 16 Vel. Design	$Q/Q_f = 6.75/7.12 = 0.95$ $V/V_f = 1.15$ $V = (1.15)\ (2.28)$ $V = 2.6$ ft/s	Figure 1.19a
Col. 17 Sect. Time	$t_s = L/V = 14.1 / 2.6 / 60$ $t_s = 0.09$ min, use 0.0 min	Col. 3 divided by Col. 16
Col. 18 U/S Invert	= 344.23 - 0.16 = 344.07 ft	Downstream invert of upstream conduit minus estimated structure loss (drop)
Col. 19 D/S Invert	= 344.07 - (14.1)(0.001) = 344.06 ft	Col. 18 - (Col. 3)(Col. 21)
Col. 20 Crown Drop	$= H_{ah} = K_{ah}\ (V^2 / 2g)$ $= (1.5)(2.6)^2 / [(2)(32.2)]$ = 0.16 ft	Equation 1.36 and Table 1.15; K_{ah} =1.5 for inlet - angled through 90 degrees
Col. 21 Slope	S = 0.001	Select desired pipe slope

Structure 43 to 44

Col. 1 From	= 43	
Col. 2 To	= 44	
Col. 3 Run Length	L = 55.8 ft	Figure 1.30
Col. 4 Inlet Area	$A_i = 0.0$ ac	Table 1.18

Col. 5 Total Area	$A_t = 1.31$ ac	Col. 4 plus previous Col. 5
Col. 6 "C"	C = n/a	Table 1.18
Col. 7 Inlet CA	CA = 0.0	Col. 4 times Col. 6
Col. 8 Sum CA	$\Sigma CA = 0.00 + 0.95$ $\Sigma CA = 0.95$ ac	Col. 7 plus previous Col. 8
Col. 9 Inlet Time	n/a	No inlet
Col. 10 Sys. Time	$t_c = 5$ min	Col. 10 + Col. 17 for line 42-43
Col. 11 Intensity	I = 7.1 in/hr	Table 1.18
Col. 12 Runoff	$Q = C_f\,(CA)\,I$ $Q = 1.0\,(0.95)\,(7.1)$ $Q = 6.75$ ft^3/sec	Col. 8 times Col. 11 times C_f
Col. 13 Pipe Dia.	$D = [(Qn)/(K_Q\,S_o^{0.5})]^{0.375}$ $D = [(6.75)(0.013)/(0.46)(0.01)^{0.5}]^{0.375}$ $D = 1.27$ ft $D = 2.0$ ft	Equation 1.18 or Figure 1.17 U/S conduit was 2.0 ft. - Do not reduce size inside the system
Col. 14 Full Cap.	$Q_f = (K_Q/n)(D^{2.67})(S_o^{0.5})$ $Q_f = (0.46)/(0.013)(2.0)^{2.67}\,(0.01)^{0.5}$ $Q_f = 22.52$ ft^3/s	Equation 1.18 or Figure 1.17
Col. 15 Vel. Full	$V_f = (K_v/n)\,D^{0.67}\,S_o^{0.5}$ $V_f = (0.59)/(0.013)(2.0)^{0.67}\,(0.01)^{0.5}$ $V_f = 7.22$ ft/s	Equation 1.17 or Figure 1.17
Col. 16 Vel. Design	$Q/Q_f = 6.75/22.52 = 0.30$ $V/V_f = 0.84$ $V = (0.84)\,(7.22)$ $V = 6.1$ ft/s	Figure 1.19a
Col. 17 Sect. Time	$t_s = 55.8 / 6.1 / 60$ $t_s = 0.15$ min, use 0.0 min	Col. 3 divided by Col. 16
Col. 19 D/S Invert	= 330.71 ft	Invert at discharge point in ditch
Col. 18 U/S Invert	= 330.71 + (55.8)(0.01) = 331.27 ft	Col. 19 + (Col. 3)(Col. 21)
Col. 20 Crown Drop	= 344.06 - 331.27 = 12.79 ft straight run	Col. 19 previous run - Col. 18
Col. 21 Slope	S = 0.01	Select desired pipe slope

(2) Energy Grade Line Evaluation Computations - English Units

The following computational procedure follows the steps outlined in *Section 1.2.11* above. Starting at structure 44, computations proceed in the upstream direction. A summary tabulation of the computational process is provided in Figure 1.32 English and Figure 1.33 English. The column by column computations for each section of storm drain follow:

RUN FROM STRUCTURE 44 TO 43

Outlet

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

Structure 44

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

Structure 43

Step 4	Col. 1A, 1B	Str. ID = 43	Next Structure
	Col. 2A	D = 2.0 ft	Pipe Diameter
	Col. 3A	Q = 6.75 cfs	Conduit discharge (design value)
	Col. 4A	L = 55.8 ft	Conduit length
Step 5	Col. 5A	V = Q/A	Velocity; use full barrel velocity since outlet is submerged.
		$V = 6.75/[(\pi/4)\ (2.0)^2]$	
		V = 2.15 ft/s	
	Col. 7A	$V^2/2g = (2.15)^2/(2)(32.2)$	Velocity head in conduit
		= 0.07 ft	
Step 6	Col. 8A	$S_f = [(Qn)/(K_Q D^{2.67})]^2$	Equation 1.18
		$S_f = [(6.75)(0.013)/(0.46)(2.0)^{2.67}]^2$	
		$S_f = 0.00090$ ft/ft	
Step 7	Col. 2B	$H_f = S_f L$	Equation 1.27
		$H_f = (0.0009)\ (55.8)$	Col. 8A x Col. 4A
		$H_f = 0.05$	
	Col. 7B &	$h_b, H_c, H_e, H_j = 0$	
	Col. 9A	Total = 0.05 ft	

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

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

Str. ID	D	Q	L	V	d	d_c	$V^2/2g$	S_f	Total Pipe Loss (table B)	EGL_o	K table B	$K(V^2/2g)$	EGL_i	HGL	U/S TOC	Surf. Elev.
	(ft)	(ft³/s)	(ft)	(fps)	(ft)		(ft)	(ft / ft)	(ft)	(ft)		(ft)	(ft)	(ft)	(ft)	(ft)
(1)	(2)	(3)	(4)	(5)	(6a)	(6b)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16
OUTLET										333.00				333.00		
44													333.07		332.71	332.71
43	2.0	6.75	55.8	2.15	FULL	n/a	0.07	0.0009	0.05	333.12	0.5	0.04	333.16	*333.16	346.06	347.76
											*(Drop Structure)					
43	(New Outlet)			2.6		0.8	0.10						345.56	345.46	346.06	347.76
		(Above calculations are for inlet end of STR 43 for conduit 42-43)														
42	2.0	6.75	14.1	2.6	1.56	0.80	0.10	0.001	0.014	345.57	0.62	0.06	346.11	346.01	345.73	349.31
41	1.5	5.10	328.0	8.65	0.56	0.85	1.16	-	0	355.79	-	-	355.98	355.10	356.17	360.0
40	1.5	3.35	361.0	7.52	0.43	0.70	0.88		0	-	0	0	366.50	366.50	367.0	370.0

Figure 1.32 Energy Grade Line Computation Sheet, Table A, for English Example

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

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

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

Figure 1.33 Energy Grade Line Computation Sheet, Table B, for English Example

Step 8	Col. 10A	$EGL_o = EGL_i$ + pipe loss EGL_o = 333.07 + 0.05 EGL_o = 333.12 ft	
		HGL = 333.12 - 0.07 = 333.05 TOC = 331.27 + 2.0 = 333.27	Check for full flow - close Assumption OK
Step 9	Col. 8B	Not applicable due to drop structure	
Step 10	Col. 9B and 11A	K_e = 0.5	Inflow pipe invert much higher than d_{aho}. Assume square edge entrance
Step 17	Col. 12A	$K(V^2/2g)$ = (0.50)(0.07) $K(V^2/2g)$ = 0.04 ft	Col. 11A times Col. 7A
Step 18	Col. 13A	$EGL_i = EGL_o$ EGL_i = 333.12 + 0.04 EGL_i = 333.16 ft	Col 10A plus 12A
Step 19	Col. 14A	HGL = EGL_i = 333.16 ft	For drop structures, the HGL is the same as the EGL
		d_{aho} = HGL- invert = 333.16 - 331.27 = 1.89 ft	Col. 8B
Step 20	Col. 15A	U/S TOC = Inv. + Dia. U/S TOC = 344.06 + 2.0 U/S TOC = 346.06 ft	From storm drain comp. sheet (Figure 1.32)
Step 21	Col. 16A	Surf. Elev. = 347.76 ft 347.76 > 333.09	From Figure 1.31. Surface elev. exceeds HGL, OK
Step 2	Col. 1A, 1B Col. 15A Col. 16A	Str. ID = 43 U/S TOC = 344.06 + 2.0 = 346.06 Surface Elev = 347.76	Drop Structure - new start
Step 3	 Col. 14A	HGL' = inv. + $(d_c$+D)/2 HGL' = 344.06 + (0.80 + 2.0)/2 HGL = 345.46 ft	Calculate new HGL - Use Case 2 d_c from Figure 1.19b
		EGL = HGL + $V^2/2g$	V = 2.6 fps from Prelim. Comp. Sht.
	 Col. 13A	EGL = 345.46 + 0.10 EGL = 345.56 ft	

Structure 42

Step 4	Col.1A	Str. ID = 42	
	Col. 2A	D = 2.0 ft	Pipe Diameter
	Col. 3A	Q = 6.75 cfs	Conduit discharge (design value)
	Col. 4A	L = 14.1 ft	Conduit length
Step 5A	Col. 5A	V = 2.6 ft/s Q/Qf = 6.75 / 7.12 = 0.95	For flow: Actual velocity from storm drain computation sheet.
	Col. 6A	d_n = 1.56 ft Chart 26	Figure 1.32
	Col. 7A	$V^2/2g = (2.6)^2/(2)(32.2)$ $V^2/2g = 0.10$ ft	Velocity head in conduit
Step 5B	Col.6bA	d_c = 0.80 ft	From HDS-5
Step 5C		$d_n < d_c$	Flow is subcritical
Step 6	Col. 8A	S_f = 0.001	Conduit not full so S_f = pipe slope d_n = 1.56 (Figure 1.19a) d_c = 0.80 (HDS-5) Flow is subcritical
Step 7		$H_f = S_f L$ H_f = (0.001) (14.1)	Equation 1.27 Col. 8A x Col. 5A
	Col. 2B	H_f = 0.014 ft h_b, H_c, H_e, H_j = 0	
	Col. 7B and 9A	Total = 0.014 ft	
Step 8		$EGL_o = EGL_i$ + total pipe loss EGL_o = 345.56 + 0.014	Col. 14A plus Col. 9A
	Col. 10A	EGL_o = 345.57 ft	
Step 9	Col. 8B	$d_{aho} = EGL_o$ - velocity head - pipe invert d_{aho} = 345.57 - 0.10 - 344.07 d_{aho} = 1.40 ft	Col. 10A - Column 7A - pipe invert
Step 10	Col. 9B	$K_o = 0.1(b/D_o)(1-\sin\Theta)+1.4(b/D_o)^{0.15}\sin(\Theta)$ b = 4.0 ft D_o = 2.0 ft $\theta = 90^o$ $K_o = 0.1(4.0/2.0)(1 - \sin 90)+$ $1.4(4.0/2.0)^{0.15} \sin 90$ K_o = 1.55	Equation 1.38 Access hole diameter. Col. 2A - outlet pipe diam Flow deflection angle
Step 11	Col. 10B	$C_D = (D_o/D_i)^3$ d_{aho} = 1.40 d_{aho}/D_o = (1.40/2.0) $d_{aho}/D_o = 0.70 < 3.2$ C_D = 1.0	Equation 1.39; pipe diameter Column 8B therefore

Step 12	Col. 11B	$C_d = 0.5\ (d_{aho}/\ D_o)^{0.6}$ $d_{aho}/D_o = 0.70 < 3.2$ $C_d = 0.5\ (1.4/2.0)^{0.6}$ $C_d = 0.40$	Equation 1.40; Flow depth correction.
Step 13	Col. 12B	$C_Q = (1\text{-}2 \sin \theta)(1\text{-}Q_i/Q_o)^{0.75}+1$ $C_Q = 1.0$	Equation 1.41; relative flow No additional pipes entering
Step 14	Col. 13B	$C_p = 1 + 0.2(h/D_o)[(h\text{-}d)/D_o]$ $C_p = 1.0$	Equation 1.42; plunging flow No plunging flow
Step 15	Col. 14B	$C_B = 1.0$	Benching Correction, flat floor (Table 1.17)
Step 16	Col. 15B and 11A	$K = K_o\ C_D\ C_d\ C_Q\ C_p\ C_B$ $K = (1.55)(1.0)(0.40)(1.0)(1.0)(1.0)$ $K = 0.62$	Equation 1.37
Step 17	Col. 12A	$K(V^2/2g) = (0.62)(0.10)$ $K(V^2/2g) = 0.06$ ft	Col. 11A times Col. 7A
Step 18	Col. 13A	$EGL_i = EGL_o + K(V_2/2g)$ $EGL_i = 346.05 + 0.06$ $EGL_i = 346.11$	Col. 10A plus 12A
Step 19	Col. 14A	$HGL = EGL_i - V^2/2g$ HGL = 346.11 - 0.10 HGL = 346.01 ft	Col. 13A minus Col. 7A
Step 20	Col 15A	U/S TOC = Inv. + Dia. U/S TOC = 344.23 + 1.5 U/S TOC = 345.73 ft	Information from storm drain comp.sheet (Figure 1.31)
Step 21	Col 16A	Surf. Elev. = 349.31 ft 349.31 > 345.96	From Figure 1.30 Surface elev. exceeds HGL, OK

Structure 41

Step 4	Col. 1A, 1B Col. 2A Col. 3A Col. 4A	Str. ID = 41 D = 1.50 ft Q = 5.10 cfs L = 328 ft	Next Structure Pipe Diameter Conduit discharge (design value) Conduit length
Step 5	Part full flow from column's 12 and 15 of storm drain computation sheet.		Continue with Step 5A
Step 5A	 Col. 6aA	$Q/Q_f = 5.1/18.1 = 0.28$ $d/d_f = 0.37$ d = (0.37) (1.5) d = 0.56 ft	 Figure 1.19a
	  Col. 5A	$V/V_f = 0.84$ V = (0.84)(10.3) V = 8.65 fps	Figure 1.19a

		$V^2/2g = (8.65)_2/(2)(32.2)$	Velocity head
	Col. 7A	$V^2/2g$ = 1.16 ft	
Step 5B	Col. 6bA	d_c = 0.85 ft	Figure 1.19b
Step 5C		0.56 < 0.85	Supercritical flow since $d_n < d_c$
Step 5D	Col. 7B	Total pipe loss = 0	

Structure 40

Step 5E	Col. 1A,1B	Str. Id. = 40	Next structure
	Col. 2A	D = 1.5 ft	Pipe diameter
	Col. 3A	Q = 3.35 cfs	Conduit discharge (design)
	Col. 4A	L = 361.0 ft	Conduit length
Step 5F		Q/Q_f = 3.3/18.1 = 0.18	
		d/d_c = 0.29	Figure 1.19a
		d = (0.29)(1.5)	
	Col. 6aA	d = 0.43 ft	
	Col. 6bA	d_c = 0.7 ft	Figure 1.19b
Step 5H		V/V_f = 0.73	Figure 1.19a
		V = (0.73)(10.3)	
	Col. 5A	V = 7.52 fps	
	Col. 7A	$V^2/2g = (7.52)^2/(2)(32.2)$	Velocity head
		$V^2/2g$ = 0.88 ft	
Step 5I		d_n = 0.43 ft < 0.70 ft= d_c	Supercritical flow since $d_n < d_c$
Step 5K	Col. 11A, and 15B	K = 0.0	Str. 41 line; supercritical flow;
	Col. 12A	$K(V^2/2g) = 0$	no structure losses

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

		HGL = Inv. + d	
		HGL = 354.07 + 0.56	D/S Invert + Flow depth
		HGL = 354.63 ft	
		EGL = 354.63 + 1.16	
		HGL + velocity head	
	Col. 10A	EGL = 355.79 ft	EGL_o of Str.41
		HGL = 354.67 + 0.43	U/S invert + Flow depth
	Col. 14A	HGL = 355.10 ft	Highest HGL
		EGL = 355.10 + 0.88	HGL + velocity head
	Col. 13A	EGL = 355.98 ft	EGL_i of Str. 41
Step 20	Col. 15A	U/S TOC = Inv. + Dia.	Information from storm drain comp
		U/S TOC = 354.67 + 1.5	Sheet (Figure 1.31) for Str. 41
		U/S TOC = 356.17 ft	
Step 21	Col. 16A	Surf. Elev. = 360.0 ft	From Figure 1.30.
		360.0 > 355.10	Surface elev. > HGL, OK

Step 10b	Col. 8B	d_{aho} = 0.67 (1.5) = 1.0 ft HGL = Str. 40 Inv. + d_{aho} HGL = 365.50 + 1.0.	Figure 1.31, HW/D = 0.67 Structure Inv. from storm drain comp. sheet
	Col. 14A	HGL = 366.50 ft	
	Col.13A	EGL = 366.50 ft	Assume no velocity in str.
Step 20	Col. 15A	U/S TOC = Inv. + Dia. U/S TOC = 365.5 + 1.5 U/S TOC = 367.0 ft	Information from storm drain comp. sheet (Figure 1.31) for Str. 40
Step 21	Col. 16A	Surf. Elev. = 370.0 ft 370.0 ft > 366.50 ft	From Figure 1.30 Surface Elev. > HGL, OK

See Figures 1.32 and 1.33 for the tabulation of results. The final HGL values are indicated in Figure 1.30.

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2.0 Storage Design

2.1 General Storage Concepts

2.1.1 Introduction

This section provides general guidance on stormwater runoff storage for meeting stormwater management control objectives (i.e., water quality protection, downstream streambank protection, and flood control).

Storage of stormwater runoff within a stormwater management system is essential to providing the extended detention of flows for water quality protection and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection. Runoff storage can be provided within an on-site system through the use of structural stormwater controls and/or nonstructural features and landscaped areas. Figure 2.1 illustrates various storage facilities that can be considered for a development site.

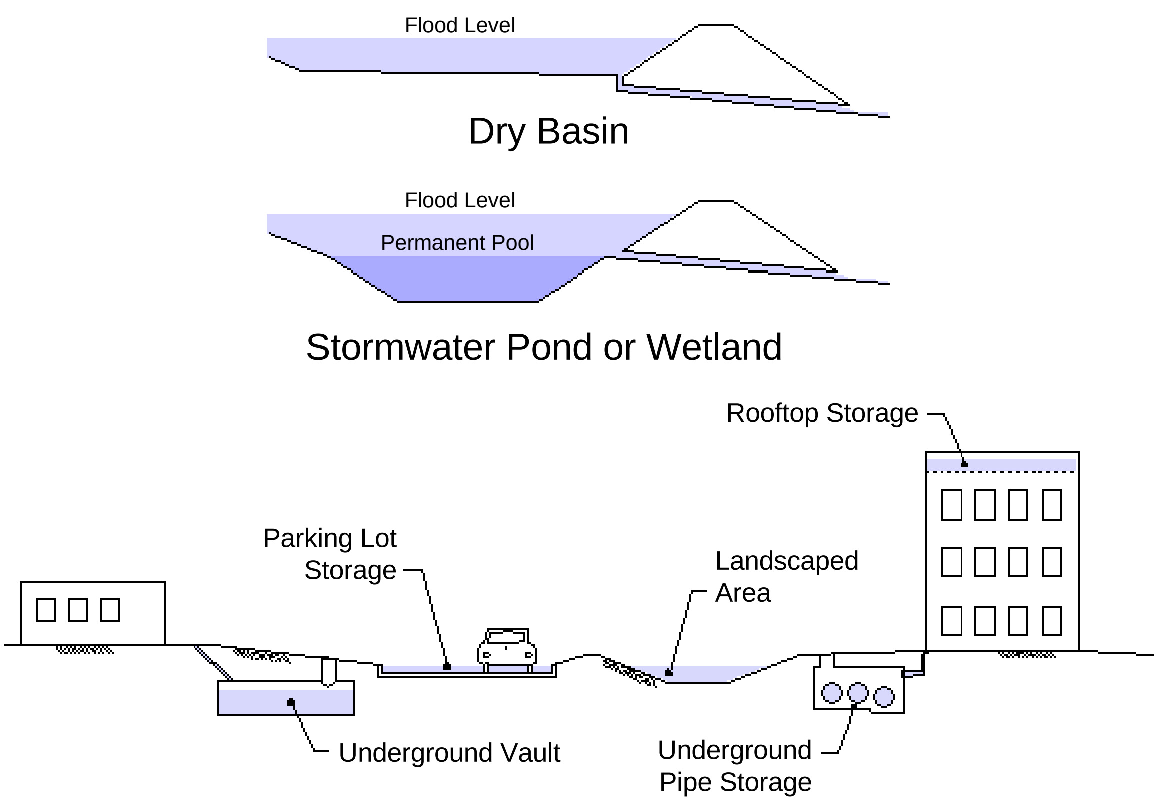

Figure 2.1 Examples of Typical Stormwater Storage Facilities

2.1.2 Storage Classification

Stormwater storage(s) can be classified as either detention, extended detention or retention. Some facilities include one or more types of storage.

Stormwater *detention* is used to reduce the peak discharge and detain runoff for a specified short period of time. Detention volumes are designed to completely drain after the design storm has passed. Detention is used to meet streambank protection criteria, and flood criteria where required.

Extended detention (ED) is used to drain a runoff volume over a specified period of time, typically 24 hours, and is used to meet streambank protection criteria. Some structural control designs (wet ED pond, micropool ED pond, and shallow ED marsh) also include extended detention storage of a portion of the water quality protection volume.

Retention facilities are designed to contain a permanent pool of water, such as stormwater ponds and wetlands, which is used for water quality protection.

Storage facilities are often classified on the basis of their location and size. *On-site* storage is constructed on individual development sites. *Regional* storage facilities are constructed at the lower end of a subwatershed and are designed to manage stormwater runoff from multiple projects and/or properties. A discussion of regional stormwater controls is found in *Section 1.0 of the Site Development Controls Technical Manual*.

Storage can also be categorized as *on-line* or *off-line*. On-line storage uses a structural control facility that intercepts flows directly within a conveyance system or stream. Off-line storage is a separate storage facility to which flow is diverted from the conveyance system. Figure 2.2 illustrates on-line versus off-line storage.

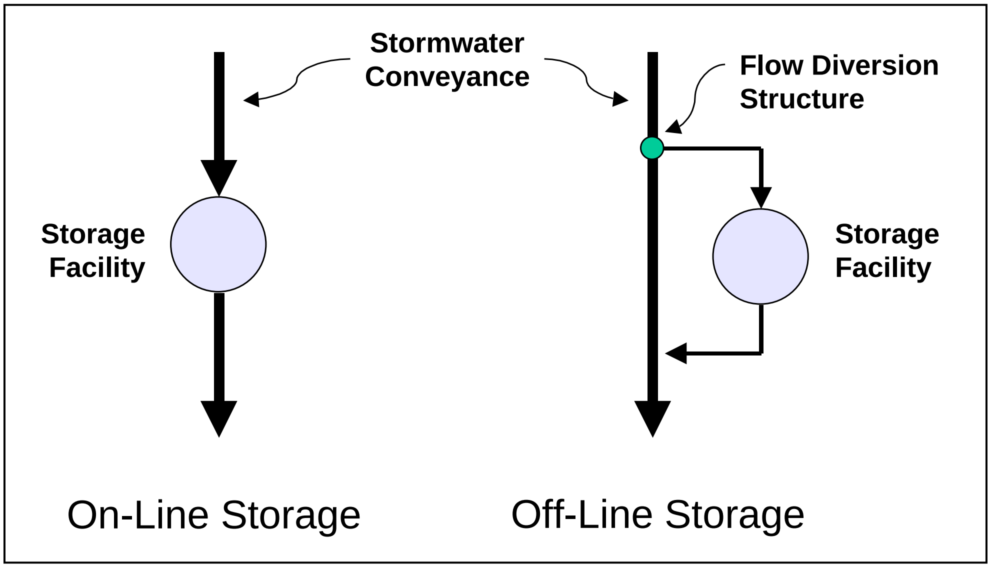

Figure 2.2 On-Line versus Off-Line Storage

2.1.3 Stage-Storage Relationship

A stage-storage curve defines the relationship between the depth of water and storage volume in a storage facility (see Figure 2.3). The volume of storage can be calculated by using simple geometric formulas expressed as a function of depth.

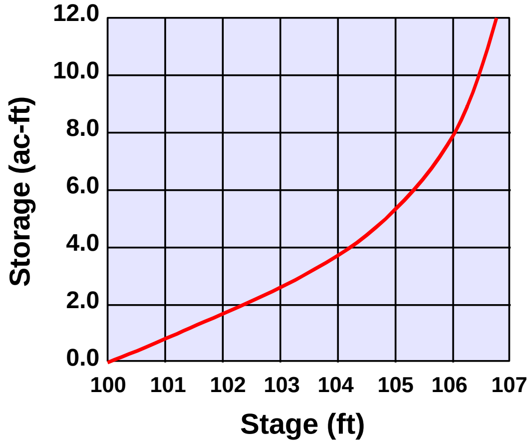

Figure 2.3 Stage-Storage Curve

The storage volume for natural basins may be developed using a topographic map and the double-end area, frustum of a pyramid, prismoidal or circular conic section formulas.

The double-end area formula (see Figure 2.4) is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad (2.1)$$

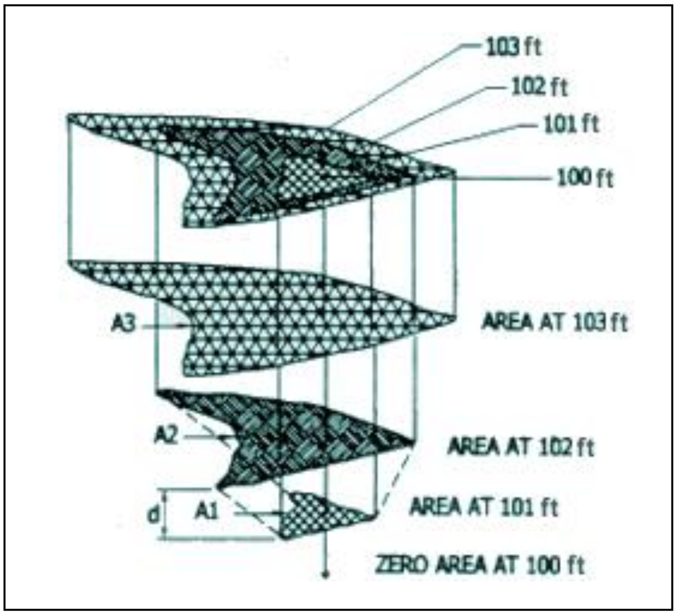

Figure 2.4 Double-End Area Method

where:

$V_{1,2}$ = storage volume (ft^3) between elevations 1 and 2
A_1 = surface area at elevation 1 (ft^2)
A_2 = surface area at elevation 2 (ft^2)
d = change in elevation between points 1 and 2 (ft)

The frustum of a pyramid formula is expressed as:

$$\mathbf{V = d/3\ [A_1 + (A_1 \times A_2)^{0.5} + A_2]} \quad \mathbf{(2.2)}$$

where:

V = volume of frustum of a pyramid (ft^3)
d = change in elevation between points 1 and 2 (ft)
A_1 = surface area at elevation 1 (ft^2)
A_2 = surface area at elevation 2 (ft^2)

The prismoidal formula for trapezoidal basins is expressed as:

$$\mathbf{V = LWD + (L + W)\ ZD^2 + 4/3\ Z^2\ D^3} \quad \mathbf{(2.3)}$$

where:

V = volume of trapezoidal basin (ft^3)
L = length of basin at base (ft)
W = width of basin at base (ft)
D = depth of basin (ft)
Z = side slope factor, ratio of horizontal to vertical

The circular conic section formula is:

$$\mathbf{V = 1.047\ D\ (R_1^2 + R_2^2 + R_1R_2)} \quad \mathbf{(2.4)}$$

$$\mathbf{V = 1.047\ D\ (3\ R_1^2 + 3ZDR_1 + Z_2D^2)} \quad \mathbf{(2.5)}$$

where:

R_1, R_2 = bottom and surface radii of the conic section (ft)
D = depth of basin (ft)
Z = side slope factor, ratio of horizontal to vertical

2.1.4 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility (see Figure 2.5). A typical storage facility has two outlets or spillways: a principal outlet and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity sufficient to convey the design flows without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. This spillway should be designed taking into account the potential threat to downstream areas if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway. For more details, see *Section 2.2*.

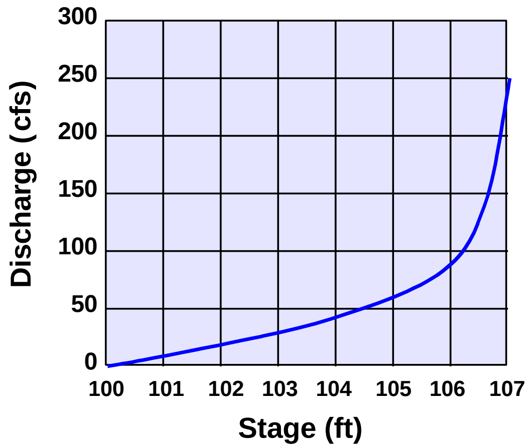

Figure 2.5 Stage-Discharge Curve

2.1.5 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.1 will be used. These symbols were selected because of their wide use in technical publications. 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 2.1 Symbols and Definitions

Symbol	Definition	Units
A	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
t	Routing time period	sec
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
K	Coefficient	-
I	Inflow rate	cfs
L	Length	ft
Q, q	Peak inflow or outflow rate	cfs, in
R	Surface Radii	ft
S, V_S	Storage volume	ft^3
t_b	Time base on hydrograph	hrs
T_I	Duration of basin inflow	hrs
t_P	Time to peak	hrs
V_S, S	Storage volume	ft^3, in, acre-ft
V_r	Volume of runoff	ft^3, in, acre-ft
W	Width of basin	ft
Z	Side slope factor	-

2.1.6 General Storage Design Procedures

Introduction

This section discusses the general design procedures for designing storage to provide standard detention of stormwater runoff for flood control (Q_f).

The design procedures for all structural control storage facilities are the same whether or not they include a permanent pool of water. In the latter case, the permanent pool elevation is taken as the "bottom" of storage and is treated as if it were a solid basin bottom for routing purposes.

It should be noted that the location of structural stormwater controls is very important as it relates to the effectiveness of these facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Therefore, a downstream peak flow analysis should be performed as part of the storage facility design process (see *Section 2.0 of the Hydrology Technical Manual*).

In multi-purpose multi-stage facilities such as stormwater ponds, the design of storage must be integrated with the overall design for water quality protection objectives. See *Section 1.0 of the Site Development Controls Technical Manual* for further guidance and criteria for the design of structural stormwater controls.

Data Needs

The following data are needed for storage design and routing calculations:

- Inflow hydrograph for all selected design storms (this can be generated using a unit hydrograph method or the Modified Rational Method, see the *Hydrology Technical Manual* for more details)
- Stage-storage curve for proposed storage facility
- Stage-discharge curve for all outlet control structures

Design Procedure

A general procedure for using the above data in the design of storage facilities is presented below.

Step 1 Compute inflow hydrograph for runoff from the "Conveyance" (e.g., Q_{p25}) and flood mitigation (Q_{p100}) design storms using the hydrologic methods outlined in *Section 1.0 of the Hydrology Technical Manual*. Both existing- and post-development hydrographs are required for both the "Conveyance" and flood mitigation design storms.

Step 2 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see *Section 2.1.7*).

Step 3 Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used. From the selected shape determine the maximum depth in the pond.

Step 4 Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 5 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed post-development peak discharges from the "Conveyance" design storm exceed the existing-development peak discharges, then revise the available storage volume, outlet device, etc., and return to Step 3.

Step 6 Perform routing calculations using the flood mitigation hydrograph to determine if any increases in downstream flows from this hydrograph will cause damages and/or drainage and flooding problems. If problems will be created (e.g., flooding of habitable dwellings, property damage, or public access and/or utility interruption) then the storage facility must be designed to control the increased flows from the flood mitigation storm. If not then consider emergency overflow from runoff due to the flood mitigation (or larger) design storm and established freeboard requirements.

Step 7 Evaluate the downstream effects of detention outflows for the "Conveyance" and flood mitigation storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system to the location where the discharge from the proposed development no longer has a significant impact upon the receiving stream or storm drainage system (see *Section 2.0 of the Hydrology Technical Manual*).

Step 8 Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use in designing detention facilities has not produced acceptable results in many areas of the country, including North Central Texas.

Although hand calculation procedures are available for routing hydrographs through storage facilities, they are very time consuming, especially when several different designs are evaluated. Many standard hydrology and hydraulics textbooks give examples of hand-routing techniques. For this Manual, it assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

2.1.7 Preliminary Detention Calculations

Introduction

Procedures for preliminary detention calculations are included here to provide a simple method that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs. Standard routing should be used for actual (final) storage facility calculations and design.

Storage Volume

For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 2.6.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5T_i\,(Q_i - Q_O) \tag{2.6}$$

where:

V_S = storage volume estimate (ft^3)

Q_i = peak inflow rate (cfs)

Q_O = peak outflow rate (cfs)
T_i = duration of basin inflow (s)

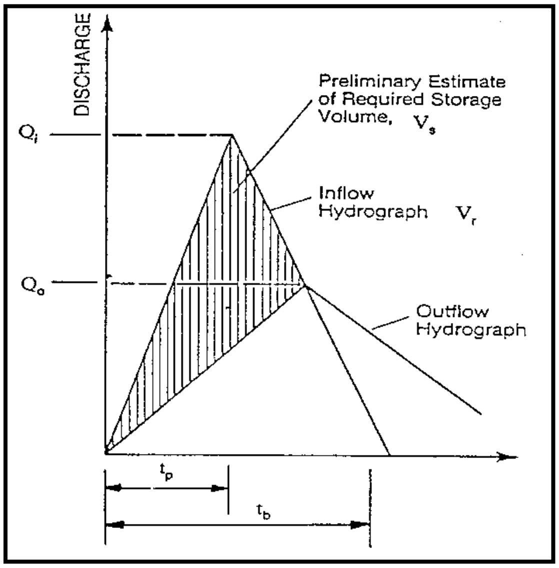

Figure 2.6 Triangular-Shaped Hydrographs
(For Preliminary Estimate of Required Storage Volume)

Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff and Singh, 1976).

Determine input data, including the allowable peak outflow rate, Q_O, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and the following equation:

$$V_s/V_r = \frac{1.291(1 - Q_o/Q_i)^{0.753}}{(t_b/t_p)^{0.411}} \qquad (2.7)$$

where:

V_S = volume of storage (in)
V_r = volume of runoff (in)
Q_O = outflow peak flow (cfs)
Q_i = inflow peak flow (cfs)
t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak]
t_p = time to peak of the inflow hydrograph (hr)

Multiply the volume of runoff, V_r, times the ratio V_S/V_r, calculated in Step 2 to obtain the estimated storage volume V_S.

Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

Determine volume of runoff, V_r, peak flow rate of the inflow hydrograph, Q_i, time base of the inflow hydrograph, t_b, time to peak of the inflow hydrograph, t_p, and storage volume V_S.

Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Wycoff and Singh, 1976):

$$Q_O/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546} \qquad (2.8)$$

where:

- Q_O = outflow peak flow (cfs)
- Q_i = inflow peak flow (cfs)
- V_S = volume of storage (in)
- V_r = volume of runoff (in)
- t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak]
- t_p = time to peak of the inflow hydrograph (hr)

Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_O, for the selected storage volume.

2.2 Outlet Structures

2.2.1 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.2 will be used. These symbols were selected because of their wide use in technical publications. 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 2.2 Symbols and Definitions

Symbol	Definition	Units
A, a	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
B	Breadth of weir	ft
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
K, k	Coefficient	-
L	Length	ft
n	Manning's n	-
Q, q	Peak inflow or outflow rate	cfs, in
V_u	Approach velocity	ft/s
WQ_v	Water quality protection volume	ac ft
w	Maximum cross sectional bar width facing the flow	in
x	Minimum clear spacing between bars	in
θ	Angle of v-notch	degrees
θ_g	Angle of the grate with respect to the horizontal	degrees

2.2.2 Primary Outlets

Introduction

Primary outlets provide the critical function of the regulation of flow for structural stormwater controls. There are several different types of outlets that may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the stormwater facility can be designed as a simple pipe or culvert. For multistage control structures, the inlet is designed considering a range of design flows.

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

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

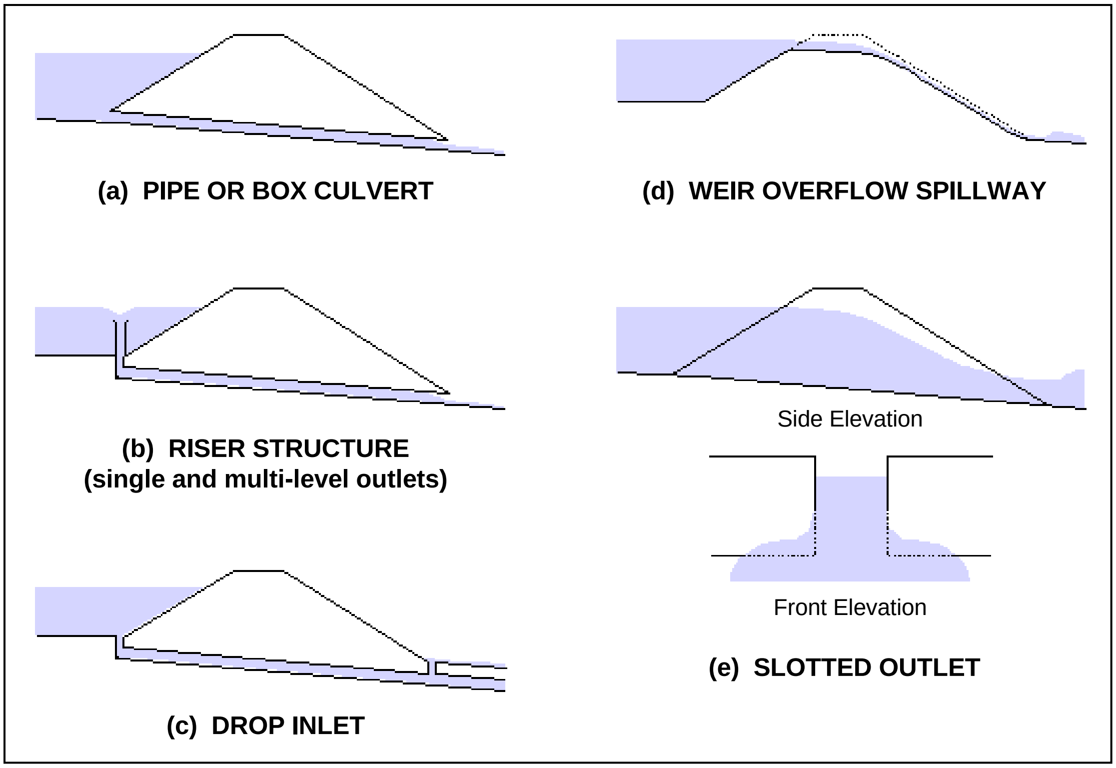

Figure 2.7 Typical Primary Outlets

Outlet Structure Types

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

- Orifices
- Perforated risers
- Pipes / Culverts
- Sharp-crested weirs
- Broad-crested weirs
- V-notch weirs
- Proportional weirs
- Combination outlets

The design professional must pay attention to material types and construction details when designing an outlet structure or device. Non-corrosive material and mounting hardware are key to device longevity, ease of operation, and low cost maintenance. Special attention must also be paid to not placing dissimilar metal materials together where a cathodic reaction will cause deterioration and destruction of metal parts.

Protective coatings, paints, and sealants must also be chosen carefully to prevent contamination of the stormwater flowing through the structure/device. This is not only important while they are being applied, but also as these coating deteriorate and age over the functional life of the facility.

Each of these outlet types has a different design purpose and application:

- Water quality and streambank protection flows are normally handled with smaller, more protected outlet structures such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs.

- Larger flows, such as flood flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway through the embankment. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice.

For a single orifice, as illustrated in Figure 2.8(a), the orifice discharge can be determined using the standard orifice equation below.

$$Q = CA\,(2gH)^{0.5} \quad \textbf{(2.9)}$$

where:

Q = the orifice flow discharge (cfs)

C = discharge coefficient

A = cross-sectional area of orifice or pipe (ft^2)

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

H = effective head on the orifice, from the center of orifice to the water surface (ft)

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in Figure 2.8(b).

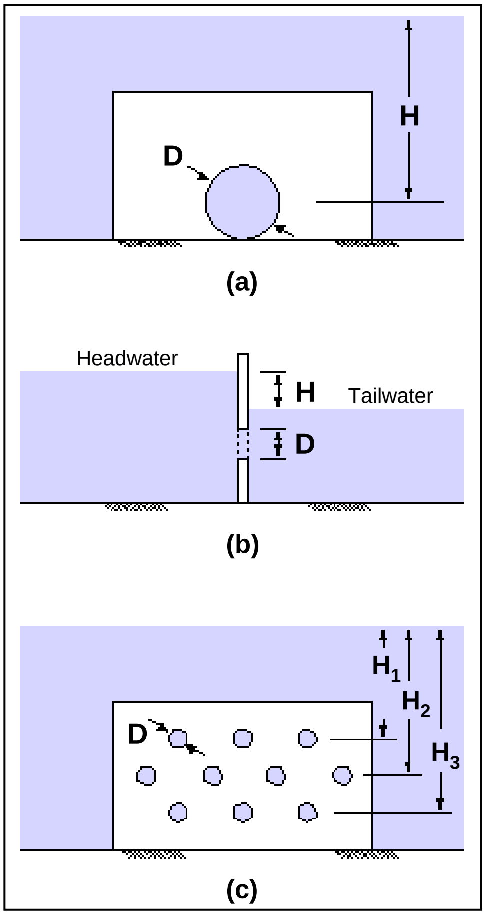

Figure 2.8 Orifice Definitions

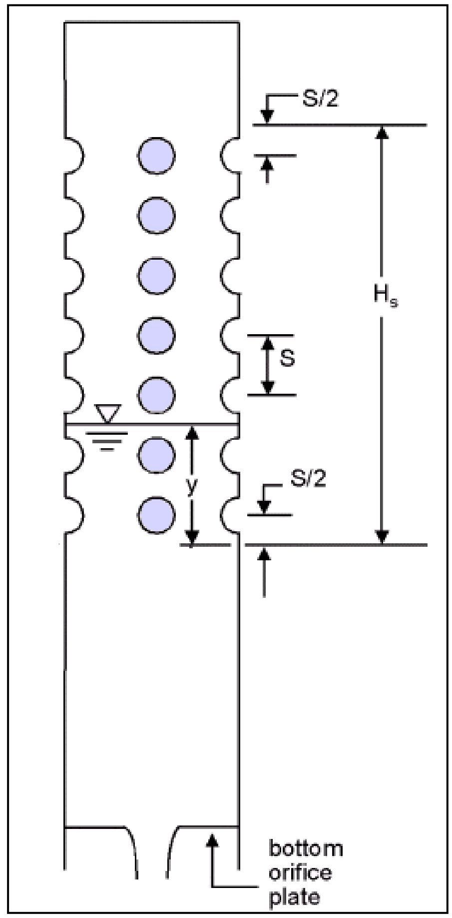

Figure 2.9 Perforated Riser

When the material is thinner than the orifice diameter, with sharp edges, a coefficient of 0.6 should be used. For square-edged entrance conditions the generic orifice equation can be simplified:

$$Q = 0.6A\,(2gH)^{0.5} = 3.78D^2H^{0.5} \qquad (2.10)$$

where:

D = diameter of orifice or pipe (ft)

When the material is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used.

Flow through multiple orifices, such as the perforated plate shown in Figure 2.8(c), can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

Perforated orifice plates for the control of discharge can be of any size and configuration. However, the Denver Urban Drainage and Flood Control District has developed standardized dimensions that have worked well. Table 2.3 gives appropriate dimensions. The vertical spacing between hole centerlines is always 4 inches.

Table 2.3 Circular Perforation Sizing

Hole Diameter (in)	Minimum Column Hole Centerline Spacing (in)	Flow Area per Row (in²)		
		1 column	2 columns	3 columns
1/4	1	0.05	0.1	0.15
5/16	2	0.08	0.15	0.23
3/8	2	0.11	0.22	0.33
7/16	2	0.15	0.3	0.45
1/2	2	0.2	0.4	0.6
9/16	3	0.25	0.5	0.75
5/8	3	0.31	0.62	0.93
11/16	3	0.37	0.74	1.11
3/4	3	0.44	0.88	1.32
13/16	3	0.52	1.04	1.56
7/8	3	0.6	1.2	1.8
15/16	3	0.69	1.38	2.07
1	4	0.79	1.58	2.37
1 1/16	4	0.89	1.78	2.67
1 1/8	4	0.99	1.98	2.97
1 3/16	4	1.11	2.22	3.33
1 1/4	4	1.23	2.46	3.69
1 5/16	4	1.35	2.7	4.05
1 3/8	4	1.48	2.96	4.44
1 7/16	4	1.62	3.24	4.86
1 1/2	4	1.77	3.54	5.31
1 9/16	4	1.92	3.84	5.76
1 5/8	4	2.07	4.14	6.21
1 11/16	4	2.24	4.48	6.72
1 3/4	4	2.41	4.82	7.23
1 13/16	4	2.58	5.16	7.74
1 7/8	4	2.76	5.52	8.28
1 15/16	4	2.95	5.9	8.85
2	4	3.14	6.28	9.42
Number of columns refers to parallel columns of holes				
Minimum plate thickness		1/4”	5/16”	3/8”

Source: Urban Drainage and Flood Control District, Denver, CO

For rectangular slots the height is normally 2 inches with variable width. Only one column of rectangular slots is allowed.

Figure 2.10 provides a schematic of an orifice plate outlet structure for a wet extended detention pond showing the design pool elevations and the flow control mechanisms.

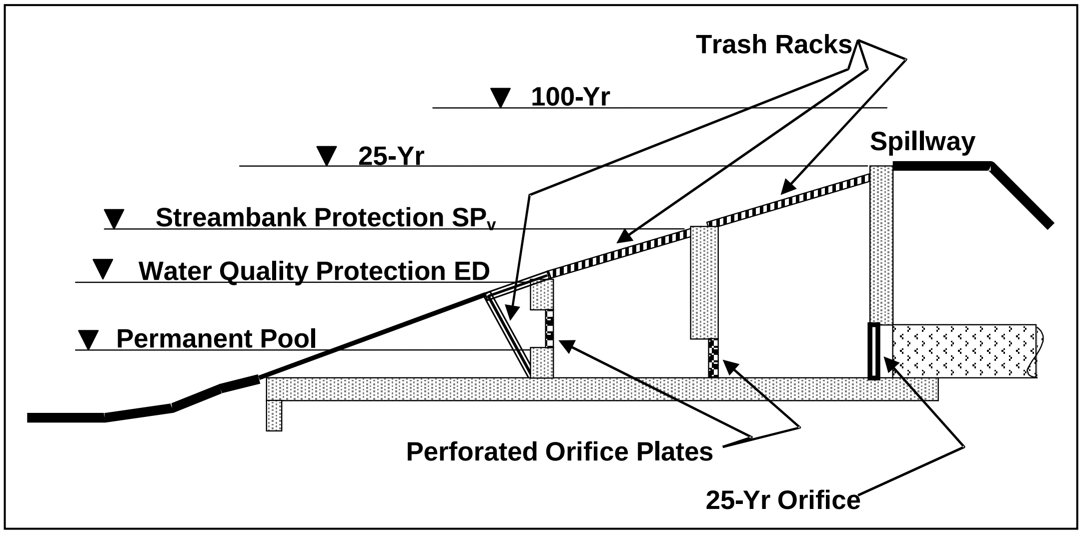

Figure 2.10 Schematic of Orifice Plate Outlet Structure

Perforated Risers

A special kind of orifice flow is a perforated riser as illustrated in Figure 2.9. In the perforated riser, an orifice plate at the bottom of the riser, or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so as not to become the control.

Referring to Figure 2.9, a shortcut formula has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988):

$$Q = C_p[(2A_p)/(3H_s)]*(\sqrt{2g})*H^{3/2} \qquad (2.11)$$

where:

Q = discharge (cfs)

C_p = discharge coefficient for perforations (normally 0.61)

A_p = cross-sectional area of all the holes (ft²)

H_s = distance from S/2 below the lowest row of holes to S/2 above the top row (ft)

Pipes and Culverts

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

Pipes smaller than 12 inches in diameter may be analyzed as a submerged orifice as long as H/D is greater than 1.5. Note: For low flow conditions when the flow reaches and begins to overflow the pipe, weir flow controls. As the stage increases the flow will transition to orifice flow.

Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in *Section 3.3*, or by using Equation 2.12 (NRCS, 1984).

The following equation is a general pipe flow equation derived through the use of the Bernoulli and continuity principles.

$$Q = a[(2gH) / (1 + k_m + k_pL)]0.5 \quad \textbf{(2.12)}$$

where:

Q = discharge (cfs)
a = pipe cross sectional area (ft^2)
g = acceleration of gravity (ft/s^2)
H = elevation head differential (ft)
k_m = coefficient of minor losses (use 1.0)
k_p = pipe friction coefficient = $5087n^2/D^{4/3}$
L = pipe length (ft)

Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear as it overflows, the overflow is termed a *sharp-crested* weir. If the sides of the weir also cause the through flow to contract, it is termed an *end-contracted* sharp-crested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a measurement device. A sharp-crested weir with compensation for end contractions is illustrated in Figure 2.11(a). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$Q = [(3.27 + 0.4(H/H_C)]\ LH^{1.5} \quad \textbf{(2.13)}$$

were:

Q = discharge (cfs)
H = head above weir crest excluding velocity head (ft)
H_C = height of weir crest above channel bottom (ft)
L = horizontal weir length (ft)

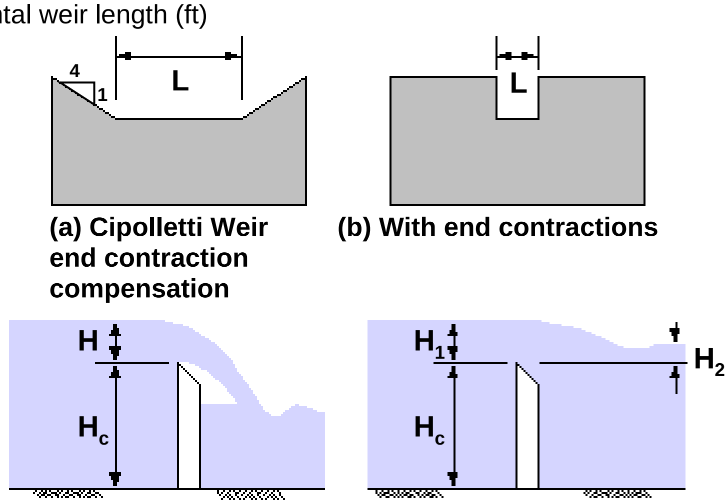

Figure 2.11 Sharp-Crested Weir

The discharge equation for the Cipolletti Weir is $Q = 3.367\ LH^{1/2}$

A sharp-crested weir with two end contractions is illustrated in Figure 2.11(b). The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$\mathbf{Q = [(3.27 + 0.4(H/H_C)]\ (L - 0.2H)\ H^{1.5}} \qquad \textbf{(2.14)}$$

where:

- Q = discharge (cfs)
- H = head above weir crest excluding velocity head (ft)
- H_C = height of weir crest above channel bottom (ft)
- L = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$\mathbf{Q_S = Q_f\,(1 - (H_2/H_1)^{1.5})^{0.385}} \qquad \textbf{(2.15)}$$

where:

- Q_S = submergence flow (cfs)
- Q_f = free flow (cfs)
- H_1 = upstream head above crest (ft)
- H_2 = downstream head above crest (ft)

Broad-Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a *broad-crested* weir. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner. If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

The equation for the broad-crested weir is (Brater and King, 1976):

$$\mathbf{Q = CLH^{1.5}} \qquad \textbf{(2.16)}$$

where:

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

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 2.4.

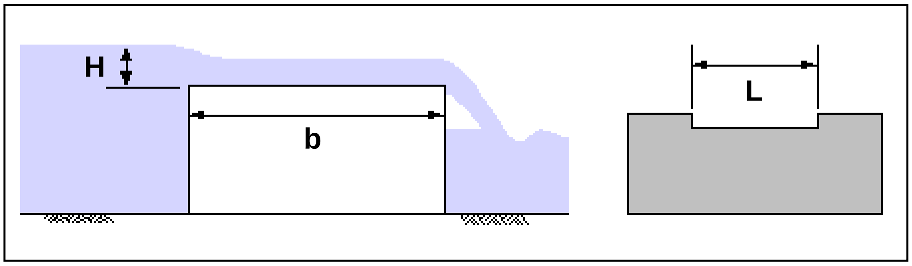

Figure 2.12 Broad-Crested Weir

Table 2.4 Broad-Crested Weir Coefficient (C) Values

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

* Measured at least 2.5H upstream of the weir.
Source: Brater and King (1976)

V-Notch Weirs

The discharge through a V-notch weir (Figure 2.13) can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan(\theta/2)\ H^{2.5} \qquad (2.17)$$

where:

Q = discharge (cfs)

θ = angle of V-notch (degrees)

H = head on apex of notch (ft)

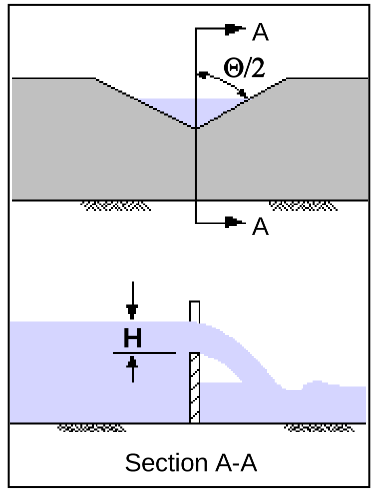

Figure 2.13 V-Notch Weir

Proportional Weirs

Although it may be more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. A typical proportional weir is shown in Figure 2.14. Design equations for proportional weirs are (Sandvik, 1985):

$$Q = 4.97\ a^{0.5}\ b\ (H - a/3) \qquad (2.18)$$

$$x/b = 1 - (1/3.17)\ (\arctan\ (y/a)^{0.5}) \qquad (2.19)$$

where:

Q = discharge (cfs)

Dimensions a, b, H, x, and y are shown in Figure 2.14

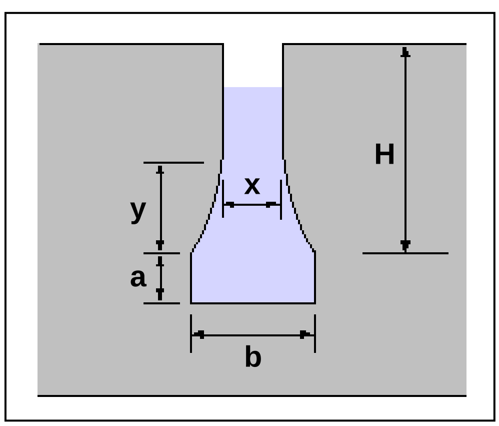

Figure 2.14 Proportional Weir Dimensions

Combination Outlets

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

They are generally two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs, or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure 2.15 shows an example of a riser designed for a wet extended detention pond. The orifice plate outlet structure in Figure 2.9 is another example of a combination outlet.

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

The use of a combination outlet requires the construction of a composite stage-discharge curve (as shown in Figure 2.16) suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed later in this section.

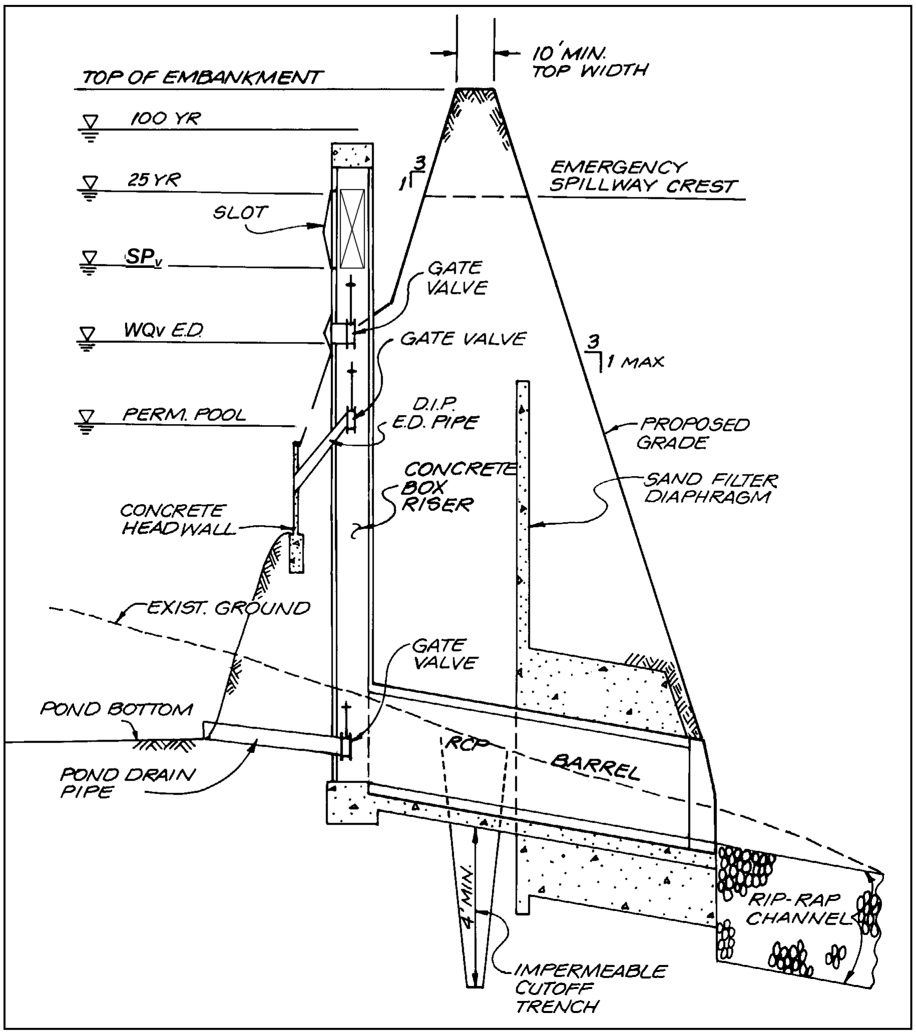

Figure 2.15 Schematic of Combination Outlet Structure

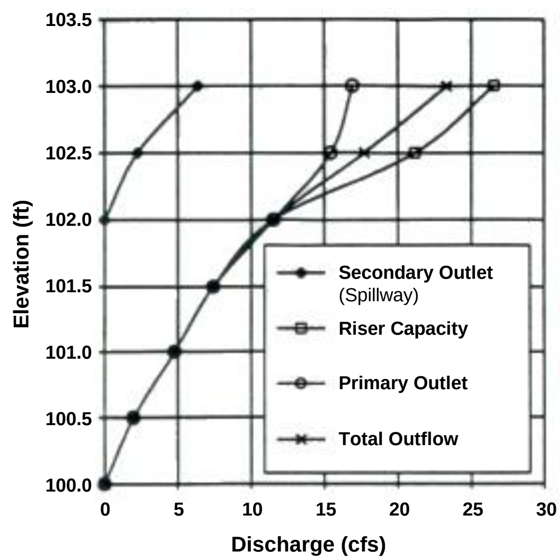

Figure 2.16 Composite Stage-Discharge Curve

2.2.3 Extended Detention (Water Quality and Streambank Protection) Outlet Design

Introduction

Extended detention (ED) orifice sizing is required in design applications that provide extended detention for downstream streambank protection or the ED portion of the water quality protection volume. The release rate for both the WQ_v and SP_v should be one that discharges the ED volume in a period of 24 hours or longer. In both cases an extended detention orifice or reverse slope pipe can be used for the outlet. For a structural control facility providing both WQ_v extended detention and SP_v control (wet ED pond, micropool ED pond, and shallow ED wetland), there will be a need to design two outlet orifices – one for the water quality control outlet and one for the streambank protection drawdown.

(The following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999)

The outlet hydraulics for peak control design (flood control) is usually straightforward in that an outlet is selected to limit the peak flow to some predetermined maximum. Since volume and the time required for water to exit the storage facility are not usually considered, the outlet design can easily be calculated and routing procedures used to determine if quantity design criteria are met.

In an extended detention facility for water quality protection or downstream streambank protection, however, the storage volume is detained and released for each over a specified amount of time (e.g., 24-hours). The release period is a "brim" drawdown time, beginning at the time of peak storage of the WQ_v or SP_v until the entire calculated volume drains out of the basin. This assumes the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following methods:

- Using the maximum hydraulic head associated with the brim storage volume and maximum discharge, calculate the orifice size needed to achieve the required drawdown time. Route the volume through the basin to verify the actual storage volume used and the drawdown time.

- Approximate the orifice size using the average hydraulic head associated with the storage volume and the required drawdown time.

These two procedures are outlined in the examples below and can be used to size an extended detention orifice for water quality and/or streambank protection.

Method 1: Maximum Hydraulic Head with Routing

A wet ED pond sized for the required water quality protection volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Given the following information, calculate the required orifice size for water quality protection design.

Given: Water Quality Protection Volume (WQ_v) = 0.76 ac ft = 33,106 ft^3
Maximum Hydraulic Head (H_{max}) = 5.0 ft (from stage vs. storage data)

Step 1 Determine the maximum discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the Water Quality Protection Volume (or Streambank Protection Volume) by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

Q_{avg} = 33,106 ft^3 / (24 hr)(3,600 s/hr) = 0.38 cfs
Q_{max} = 2 * Q_{avg} = 2 * 0.38 = 0.76 cfs

Step 2 Determine the required orifice diameter by using the orifice Equation 2.9 and Q_{max} and H_{max}:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$

$A = 0.76 / 0.6[(2)(32.2)(5.0)]^{0.5} = 0.071\ ft^2$

Determine pipe diameter from $A = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$

$D = [4(0.071)/3.14]^{0.5} = 0.30\ ft = 3.61\ in$

Use a 3.6-inch diameter water quality protection orifice.

Routing the water quality protection volume of 0.76 ac ft through the 3.6-inch water quality protection orifice will allow the designer to verify the drawdown time, as well as the maximum hydraulic head elevation. The routing effect will result in the actual drawdown time being less than the calculated 24 hours. Judgment should be used to determine whether the orifice size should be reduced to achieve the required 24 hours.

Method 2: Average Hydraulic Head and Average Discharge

Using the data from the previous example use Method 2 to calculate the size of the outlet orifice.

Given: Water Quality Protection Volume (WQ_v) = 0.76 ac ft = 33,106 ft^3
Average Hydraulic Head (h_{avg}) = 2.5 ft (from stage vs storage data)

Step 1 Determine the average release rate to release the water quality protection volume over a 24-hour time period.

Q = 33,106 ft^3 / (24 hr)(3,600 s/hr) = 0.38 cfs

Step 2 Determine the required orifice diameter by using the orifice Equation 2.9 and the average head on the orifice:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$

$A = 0.38 / 0.6[(2)(32.2)(2.5)]^{0.5} = 0.05\ ft^3$

Determine pipe diameter from $A = 3.14r^2 = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$

$D = [4(0.05)/3.14]^{0.5} = 0.252$ ft = 3.03 in

Use a 3-inch diameter water quality protection orifice.

Use of Method 1, utilizing the maximum hydraulic head and discharge and routing, results in a 3.6-inch diameter orifice (though actual routing may result in a changed orifice size) and Method 2, utilizing average hydraulic head and average discharge, results in a 3.0-inch diameter orifice.

2.2.4 Multi-Stage Outlet Design

Introduction

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements (e.g., water quality protection, streambank protection, and flood control) for stormwater ponds, stormwater wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms. Figures 2.9 and 2.15 are examples of multi-stage combination outlet systems.

A design engineer may be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite of the different outlets that are used for different elevations within the multi-stage riser (see Figure 2.16)

Multi-Stage Outlet Design Procedure

Below are the steps for designing a multi-stage outlet. Note that if a structural control facility will not control one or more of the required storage volumes (WQ_v, SP_v, and Q_f), then that step in the procedure is skipped.

Step 1 Determine Stormwater Control Volumes. Using the procedures from *Section 1.0 of the Hydrology and Water Quality Technical Manuals*, estimate the required storage volumes for water quality protection (WQ_v), streambank protection (SP_v), and flood control (Q_f).

Step 2 Develop Stage-Storage Curve. Using the structure geometry and topography, develop the stage-storage curve for the facility in order to provide sufficient storage for the control volumes involved in the design.

Step 3 Design Water Quality Protection Outlet. Design the water quality protection extended detention (WQ_v-ED) orifice using either Method 1 or Method 2 outlined in *Section 2.2.3*. If a permanent pool is incorporated into the design of the facility, a portion of the storage volume for water quality protection will be above the elevation of the permanent pool. The outlet can be protected using either a reverse slope pipe, a hooded protection device, or another acceptable method (see *Section 2.2.5*).

Step 4 Design Streambank Protection Outlet. Design the streambank protection extended detention outlet (SP_v-ED) using either method from *Section 2.2.3*. For this design, the storage needed for streambank protection will be greater than the water quality protection volume storage elevation determined in Step 3. The total stage-discharge rating curve at this point will include the water quality protection orifice and the outlet used for streambank protection. The outlet should be protected in a manner similar to that for the water quality protection orifice.

Step 5 Design Flood Control Outlet. The storage needed for flood control will be greater than the water quality protection and streambank protection storage. Establish the Q_f maximum water surface elevation using the stage-storage curve and subtract the SP_v elevation to find the maximum head. Select an outlet type and calculate the initial size and geometry based upon maintaining the predevelopment peak discharge rate. Develop a stage-discharge curve for the combined set of outlets (WQ_v, SP_v and Q_f).

Step 6 Check Performance of the Outlet Structure. Perform a hydraulic analysis of the multi-stage outlet structure using reservoir routing to ensure that all outlets will function as designed. Several iterations may be required to calibrate and optimize the hydraulics and outlets that are used. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This usually requires that the outlet structure have a larger cross-sectional area than the outlet conduit.

The hydraulic analysis of the design must take into account the hydraulic changes that will occur as depth of storage changes for the different design storms. As shown in Figure 2.17, as the water passes over the rim of a riser, the riser acts as a weir. However, when the water surface reaches a certain height over the rim of a riser, the riser will begin to act as a submerged orifice. The designer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place for an outlet where this transition will occur. Also note in Figure 2.17 that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. Figure 2.18 shows another condition where weir flow can change to orifice flow, which must be taken into account in the hydraulics of the rating curve as different design conditions results in changing water surface elevations.

Step 7 Size the Emergency Spillway. It is recommended that all stormwater impoundment structures have a vegetated emergency spillway (see *Section 2.2.7*). An emergency spillway provides a degree of safety to prevent overtopping of an embankment if the primary outlet or principal spillway should become clogged, or otherwise inoperative. The flood mitigation storm should be routed through the outlet devices and emergency spillway to ensure the hydraulics of the system will operate as designed. Also check the dam safety requirements to be sure of an adequate design.

Step 8 Design Outlet Protection. Design necessary outlet protection and energy dissipation facilities to avoid erosion problems downstream from outlet devices and emergency spillway(s). See *Section 4.0*, for more information.

Step 9 Perform Buoyancy Calculations. Perform buoyancy calculations for the outlet structure and footing. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.

Step 10 Provide Seepage Control. Seepage control should be provided for the outflow pipe or culvert through an embankment. The two most common devices for controlling seepage are (1) filter and drainage diaphragms and (2) anti-seep collars.

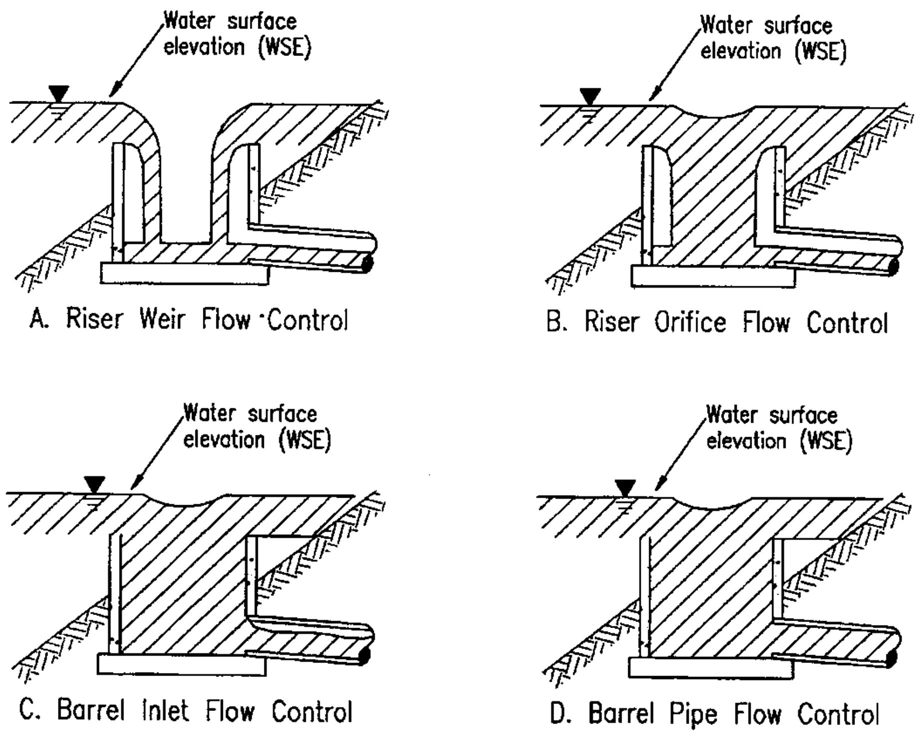

Figure 2.17 Riser Flow Diagrams
(Source: VDCR, 1999)

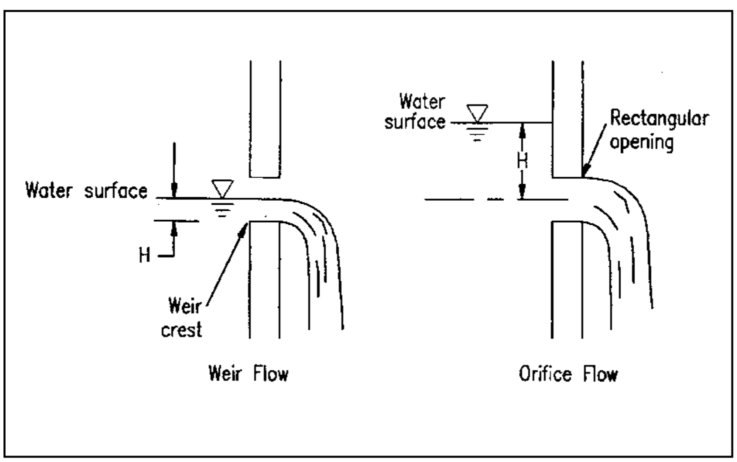

Figure 2.18 Weir and Orifice Flow
(Source: VDCR, 1999)

2.2.5 Extended Detention Outlet Protection

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

The use of a reverse slope pipe attached to a riser for a stormwater pond or wetland with a permanent pool (see Figure 2.19). The inlet is submerged a minimum of 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.

The use of a hooded outlet for a stormwater pond or wetland with a permanent pool is shown in Figures 2.20 and 2.21.

Internal orifice protection through the use of an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket (see Figure 2.22).

Internal orifice size requirements may be attained by the use of adjustable gate valves to achieve an equivalent orifice diameter.

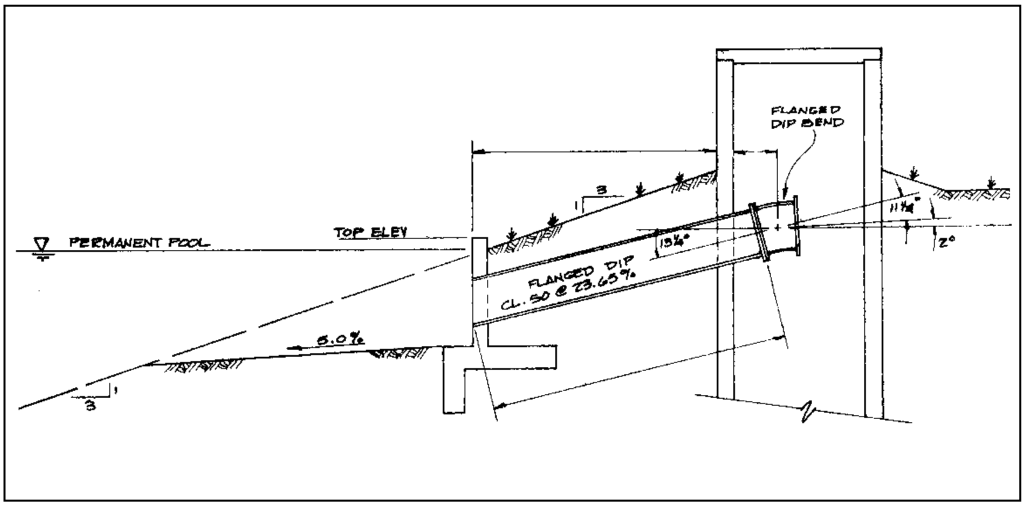

Figure 2.19 Reverse Slope Pipe Outlet

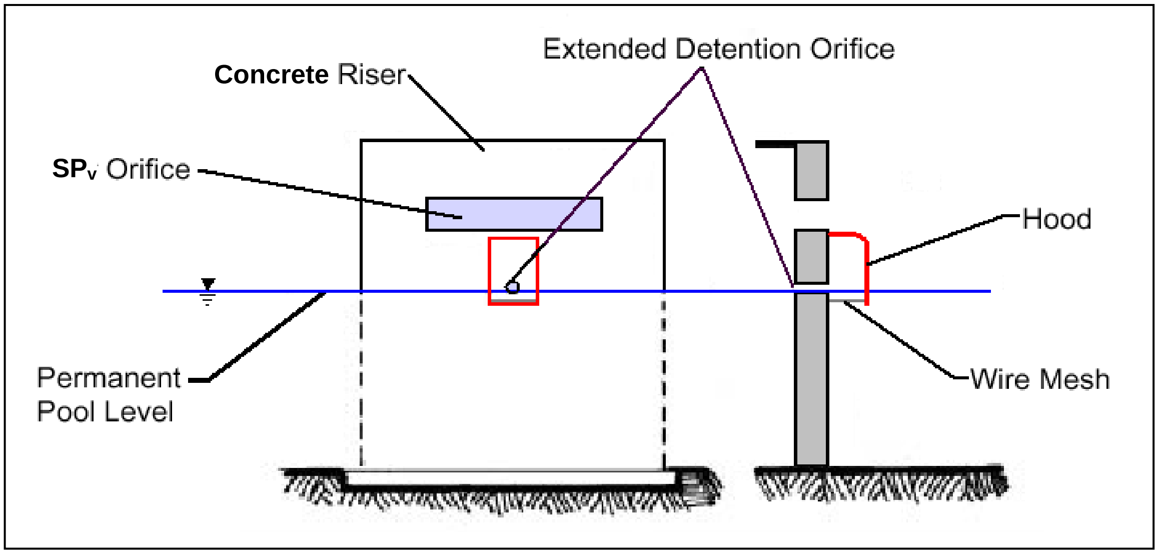

Figure 2.20 Hooded Outlet

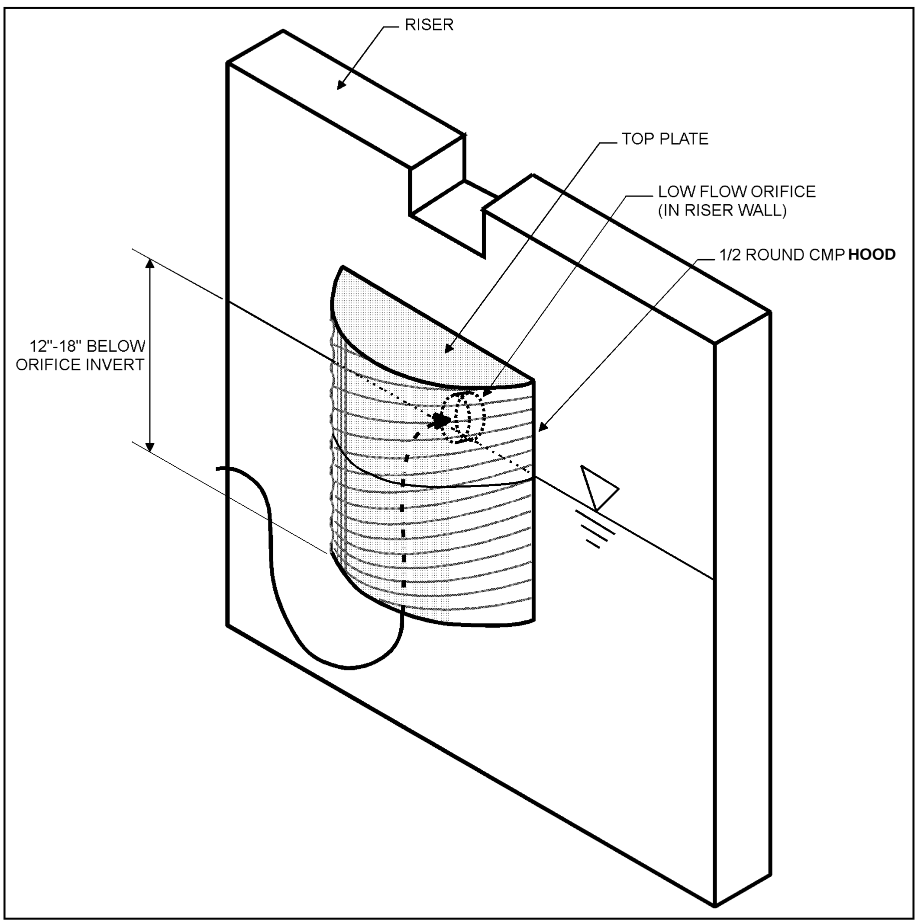

Figure 2.21 Half-Round CMP Orifice Hood

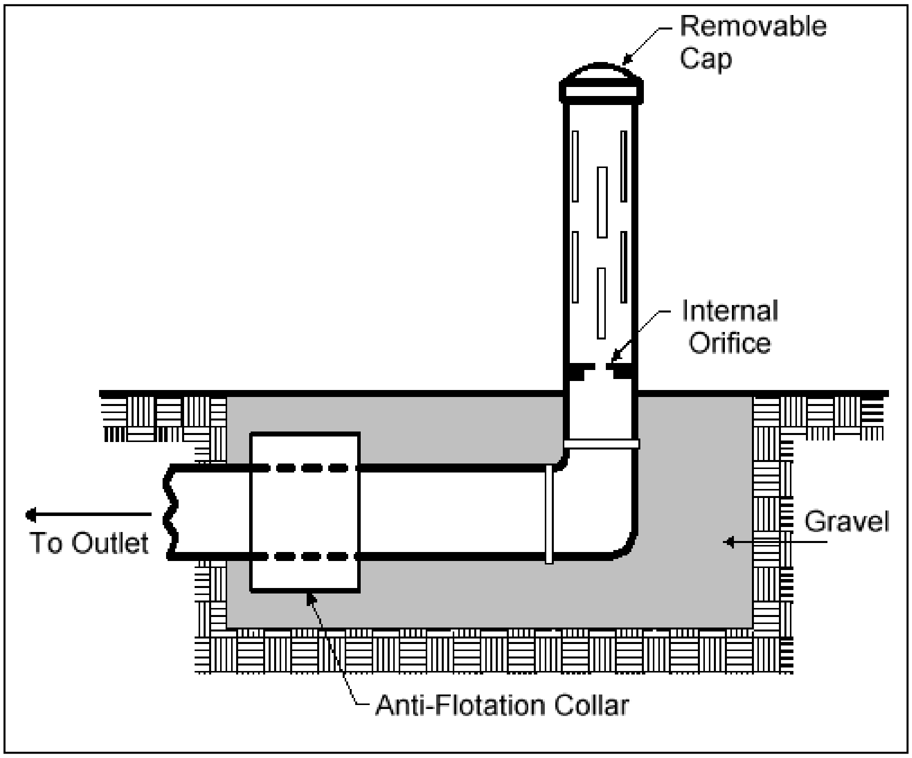

Figure 2.22 Internal Control for Orifice Protection

2.2.6 *Trash Racks and Safety Grates*

Introduction

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

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

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

An example of trash racks used on a riser outlet structure is shown in Figure 2.23. The inclined vertical bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

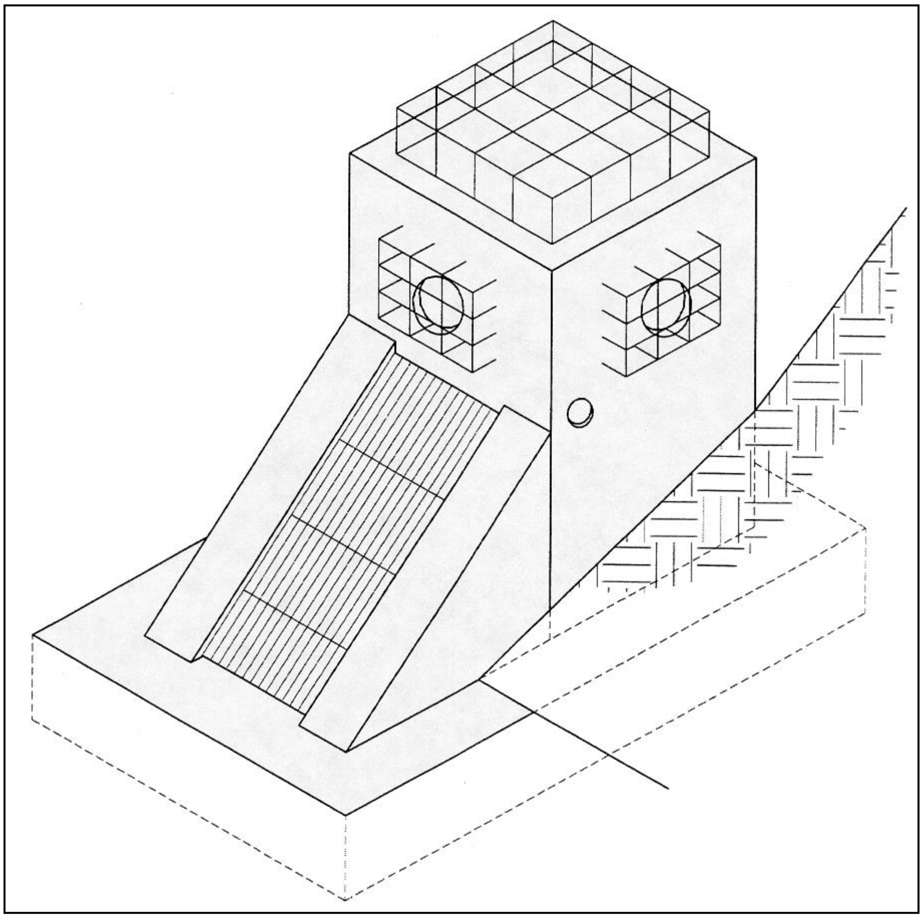

Figure 2.23 Example of Various Trash Racks Used on a Riser Outlet Structure
(Source: VDCR, 1999)

Trash Rack Design

Trash racks must be large enough so that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

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

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation.

Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level — the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb are found in literature. Figure 2.24 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack.

Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978; UDFCD, 1999). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% is chosen as a working assumption.

$$\mathbf{H_g = K_{g1}\,(w/x)^{4/3}\,(V_u^2/2g)\,\sin\theta_g} \qquad \textbf{(2.20)}$$

Where:

H_g = head loss through grate (ft)

K_{g1} = bar shape factor:
2.42 - sharp edged rectangular
1.83 - rectangular bars with semicircular upstream faces
1.79 - circular bars
1.67 - rectangular bars with semicircular up- and downstream faces

w = maximum cross-sectional bar width facing the flow (in)

x = minimum clear spacing between bars (in)

V_u = approach velocity (ft/s)

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

θ_g = angle of the grate with respect to the horizontal (degrees)

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$\mathbf{H_g = \frac{K_{g2}V_u^2}{2g}} \qquad \textbf{(2.21)}$$

Where:

K_{g2} is defined from a series of fit curves as:

- sharp edged rectangular (length/thickness = 10)
  $K_{g2} = 0.00158 - 0.03217\,A_r + 7.1786\,A_r^2$
- sharp edged rectangular (length/thickness = 5)
  $K_{g2} = -0.00731 + 0.69453\,A_r + 7.0856\,A_r^2$

- round edged rectangular (length/thickness = 10.9)
  $K_{g2} = -0.00101 + 0.02520\ A_r + 6.0000\ A_r^2$
- circular cross section
  $K_{g2} = 0.00866 + 0.13589\ A_r + 6.0357\ A_r^2$

and A_r is the ratio of the area of the bars to the area of the grate section.

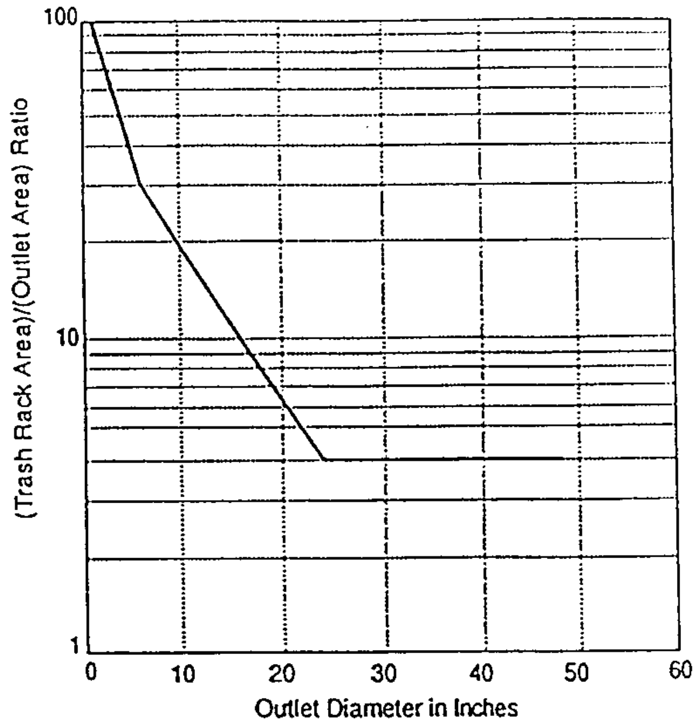

Figure 2.24 Minimum Rack Size vs. Outlet Diameter
(Source: UDCFD, 1992)

2.2.7 Secondary Outlets

Introduction

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 2.25 shows an example of an emergency spillway.

In many cases, on-site stormwater storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

Emergency Spillway Design

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 2.25). The emergency spillway is proportioned to pass flows in excess of the design flood (typically the flood mitigation storm or greater) without allowing excessive velocities and without overtopping of the embankment. Any dam, six feet or higher, must meet appropriate state and federal design standards, especially those regarding spillway design requirements related to passage of the probable maximum flood. In any case, the flood mitigation storm discharge, assuming blockage of outlet works, must be conveyed with some freeboard as specified by local criteria. Flow in the emergency spillway is open channel flow (see *Section 3.2*, for more information). Normally, it is assumed that critical depth occurs at the control section.

NRCS (SCS) manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper the 3:1 horizontal to vertical.

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated below.

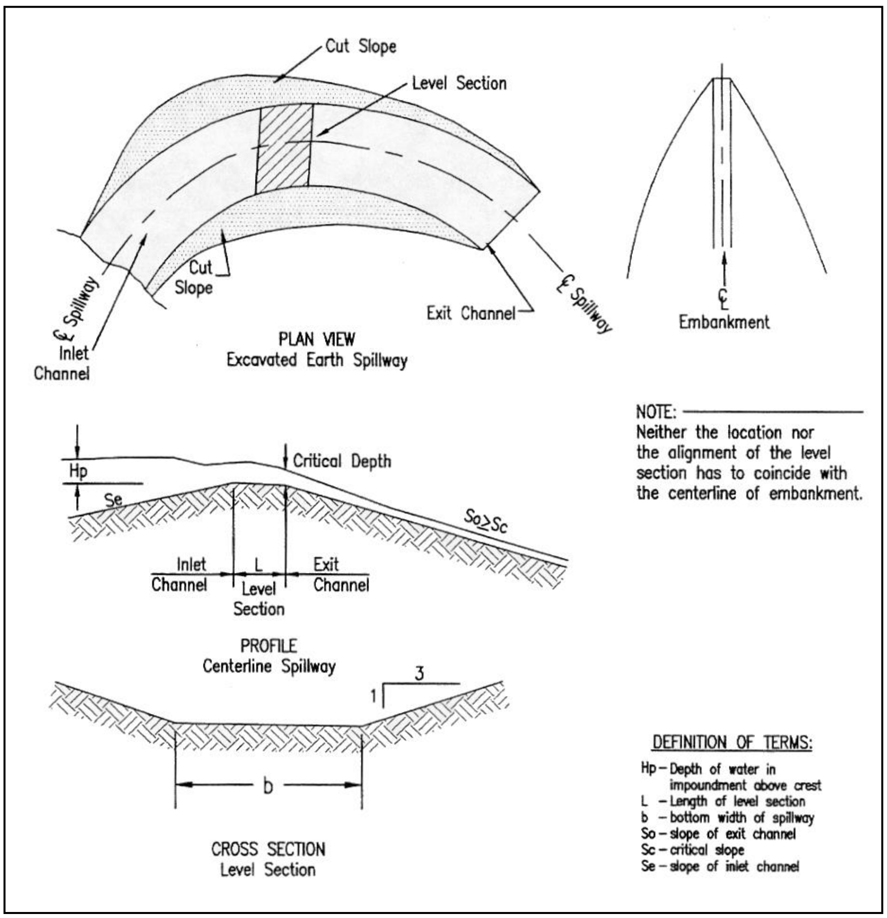

Figure 2.25 Emergency Spillway
(Source: VDCR, 1999)

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3.0 Open Channels, Culverts and Bridges

3.1 Open Channels, Culverts and Bridges Overview

3.1.1 Key Issues in Stormwater System Design

Introduction

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

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

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

General Design Considerations

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

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 and super elevation.

- Compound sections can be developed to carry the annual flow in the lower section and higher flows above them. Figure 3.1 illustrates a compound section that carries the 2-year and 100-year flows within banks. This reduces channel erosion at lower flows, and meandering, self-forming low flow channels that attack banks. The shelf in the compound section should have a minimum 1:12 slope to ensure drainage.

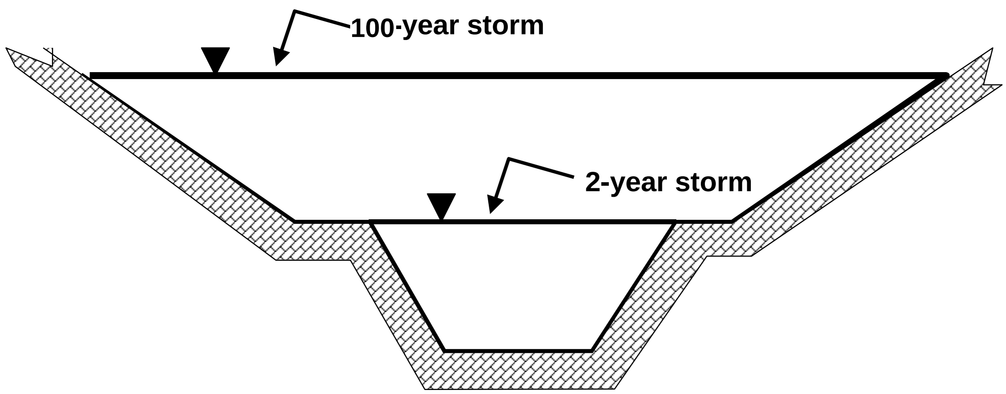


Figure 3.1 Compound Channel

- Flow control structures can be placed in the channels to increase residence time. Higher flows should be calculated using a channel slope from the top of the cross piece to the next one if it is significantly different from the channel bottom for normal depth calculations. 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 keep the channels from aggrading and losing capacity for larger flows.

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 depths exceed several feet, and to ensure safety during flows.
- Improved entrance designs can absorb considerable slope and energy for steeper sloped designs, thus helping to protect channels.

Bridges

- Bridges enable streams to maintain flow conveyance.
- Bridges are usually designed so that they are not submerged.
- Bridges may be vulnerable to failure from flood-related causes.
- Flow velocities through bridge openings should not cause scour within the bridge opening or in the stream reaches adjacent to the bridge.

Storage Design

- Stormwater storage within a stormwater system is essential to providing the extended detention of flows for water quality treatment and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection.

- Runoff storage can be provided within an on-site flood control system through the use of structural stormwater controls and/or nonstructural features.
- Stormwater storage can be provided by detention, extended detention, or retention.
- Storage facilities may be provided on-site, or as regional facilities designed to manage stormwater runoff from multiple projects.

Outlet Structures

- Outlet structures provide the critical function of the regulation of flow for structural stormwater controls.
- Outlet structures may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.
- Smaller, more protected outlet structures should be used for water quality and streambank protection flows.
- Large flows, such as flood flows, are typically handled through a broad crested weir, a riser with different sized openings, a drop inlet structure, or a spillway through an embankment.

Energy Dissipators

- Energy dissipators should be designed to return flows to non-eroding 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 their proper function.

3.2 Open Channel Design

3.2.1 Overview

Introduction

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 controls. Open channels include drainage ditches, grass channels, dry and wet enhanced swales, stone riprap channels and concrete-lined channels.

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

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. Stone riprap and some forms of flexible man-made linings or gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

Vegetative Linings – Vegetation, where practical, is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, provides habitat, and provides water quality benefits (see *Section 3.6.3 of the Criteria Manual* and the *Site Development Controls Technical Manual* for more details on using enhanced swales and grass channels for water quality purposes).

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

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

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

If low flows are prevalent, a hard lined pilot channel may be needed, and it should be wide enough to accommodate maintenance equipment.

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

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

3.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 3.1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3.1 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	Super-elevation 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 of gravity	32.2 ft/s^2
h_{loss}	Head loss	ft
K	Channel conveyance	-
k_e	Eddy head loss coefficient	ft
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 or 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	-

3.2.3 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 stone riprap linings are given in Table 3.2. Recommended values for vegetative linings should be determined using Figure 3.2, 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 3.6). Figure 3.2 is used iteratively as described in *Section 3.2.5*. Recommended Manning's values for natural channels that are either excavated or dredged, and natural are given in Table 3.2. 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*, FHWA-TS-84-204, 1984, FHWA HEC-15, 1988, or Chow, 1959. When designing open channels, the usual choice of Manning's roughness coefficients may be found in Table 3.5. The local jurisdiction may choose to vary from these values.

Table 3.2 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels

Channel Description	Manning's n	Maximum Permissible Channel Velocity (ft/s)
MINOR NATURAL STREAMS		
Fairly regular section		
1. Some grass and weeds; little or no brush	0.030	3 to 6
2. Dense growth of weeds, depth of flow materially greater than weed height	0.035	3 to 6
3. Some weeds, light brush on banks	0.035	3 to 6
4. Some weeds, heavy brush on banks	0.050	3 to 6
5. Some weeds, dense willows on banks	0.060	3 to 6
For trees within channels with branches submerged at high stage, increase above values by	0.010	
Irregular section with pools, slight channel meander, increase above values by	0.010	
Floodplain – Pasture		
1. Short grass	0.030	3 to 6
2. Tall grass	0.035	3 to 6
Floodplain – Cultivated Areas		
1. No crop	0.030	3 to 6
2. Mature row crops	0.035	3 to 6
3. Mature field crops	0.040	3 to 6
Floodplain – Uncleared		
1. Heavy weeds scattered brush	0.050	3 to 6
2. Wooded	0.120	3 to 6

Table 3.2 Roughness Coefficients (Manning's n) and Allowable Velocities for Natural Channels

Channel Description	Manning's n	Maximum Permissible Channel Velocity (ft/s)
MAJOR NATURAL STREAMS		
Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of "n" for larger streams of mostly regular sections, with no boulders or brush	Range from 0.028 to 0.060	3 to 6
UNLINED VEGETATED CHANNELS		
Clays (Bermuda Grass)	0.035	5 to 6
Sandy and Silty Soils (Bermuda Grass)	0.035	3 to 5
UNLINED NON-VEGETATED CHANNELS		
Sandy Soils	0.030	1.5 to 2.5
Silts	0.030	0.7 to 1.5
Sandy Silts	0.030	2.5 to 3.0
Clays	0.030	3.0 to 5.0
Coarse Gravels	0.030	5.0 to 6.0
Shale	0.030	6.0 to 10.0
Rock	0.025	15

Table 3.3 Maximum Velocities for Vegetative Channel Linings

Vegetation Type	Slope Range (%)[1]	Maximum Velocity[2] (ft/s)
Bermuda grass	0-5	6
Bahia		4
Tall fescue grass mixtures[3]	0-10	4
Kentucky bluegrass	0-5	6
Buffalo grass	5-10 >10	5 4
Grass mixture	0-5[1] 5-10	4 3
Sericea lespedeza, Weeping lovegrass, Alfalfa	0-5[4]	3
Annuals[5]	0-5	3
Sod		4
Lapped sod		5

[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.

Source: Manual for Erosion and Sediment Control in Georgia, 1996

Table 3.4 Manning's Roughness Coefficients (n) for Artificial Channels

		Depth Ranges		
Category	**Lining Type**	**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.

Source: HEC-15, 1988.

Table 3.5 Manning’s Roughness Coefficients for Design

Lining Type	**Manning’s n**	**Comments**
Grass Lined	0.035	Use for velocity check.
	0.050	Use for channel capacity check (freeboard check)
Concrete Lined	0.015	
Gabions	0.030	
Rock Riprap	0.040	$n = 0.0395 d_{50}^{1/6}$ where d_{50} is the stone size of which 50% of the sample is smaller
Grouted Riprap	0.028	FWHA

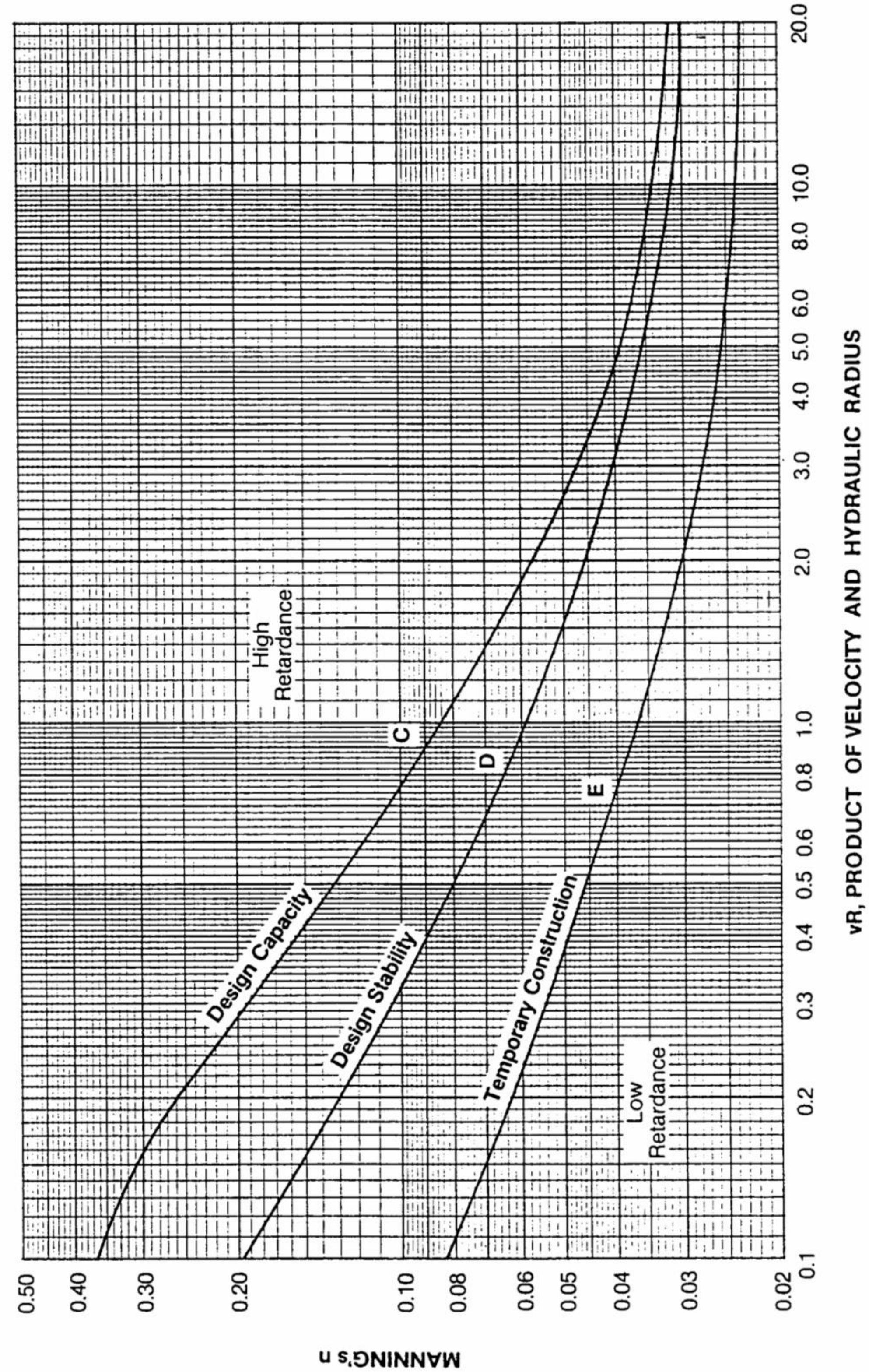


Figure 3.2 Manning's n Values for Vegetated Channels
(Source: USDA, TP-61, 1947)

Table 3.6 Classification of Vegetal Covers as to Degrees of Retardance

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 grasses	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

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.
Source: HEC-15, 1988

3.2.4 Uniform Flow Calculations

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. Examples of these charts and instructions for their use are given in *Section 3.2.11*.

Manning's Equation

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$\mathbf{v = (1.49/n)\, R^{2/3}\, S^{1/2}} \qquad \mathbf{(3.1)}$$

$$\mathbf{Q = (1.49/n)\, A\, R^{2/3}\, S^{1/2}} \qquad \mathbf{(3.2)}$$

$$\mathbf{S = [Q_n/(1.49\, A\, R^{2/3})]^2} \qquad \mathbf{(3.3)}$$

where:

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

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

For a more comprehensive discussion of open channel theory and design, see the reference USACE, 1991/1994.

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.

Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from Equation 3.2. The slope can be calculated using Equation 3.3 when the discharge, roughness coefficient, area, and hydraulic radius are known.

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 3.3 and 3.4. Figure 3.3 provides a general solution for the velocity form of Manning's Equation, while Figure 3.4 provides a solution of Manning's Equation for trapezoidal channels.

General Solution Nomograph

The following steps are used for the general solution nomograph in Figure 3.3:

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.

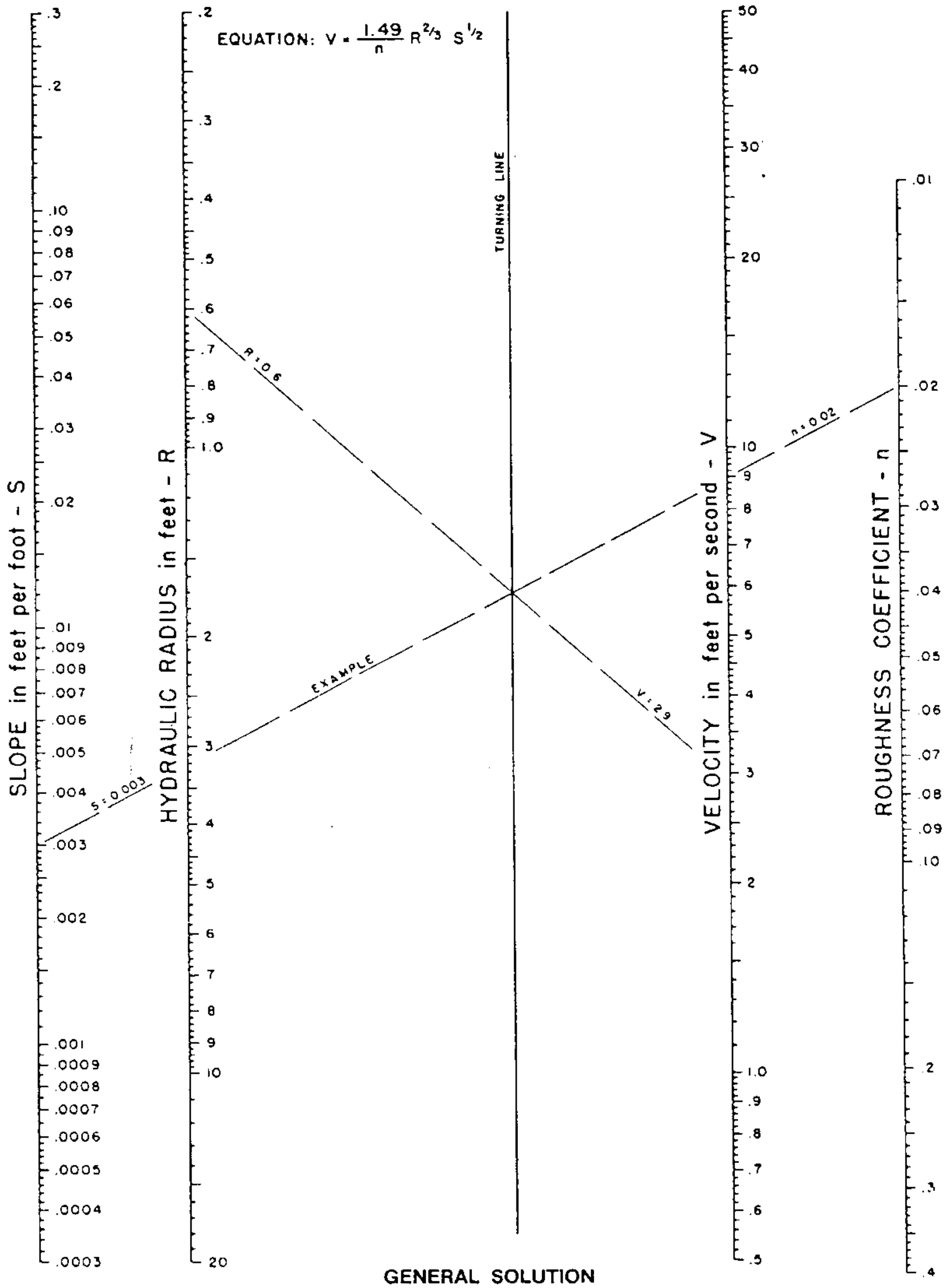
Step 4 Extend the line from Step 3 to the velocity scale to obtain the velocity in ft/s.

Trapezoidal Solution Nomograph

The trapezoidal channel nomograph solution to Manning's Equation in Figure 3.4 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.

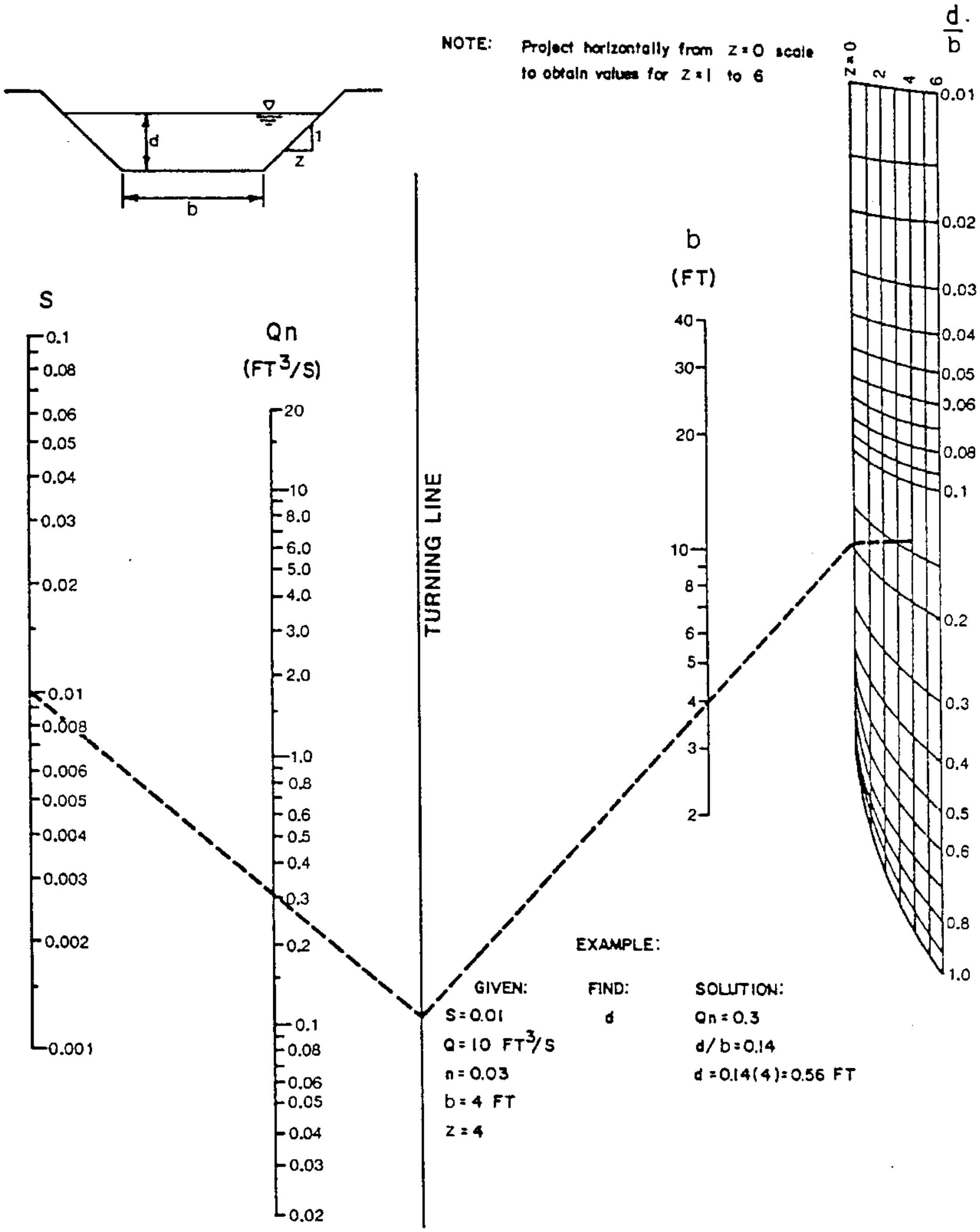
Determine input data, including slope in ft/ft, Manning's n value, bottom width in ft, and side slope in ft/ft.

- 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.
- 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.



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

Figure 3.3 Nomograph for the Solution of Manning's Equation



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3.4 Solution of Manning's Equation for Trapezoidal Channels

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:

$$AR^{2/3} = (Qn)/(1.49\ S^{1/2}) \qquad (3.4)$$

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 3.4 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 3.5 for trapezoidal channels. Computer programs are also available for these calculations.

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 the equation:

$$K_T = (Qn)/(b^{8/3}S^{1/2}) \qquad (3.5)$$

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 3.5 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.

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 3.11

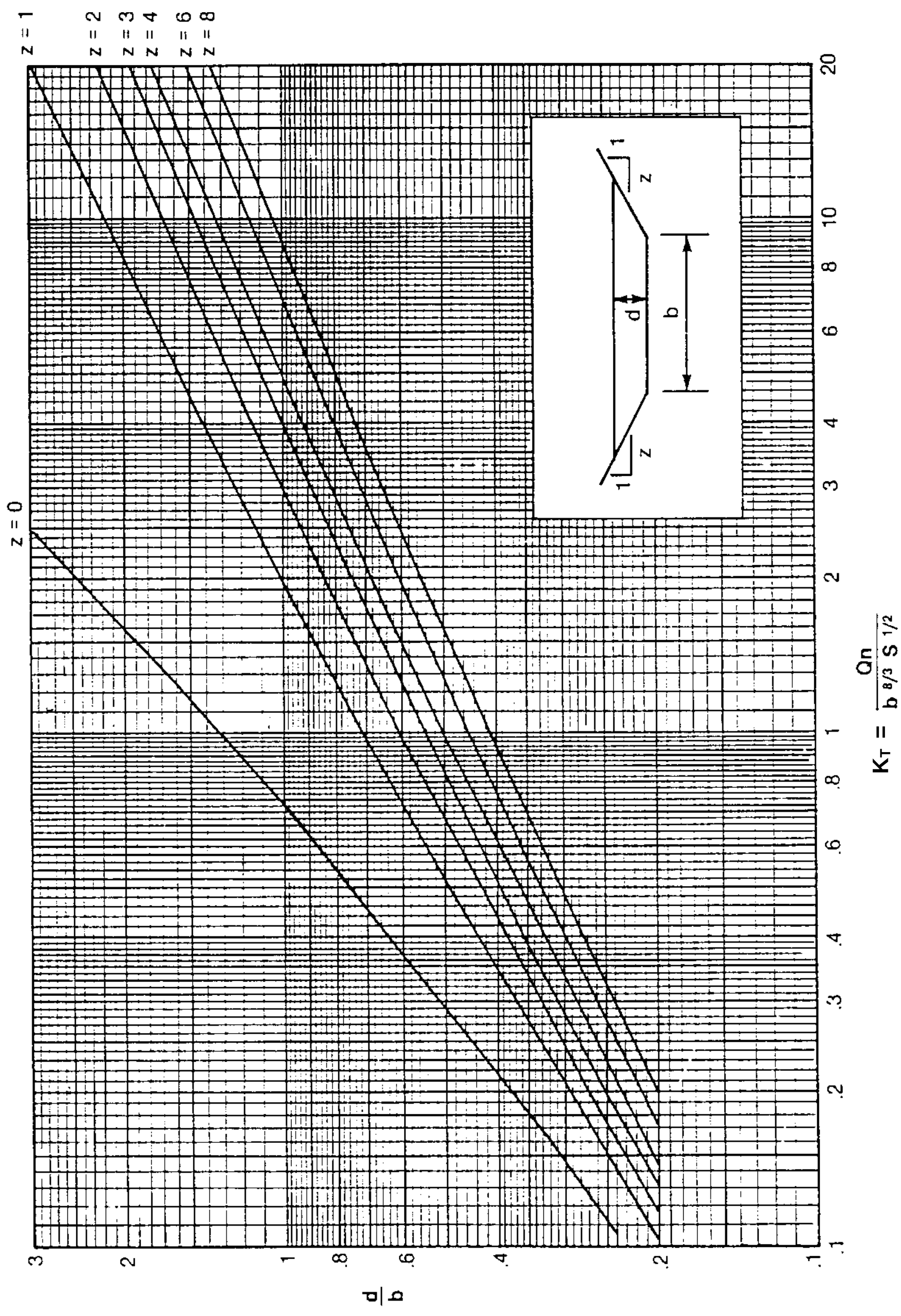


Figure 3.5 Trapezoidal Channel Capacity Chart
(Source: Nashville Stormwater Management Manual, 1988)

3.2.5 Critical Flow Calculations

Background

In the design of open channels, it is important to calculate the critical depth in order to determine if the flow in the channel will be subcritical or supercritical. If the flow is subcritical 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. Hydraulic jumps are possible under these conditions and consideration should be given to stabilizing the channel.

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$$Q^2/g = A^3/T \qquad \textbf{(3.6)}$$

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 3.6.

Semi-Empirical Equations

Semi-empirical equations (as presented in Table 3.7) or section factors (as presented in Figure 3.6) can be used to simplify trial and error critical depth calculations. The following equation is used to determine critical depth with the critical flow section factor, Z:

$$Z = Q/(g^{0.5}) \qquad \textbf{(3.7)}$$

where:

Z = critical flow section factor
Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.3 ft/sec^2)

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

- A normal depth of uniform flow within about 10% of critical depth is unstable and should be avoided in design, if possible.
- If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.
- If the velocity head is equal to one-half the mean depth of flow, the flow is critical.
- 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 ($V^2/2g$).

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

$$\mathbf{Fr = v/(gA/T)^{0.5}} \qquad \textbf{(3.8)}$$

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)

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.

Table 3.7 Critical Depth Equations for Uniform Flow in Selected Channel Cross Sections

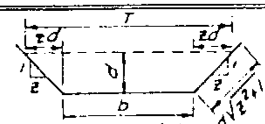
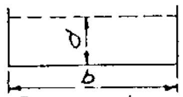
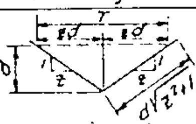
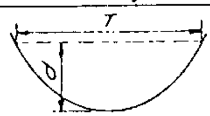
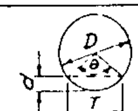
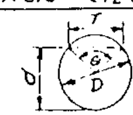
Channel Type[1]	Semi-Empirical Equations[2] for Estimating Critical Depth	Range of Applicability
1. Rectangular[3]	$d_c = [Q^2/(gb^2)]^{1/3}$	N/A
2. Trapezoidal[3]	$d_c = 0.81[Q^2/(gz^{0.75}b^{1.25})]^{0.27} - b/30z$	$0.1 < 0.5522\ Q/b^{2.5} < 0.4$ For $0.5522\ Q/b^{2.5} < 0.1$, use rectangular channel 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 (32.3 ft/s^2)
b = bottom width of channel (ft)
z = side slopes of a channel (horizontal to vertical)
D = diameter of circular conduit (ft)
A = cross-sectional area of flow (ft^2)
T = top width of water surface (ft)

[1] See Figure 3.6 for channel sketches
[2] Assumes uniform flow with the kinetic energy coefficient equal to 1.0
[3] Reference: French (1985)
[4] Reference: USDOT, FHWA, HDS-4 (1965)
[5] Reference: Brater and King (1976)

If the water surface profile in a channel transitions from supercritical flow to subcritical flow, a hydraulic jump must occur. The location of the hydraulic jump and its sequent depth are critical to proper design of free flow conveyances. To determine the location of a hydraulic jump, the standard step method is used to compute the water surface profile and specific force (momentum principle) and specific energy relationships are used. For computational methods refer to Chow, 1959, TxDOT, 2002, and Mays, 1999. The HEC-RAS computer program can be used to compute water surface profiles for both subcritical and supercritical flow regimes.

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

[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$

When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$

[3] $\theta = 4\cos^{-1}\sqrt{d/D}$

Insert θ in degrees in above equations

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

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

Figure 3.6 Open Channel Geometric Relationships for Various Cross Sections

3.2.6 *Vegetative Design*

Introduction

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 3.6. 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 3.6. If 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 not presented below are required. References include HEC-15 (USDOT, FHWA, 1986) and HEC-14 (USDOT, FHWA, 1983).

Design Stability

The following are the steps for design stability calculations:

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

Step 2 Use Table 3.3 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 3.2. Use retardance Class D for permanent vegetation and E for temporary construction.

Step 4 Calculate the hydraulic radius using the equation:

$$\mathbf{R = (vR)/v_m} \qquad \textbf{(3.9)}$$

where:

R = hydraulic radius of flow (ft)
vR = value obtained from Figure 3.2 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:

$$\mathbf{vR = (1.49\ R^{5/3}\ S^{1/2})/n} \qquad \textbf{(3.10)}$$

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 3.2 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 3.4 or 3.5, as described in *Section 3.2.4*. The depth of flow for other channel shapes can be evaluated using the trial and error procedure described in *Section 3.2.4*.

Step 8 If bends are considered, calculate the length of downstream protection, L_p, for the bend, using Figure 3.7. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

Design Capacity

The following are the steps for design capacity calculations:

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 3.6 for equations).

Step 2 Divide the design flow rate, obtained using appropriate procedures from the *Hydrology Technical Manual*, 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 3.2 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 3.1) or Figure 3.3 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 an appropriate freeboard to the final depth from Step 6. Generally, 20% is adequate.

Step 8 If bends are considered, calculate super-elevation of the water surface profile at the bend using the equation:

$$\Delta d = (v^2 T)/(g R_c) \qquad (3.11)$$

where:

Δd = super-elevation of the water surface profile due to the bend (ft)

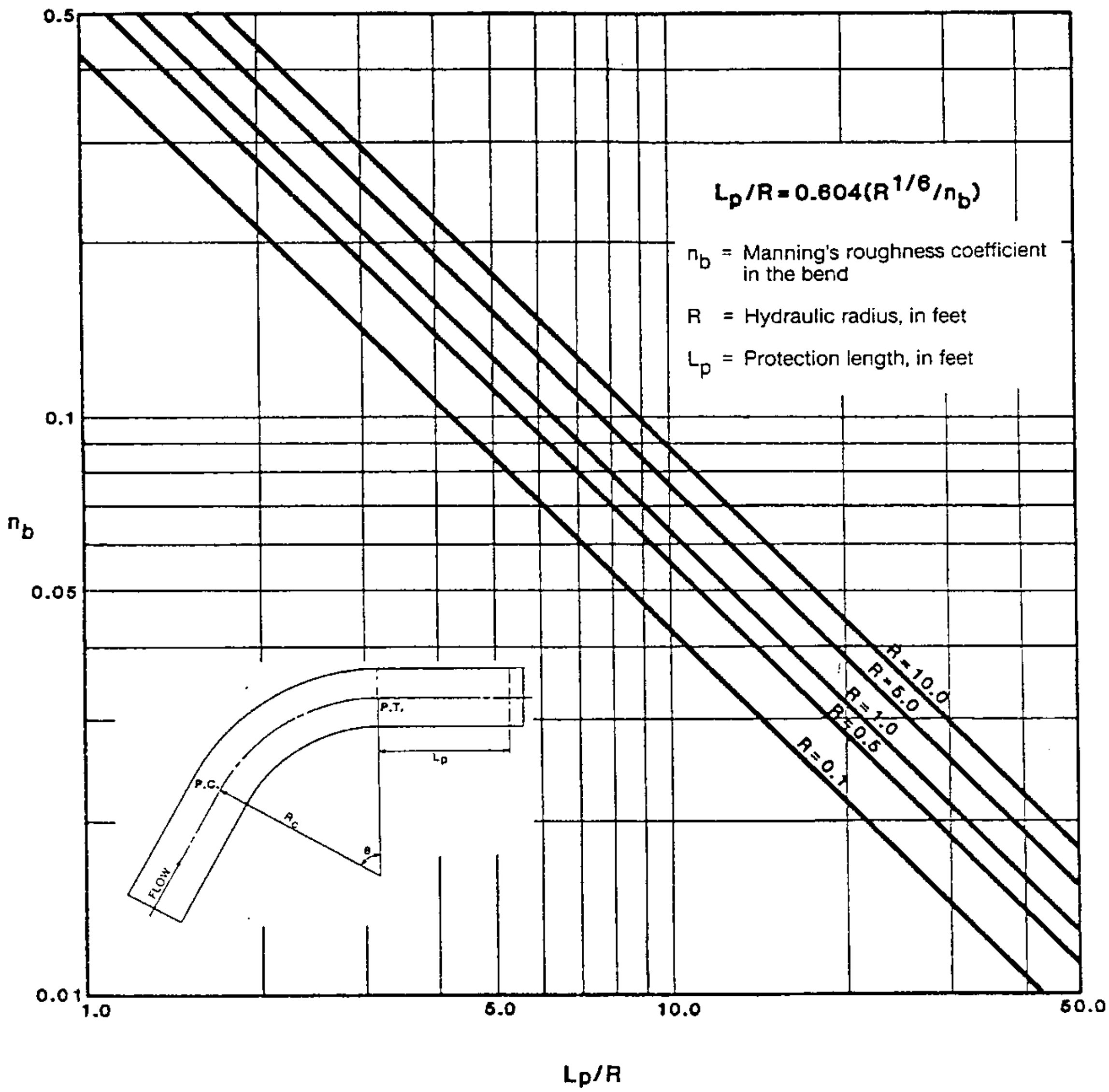
v = average velocity from Step 6 (ft/s)

T = top width of flow (ft)

g = acceleration of gravity (32.2 ft/sec^2)

R_c = mean radius of the bend (ft)

Note: Add freeboard consistent with the calculated Δd.



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3.7 Protection Length, Lp, Downstream of Channel Bend

3.2.7 Stone Riprap Design

Introduction

A number of agencies and researchers have studied and developed empirical equations to estimate the required size of rock riprap to resist various hydraulic conditions, including the U.S. Army Corps of Engineers (USACE), Natural Resource Conservation Service (NRCS), and the Federal Highway Administration (FHWA). As with all empirical equations based on the results of laboratory experiments, they must be used with an understanding of the range of data on which they are based.

The following methods give design guidance for designing stone riprap for open channels. Design guidance for designing stone riprap for culvert outfall protection is also provided in this section. *Section 4.0* gives additional guidance on the design of riprap aprons for erosion protection at outfalls, and the design of riprap basins for energy dissipation.

Method #1: Maynord & Reese

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
- If significant turbulence is caused by boundary irregularities, such as vertical drops, obstructions, or structures, this procedure is not applicable.

<u>Procedure</u>

Following are the steps in the procedure for riprap design using the method by Maynord & Reese:

Step 1 Determine the average velocity in the main channel for the design condition. Manning's n values for riprap can be calculated from the equation:

$$\mathbf{n = 0.0395\ (d_{50})^{1/6}} \qquad \textbf{(3.12)}$$

where:

n = Manning's 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 3.8 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 3.9.

Step 4 Determine the required minimum d_{30} value from Figure 3.10, or from the equation:

$$d_{30}/D = 0.193\ Fr^{2.5} \qquad (3.13)$$

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 3.8), dimensionless
v = mean velocity above the stone (ft/s)
g = acceleration of gravity (32.2 ft/sec)

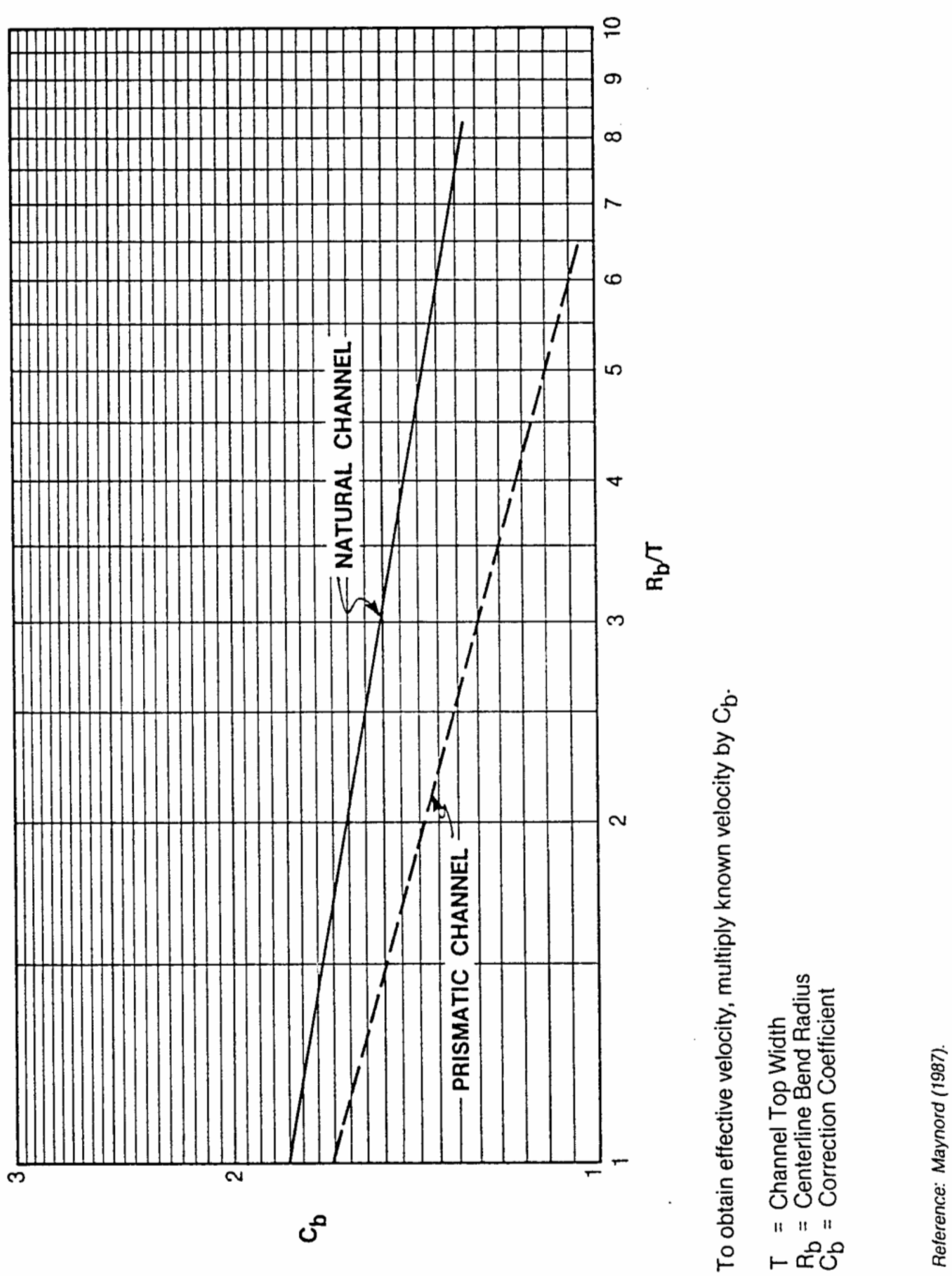


Figure 3.8 Riprap Lining Bend Correction Coefficient

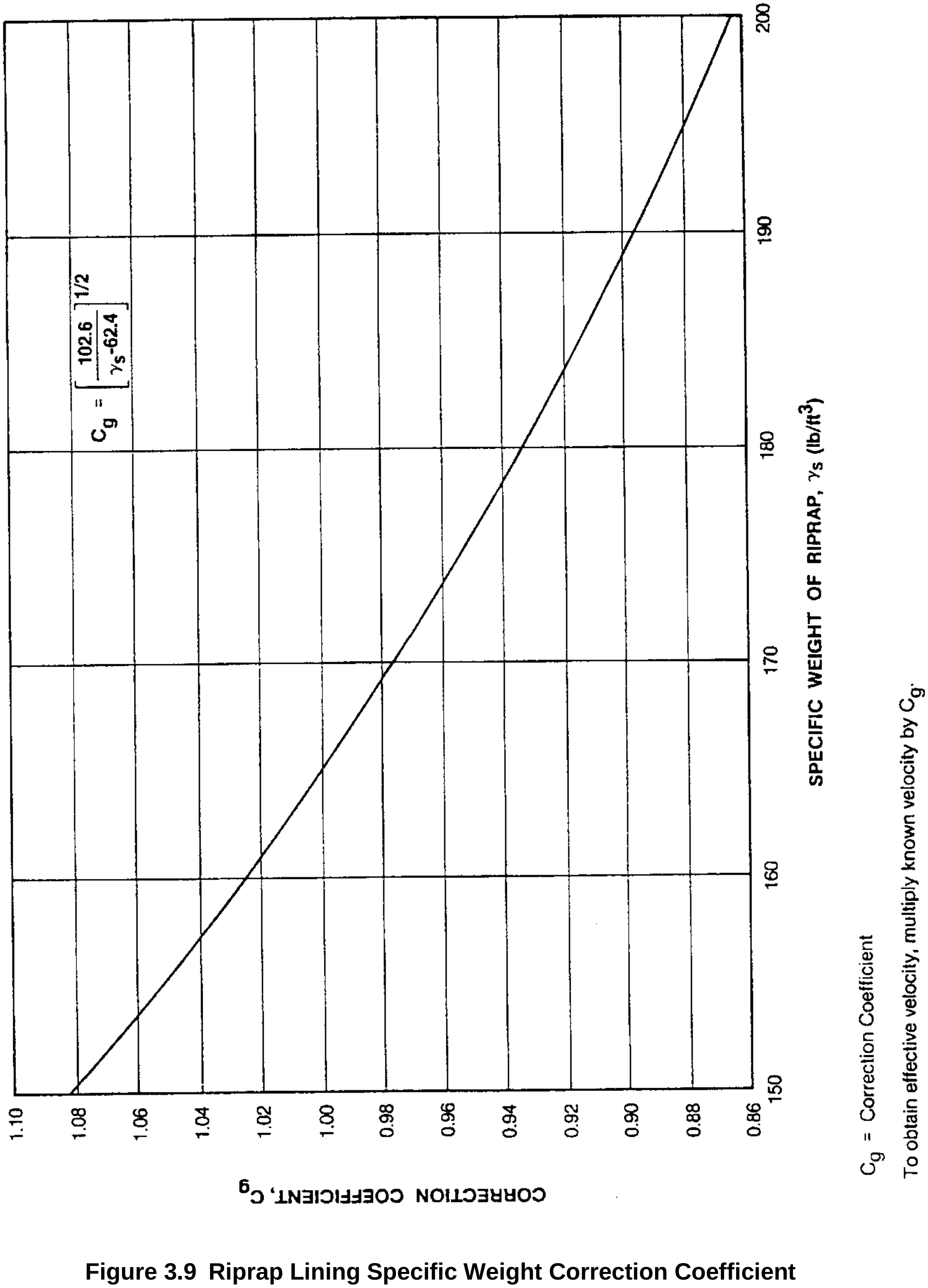
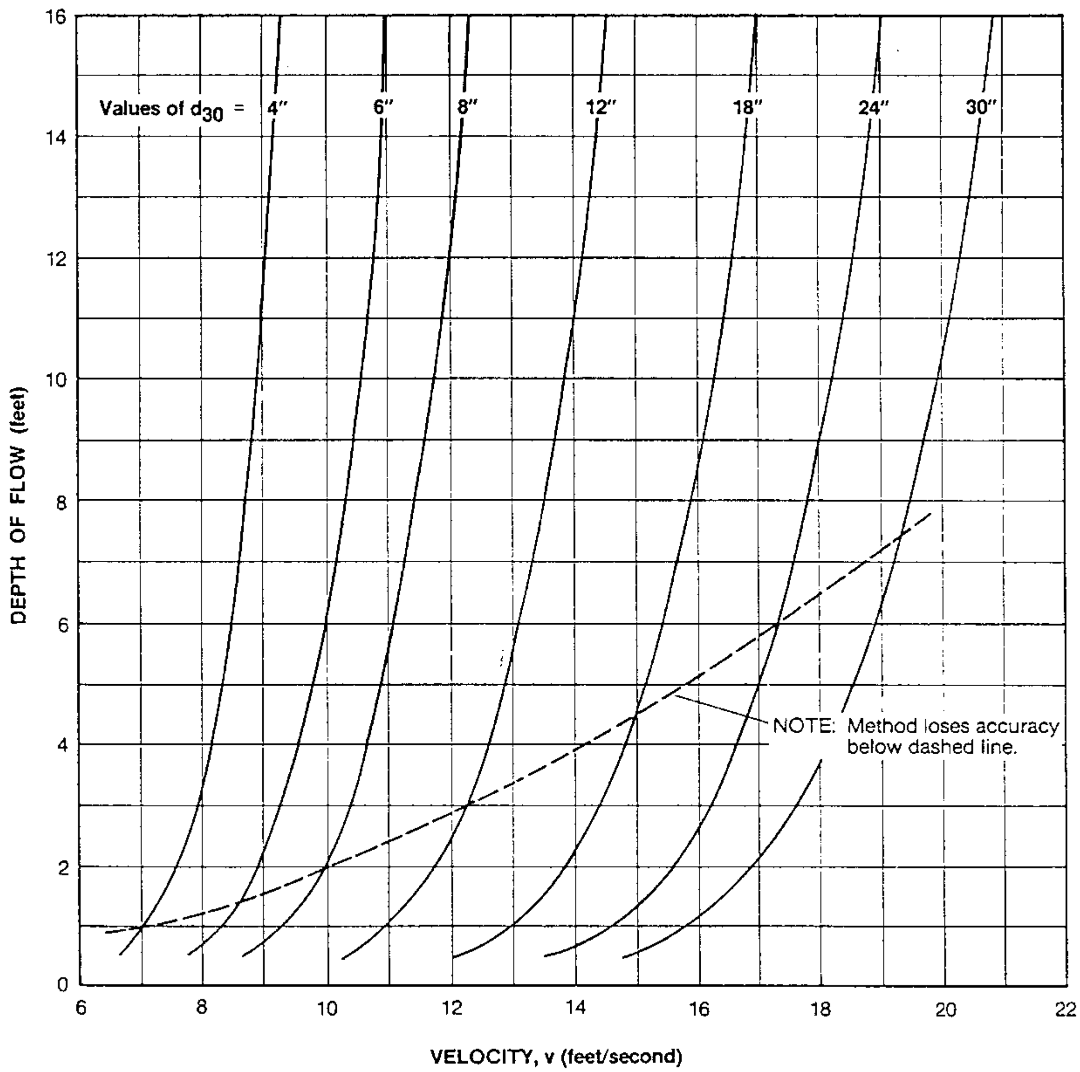


Figure 3.9 Riprap Lining Specific Weight Correction Coefficient

(Source: Nashville Stormwater Management Manual, 1988)



Reference: Reese (1988).

Figure 3.10 Riprap Lining d_{30} Stone Size – Function of Mean Velocity and Depth

Step 5 Determine available riprap gradations. A well graded riprap is required. 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:

$$\mathbf{W = 0.5236\ SW_s\ d^3} \qquad \mathbf{(3.14)}$$

where:

W = stone weight (lbs)

d = selected stone diameter (ft)

SW_s = specific weight of stone (lbs/ft^3)

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.

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 3.11 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 that toe protection is provided to avoid riprap undermining.

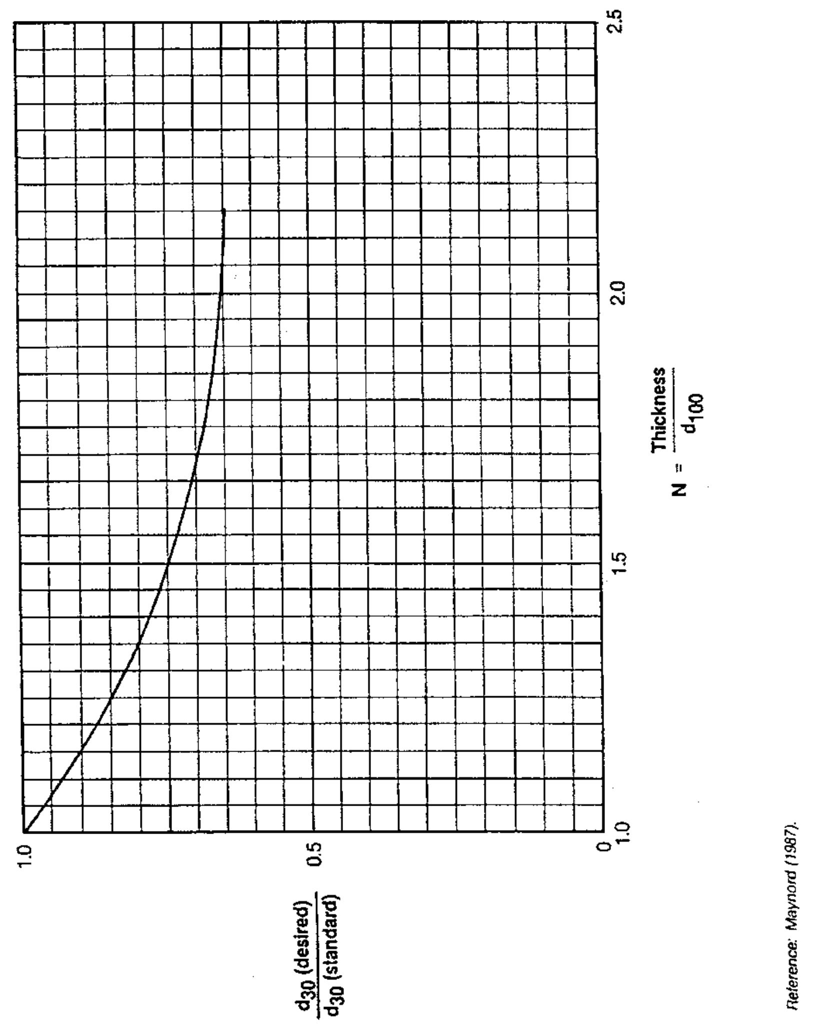


Figure 3.11 Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 1.0 to 2.3
(Source: Maynord, 1987)

Method #2: Gregory

The following procedure is based on excerpts from a paper prepared by Garry Gregory (June, 1987) and has been widely used in the Dallas-Fort Worth area for riprap design.

Procedure

Following are the steps in the procedure for riprap design using the method by Gregory:

Step 1 Calculate the boundary shear (tractive stress or tractive force) by:

$$\mathbf{T_o = \Upsilon_w RS} \qquad \textbf{(3.15)}$$

where:

T_o = average tractive stress on channel bottom (lb/ft²)

Υ_w = unit weight of water (62.4 lb/ft³)

R = hydraulic radius of channel (ft)

S = slope of energy gradient (ft/ft)

$$\mathbf{T_o' = T_o(1-(\sin^2\Phi/\sin^2\Theta))^{0.5}} \qquad \textbf{(3.16)}$$

where:

T_o' = average tractive stress on channel side slopes (lb/ft²)

Φ = angle of side slope with the horizontal

Θ = angle of repose of riprap (approximately 40°)

The greater value of T_o or T_o' governs.

Step 2 Determine the tractive stress in a bend in the channel by:

$$\mathbf{T_b = 3.15T(r/w)^{-0.5}} \qquad \textbf{(3.17)}$$

where:

T_b = local tractive stress in the bend (lb/ft²)

T = the greater of T_o or T_o' from Equations 3.15 and 3.16

r = center-line radius of the bend (ft)

w = water surface width at upstream end of bend (ft)

Step 3 Determine D_{50} size of riprap stone (size at which 50% of the gradation is finer weight) from:

$$\mathbf{D_{50} = T/0.04(\Upsilon_s-\Upsilon_w)} \qquad \textbf{(3.18)}$$

where:

D_{50} = required average size of riprap stone (ft)

T = the greater of T_o or T_o' from Equations 3.15 and 3.16

Υ_s = saturated surface dry (SSD) unit weight of stone (lb/ft³)

Υ_w = unit weight of water (62.4 lb/ft³)

Step 4 Select minimum riprap thickness required from grain size curves in Figures 3.12 through 3.17. Select from smaller side of band at 50% finer gradation.

Step 5 Select riprap gradations table (Figures 3.18 and 3.19) based on riprap thickness selected in Step 4.

Step 6 Select bedding thickness from grain size curves in Figures 3.12 through 3.17, which was used to select the riprap thickness in Step 4. Note: The bedding thicknesses included in Figures 3.12 through 3.17 are based on using a properly designed geotextile underneath the bedding. If a geotextile is not used, the bedding thickness must be increased to a minimum of 9 inches for 24 inch and 30 inch riprap and a minimum of 12 inches for the 36 inch riprap.

Step 7 To provide stability in the riprap layer the riprap gradations should meet the following criteria for GRADATION INDEX:

$$\textbf{GRADATION INDEX: } [D_{85}/D_{50} + D_{50}/D_{15}] \leq 5.5 \qquad (3.19)$$

where: D_{85}, D_{50}, and D_{15} are the riprap grain sizes (mm) of which 85%, 50%, and 15% respectively are finer by weight.

Step 8 To provide stability of the bedding layer the bedding should meet the following filter criteria with respect to the riprap:

$$D_{15}/d_{85} < 5 < D_{15}/d_{15} < 40 \qquad (3.20)$$

$$D_{50}/d_{50} < 40 \qquad (3.21)$$

where: D refers to riprap sizes, and d refers to bedding sizes, both in mm.

Step 9 The geotextile underneath the bedding should be designed as a filter to the soil.

Step 10 Typical riprap design sections are shown in Figures 3.20 and 3.21, from the USACE publication EM1110-2-160.

Culvert Outfall Protection

The following procedure is used to design riprap for protection at culvert outfalls.

Step 1 Determine D_{50} size of riprap determined from:

$$D_{50} = \sqrt{V}/[1.8\sqrt{(2g(\Upsilon_s-\Upsilon_w)/\Upsilon_w)}] \qquad (3.22)$$

Step 2 Select riprap and bedding from Figures 3.12 through 3.17 using D_{50} from Equation 3.22.

Step 3 Select gradations from tables in Figures 3.18 and 3.19.

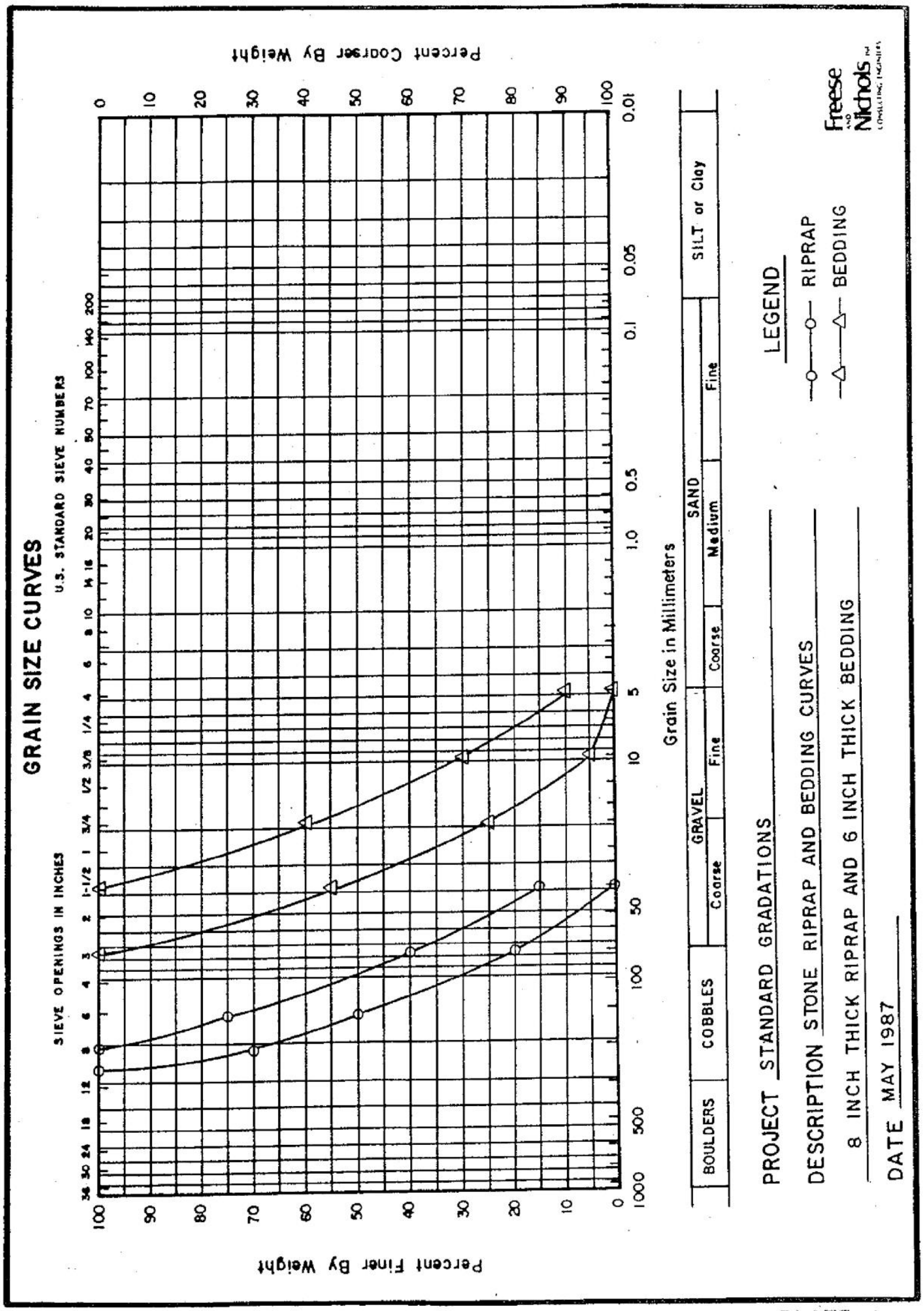


Figure 3.12 Grain Size Curve for 8" Riprap and 6" Bedding

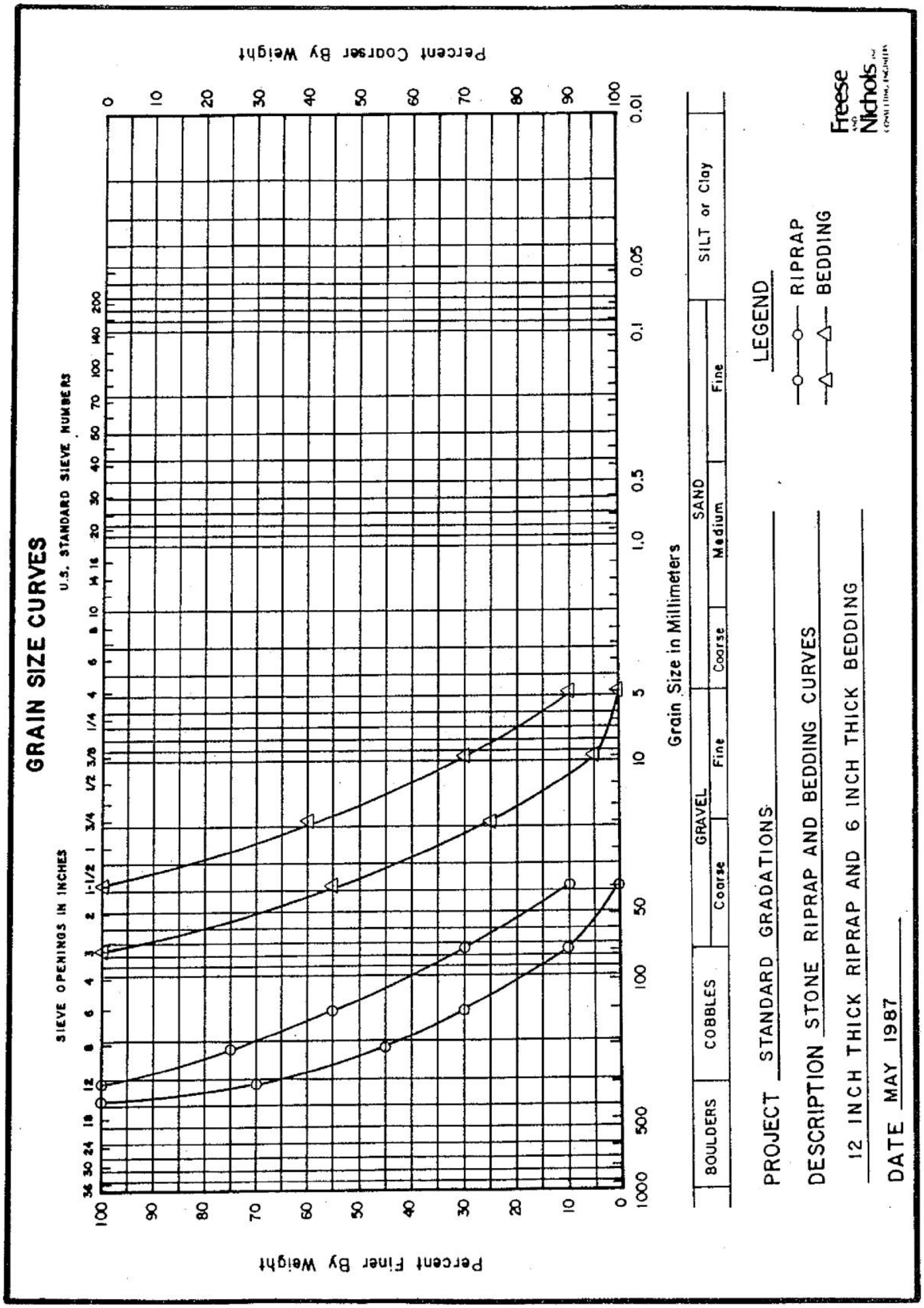


Figure 3.13 Grain Size Curve for 12" Riprap and 6" Bedding

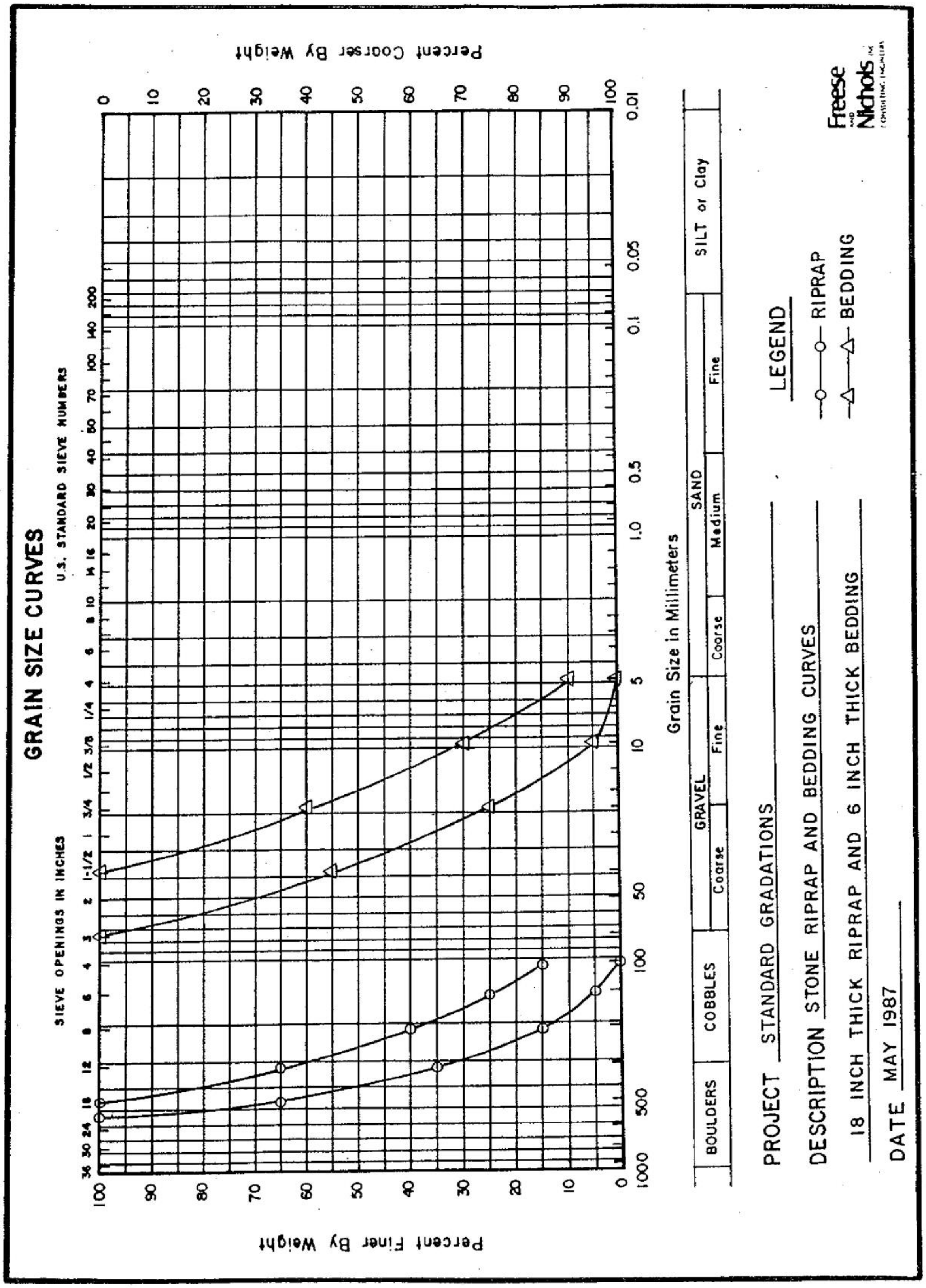


Figure 3.14 Grain Size Curve for 18" Riprap and 6" Bedding

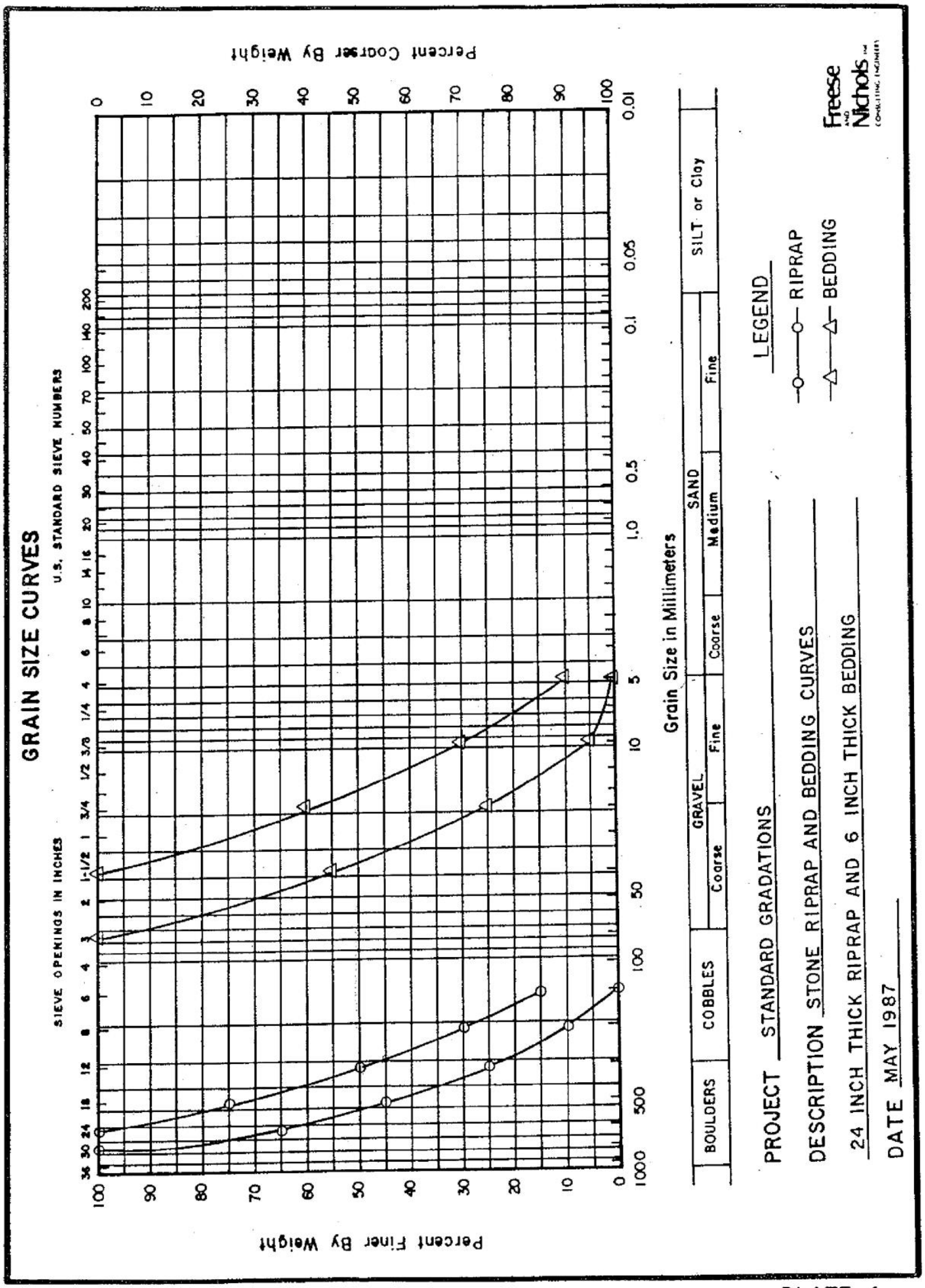


Figure 3.15 Grain Size Curve for 24" Riprap and 6" Bedding

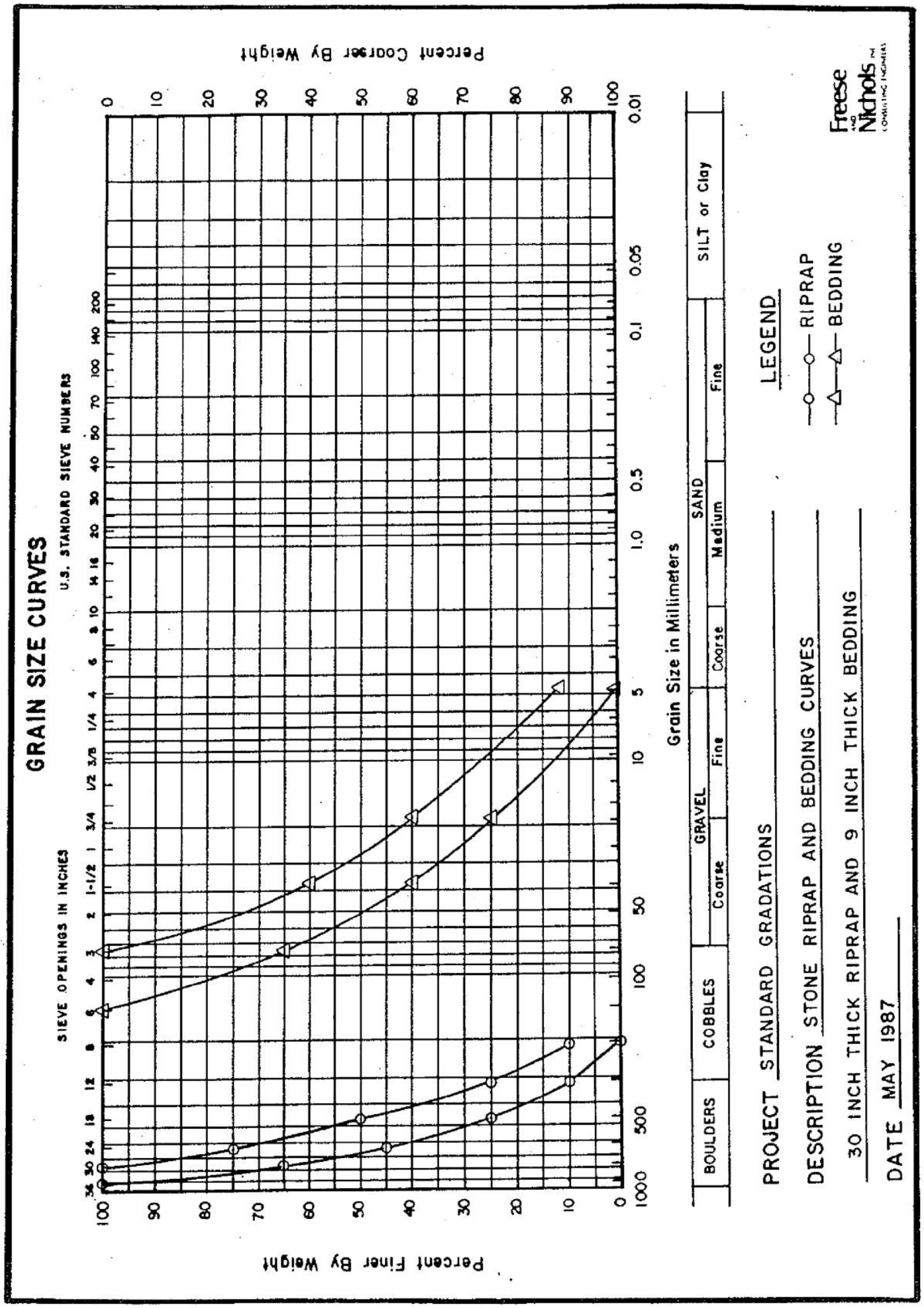


Figure 3.16 Grain Size Curve for 30" Riprap and 9" Bedding

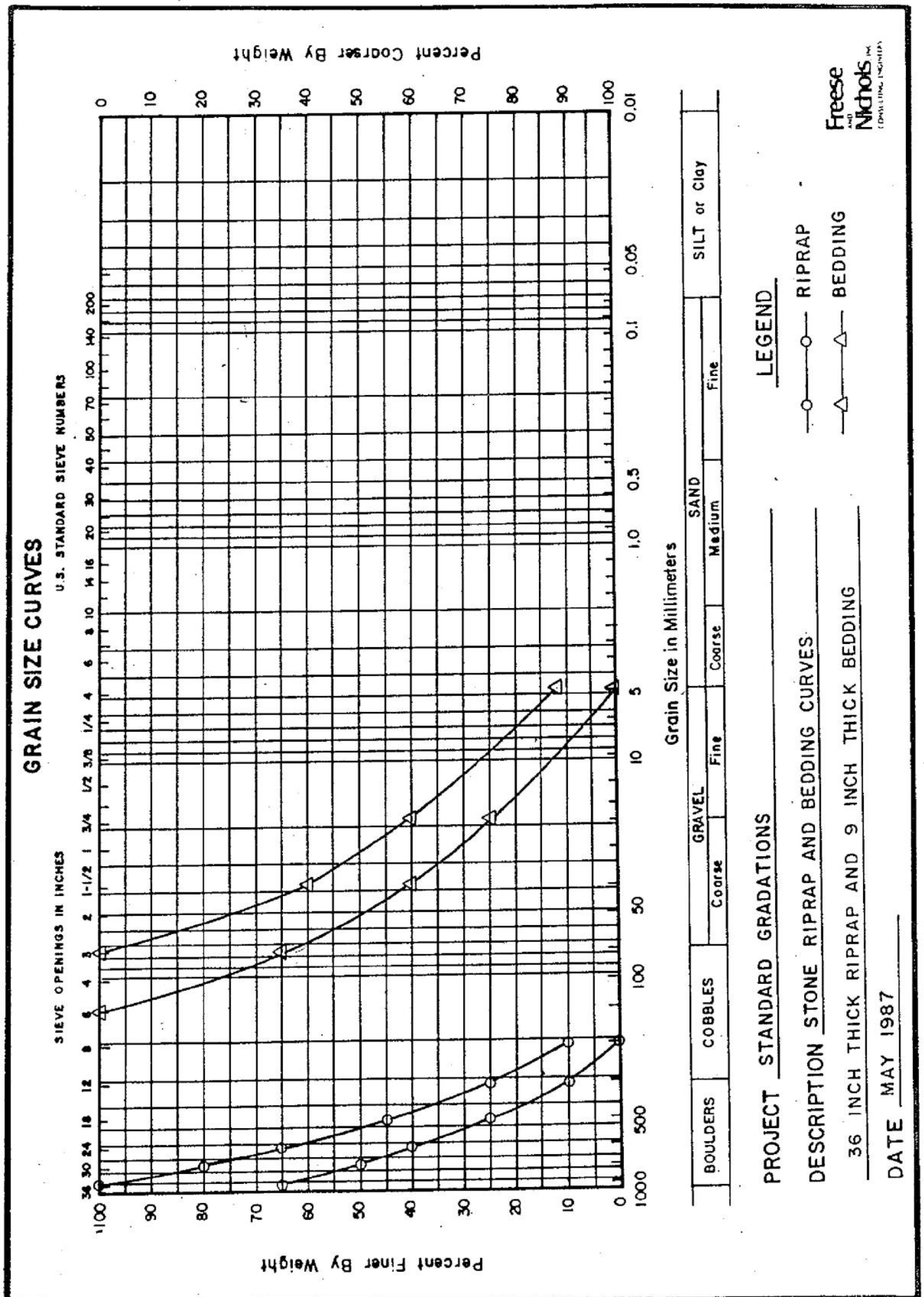

GRAIN SIZE CURVES
SIEVE OPENINGS IN INCHES
U.S. STANDARD SIEVE NUMBERS
Percent Coarser By Weight
Percent Finer By Weight
Grain Size in Millimeters
BOULDERS
COBBLES
GRAVEL Coarse Fine
SAND Coarse Medium Fine
SILT or Clay
LEGEND
RIPRAP
BEDDING
PROJECT STANDARD GRADATIONS
DESCRIPTION STONE RIPRAP AND BEDDING CURVES
36 INCH THICK RIPRAP AND 9 INCH THICK BEDDING
DATE MAY 1987
Freese and Nichols

Figure 3.17 Grain Size Curve for 36" Riprap and 9" Bedding

RIPRAP GRADATIONS	
12" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
15 INCH	100
12 INCH	70 – 100
8 INCH	45 – 75
6 INCH	30 – 55
3 INCH	10 – 30
1-1/2 INCH	0 – 10

RIPRAP GRADATIONS	
8" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
10 INCH	100
8 INCH	70 – 100
6 INCH	50 – 75
3 INCH	20 – 40
1-1/2 INCH	0 – 15

BEDDING GRADATIONS	
6" THICKNESS OF BEDDING	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
3 INCH	100
1-1/2 INCH	55 – 100
3/4 INCH	25 – 60
3/8 INCH	5 – 30
No.4	0 – 10

BEDDING GRADATIONS	
9" THICKNESS OF BEDDING	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
6 INCH	100
3 INCH	65 – 100
1-1/2 INCH	40 – 60
3/4 INCH	25 – 40
No.4	0 – 12

Figure 3.18 Riprap Gradation Tables for 6", 8", 9", and 12" Thickness of Riprap

RIPRAP GRADATIONS — 36" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
44 INCH	100
36 INCH	65 – 100
30 INCH	50 – 80
18 INCH	25 – 45
12 INCH	10 – 25
8 INCH	0 – 10

RIPRAP GRADATIONS — 24" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
30 INCH	100
24 INCH	65 – 100
18 INCH	45 – 75
12 INCH	25 – 50
8 INCH	10 – 30
6 INCH	0 – 15

RIPRAP GRADATIONS — 30" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
36 INCH	100
30 INCH	65 – 100
24 INCH	45 – 75
18 INCH	25 – 50
12 INCH	10 – 25
8 INCH	0 – 10

RIPRAP GRADATIONS — 18" THICKNESS OF RIPRAP

SIEVE SIZE SQUARE MESH	PERCENT PASSING
21 INCH	100
18 INCH	65 – 100
12 INCH	35 – 65
8 INCH	15 – 40
6 INCH	5 – 25
4 INCH	0 – 15

Figure 3.19 Riprap Gradation Tables for 18", 24", 30", and 36" Thickness of Riprap

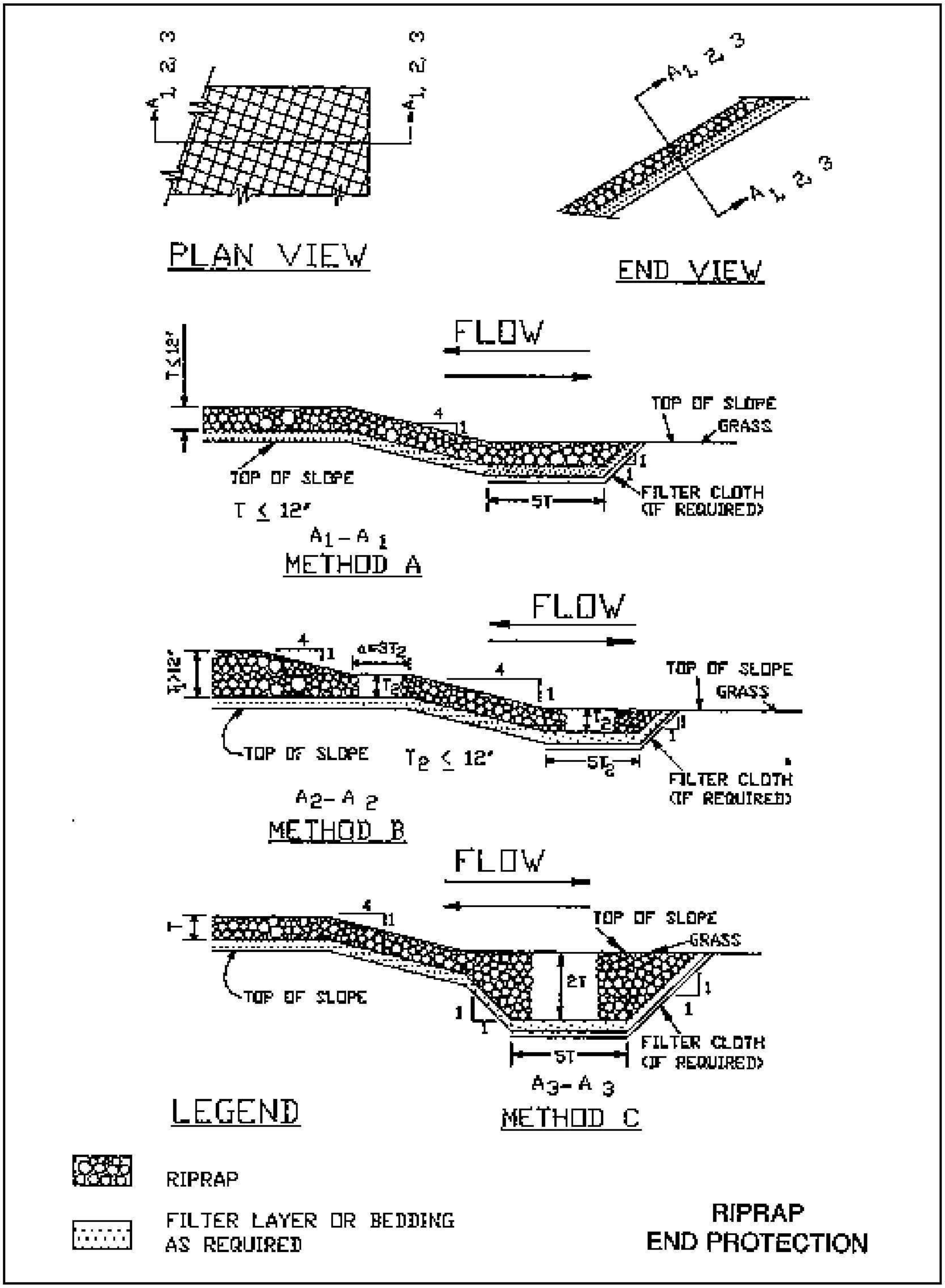


Figure 3.20 Typical Riprap Design Cross Sections

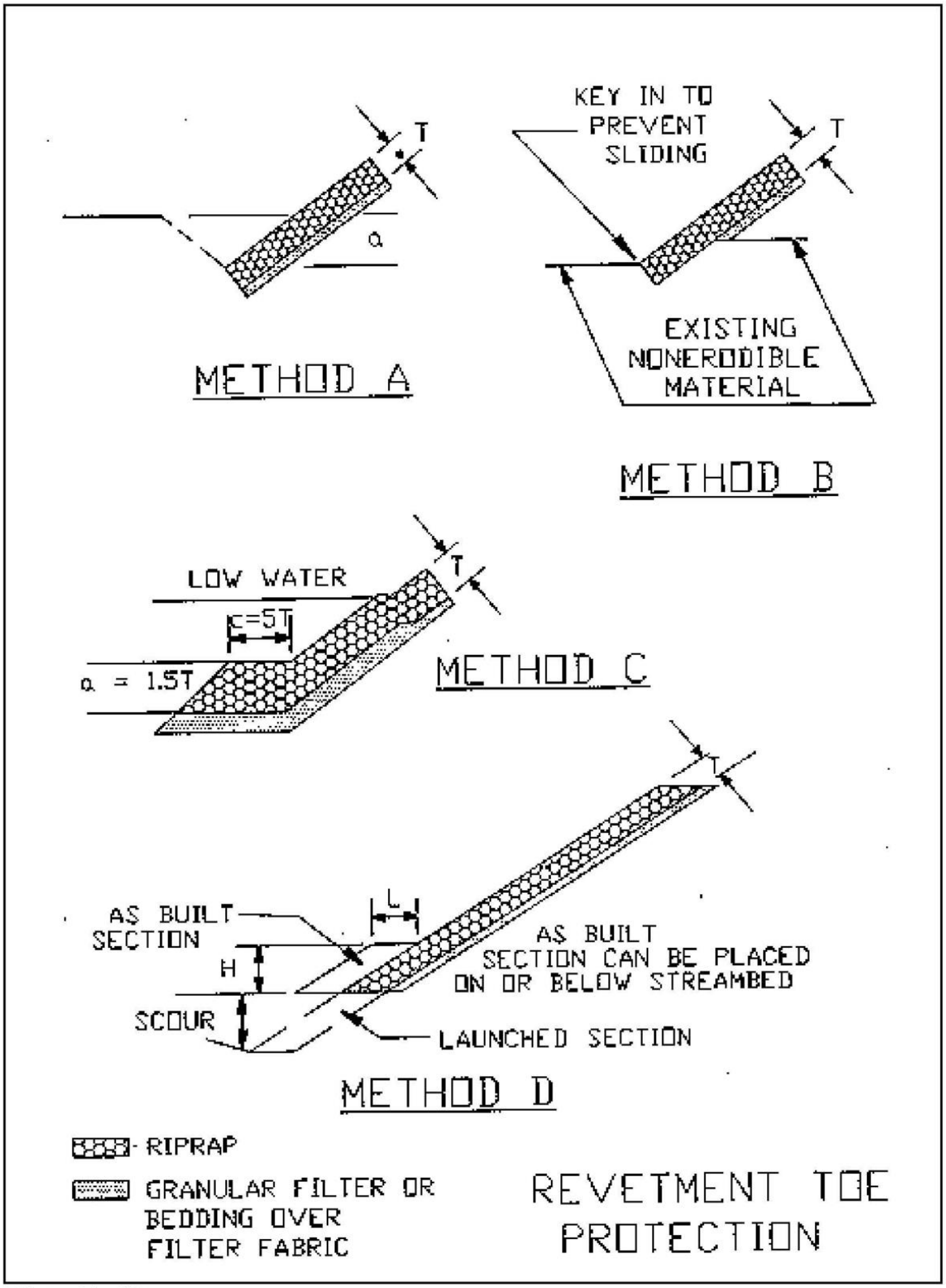


Figure 3.21 Typical Riprap Design for Revetment Toe Protection

3.2.8 Gabion Design

Introduction

Gabions come in three basic forms, the gabion basket, gabion mattress, and sack gabion. All three types consist of wire mesh baskets filled with cobble or small boulder material. The fill normally consists of rock material but other materials such as bricks have been used to fill the baskets. The baskets are used to maintain stability and to protect streambanks and beds.

The difference between a gabion basket and a gabion mattress is the thickness and the aerial extent of the basket. A sack gabion is, as the name implies, a mesh sack that is filled with rock material. The benefit of gabions is that they can be filled with rocks that would individually be too small to withstand the erosive forces of the stream. The gabion mattress is shallower (0.5 to 1.5 ft) than the basket and is designed to protect the bed or banks of a stream against erosion.

Gabion baskets are normally much thicker (about 1.5 to 3 ft) and cover a much smaller area. They are used to protect banks where mattresses are not adequate or are used to stabilize slopes (Figure 3.22), construct drop structures, pipe outlet structures, or nearly any other application where soil must be protected from the erosive forces of water. References to gabions in this manual refer generally to both mattresses and baskets. Sack gabions are rarely used in the United States and are not discussed.

Gabion baskets can be made from either welded or woven wire mesh. The wire is normally galvanized to reduce corrosion but may be coated with plastic or other material to prevent corrosion and/or damage to the wire mesh containing the rock fill. New materials such as Tensar, a heavy-duty polymer plastic material, have been used in some applications in place of the wire mesh. If the wire baskets break, either through corrosion, vandalism, or damage from debris or bed load, the rock fill in the basket can be lost and the protective value of the method endangered. Gabions are often used where available rock size is too small to withstand the erosive and tractive forces present at a project site. The available stone size may be too small due to the cost of transporting larger stone from remote sites, or the desire to have

Figure 3.22 Gabion Baskets Installed for Slope Stabilization along a Stream

a project with a smoother appearance than obtained from riprap or other methods. Gabions also require about one third the thickness of material when compared to riprap designs. Riprap is often preferred, however, due to the low labor requirements for its placement.

The science behind gabions is fairly well established, with numerous manufacturers providing design methodology and guidance for their gabion products. Dr. Stephen T. Maynord of the U.S. Army Corps of Engineers Research and Development Center in Vicksburg, Mississippi, has also conducted research to develop design guidance for the installation of gabions. Two general methods are typically used to determine the stability of gabion baskets in stream channels, the critical shear stress calculation and the critical velocity calculation. A software package known as CHANLPRO has been developed by Dr. Maynord (Maynord et al. 1998).

Manufacturers have generated extensive debate regarding the use and durability of welded wire baskets versus woven wire baskets in project design and construction. Project results seem to indicate that performance is satisfactory for both types of mesh.

The rocks contained within the gabions provide substrates for a wide variety of aquatic organisms. Organisms that have adapted to living on and within the rocks have an excellent home, but vegetation may be difficult to establish unless the voids in the rocks contained within the baskets are filled with soil or a planting bed mixture.

If large woody vegetation is allowed to grow in the gabions, there is a risk that the baskets will break when the large woody vegetation is uprooted or as the root and trunk systems grow. Thus, it is normally not acceptable to allow large woody vegetation to grow in the baskets. The possibility of damage must be weighed against the desirability of vegetation on the area protected by gabions and the stability of the large woody vegetation. If large woody vegetation is kept out of the baskets, grasses and other desirable vegetation types may be established and provide a more aesthetic and ecologically desirable project than gabions alone.

Design

Primary design considerations for gabions and mattresses are: 1) foundation stability; 2) sustained velocity and shear-stress thresholds that the gabions must withstand; and 3) toe and flank protection. The base layer of gabions should be placed below the expected maximum scour depth. Alternatively, the toe can be protected with mattresses that will fall into any scoured areas without compromising the stability of the bank or bed protection portion of the project. If bank protection does not extend above the expected water surface elevation for the design flood, measures such as tiebacks to protect against flanking should be installed.

The use of a filter fabric behind or under the gabion baskets to prevent the movement of soil material through the gabion baskets is an extremely important part of the design process. This migration of soil through the baskets can cause undermining of the supporting soil structure and failure of the gabion baskets and mattresses.

Primary Design Considerations

The major consideration in the design of gabion structures is the expected velocity at the gabion face. The gabion must be designed to withstand the force of the water in the stream.

Since gabion mattresses are much shallower and more subject to movement than gabion baskets, care should be taken to design the mattresses such that they can withstand the forces applied to them by the water. However, mattresses have been used in application where very high velocities are present and have performed well. But, projects using gabion mattresses should be carefully designed.

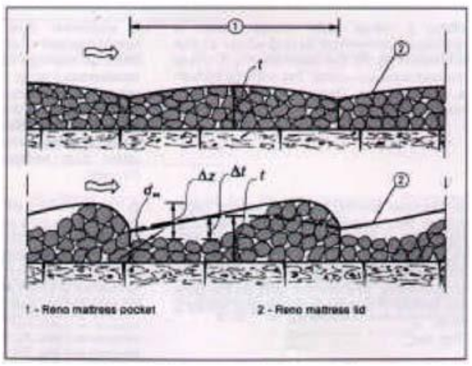
The median stone size for a gabion mattress can be determined from the following equation:

$$d_m = S_f C_s C_v d[(\Upsilon_w/(\Upsilon_s - \Upsilon_w))^{0.5}(V/\sqrt{(gdK_1)})]^{2.5} \qquad (3.23)$$

where:

d_m = average rock diameter in gabions (ft)
S_f = safety factor (1.1 minimum)
C_s = stability coefficient (usually 0.1)
C_v = velocity distribution coefficient = 1.283-0.2log(r/w) (minimum of 1.0) and equals 1.25 at end of dikes and concrete channels
r = center-line bend radius of main channel flow (ft)
w = water surface width of main channel (ft)
d = local flow depth at V (ft)
g = acceleration due to gravity (32.2 ft/s^2)
V = depth-averaged velocity (ft/s)
K_1 = side slope correction factor (Table 3.8)
Υ_w = unit weight of water (62.4 lb/ft^3)
Υ_s = unit weight of stone (lb/ft^3)

Table 3.8 Values of K_1 for various Side Slopes to be used in Equation 3.23	
Side Slope	**K_1**
1V : 1H	0.46
1V : 1.5H	0.71
1V : 2H	0.88
1V : 3H	0.98
<1V : 4H	1.0



Equation 3.23 was developed to design stone size such that the movement of filler stone in the mattresses is prevented. This eliminates deformation that can occur when stone sizes are not large enough to withstand the forces of the water. The result of mattresses deformation is stress on the basket wire and increases in resistance to flow and the likelihood of basket failure. The upper portion of Figure 3.23 shows an undeformed gabion, while the lower portion shows how gabions deform under high-velocity conditions. Maccaferri Gabions gives guidance on sizing stone and allowable velocities for gabion baskets and mattresses, shown in Table 3.9.

Figure 3.23 Gabion Mattress Showing Deformation of Mattress Pockets under High Velocities

Table 3.9 Stone Sizes and Allowable Velocities for Gabions

Type	Thickness (ft)	Filling Stone Range	D_{50}	Critical Velocity	Limit Velocity
Mattress	0.5	3 – 4”	3.4”	11.5	13.8
	0.5	3 – 6”	4.3”	13.8	14.8
	0.75	3 – 4”	3.4”	14.8	16
	0.75	3 – 6”	4.7”	14.8	20
	1.0	3 – 5”	4”	13.6	18
	1.0	4 – 6”	5”	16.4	21
Basket	1.5	4 – 8”	6”	19	24.9
	1.5	5 – 10”	7.5”	21	26.2

When the data in Table 3.9 are compared to Equation 3.22, if V = 11.5, C_s = 0.1, C_v = 1.0, K_1 = 0.71, , γ_s = 150 lb/ft^3 and S_f = 1.1, the local flow depth must be on the order of 25 ft in order to arrive at the stone diameter of 3.4 in. shown in Table 3.9. Designers should use Equation 3.23 to take the depth of flow into account. Table 3.9 does, however, give some general guidelines for fill sizes and is a quick reference for maximum allowable velocities.

Maccaferri also gives guidance on the stability of gabions in terms of shear stress limits. The following equation gives the shear for the bed of the channel as:

$$\tau_b = \Upsilon_w S d \qquad (3.24)$$

where S = bed or water surface slope through the reach (ft/ft)

The bank shear is generally taken as 75 percent of the bed shear, i.e.,

$$\tau_m = 0.75\ \tau_m \qquad (3.25)$$

These values are then compared to the critical stress for the bed calculated by the following equation:

$$\tau_c = 0.10(\Upsilon_s - \Upsilon_w)d_m \qquad (3.26)$$

with critical shear stress for the banks given as:

$$\tau_s = \tau_c\sqrt{(1-(\sin^2\Theta/0.4304))} \qquad (3.27)$$

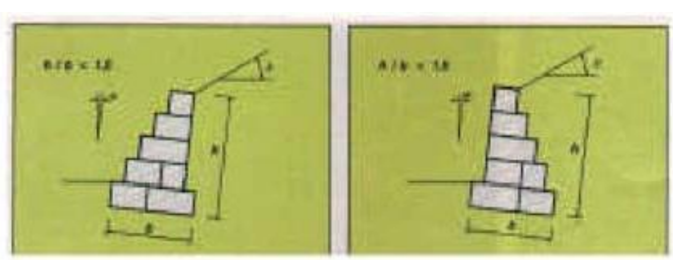
where Θ = angle of the bank rotated up from horizontal.

A design is acceptable if $\tau_b < \tau_c$ and $\tau_m < \tau_s$. If either $\tau_b > \tau_c$ or $\tau_m > \tau_s$ then a check must be made to see if they are less than 120 percent of τ_b and τ_s. If the values are less than 120 percent of τ_b and τ_s the gabions will not be subject to more than what Maccaferri defines as "acceptable" deformation. However it is recommended that the stone size be increased to limit deformation if possible.

Research has indicated that stone in the gabion mattress should be sized such that the largest stone diameter is not more than about two times the diameter of the smallest stone diameter and the mattress should be at least twice the depth of the largest stone size. The size range should, however, vary by about a factor of two to ensure proper packing of the stone material into the gabions. Since the mattresses normally come in discrete sizes, i.e. 0.5, 1.0, and 1.5 ft in depth, normal practice is to size the stone and then select the basket depth that is deep enough to be at least two times the largest stone diameter. The smallest stone should also be sized such that it cannot pass through the wire mesh.

Stability of Underlying Bed and Bank Material

Another critical consideration is the stability of the gabion foundation. This includes both geotechnical stability and the resistance of the soil under the gabions to the erosive forces of the water moving through the gabions. If there is any question regarding the stability of the foundation, i.e. possibility of rotational failures, slip failures, etc., a qualified geotechnical engineer should be consulted prior to and during the design of the bank/channel protection. Several manufacturers give guidance on how to check for geotechnical failure.



One of the critical factors in determining stability is the velocity of the water that passes through the gabions and reaches the soil behind the gabion. The water velocity under the filter fabric, i.e. water that moves through the gabions and filter fabric, is estimated to be one-fourth to one-half of the velocity at the mattress/filter interface.

Figure 3.24 Front-step and Rear-step Gabion Layout

The velocity at the mattress/filter interface, V_b, is estimated to be

$$V_b = 1.486\sqrt{S}(d_m/2)^{2/3}/n_f \qquad (3.28)$$

where n_f = 0.02 for filter fabric, 0.022 for gravel filter material

If the underlying soil material is not stable, additional filter material must be installed under the gabions to ensure soil stability.

The limit for velocity on the soil is different for each type of soil. The limit for cohesive soils is obtained from a chart, while maximum allowable velocities for other soil types are obtained by calculating V_e, the maximum velocity allowable at the soil interface, and comparing it to V" the residual velocity on the bed, i.e. under the gabion mattress and under the filter fabric. V_e for loose soils is equal to $16.1d^2$ while V_f is calculated by:

$$V_f = 1.486S\sqrt{V_a}(d_m/2)^{2/3}/n_f \qquad (3.29)$$

where V_a = average channel velocity (ft/s)

If V_f is larger than two to four times V_e, a gravel filter is required to further reduce the water velocity at the soil interface under the gabions until V_f is in an acceptable range. To check for the acceptability of the filter use the average gravel size for d_m in Equation 5.28. If the velocity V_f is still too high, the gravel size should be reduced to obtain an acceptable value for V_f.

Other Design Considerations

It may be possible to combine gabions with less harsh methods of bank protection on the upper bank and still achieve the desired result of a stable channel. Provisions for large woody vegetation and a more aesthetically pleasing project may also be used on the upper banks or within the gabions. However, the stability of vegetation or other upper bank protection should be carefully analyzed to ensure stability of the upper bank area. A failure in the upper bank region can adversely affect gabion stability and lead to project failure.

3.2.9 Uniform Flow – Example Problems

Example 1 -- 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 3.3:

- 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.
- 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.

Example 2 -- Grassed Channel Design Stability

A trapezoidal channel 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.

From Table 3.3, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5% is 4 ft/s.

Assume an n value of 0.035 and find the value of vR from Figure 3.2, vR = 5.4

- Use Equation 3.9 to calculate the value of R: R = 5.4/4 = 1.35 ft
- Use Equation 3.10 to calculate the value of vR:
  $vR = [1.49\,(1.35)^{5/3}\,(0.015)^{1/2}]/0.035 = 8.60$

- 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 3.2)	R (Eq. 3.9)	vR (Eq. 3.10)
0.035	5.40	1.35	8.60
0.038	3.8	0.95	4.41
0.039	3.4	0.85	3.57
0.040	3.2	0.80	3.15

Select n = 0.040 for stability criteria.

- Use Figure 3.4 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

  Qn = (50) (0.040) = 2.0, S = 0.015
  For b = 10 ft, d = (10) (0.098) = 0.98 ft, b = 8 ft, d = (8) (0.14) = 1.12 ft

  Select:

  b = 10 ft, such that R is approximately 0.80 ft

  z = 3

  d = 1 ft

  v = 3.9 ft/s (Equation 3.1)

  Fr = 0.76 (Equation 3.8)

  Flow is subcritical

  Design capacity calculations for this channel are presented in Example 3 below.

Example 3 -- Grassed Channel Design Capacity

Use a 10-ft bottom width and 3:1 side-slopes for the trapezoidal channel sized in Example 2 and find the depth of flow for retardance Class C.

- Assume a depth of 1.0 ft and calculate the following (see Figure 3.6):

  $A = (b + zd)\,d = [10 + (3)(1)](1) = 13.0$ square ft

  $R = [(b + zd)\,d]/\{b + [2d(1 + z^2)^{0.5}]\} = \{[10+(3)(1)]1\}/\{10+[(2)(1)(1+3^2)^{0.5}]\}$

  $R = 0.796$ ft

- Find the velocity: v = Q/A = 50/13.0 = 3.85 ft/s
- Find the value of vR: vR = (3.85) (0.796) = 3.06
- Using the vR product from Step 3, find Manning's n from Figure 3.2 for retardance Class C (n = 0.047)
- Use Figure 3.3 or Equation 3.1 to find the velocity for S = 0.015, R = 0.796, and n = 0.047: <u>v = 3.34 ft/s</u>
- 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 (ft)	Area (ft^2)	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's n (Fig. 5.2)	Velocity (Eq. 5.1)
1.0	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
1.1	14.63	0.863	3.42	2.95	0.048	3.45
1.2	16.32	0.928	3.06	2.84	0.049	3.54

- Select a depth of 1.1 with an n value of 0.048 for design capacity requirements. Add at least 0.2 ft for freeboard to give a design depth of 1.3 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 = 1.3 ft, z = 3, S = 0.015
  Top width = (10) + (2) (3) (1.3) = 17.8 ft

  n (stability) = 0.040, d = 1.0 ft, v = 3.9 ft/s, Froude number = 0.76 (Equation 3.8)
  n (capacity) = 0.048, d = 1.1 ft, v = 3.45 ft/s, Froude number = 0.64 (Equation 3.8)

Example 4 -- Riprap Design

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.

- Use 8 ft/s as the design velocity, because the reach is short and the bend is not protected.
- Determine the bend correction coefficient for the ratio of R_b/T = 50/20 = 2.5. From Figure 3.8, C_b = 1.55. The adjusted effective velocity is (8) (1.55) = 12.4 ft/s.
- Determine the correction coefficient for the specific weight of 170 lbs from Figure 3.9 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 ft/s.
- Determine minimum d_{30} from Figure 3.10 or Equation 3.13 as about 10 inches.
- Use a gradation with a minimum d_{30} size of 10 inches.
- *(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 3.11, 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).
- 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 3.7.

3.2.10 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer 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 and Bridge Waterways Analysis Model (WSPRO) developed for the Federal Highway Administration. 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 (Chow, 1959, TxDOT, 2002). In the Direct Step method an increment of water depth is chosen, and the distance over which the depth change occurs is computed. This method is often used in association with culvert hydraulics. It is most accurate when the slope and depth increments are small. It is appropriate for prismatic channel sections which occur in most conduits, and can be useful when estimating both supercritical and subcritical profiles. For supercritical flow, the water surface profile is computed downstream. For subcritical flow, the water surface profile is computed upstream.

For an irregular nonuniform 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 the designed waterway. Channel cross sections will be required at each location along the waterway where there are changes in channel shape or dimension, changes in the flowline slope, and changes in vegetation or channel lining. These sections are in addition to any sections necessary to define obstructions such as culverts, bridges, damns, energy dissipation features, or aerial crossings (pipelines). Sections should usually be no more than 4 to 5 channel widths apart or 100 feet apart for ditches or streams and 500 feet apart for floodplains, unless the channel is very regular.

3.2.11 Rectangular, Triangular and Trapezoidal Open Channel Design

Introduction

The Federal Highway Administration has prepared numerous design figures to aid in the design of open channels. Copies of these figures, a brief description of their use, and several example design problems are presented. For design conditions not covered by the figures, a trial and error solution of Manning's Equation must be used. However, it is anticipated that available software programs will be the first choice for solving these design computations.

Description of Figures

Figures given in FHWA, HDS No. 3, 1973 and Atlanta Regional Commission, 2001 are for the direct solution of the Manning's Equation for various sized open channels with rectangular, triangular, and trapezoidal cross sections. Each figure (except for the triangular cross section) is prepared for a channel of given bottom width and a particular value of Manning's n.

The figures for rectangular and trapezoidal cross section channels are used the same way. The abscissa scale of discharge in cubic feet per second (cfs), and the ordinate scale is velocity in feet per second (ft/s). Both scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth (ft), and slightly inclined lines representing channel slope (ft/ft). A heavy dashed line on each figure shows critical flow conditions. Auxiliary abscissa and ordinate scales are provided for use with other values of n and are explained in the example problems. In the figures, interpolations may be made not only on the ordinate and abscissa scales but also between the inclined lines representing depth and slope.

The chart for a triangular cross section (see Figure 1.2) is in nomograph form. It may be used for street sections with a vertical (or nearly vertical) curb face. The nomograph also may be used for shallow V-shaped sections by following the instructions on the chart.

Instructions for Rectangular and Trapezoidal Figures

Figures such as Figure 3.25 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.

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 figure 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 figures 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.

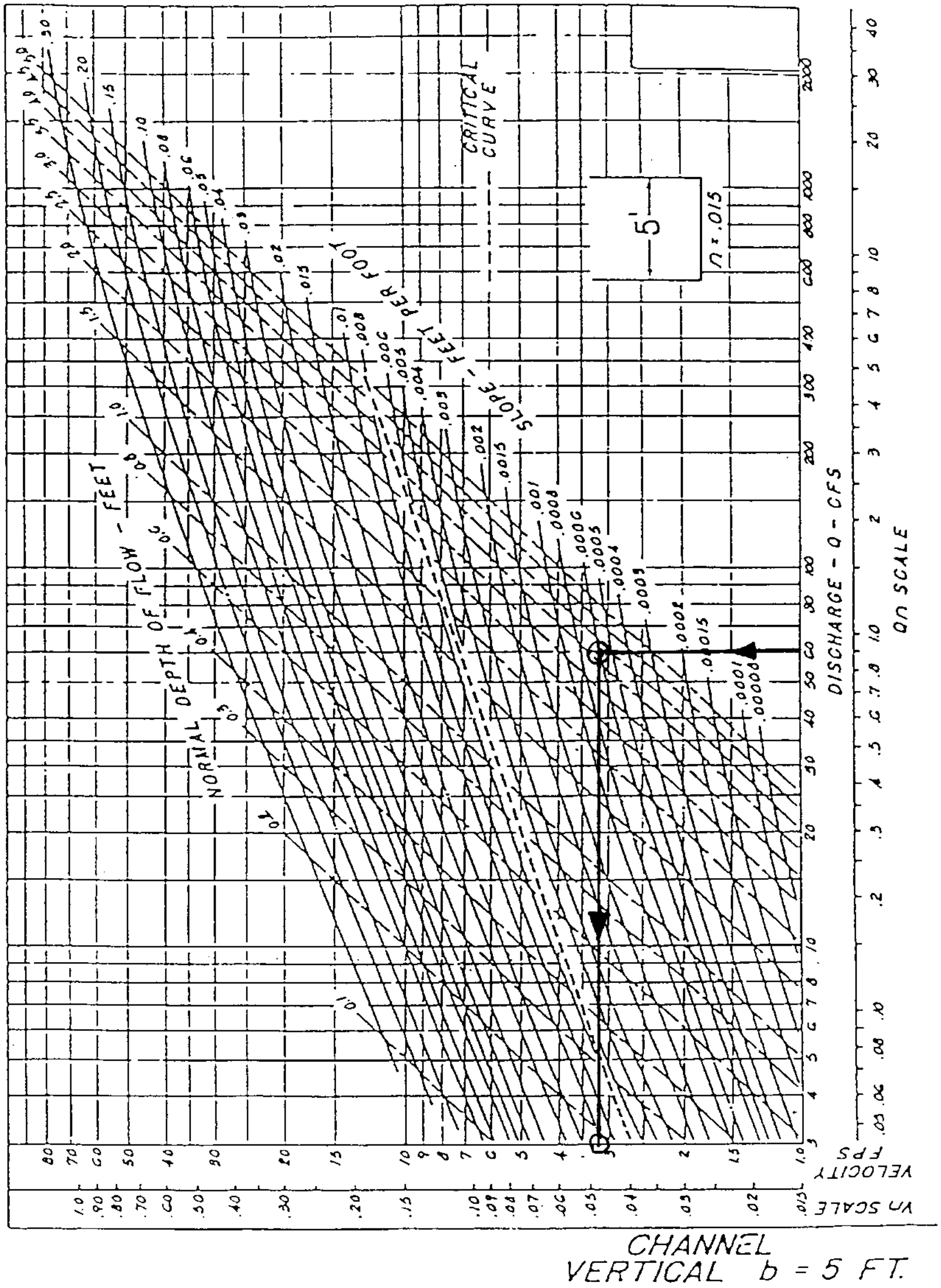
Example Design Problem 1

Given: A rectangular concrete channel 5 ft wide with n = 0.015, .06 percent slope (S = .0006), discharging 60 cfs.

Find: Depth, velocity, and type of flow

Procedure:

1. From *Section 3.2.11*, select the rectangular figure for a 5-ft width (Figure 3.25).
2. From 60 cfs on the Q scale, move vertically to intersect the slope line S = .0006, and from the depth lines read d_n = 3.7 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 3.2 ft/s, on the ordinate scale.
4. The intersection lies below the critical curve, and the flow is therefore in the subcritical range.



Source: Federal Highway Administration

Figure 3.25 Example Nomograph #1

Example Design Problem 2

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:

1. Select the trapezoidal figure for b = 4 ft (see Figure 3.26).
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.
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, and the flow is therefore subcritical.

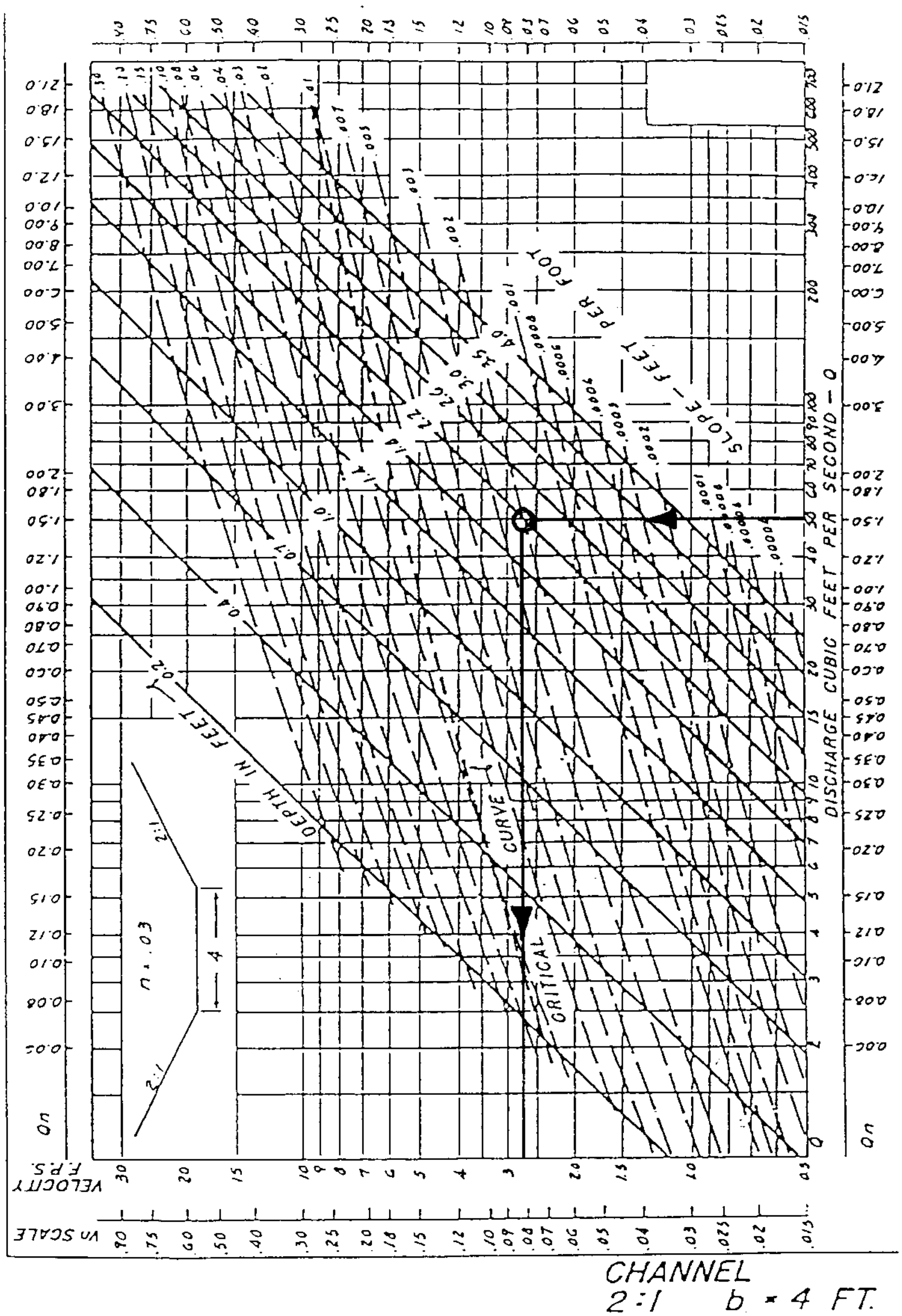
Example Design Problem 3

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

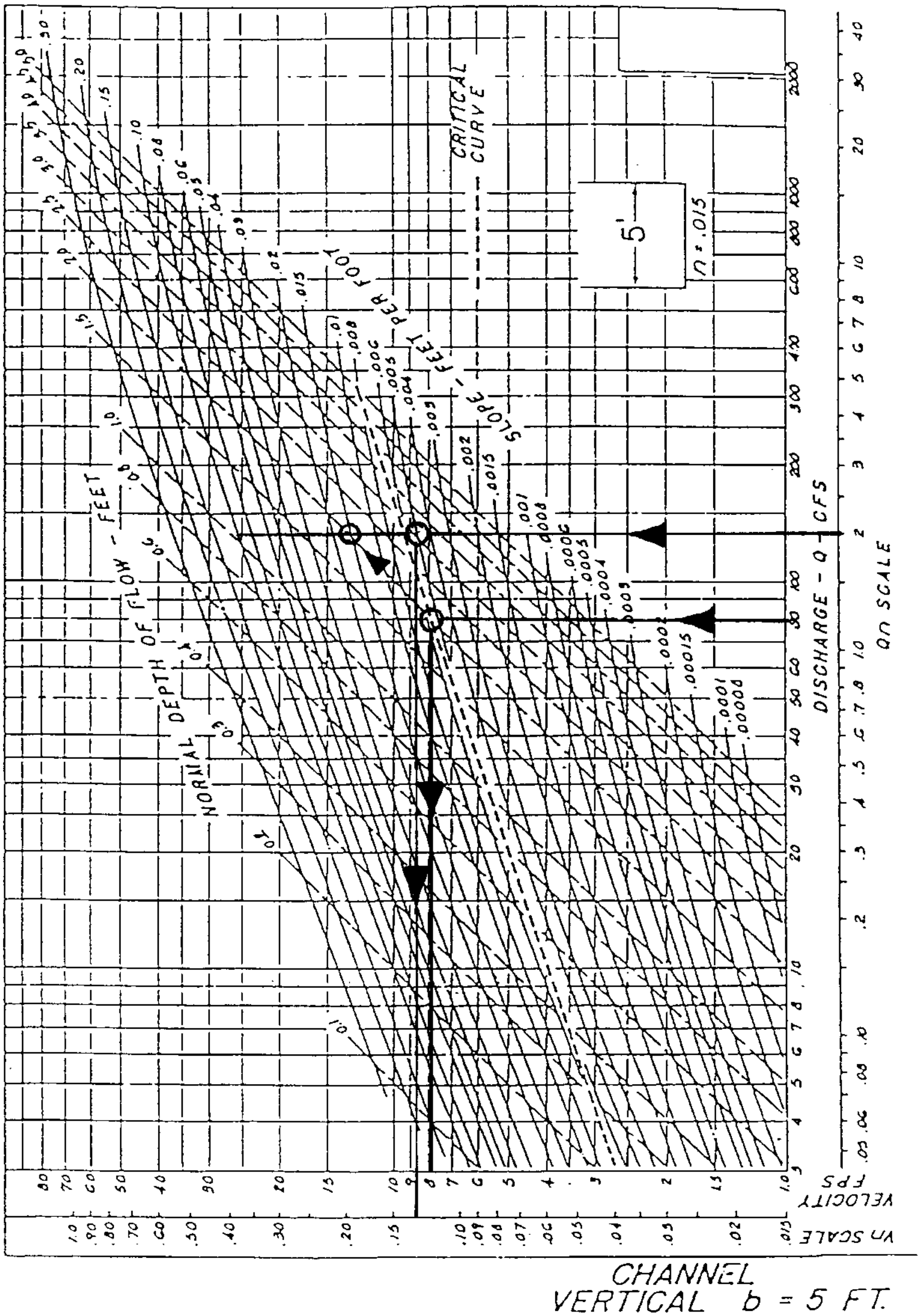
Procedure:

1. Select the rectangular figure for a 5 ft width (Figure 3.27).
2. Multiply Q by n to obtain Qn: 80 x 0.025 = 2.0.
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.
4. Move horizontally from the intersection and read Vn = .13, then Vn/n = 0.13/0.025 = 5.2 ft/s.
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 3.14, 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.
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.



Source: Federal Highway Administration

Figure 3.26 Example Nomograph #2



Source: Federal Highway Administration

Figure 3.27 Example Nomograph #3

3.2.12 Grassed Channel Figures

The Manning equation can be used to determine the capacity of a grass-lined channel, but the value of n varies with the type of grass, development of the grass cover, depth, and velocity of flow. The variable value of 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.

To aid in the design of grassed channels, the Federal Highway Administration has prepared numerous design figures. Copies of these figures are in *Section 3.2.11*. Following is a brief description of general design criteria, 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.

Description of Figures

A set of figures in FHWA, NDS No. 3, 1973 and Atlanta Regional Commission, 2001 are designed for use in the direct solution of the Manning equation for various channel sections lined with grass. The figures are similar in appearance and use to those for trapezoidal cross sections described earlier. However, their construction is much more difficult because the roughness coefficient (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 3.6) and the product V times R is shown in Figure 3.2. As indicated in Table 3.6, 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 3.6 is used to determine retardance classification.

The grassed channel figures 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.

Instructions for Grassed Channel Figures

The grassed channel figures like those in Figure 3.12 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 length sufficient to establish uniform flow. The figures 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 3.2 and 3.3. The use of the figures is explained in the following steps:

Step 1 Select the channel cross section to be used and find the appropriate figure.

Step 2 Enter the lower graph (for retardance Class C) on the figure 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.

Step 3 To check the velocity developed against the permissible velocities (Tables 3.2 and 3.3), 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 Design Problem 1

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=.02), discharging 20 cfs.

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

Procedure:

1. Select figure for 4:1 side slopes (see Figure 3.28).
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.
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 3.3.

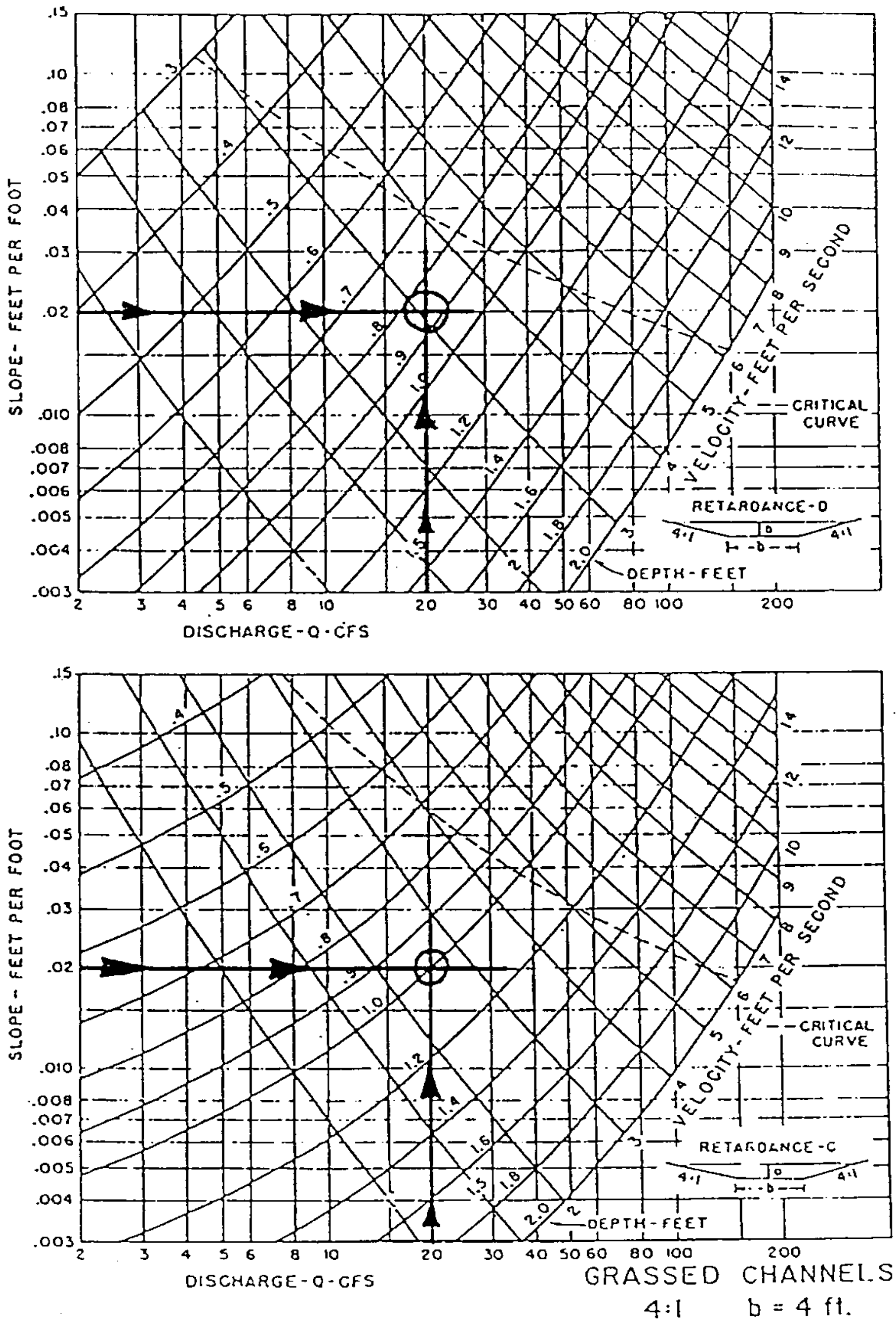
Example Design Problem 2

Given: The channel and discharge of Example 1.

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

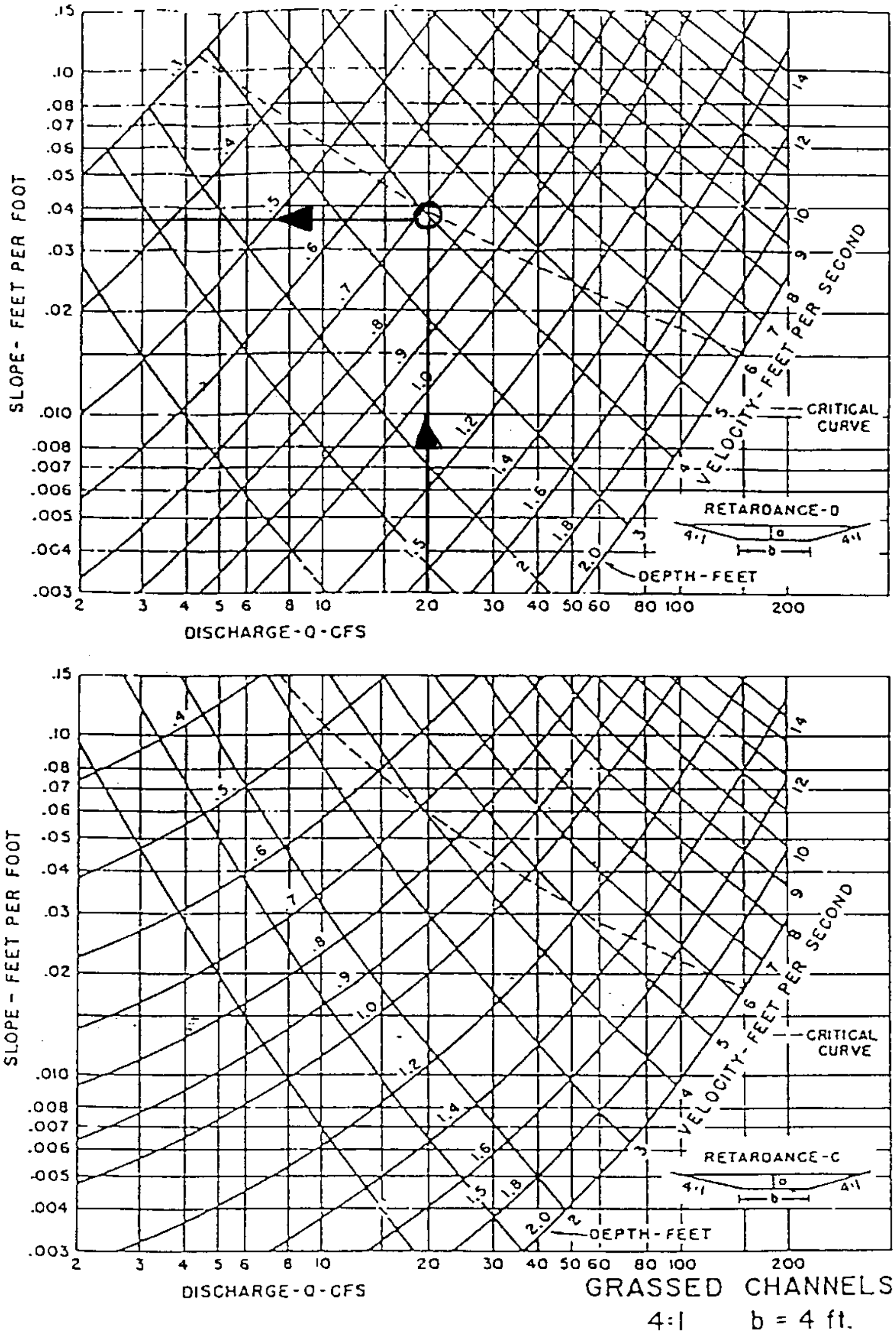
Procedure:

With an increase in slope (but still less than 5%), the allowable velocity is estimated to be 4 ft/s (see Table 3.3). On the upper graph of Figure 3.29 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.



Source: Federal Highway Administration

Figure 3.28 Example Nomograph #4



Source: Federal Highway Administration

Figure 3.29 Example Nomograph #5

3.3 Culvert Design

3.3.1 Overview

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

The hydraulic and structural designs of a culvert must be such that minimal risks to traffic, property damage, and failure from floods prove the results of good engineering practice and economics. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Design considerations include site and roadway data, design parameters (including shape, material, and orientation), hydrology (flood magnitude versus frequency relation), and channel analysis (stage versus discharge relation).

3.3.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual the symbols listed in Table 3.10 will be used. These symbols were selected because of their wide use.

Table 3.10 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 depth	in or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_u	Uniform depth of flow	ft
g	Acceleration of gravity	ft/s
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 invert 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/f
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
V_c	Critical velocity	ft/s

3.3.3 Design Considerations

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following list of design recommendations should be considered for all culvert designs as applicable. Refer to *Section 3.6.3 of the Criteria Manual* or the local review authority for design criteria details.

- **Frequency Flood**
- **Velocity Limitations**
- **Buoyancy Protection**
  - Headwalls, endwalls, slope paving, or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts.
- **Length and Slope**
- **Debris Control**
  - In designing debris control structures, it is recommended that the Hydraulic Engineering Circular No. 9 entitled Debris Control Structures be consulted.
- **Headwater Limitations**
- **Tailwater Considerations**
- **Storage**
- **Culvert Inlets**
  - Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient Ke, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 3.11.
- **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 should 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.
- **Wingwalls and Aprons**
  - Wingwalls are 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.
- **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.

- **Material Selection**
  - Reinforced concrete pipe (RCP), pre-cast and cast in place concrete boxes are recommended for use (1) under a roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams. RCP and fully coated corrugated metal pipe can be used in all other cases. High-density polyethylene (HDPE) pipe may also be used as specified in the municipal regulations. Table 3.12 gives recommended Manning's n values for different materials.

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

- **Weep Holes**
  - Weep holes are sometimes used to relieve uplift pressure on headwalls and concrete rip-rap. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and prevent the formation of piping channels through the fill embankment. The filter materials should 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.

- **Outlet Protection**
  - See *Section 2.2* for information on the design of outlet protection.

- **Erosion and Sediment Control**

- **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 or wetlands. This selection must consider the entire site, including any necessary lead channels.

- **Safety Considerations**
  - Roadside safety should be considered for culverts crossing under roadways. Guardrails or safety end treatments may be needed to enhance safety at culvert crossings. The AASHTO roadside design guide should be consulted for culvert designs under and adjacent to roadways.

Table 3.11 Inlet Coefficients	
Type of Structure and Design of Entrance	**Coefficient K_e**
Pipe, Concrete	
Projecting from fill, socket end (grove-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, 33.7º or 45º bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal[1]	
Projecting form fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
*End Section conforming to fill slope	0.5
Beveled edges, 33.7º or 45º bevels	0.2
Slide- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of [1/12(D)] or [1/12(B)] or beveled edges on 3 sides	0.2
Wingwalls at 30º to 75º 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 10º or 25º to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

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

* Note: “End Section conforming to fill slope”, made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Source: HDS No. 5, 2001

Table 3.12 Manning's n Values		
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 Corrugations	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

Source: HDS No. 5, 2001

Note: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, 2001, HDS No. 5, pages 201 - 208.

3.3.4 Design Procedures

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:

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.

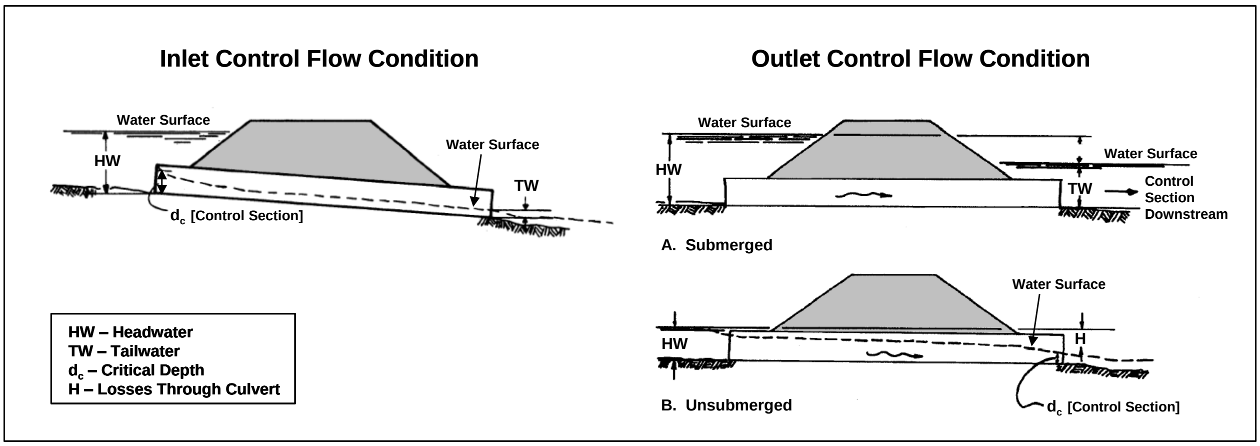


Figure 3.30 Culvert Flow Conditions
(Adapted from: HDS-5, 2001)

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control, see the FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001.

Procedures

The culvert design process includes the following basic stages:

1. Define the location, orientation, shape, and material for the culvert to be designed. In many instances, consider more than single shape and material.
2. With consideration of the site data, establish allowable outlet velocity and maximum allowable depth of barrel.
3. Based on upon subject discharges, associated tailwater levels, and allowable headwater level, define an overall culvert configuration to be analyzed (culvert hydraulic length, entrance conditions, and conduit shape and material).
4. Determine the flow type (supercritical or subcritical) to establish the proper path for determination of headwater and outlet velocity.
5. Optimize the culvert configuration.
6. Treat any excessive outlet velocity separately from headwater.

There are three procedures for designing culverts: inlet control design equations, manual use of inlet and outlet control nomographs, and the use computer programs such as HY8. It is recommended that the HY8 computer model or equivalent be used for culvert design. The computer software package HYDRAIN, which includes HY8, uses the theoretical basis from the nomographs to size culverts. In addition, this software 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.

Inlet Control Design Equations

This section contains explanations of the equations and methods used to develop the design charts in HDS No. 5, where those equations and methods are not fully described in the main text. The following topics are discussed: the design equations for the unsubmerged and submerged inlet control nomographs, the dimensionless design curves for culvert shapes and sizes without nomographs, and the dimensionless critical depth charts for long span culverts and corrugated metal box culverts.

Inlet Control Nomograph Equations: The design equations used to develop the inlet control nomographs are based on the research conducted by the National Bureau of Standards (NBS) under the sponsorship of the Bureau of Public Roads (now the Federal Highway Administration). Seven progress reports were produced as a result of this research. Of these, the first and fourth through seventh reports dealt with the hydraulics of pipe and box culvert entrances, with and without tapered inlets (4, 7, to 10). These reports were one source of the equation coefficients and exponents, along with other references and unpublished FHWA notes on the development of the nomographs (56 and 57).

The two basic conditions on inlet control depend upon whether the inlet end of the culvert is or is not submerged by the upstream headwater. If the inlet is not submerged, the inlet performs as a weir. If the inlet is submerged, the inlet performs as an orifice. Equations are available for each of the above conditions.

Between the unsubmerged and the submerged conditions, there is a transition zone for which the NBS research provided only limited information. The transition zone is defined empirically by drawing a curve between and tangent to the curves defined by the unsubmerged and submerged equations. In most cases, the transition zone is short and the curve is easily constructed.

Table 3.13 contains the unsubmerged and submerged inlet control design equations. Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at critical depth, adjusted with tow correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but Form (2) is easier to apply and is the only documented form of equation for some of the inlet control nomographs.

The constants and the corresponding equation form are given in Table 3.14. Table 3.14 is arranged in the same order as the design nomographs shown later in this section, and provides the unsubmerged and submerged equation coefficients for each shape, material, and edge configuration. For the unsubmerged equations, the form of the equation is also noted.

The equations may be used to develop design curves for any conduit shape or size. Careful examination of the equation constants for a given form of equation reveals that there is very little difference between the constants for a given inlet configuration. Therefore, given the necessary conduit geometry for a new shape from the manufacturer, a similar shape is chosen from Table 3.14, and the constants are used to develop new design curves. The curves may be quasi-dimensionless, in terms of $Q/AD^{0.5}$ and HW_i/D, or dimensional, in terms of Q and HW_i for a particular conduit size. To make the curves truly dimensionless, $Q/AD^{0.5}$ must be divided by $g^{0.5}$, but this results in small decimal numbers. Note that coefficients for rectangular (Box) shapes should not be used for nonrectangular (circular, arch, pipe-arch, etc.) shapes and vice-versa. A constant slope value of 2 percent (0.02) is usually selected for the development of design curves. This is because the slope effect is small and the resultant headwater is conservatively high for sites with slopes exceeding 2 percent (except for mitered inlets).

Table 3.13 Inlet Control Design Equations

Unsubmerged*		
Form (1)	$\frac{HW_i}{D} = \frac{H_c}{D} + K\left(\frac{K_u Q}{AD^{0.5}}\right)^M - 0.5S$ ***	**(3.30)**
Form (2)	$\frac{HW_i}{D} = K\left(\frac{K_u Q}{AD^{0.5}}\right)^M$	**(3.31)**
Submerged**		
	$\frac{HW_i}{D} = c\left(\frac{K_u Q}{AD^{0.5}}\right)^2 + Y - 0.5S$ ***	**(3.32)**
Definitions		
HW_i	Headwater depth above inlet control section invert, m (ft)	
D	Interior height of culvert barrel, m (ft)	
H_c	Specific head at critical depth ($d_c + V_c^2/2g$), m^2 (ft^2)	
Q	Discharge, m^3/s (ft^3/s)	
A	Full cross sectional area of culvert barrel, m^2 (ft^2)	
S	Culvert barrel slope, m/m (ft/ft)	
K, M, c, Y	Constants from Table 3.14	
K_u	1.811 SI (1.0 English)	
* Equations 3.30 and 3.31 (unsubmerged) apply to about $Q/AD^{0.5}$ = 1.93 (3.5 English)		
** Equation 3.32 (submerged) above applies to about $Q/AD^{0.5}$ = 2.21 (4.0 English)		
*** For mitered inlets use +0.7 S instead of -0.5 S as the slope correction factor.		

Table 3.14 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged		Submerged		References*
					K	M	c	Y	
1	Circular Concrete	1 2 3	Square edge w/ headwall Groove end w/ headwall Groove end projecting	1	.0098 .0018 .0045	2.0 2.0 2.0	.0398 .0292 .0317	.67 .74 .69	56/57
2	Circular CMP	1 2 3	Headwall Mitered to slope Projecting	1	.0078 .0210 .0340	2.0 1.33 1.50	.0379 .0463 .0553	.69 .75 .54	56/57
3	Circular	A B	Beveled ring, 45º bevels Beveled ring, 33.7º bevels	1	.0018 .0018	2.50 2.50	.0300 .0243	.74 .83	57
8	Rectangular Box	1 2 3	30º to 75º wingwall flares 90º and 15º wingwall flares 0º wingwall flares	1	.026 .061 .061	1.0 .75 .75	.0347 .0400 .0423	.81 .80 .82	56 56 8
9	Rectangular Box	1 2	45º wingwall flare d = .043D 18º to 33.7º wingwall flare d = .083D	2	.510 .486	.667 .667	.0309 .0249	.80 .83	8

Table 3.14 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged K	Unsubmerged M	Submerged c	Submerged Y	References*
10	Rectangular Box	1 2 3	90º headwall w/ 3/4" chamfers 90º headwall w/ 45º bevels 90º headwall w/ 33.7º bevels	2	.515 .495 .486	.667 .667 .667	.0375 .0314 .0252	.79 .82 .865	8
11	Rectangular Box	1 2 3 4	3/4" chamfers; 45º skewed headwall 3/4" chamfers; 30º skewed headwall 3/4" chamfers; 15º skewed headwall 45º bevels; 10º-45º skewed headwall	2	.545 .533 .522 .498	.667 .667 .667 .667	.0451 .0425 .0402 .0327	.73 .705 .68 .75	8
12	Rectangular Box 3/4" chamfers	1 2 3	45º non-offset wingwall flares 18.4º non-offset wingwall flares 18.4º non-offset wingwall flares 30º skewed barrel	2	.497 .493 .495	.667 .667 .667	.0339 .0361 .0386	.803 .806 .71	8
13	Rectangular Box Top Bevels	1 2 3	45º wingwall flares - offset 33.7º wingwall flares - offset 18.4º wingwall flares - offset	2	.497 .495 .493	.667 .667 .667	.0302 .0252 .0227	.835 .881 .887	8
16-19	CM Boxes	2 3 5	90º headwall Thick wall projecting Thin wall projecting	1	.0083 .0145 .0340	2.0 1.75 1.5	.0379 .0419 .0496	.69 .64 .57	57
29	Horizontal Ellipse Concrete	1 2 3	Square edge w/ headwall Groove end w/ headwall Groove end projecting	1	.0100 .0018 .0045	2.0 2.5 2.0	.0398 .0292 .0317	.67 .74 .69	57
30	Vertical Ellipse Concrete	1 2 3	Square edge w/ headwall Groove end w/ headwall Groove end projecting	1	.0100 .0018 .0095	2.0 2.5 2.0	.0398 .0292 .0317	.67 .74 .69	57
34	Pipe Arch 18" Corner Radius CM	1 2 3	90º headwall Mitered to slope Projecting	1	.0083 .0300 .0340	2.0 1.0 1.5	.0379 .0463 .0496	.69 .75 .57	57
35	Pipe Arch 18" Corner Radius CM	1 2 3	Projecting No Bevels 33.7º Bevels	1	.0300 .0088 .0030	1.5 2.0 2.0	.0496 .0368 .0269	.57 .68 .77	56
36	Pipe Arch 31" Corner Radius CM	1 2 3	Projecting No Bevels 33.7º Bevels	1	.0300 .0088 .0030	1.5 2.0 2.0	.0496 .0368 .0269	.57 .68 .77	56
41-43	Arch CM	1 2 3	90º headwall Mitered to slope Thin wall projecting	1	.0083 .0300 .0340	2.0 1.0 1.5	.0379 .0463 .0496	.69 .75 .57	57
55	Circular	1 2	Smooth tapered inlet throat Rough tapered inlet throat	2	.534 .519	.555 .64	.0196 .0210	.90 .90	3
56	Elliptical Inlet Face	1 2 3	Tapered inlet-beveled edges Tapered inlet-square edges Tapered inlet-thin edge projecting	2	.536 .5035 .547	.622 .719 .80	.0368 .0478 .0598	.83 .80 .75	3
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97	3

Table 3.14 Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged K	Unsubmerged M	Submerged c	Submerged Y	References*
58	Rectangular Concrete	1	Side tapered-less favorable edges	2	.56	.667	.0446	.85	3
		2	Side tapered-more favorable edges		.56	.667	.0378	.87	
59	Rectangular Concrete	1	Slope tapered-less favorable edges	2	.50	.667	.0446	.65	3
		2	Slope tapered-more favorable edges		.50	.667	.0378	.71	

* These references are cited in FHWA, 2001, HYD-5. They can be accessed at the Federal Highway Administration web site: www.fhwa.dot.gov/bridge/hydpub.htm.

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 3.31 (a) and (b) show examples of an inlet control and outlet control nomographs for the design of concrete pipe culverts. For other culvert designs, refer to the complete set of nomographs in FHWA Hydraulic Design of Highway Culverts, HDS-5, 2001, Second Edition.

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

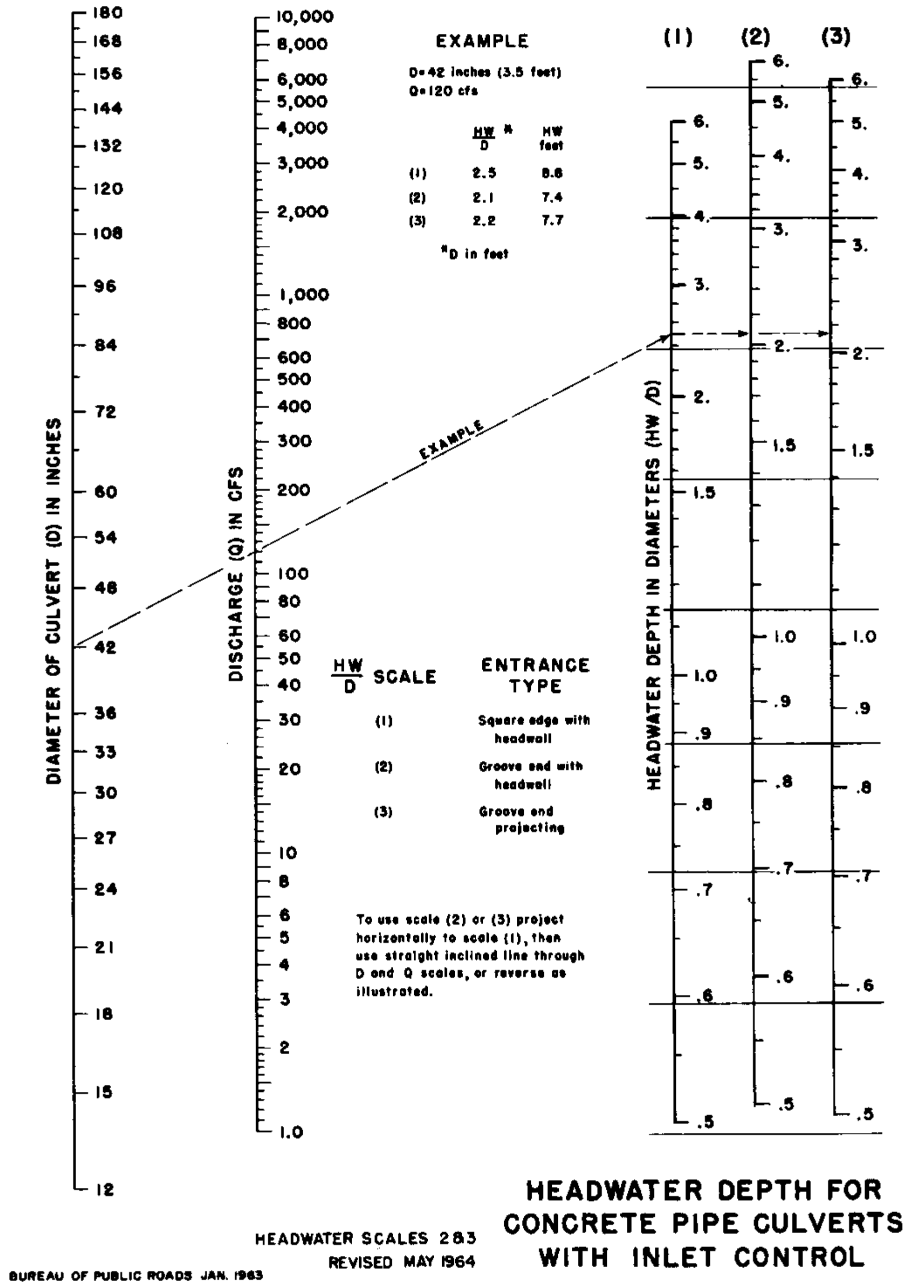


Figure 3.31a Headwater Depth for Concrete Pipe Culvert with Inlet Control

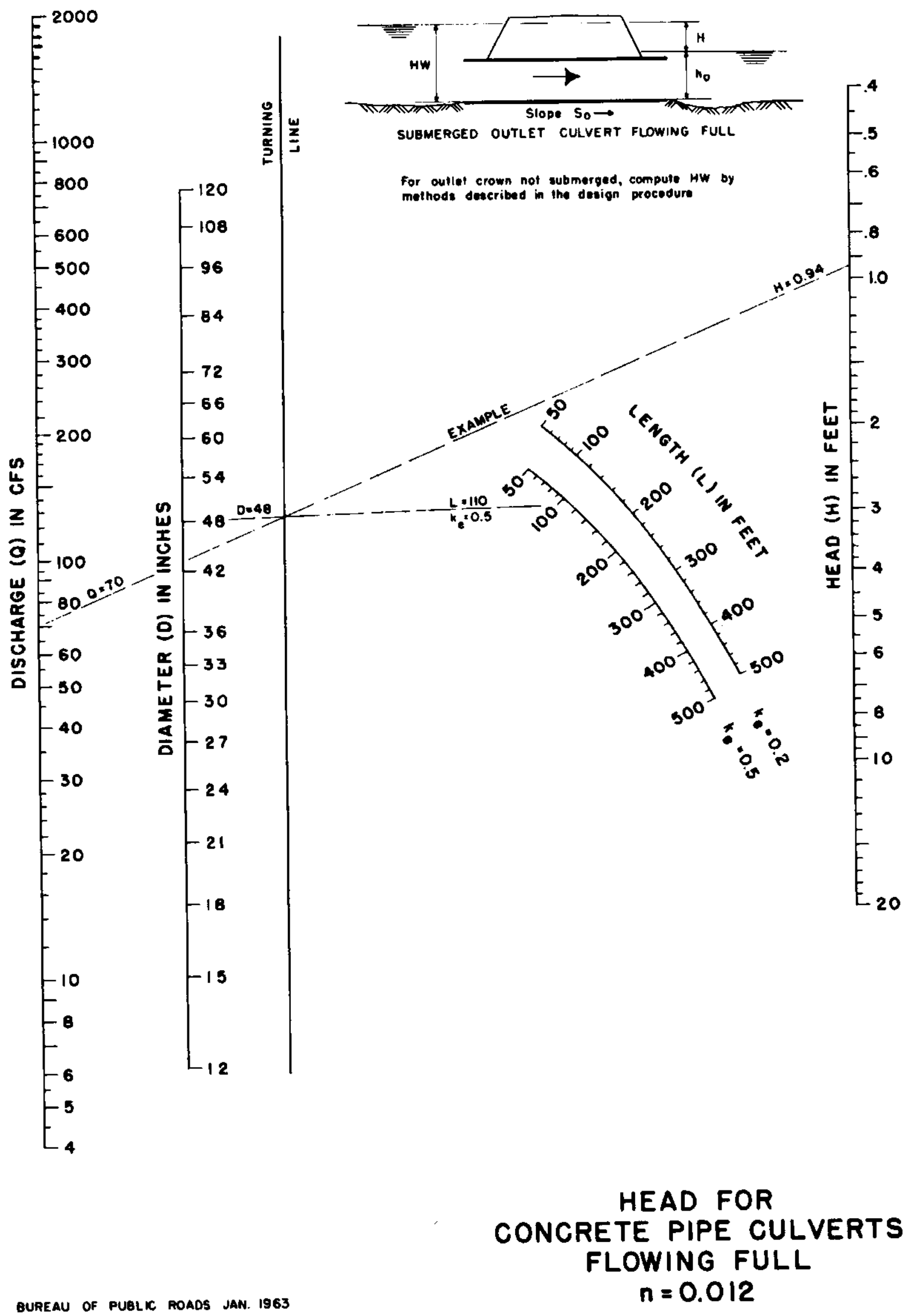


Figure 3.31b Head for Concrete Pipe Culverts Flowing Full

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)

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

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 the equation:

$$\mathbf{HW = H + h_o - LS} \qquad \mathbf{(3.33)}$$

where:

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

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 4.0* for appropriate energy dissipation designs. Energy dissipation designs may affect the outlet hydraulics of the culvert.

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, velocities, and scour depths versus discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes. Such computations are made much easier by the use of computer programs.

To complete the culvert design, roadway overtopping should 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 as follows:

Step 1 Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters 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 3.34 to calculate flow rates across the roadway.

$$\mathbf{Q = C_d L (HW)^{1.5}} \qquad \textbf{(3.34)}$$

where:

Q = overtopping flow rate (ft^3/s)

C_d = overtopping discharge coefficient

L = length of roadway (ft)

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

Note: See Figure 3.32 on the next page for guidance in determining a value for C_d. For more information on calculating overtopping flow rates see pages 38 - 44 in HDS No. 5, 2001.

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

Storage Routing

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 3.3.7* for more information on routing. Additional routing procedures are outlined in Hydraulic Design of Highway Culverts, Section V - Storage Routing, HDS No. 5, 2001, Federal Highway Administration, pages 123 - 142.

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.

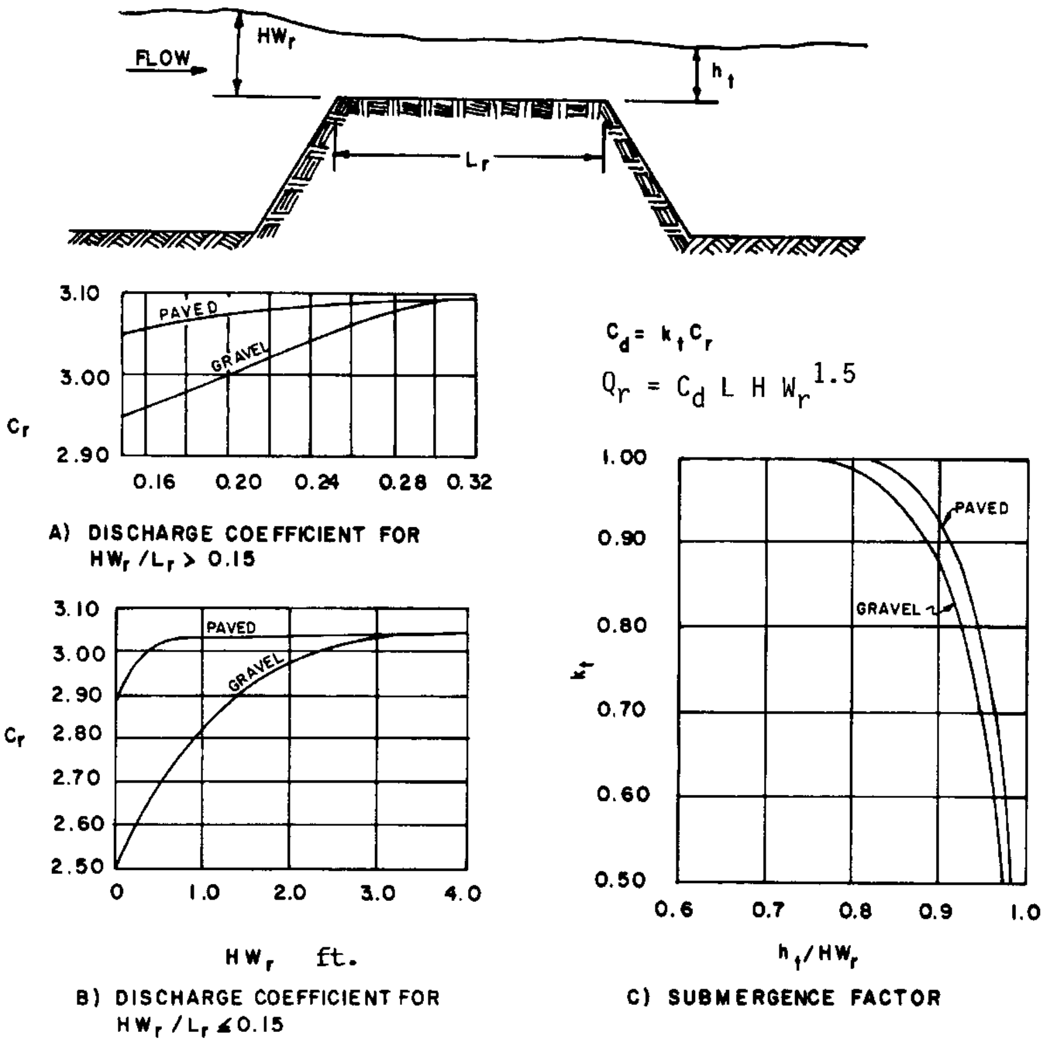


Figure 3.32 Discharge Coefficients for Roadway Overtopping
(Source: HDS No. 5, 2001)

3.3.5 Culvert Design Example

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

Example

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

Input Data

Discharge for 2-yr flood = 35 cfs

Discharge for 25-yr flood = 70 cfs

Allowable H_w 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

Entrance type = Groove end with headwall

Computations

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).

2. The corresponding culvert diameter is about 48 in. This can be calculated by using the formula for area of a circle: Area = (3.14D^2)/4 or D = (Area times 4/3.14)$^{0.5}$. Therefore: D = ((14 sq ft x 4)/3.14) $^{0.5}$ x 12 in/ft) = 50.7 in

3. A grooved end concrete culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 3.31a), with a pipe diameter of 48 inches and a discharge of 70 cfs; read a HW/D value of 0.93.

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 small culvert.

5. Using the same procedures outlined in steps 4 and 5 the following results were obtained.

   42-inch culvert – HW = 4.13 ft

   36-inch culvert – HW = 5.04 ft

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

6. The culvert is checked for outlet control by using Figure 3.31b.

   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

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

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

   Note: critical depth is obtained from Figure 1.19(b).

   Therefore: h_o = 3.5 ft

   The headwater depth for outlet control is:

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

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

8. Estimate outlet exit velocity. Since this culvert is an 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 4.0*.

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 - OK

10. The flood mitigation storm flow should be routed through the culvert to determine if any flooding problems will be associated with this flood.

Figure 3.33 provides a convenient form to organize culvert design calculations.

CULVERT DESIGN FORM

PROJECT: ____________________
____

STATION: ____________

SHEET ______ OF ______

DESIGNER / DATE: ______ OF ______

REVIEWER / DATE: ______ OF ______

HYDROLOGICAL DATA

☐ METHOD: ____________
☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______
☐ CHANNEL SHAPE: ____________
☐ ROUTING: ______ ☐ OTHER: ______

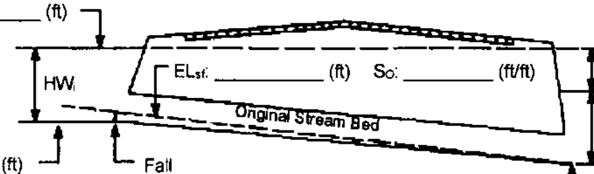
SEE ADD'L SHTS.

DESIGN FLOWS/TAILWATER

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

ROADWAY ELEVATION: ________ (ft)

EL_{hi}: ________ (ft)



EL_i: ________ (ft)

Fall

EL_o: ________ (ft)

$S = S_o - \text{Fall}/L_o$

$S =$ ________ (ft/ft)

$L_o =$ ________ (ft)

CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/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: ____________

Figure 3.33 Culvert Design Calculation Form
(Source: HDS No. 5, 2001)

3.3.6 Design Procedures for Beveled-Edged Inlets

Introduction

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side-tapered, and slope-tapered inlets. Those designers interested in using side- and slope-tapered inlets should consult the detailed design criteria and example designs outlined in the U. S. Department of Transportation publication Hydraulic Design Series No. 5 entitled, Hydraulic Design of Highway Culverts.

Design Figures

Four inlet control figures for culverts with beveled edges are found in Appendix D of HDS No. 5.

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

The following symbols are used in these figures:

B – Width of culvert barrel or diameter of pipe culvert

D – Height of box culvert or diameter of pipe culvert

H_f – Depth of pool or head, above the face section of invert

N – Number of barrels

Q – Design discharge

Design Procedure

The figures for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier. Note that Charts 10, 11, and 12 found in Appendix D of HDS No. 5 apply only to bevels having either a 33º angle (1.5:1) or a 45º angle (1:1).

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

Top Bevel = d = 6 ft x 0.5 inch/ft = 3 inches

Side Bevel = b = 8 ft x 0.5 inch/ft = 4 inches

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

Design Figure Limits

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

For example, in a double 8 ft by 8 ft box culvert:

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.

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.

Multi-barrel Installations

For multi-barrel 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 should be designed as a series of single barrel installations since each pipe requires a separate bevel.

Skewed Inlets

It is recommended that Chart 11 found in Appendix D of HDS No. 5 for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in underdesign due to a greatly reduced capacity. 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.

3.3.7 Flood Routing and Culvert Design

Introduction

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. By not considering flood routing 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.

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.

A multiple trial and error procedure is required for culvert flood routing. In general:

Step 1 A trial culvert(s) is selected

Step 2 A trial discharge for a particular hydrograph time increment (selected time increment to estimate discharge from the design hydrograph) is selected

Step 3 Flood routing computations are made with successive trial discharges until the flood routing equation is satisfied

Step 4 The hydraulic findings are compared to the selected site criteria

Step 5 If the selected site criteria are satisfied, then a trial discharge for the next time increment is selected and this procedure is repeated; if not, a new trial culvert is selected and the entire procedure is repeated.

Comprehensive Design Guidance

Comprehensive design discussions and guidance may be found in the Federal Highway Administration, National Design Series No. 5, document entitled Hydraulic Design of Highway Culverts, Second Edition, published in 2001. This document is available from the National Technical Information Service as Item Number PB2003102411*DL. (http://www.ntis.gov/search.htm) Search for this document using the Item Number.

3.4 Bridge Design

3.4.1 Overview

Bridges enable streams to maintain flow conveyance and to sustain aquatic life. They are important and expensive highway hydraulic structures vulnerable to failure from flood related causes. In order to minimize the risk of failure, the hydraulic requirements of a stream crossing must be recognized and considered during the development, construction, and maintenance phases.

This section addresses structures designed hydraulically as bridges, regardless of length. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Bridges, on the other hand, are not covered with embankment or designed to take advantage of submergence to increase hydraulic capacity, even though some are designed to be inundated under flood conditions. This discussion of bridge hydraulics considers the total crossing, including approach embankments and structures on the floodplains.

The following subsections present considerations related to the hydraulics of bridges. It is generally excerpted from Chapter 9 of the Texas Department of Transportation (TxDOT) Hydraulics Design Manual dated March 2004.

Bridge Hydraulics Considerations

When beginning analysis for a cross-drainage facility, the flood frequency and stage-discharge curves should first be established, as well as the type of cross-drain facility. The choice is usually between a bridge and a culvert. Bridges are usually chosen if the discharge is significant or if the stream to be crossed is large in extent. Both types of facilities should be evaluated and a choice made based on performance and economics. If the stream crossing is wide with multiple concentrations of flow, a multiple opening facility may be in order.

Highway-Stream Crossing Analysis

The hydraulic analysis of a highway-stream crossing for a particular flood frequency involves:

- Determining the backwater associated with each alternative profile and waterway opening(s)
- Determining the effects on flow distribution and velocities
- Estimating scour potential

The hydraulic design of a bridge over a waterway involves the following such that the risks associated with backwater and increased velocities are not excessive:

- Establishing a location
- Bridge length
- Orientation
- Roadway and bridge profiles

A hydrologic and hydraulic analysis is recommended for designing all new bridges over waterways, bridge widening, bridge replacement, and roadway profile modifications that may adversely affect the floodplain, even if no structural modifications are necessary. Typically, this should include the following:

- An estimate of peak discharge (sometimes complete runoff hydrographs)
- Existing and proposed condition water surface profiles for design and check flood conditions
- Consideration of the potential for stream stability problems and scour potential.

Freeboard

Navigational clearance and other reasons notwithstanding, the low chord elevation is defined as the sum of the design normal water surface elevation (high water) and a *freeboard*. For on system TxDOT bridges, TxDOT recommends a minimum freeboard of 2 ft to allow for passage of floating debris and to provide a safety factor for design flood flow. Higher freeboards may be appropriate over streams that are prone to heavy debris loads, such as large tree limbs, and to accommodate other clearance needs. Other constraints may make lower freeboards desirable, but the low chord should not impinge on the design high water. Generally, for off-system bridge replacement structures, the low chord should approximate that of the structure to be replaced, unless the results of a risk assessment indicate a different structure is the most beneficial option.

Roadway/Bridge Profile

A bridge is integrated into both the stream and the roadway and must be fully compatible with both. Therefore, the alignment of the roadway and the bridge are the same between the ends of the bridge. Hydraulically, the complete bridge profile can be any part of the structure that stream flow can strike or impact in its movement downstream. If the stream gets high enough to inundate the structure, then all parts of the roadway and the bridge become part of the complete bridge profile.

For TxDOT design, the roadway must not be inundated by the design flood, but inundation by the flood mitigation storm is allowed. Unless the route is an emergency escape route, it is often desirable to allow floods in excess of the design flood to overtop the road. This helps minimize both the backwater and the required length of structure.

Several vertical alignment alternatives are available for consideration, depending on site topography, traffic requirements, and flood damage potential. The alternatives range from crossings that are designed to overtop frequently to crossings that are designed to rarely or never overtop.

Crossing Profile

The horizontal alignment of a highway at a stream crossing should be taken into consideration when selecting the design and location of the waterway opening as well as the crossing profile. Every effort should be made to align the highway so that the crossing will be normal to the stream flow direction (highway centerline perpendicular to the streamline).

Often, this is not possible because of the highway or stream configuration. When a skewed structure is necessary, it should be ensured that substructure fixtures such as foundations, columns, piers, and bent caps offer minimum resistance to the stream flow.

Bent caps should be oriented as near to the skew of the streamlines at flood stage as possible. Headers should be skewed to minimize eddy-causing obstructions. A relief opening may be provided to reduce the likelihood of trapped flow and minimize the amount of flow that would have to travel up against the general direction of flow along the embankment.

Single Versus Multiple Openings

For a single structure, the flow will find its way to an opening until the roadway is overtopped. If two or more structures have flow area available, after accumulating a head, the flow will divide and proceed to the structures offering the least resistance. The point of division is called a stagnation point.

In usual practice, the TxDOT recommends that the flood discharge be forced to flow parallel to the highway embankment for no more than about 800 ft. If flow distances along the embankment are greater than recommended, an additional relief structure or opening should be considered. A possible alternative to the provision of an additional structure is a guide bank (spur dike) to control the turbulence at the header. Also, natural vegetation between the toe of slope and the right-of-way line is useful in controlling flow along the embankment. Therefore, special efforts should be made to preserve any natural vegetation in such a situation.

Factors Affecting Bridge Length

The discussions of bridge design assume normal cross sections and lengths (perpendicular to flow at flood stage). Usually one-dimensional flow is assumed, and cross sections and lengths are considered 90° to the direction of stream flow at flood stage.

If the crossing is skewed to the stream flow at flood stage, all cross sections and lengths should be normalized before proceeding with the bridge length design. If the skew is severe and the floodplain is wide, the analysis may need to be adjusted to offset the effects of elevation changes within the same cross section.

The following examples illustrate various factors that can cause a bridge opening to be larger than that required by hydraulic design.

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

3.4.2 Symbols and Definitions

The hydraulics of bridge openings are basically the same as those of open channel flow. Therefore, the symbols and definitions are essentially the same as those of in Table 3.1. There are other definitions unique to bridges which are presented here. They are defined in the TxDOT Hydraulic Design Manual.

Flow Zones and Energy Losses

Figure 3.34 shows a plan of typical cross section locations that establish three flow zones that should be considered when estimating the effects of bridge openings.

Zone 1 represents the area between the downstream face of the bridge and a cross section downstream of the bridge within which expansion of flow from the bridge is expected to occur. The distance over which this expansion occurs can vary depending on the flow rate and the floodplain characteristics. No detailed guidance is available, but a distance equal to about four times the length of the average embankment constriction is reasonable for most situations. Section 1 represents the effective channel flow geometry at the end of the expansion zone, which is also called the “exit” section. Cross sections 2 and 3 are at the toe of roadway embankment and represent the portion of unconstricted channel geometry that approximates the effective flow areas near the bridge opening as shown in Figure 3.35.

Zone 2 represents the area under the bridge opening through which friction, turbulence, and drag losses are considered. Generally, the bridge opening is obtained by superimposing the bridge geometry on cross sections 2 and 3.

Zone 3 represents an area from the upstream face of the bridge to a distance upstream where the contraction of flow must occur. A distance upstream of the bridge equal to the length of the average embankment constriction is a reasonable approximation of the location at which contraction begins. Cross section 4 represents the effective channel flow geometry where contraction begins. This is sometimes referred to as the “approach” cross section.

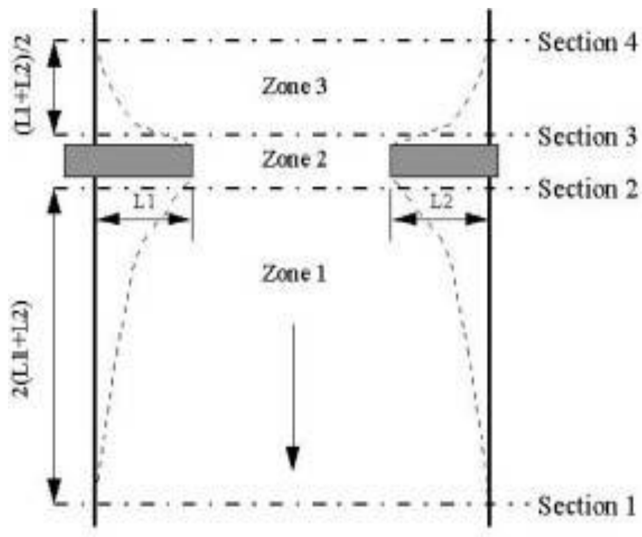


Figure 3.34 Flow Zones at Bridges
(TxDOT Hydraulic Design Manual)

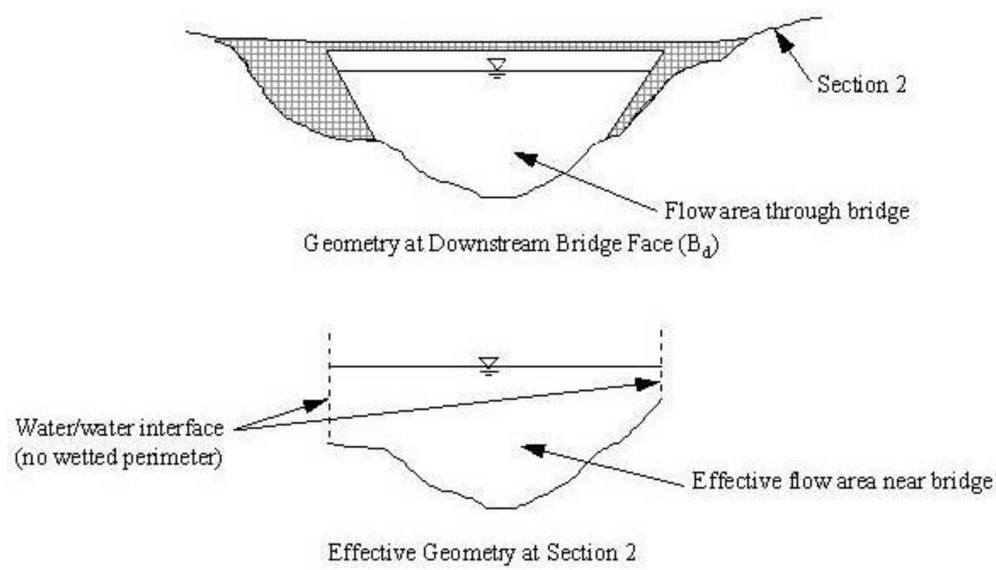


Figure 3.35 Effective Geometry for Bridge (Section 2 shown, Section 3 similar)
(TxDOT Hydraulic Design Manual)

Bridge Flow Class

The losses associated with flow through bridges depend on the hydraulic conditions of low or high flow.

Low flow describes hydraulic conditions in which the water surface between Zones 1, 2, and 3 is open to atmospheric pressure. That means the water surface does not impinge upon the superstructure. (This condition should exist for the design frequency of all new on-system bridges.) Low flow is divided into categories as described in the "Low Flow Classes" table below Type I is the most common in Texas, although severe constrictions compared to the flow conditions could result in Types IIA and IIB. Type III is likely to be limited to steep hills and mountainous regions.

Table 3.15 Low Flow Classes

Low Flow Class	Description
I	Subcritical flow through all Zones
IIA	Subcritical flow through Zones 1 and 3; flow through critical depth in Zone 2
IIB	Subcritical flow through Zone 3; flow through critical depth in Zone 2, hydraulic jump in Zone 1
III	Supercritical flow through all Zones

High flow refers to conditions in which the water surface impinges on the bridge superstructure:

- When the tailwater does not submerge the low chord of the bridge, the flow condition is comparable to a pressure flow sluice gate.
- When the tailwater submerges the low chord but does not exceed the elevation of critical depth over the road, the flow condition is comparable to orifice flow.
- If the tailwater overtops the roadway, neither sluice gate flow nor orifice flow is reasonable, and the flow is either weir flow or open flow.

3.4.3 Design Recommendations

The design of a bridge should take into account many different engineering and technical aspects at the bridge site and adjacent areas. The following design recommendations should be considered for all bridge designs as applicable. See the design criteria of the local jurisdiction for specific requirements.

Frequency Flood

Design discharges chosen by TxDOT for bridges vary with the functional classification and structure type. For major river crossings, a return period of 50 years is recommended. Flow small bridges, the recommended return period is 25 years. In all cases the check flood is for the flood mitigation storm return period.

Freeboard

Typical freeboard, the length between the computed design water surface and the low chord, is two feet. In urban settings, it may be prudent to use the flood mitigation storm fully-developed discharge to check the bridge design. The flood mitigation storm discharge, assuming blockage of outlet works, with 6" of freeboard. Some municipalities may specify different design storms and freeboard requirements.

Loss Coefficients

The contraction and expansion of water through the bridge opening creates hydraulic losses. These losses are accounted for through the use of loss coefficients. Table 3.16 gives recommended values for the Contraction (K_c) and Expansion (K_e) Coefficients.

Table 3.16 Recommended Loss Coefficients for Bridges

Transition Type	Contraction (K_c)	Expansion (K_e)
No losses computed	0.0	0.0
Gradual transition	0.1	0.3
Typical bridge	0.3	0.5
Severe transition	0.6	0.8

3.4.4 Design Procedures

The following is a general bridge hydraulic design procedure.

1. Determine the most efficient alignment of proposed roadway, attempting to minimize skew at the proposed stream crossing.
2. Determine design discharge from hydrologic studies or available data (Municipality, Federal Emergency Management Agency (FEMA), US Army Corp of Engineers (USACE), TxDOT, or similar sources).
3. If available, obtain effective FEMA hydraulic backwater model. It is assumed that if a bridge is required instead of a culvert, the drainage area would exceed one square mile and could already be included in a FEMA study. If an effective FEMA model or other model is not available, a basic hydrologic model and backwater analysis for the stream must be prepared. The HEC-RAS computer model is routinely used to compute backwater water surface profiles.
4. Using USACE or FEMA guidelines, compute or duplicate an existing conditions water surface profile for the design storm(s). Compute a profile for the fully-developed watershed to use as a baseline for design of a new bridge/roadway crossing.
5. Use the design discharge to compute an approximate opening that will be needed to pass the design storm (for preliminary sizing, use a normal-depth design procedure, or simply estimate a required trapezoidal opening.
6. Prepare a bridge crossing data set in the hydraulic model to reflect the preliminary design opening, which includes the required freeboard and any channelization upstream or downstream to transition the floodwaters through the proposed structure.
7. Compute the proposed bridge flood profile and design parameters (velocities, flow distribution, energy grade, etc.). Review for criteria on velocities and freeboard, and revise model as needed to accommodate design flows.
8. Review the velocities and determine erosion control requirements downstream, through the structure, and upstream.
9. Finalize the design size and erosion control features, based on comparing the proposed model with the existing conditions profiles, impacts on other properties, FEMA guidelines, and local criteria.
10. Exceptions/Other Issues
    A. Conditional Letter of Map Amendment (CLOMR) may be needed for new crossings of streams studied by FEMA.
    B. If applicable, coordinate with USACE Regulatory Permit requirements.
    C. Evaluate the project with respect to iSWM policy regarding downstream impacts.
    D. Design should be for fully developed watershed conditions. If the available discharges are from FEMA existing conditions hydrology, the following options are available: (1)

obtain new hydrology, (2) extrapolate fully-developed from existing data, or (3) variance from the local jurisdiction on design discharges

E. Freeboard criteria may require an unusually expensive bridge or impracticable roadway elevation. A reasonable variance in criteria from the local jurisdiction may be available.

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4.0 Energy Dissipation

4.1 Overview

4.1.1 Introduction

The outlets of pipes and lined channels are points of critical erosion potential. Stormwater transported through man-made conveyance systems at design capacity 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.

4.1.2 General Criteria

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.

4.1.3 Recommended Energy Dissipators

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

- Riprap apron
- Riprap outlet basins
- Baffled outlets
- Grade Control Structures

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

4.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.1 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
S_v	Saturated shear strength	lbs/in^2
t	Time of scour	min.
t_c	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^2
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

4.3 Design Guidelines

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

a. Riprap aprons may be used when the outlet Froude number (Fr) is less than or equal to 2.5. In general, riprap aprons prove economical for transitions from culverts to overland sheet flow at terminal outlets, but may also be used for transitions from culvert sections to stable channel sections. Stability of the surface at the termination of the apron should be considered.

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.

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.

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

Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur. Figure 4.1 provides the riprap size recommended for use downstream of energy dissipators.

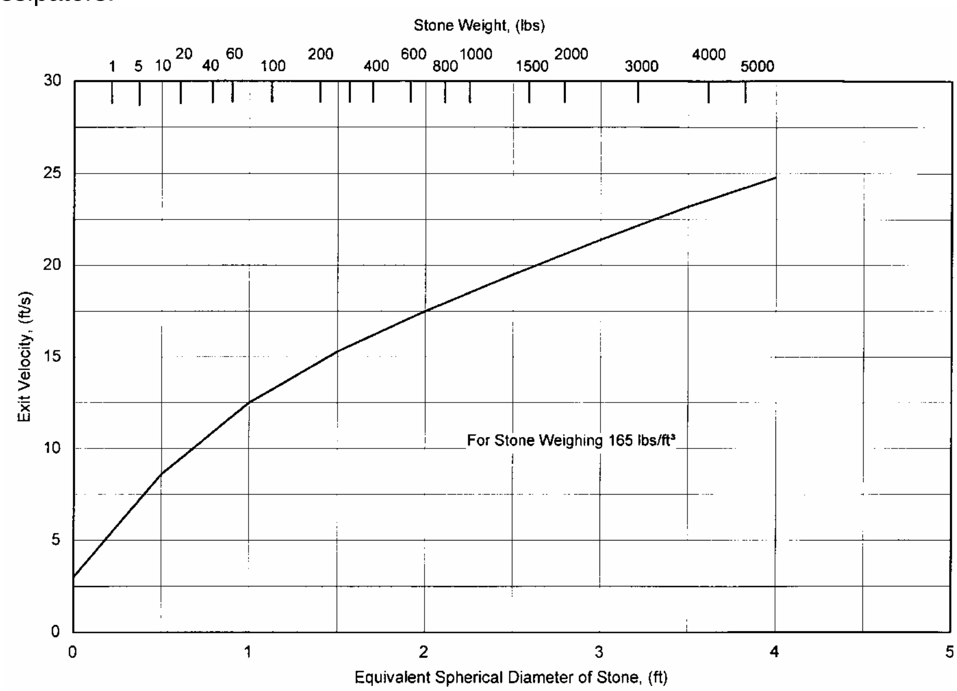


Figure 4.1 Riprap Size for Use Downstream of Energy Dissipator

(Source: Searcy, 1967)

4.4 Riprap Aprons

4.4.1 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 Fr is less than or equal to 2.5.

4.4.2 Design Procedure

The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, d_{50}. If tailwater conditions are unknown, or if both minimum and maximum conditions may occur, the apron 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:

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 4.2 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 4.3 should be used.

Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figures 4.2 and 4.3. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 4.4.

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.

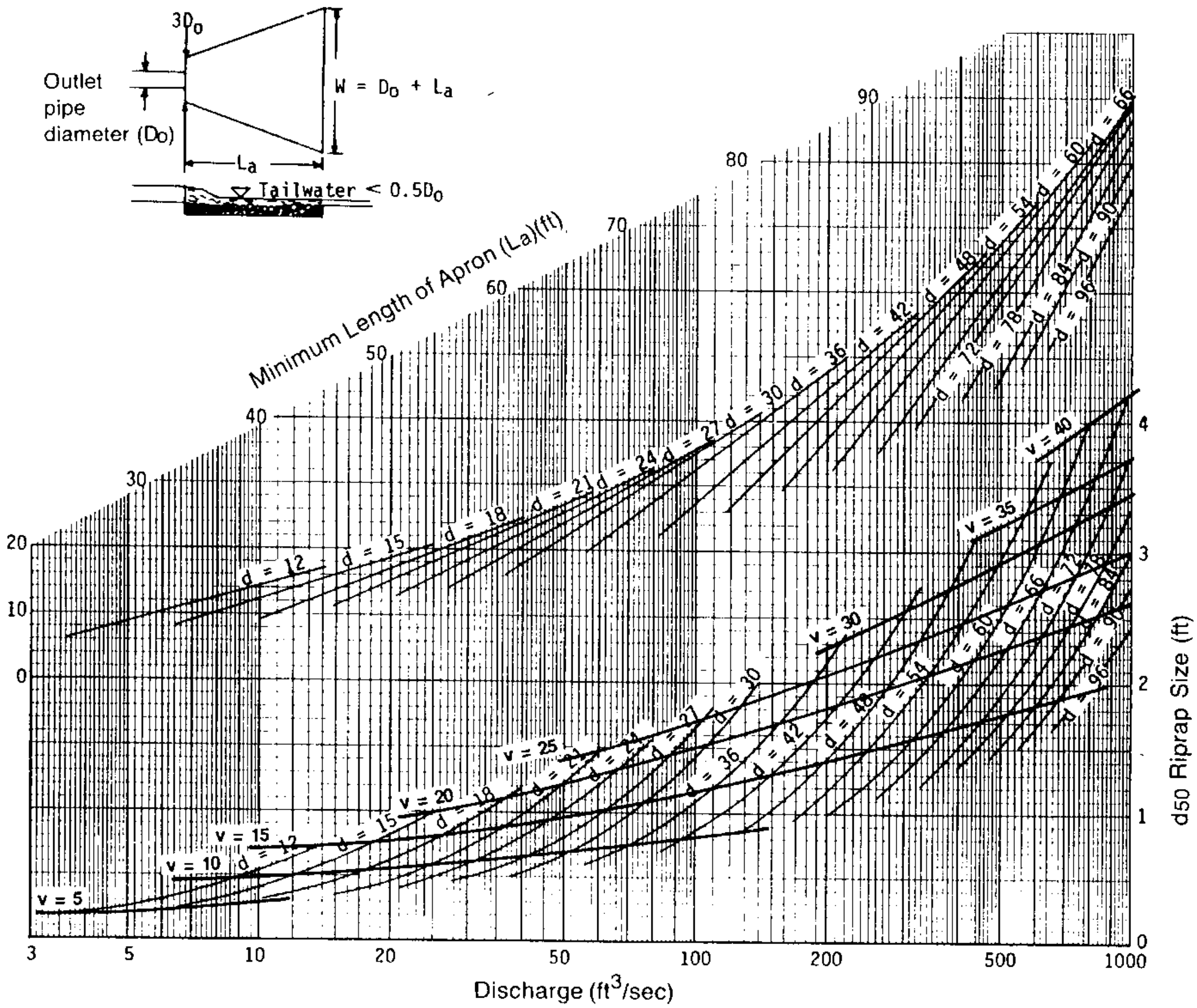
b. For pipes flowing partially full:

   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, and then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, 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, and then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth, d. Find the minimum apron length, L_a, using the scale on the left.

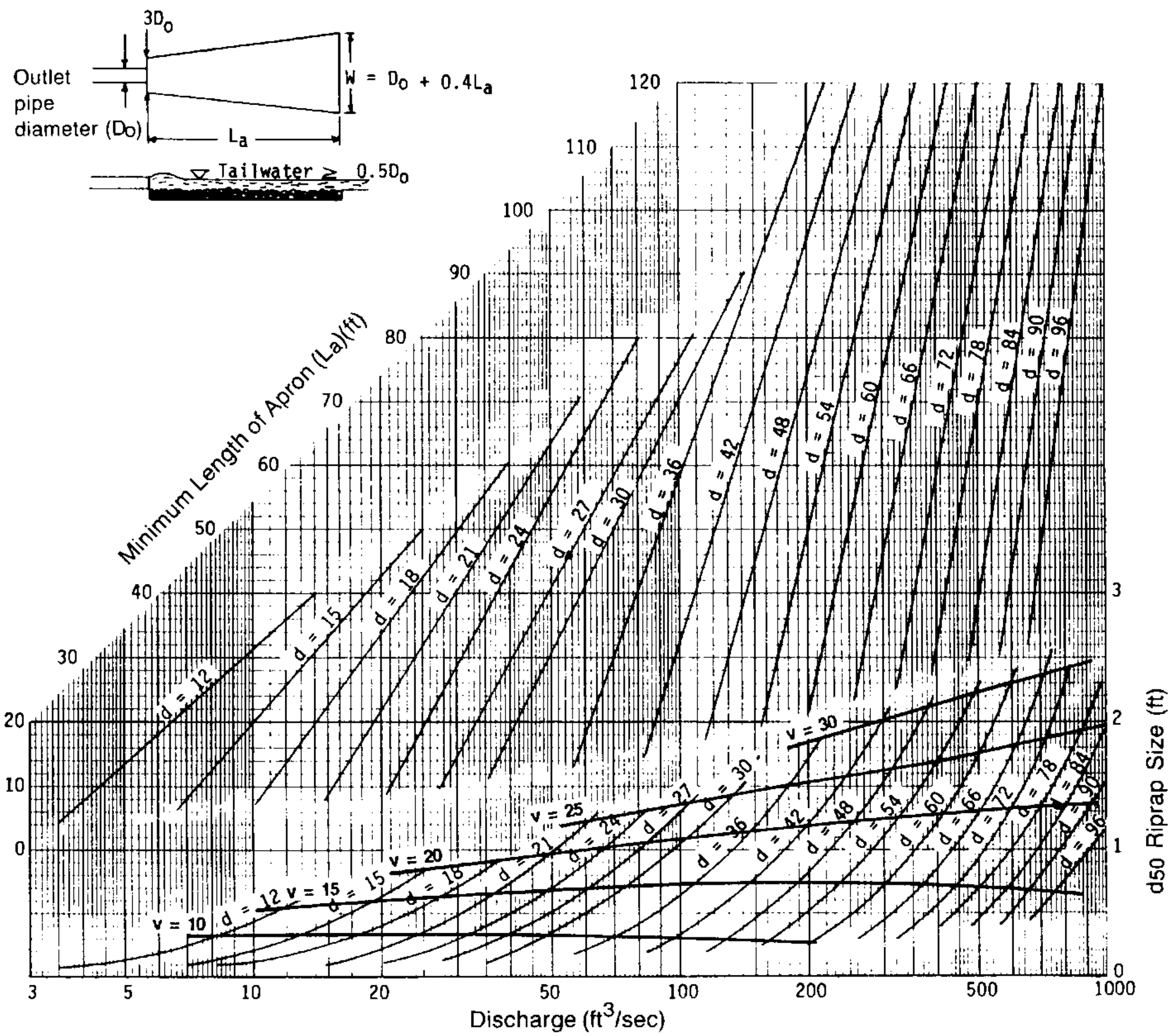
If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in Figure 4.4. This will provide protection under either of the tailwater conditions.



Curves may not be extrapolated.

Figure 4.2 Design of Riprap Apron under Minimum Tailwater Conditions

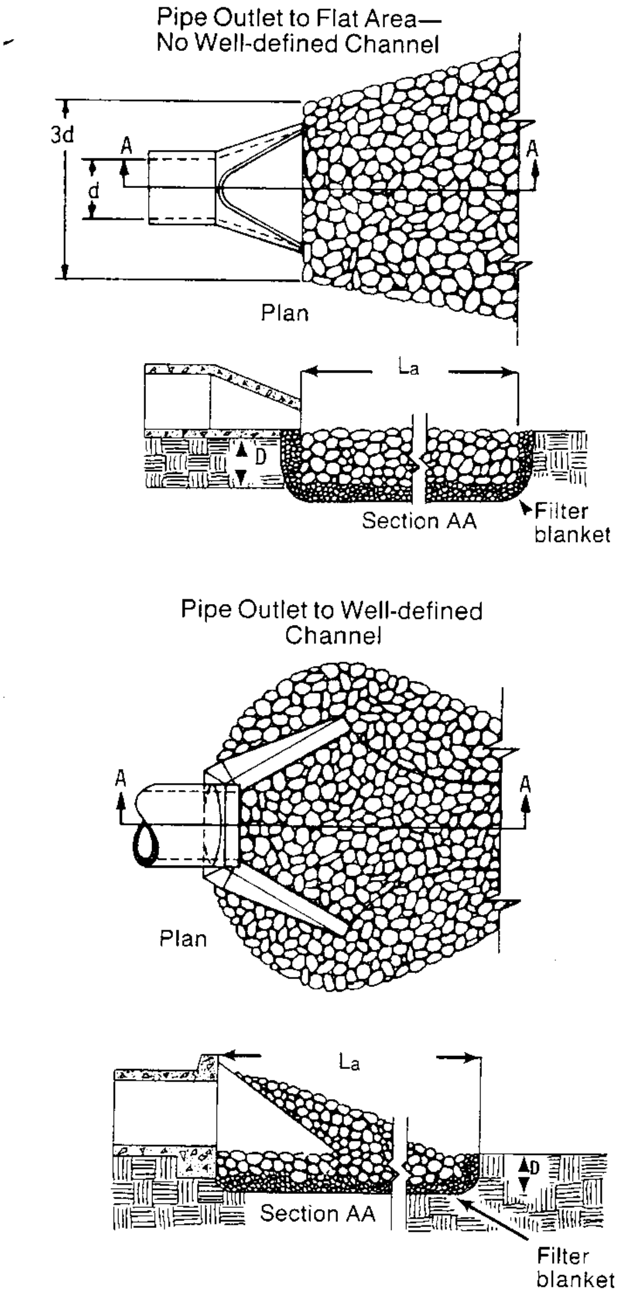
(Source: USDA, SCS, 1975)



Curves may not be extrapolated.

Figure 4.3 Design of Riprap Apron under Maximum Tailwater Conditions

(Source: USDA, SCS, 1975)



Notes

1. L_a is the length of the riprap apron.
2. D = 1.5 times the maximum stone diameter but not less than 6".
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.

Figure 4.4 Riprap Apron

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

4.4.3 Design Considerations

The following items should be considered during riprap apron design:

The maximum stone diameter should be 1.5 times the median riprap diameter.
$d_{max} = 1.5 \times d_{50}$, d_{50} = the median stone size in a well-graded riprap apron.

The riprap thickness should be 1.5 times the maximum stone diameter or 6 inches, whichever is greater.
Apron thickness = $1.5 \times d_{max}$
(Apron thickness may be reduced to $1.5 \times d_{50}$ 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 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.

4.4.4 Example Designs

<u>Example 1</u> 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.

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

Since TW = 2 ft is less than 2.75 ft, use Figure 4.2 for minimum tailwater conditions.

By Figure 4.2, the apron length, L_a, and median stone size, d_{50}, are 38 ft and 1.2 ft, respectively.

The downstream apron width equals the apron length plus the pipe diameter:

$W = d + L_a = 5.5 + 38 = 43.5$ ft

Maximum riprap diameter is 1.5 times the median stone size:

$1.5 (d_{50}) = 1.5 (1.2) = 1.8$ ft

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

<u>Example 2</u> 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.

Minimum tailwater conditions = 0.5 d_o, d_o = 0.5 (5.0) = 2.5 ft.

Since TW = 5.0 ft is greater than 2.5 ft, use Figure 4.3 for maximum tailwater conditions.

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

On Figure 4.3, at the intersection of the curve, d_o = 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.

Apron width downstream = $d_w + 0.4\ L_a = 10 + 0.4\ (40) = 26$ ft.

Maximum stone diameter = $1.5\ d_{50} = 1.5\ (0.4) = 0.6$ ft.

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

4.5 Riprap Basins

4.5.1 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 preshaped scourhole lined with riprap that functions as an energy dissipator by forming a hydraulic jump.

4.5.2 Basin Features

General details of the basin recommended in this section are shown in Figure 4.5. Principal features of the basin are:

The basin is preshaped and lined with riprap of median size (d_{50}).

The floor of the riprap basin is constructed at an elevation of h_s below the culvert invert. The dimension h_s is the approximate depth of scour that would occur in a thick pad of riprap of size d_{50} if subjected to design discharge. The ratio of h_s to d_{50} of the material 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.

4.5.3 Design Procedure

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

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

For subcritical flow conditions (culvert set on mild or horizontal slope) use Figure 4.6 or Figure 4.7 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.

For streambank protection, compute the Froude number for brink conditions with $y_e = (A/2)^{1.5}$. Select d_{50}/y_e appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25 < d_{50}/y_e < 0.45$). Obtain h_s/y_e from Figure 4.8, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if h_s/d_{50} falls out of this range.

Size basin as shown in Figure 4.5.

Where allowable dissipator exit velocity is specified:

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

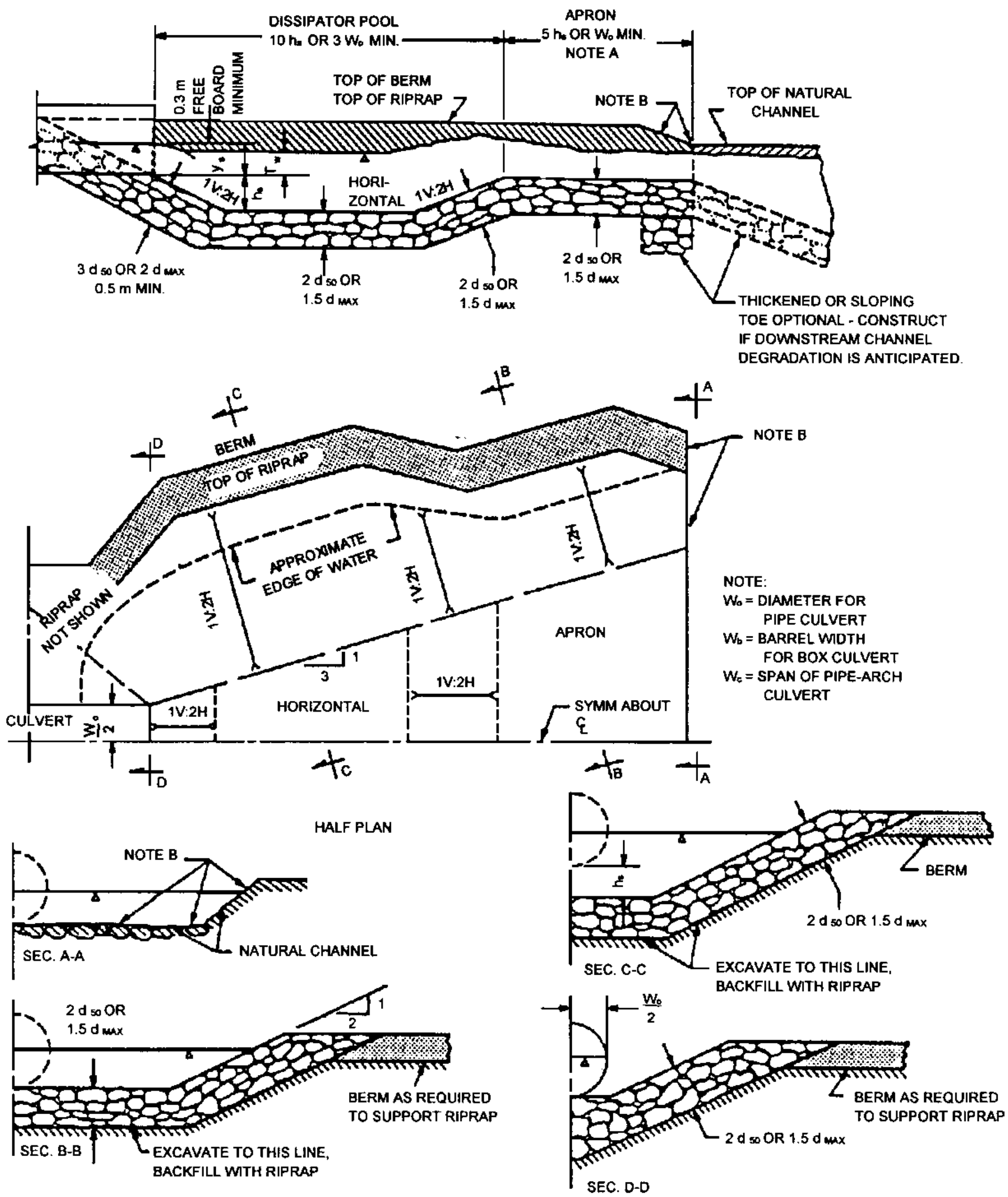
b. Extend the length of the energy basin (if necessary) so the width of the energy basin at section A-A, Figure 4.5, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so 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.

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 4.9.
- Shape downstream channel and size riprap using Figure 4.1 and the stream velocities obtained above.

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



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

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

Figure 4.5 Details of Riprap Outlet Basin

(Source: HEC-14, 1983)

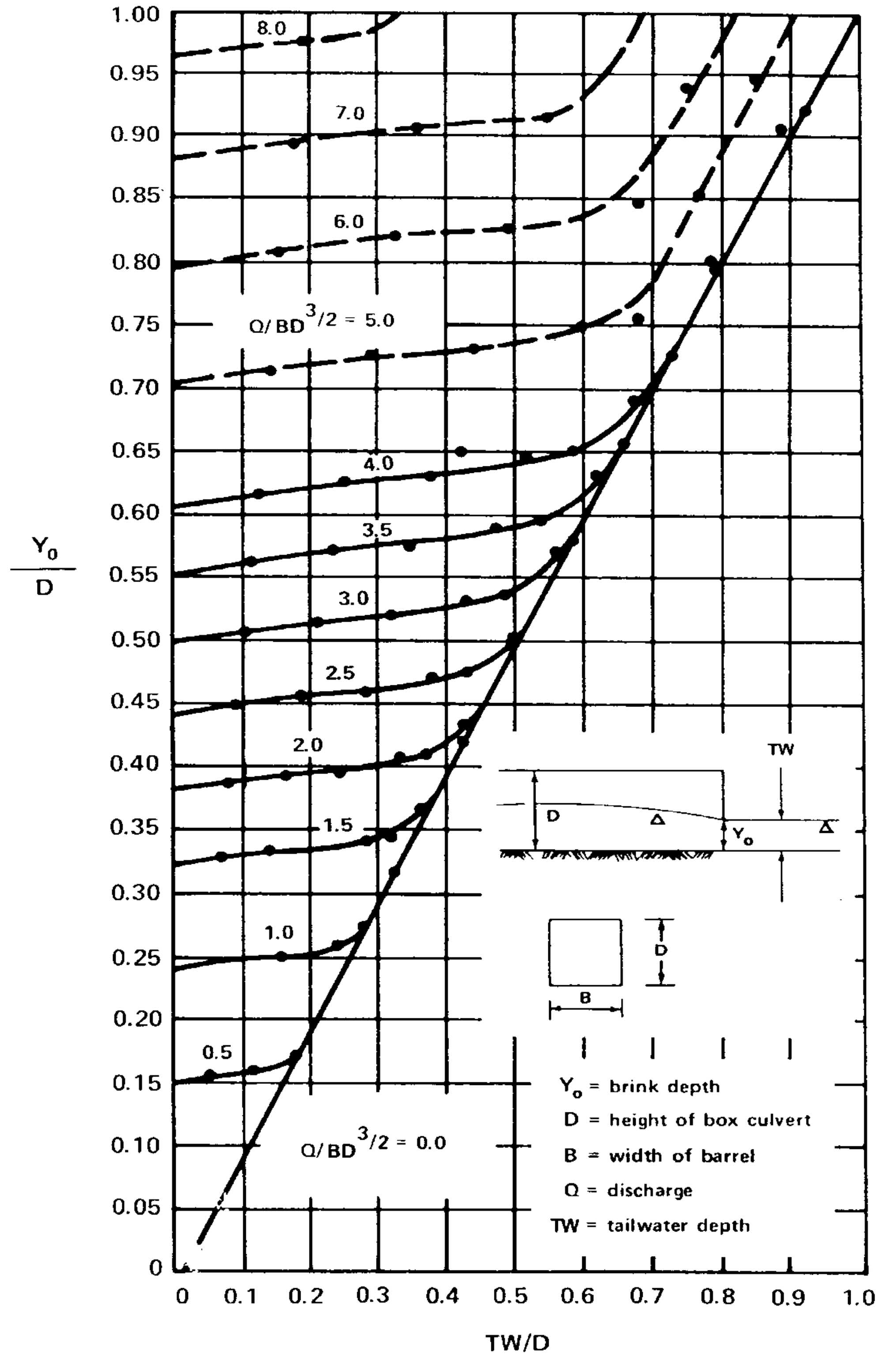


Figure 4.6 Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 1983)

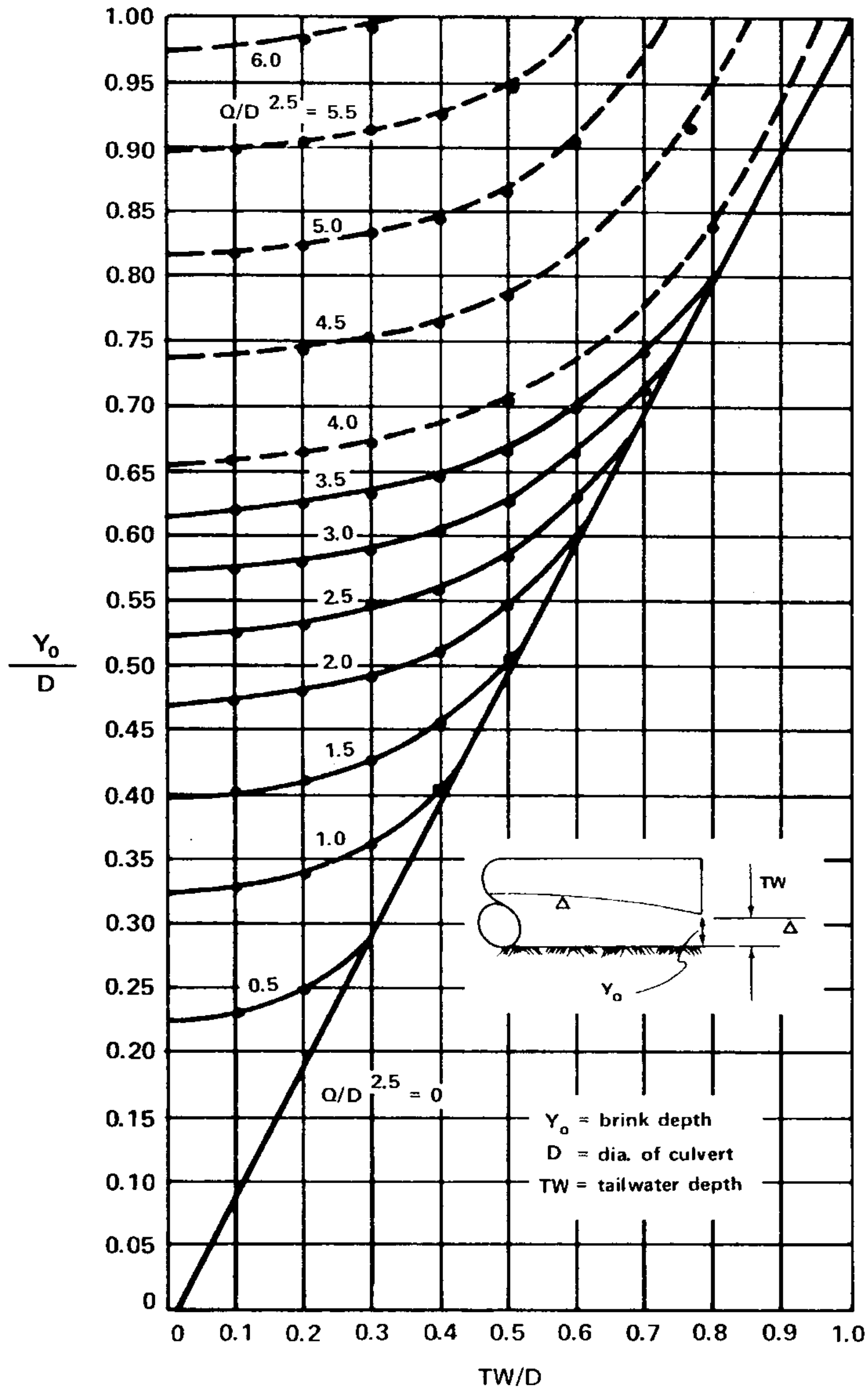


Figure 4.7 Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 1983)

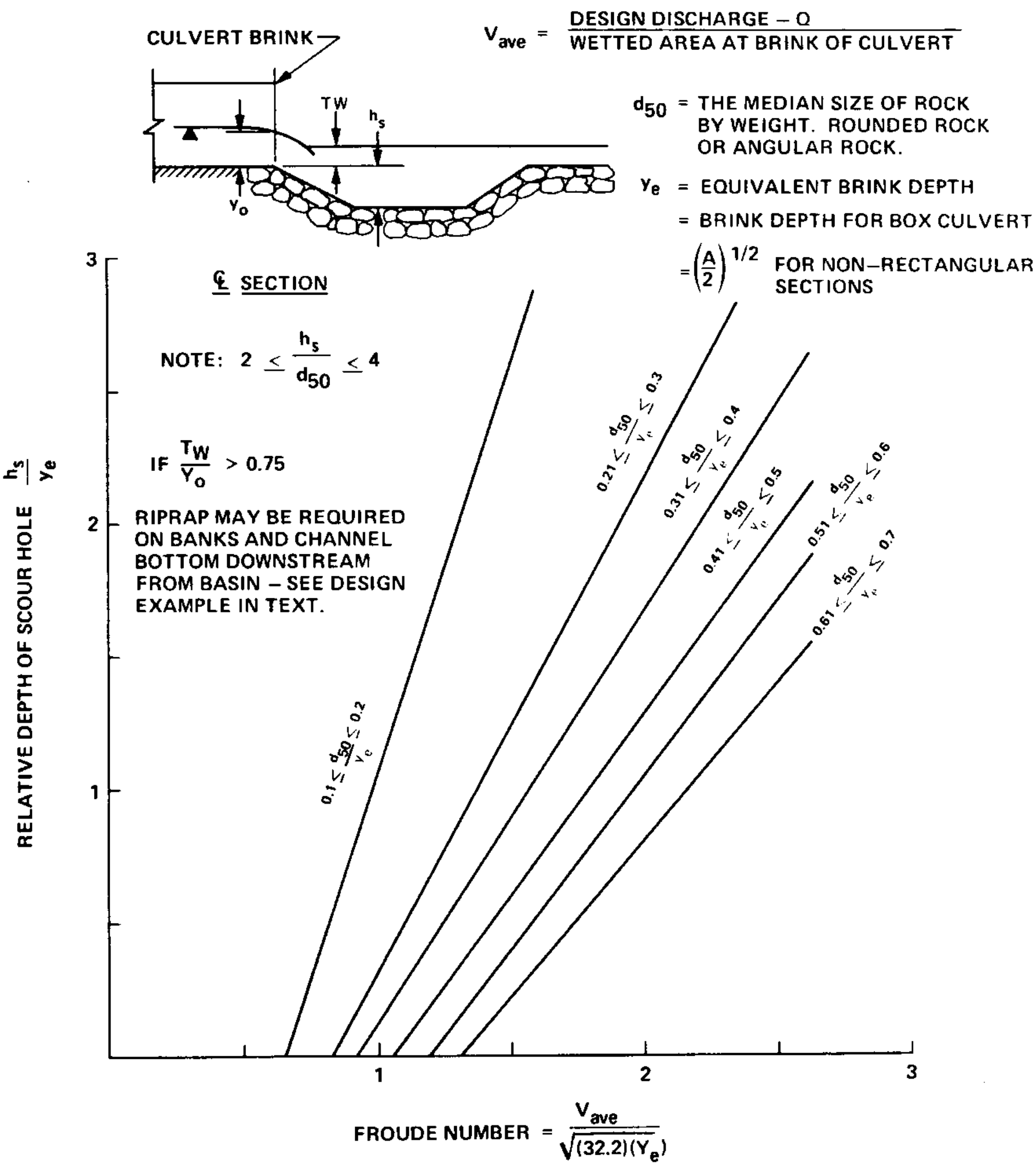


Figure 4.8 Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable

(Source: USDOT, FHWA, HEC-14, 1983)

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

Stability of the surface at the outlet of a basin should be considered using the methods for open channel flow as outlined in *Section 3.2*.

4.5.5 Example Designs

Following are some example problems to illustrate the design procedures outlined.

Example 1

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

Find: Riprap basin dimensions for these conditions

Solution: Definition of terms in Steps 1 through 5 can be found in Figures 4.5 and 4.8.

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

$V_o = Q/A = 800/(4 \times 8) = 25$ ft/s

Froude Number = $Fr = V/(g \times y_e)^{0.5}$ $(g = 32.2 \text{ ft/s}^2)$

$Fr = 25/(32.2 \times 4)^{0.5} = 2.20 < 2.5$ O.K.

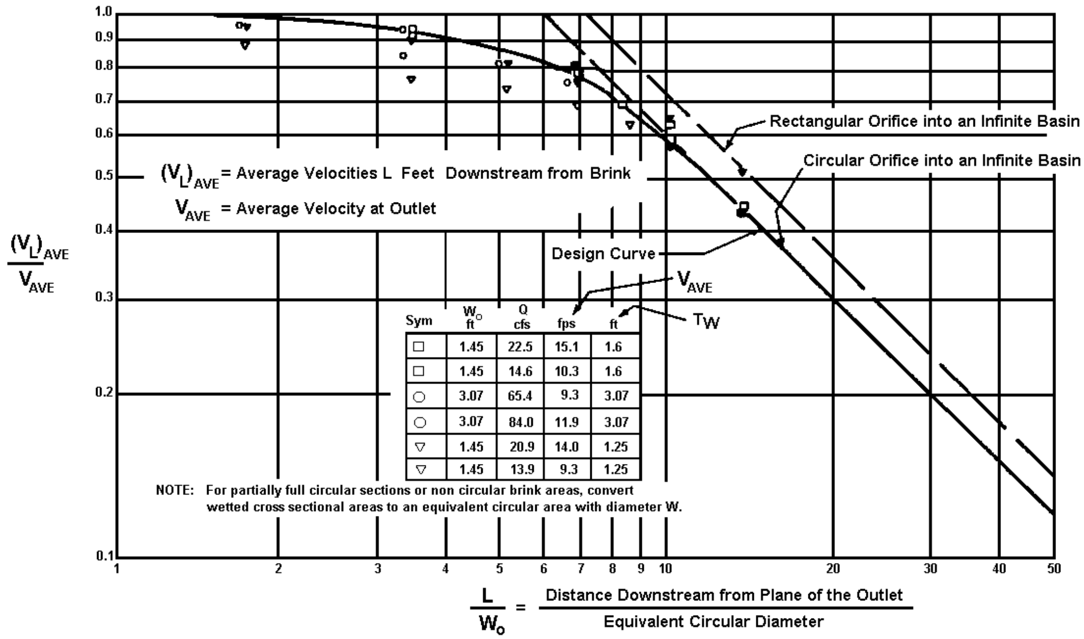


Figure 4.9 Distribution of Centerline Velocity for Flow from Submerged Outlets to Be Used for Predicting Channel Velocities Downstream from Culvert Outlet Where High Tailwater Prevails

(Source: USDOT, FHWA, HEC-14, 1983)

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

Try $d_{50}/y_e = 0.45$, $d_{50} = 0.45 \times 4 = 1.80$ ft
From Figure 4.8, $h_s/y_e = 1.6$, $h_s = 4 \times 1.6 = 6.4$ ft
$h_s/d_{50} = 6.4/1.8 = 3.6$ ft, $2 < h_s/d_{50} < 4$ OK

$L_s = 10 \times h_s = 10 \times 6.4 = 64$ ft (L_s = length of energy dissipator pool)
L_s min $= 3 \times W_o = 3 \times 8 = 24$ ft; therefore, use $L_s = 64$ ft

$L_B = 15 \times h_s = 15 \times 6.4 = 96$ ft (L_B = overall length of riprap basin)
L_B min $= 4 \times W_o = 4 \times 8 = 32$ ft; therefore, use $L_B = 96$ ft

Thickness of riprap: On the approach $= 3 \times d_{50} = 3 \times 1.8 = 5.4$ ft

Remainder $= 2 \times d_{50} = 2 \times 1.8 = 3.6$ ft
Other basin dimensions designed according to details shown in Figure 4.5.

Example 2

Given: Same design data as Example 1 except:

Tailwater depth TW = 4.2 ft
Downstream channel can tolerate only 7 ft/s discharge

Find: Riprap basin dimensions for these conditions

Solutions: Note -- High tailwater depth, $TW/y_o = 4.2/4 = 1.05 > 0.75$

From Example 1: d_{50} = 1.8 ft, h_s = 6.4 ft, L_s = 64 ft, L_B = 96 ft.

Design riprap for downstream channel. Use Figure 4.9 for estimating average velocity along the channel. Compute equivalent circular diameter D_e for brink area from:

$A = 3.14D_e^2/4 = y_o \times W_o = 4 \times 8 = 32$ ft^2

$D_e = ((32 \times 4)/3.14)^{0.5} = 6.4$ ft

V_o = 25 ft/s (From Example 1)

Set up the following table:

L/D_e	**L (ft)**	Rock Size **V_L/V_o**	**v_1 (ft/s)**	**d_{50} (ft)**
(Assume)	(Compute)	(Fig. 9)	(Fig. 1)	$D_e = W_o$
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.

Example 3

Given: 6-ft diameter CMC
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:

Determine y_o and V_o
$Q/D^{2.5} = 135/6^{2.5} = 1.53$
$TW/D = 2.0/6 = 0.33$
From Figure 4.7, $y_o/D = 0.45$
$y_o = .45 \times 6 = 2.7$ 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 Table 3.7)
For $y_o/D = d/D = 0.45$
$A/D^2 = 0.3428$; therefore, $A = 0.3428 \times 6^2 = 12.3$ ft^2
$V_o = Q/A = 135/12.3 = 11.0$ ft/s
For Froude number calculations at brink conditions,
$y_e = (A/2)^{1/2} = (12.3/2)^{1/2} = 2.48$ ft

Froude number = Fr = $V_o/(32.2 \times y_e)^{1/2} = 11/(32.2 \times 2.48)^{1/2} = 1.23 < 2.5$ OK

For most satisfactory results, $0.25 < d_{50}/y_e < 0.45$
Try $d_{50}/y_e = 0.25$
$d_{50} = 0.25 \times 2.48 = 0.62$ ft
From Figure 4.8, $h_s/y_e = 0.7$; therefore, $h_s = 0.7 \times 2.48 = 1.74$ ft

Uniform Flow in Circular Sections Flowing Partly Full (From *Section 3.2.4*)
Check: $h_s/d_{50} = 1.74/0.62 = 2.8$, $2 < h_s/d_{50} < 4$ OK

$L_s = 10 \times h_s = 10 \times 1.74 = 17.4$ ft or $L_s = 3 \times W_o = 3 \times 6 = 18$ ft;
therefore, use $L_s = 17.4$ ft

$L_B = 15 \times h_s = 15 \times 1.74 = 26.1$ ft or $L_B = 4 \times W_o = 4 \times 6 = 24$ ft;
therefore, use $L_B = 26.1$ ft

$d_{50} = 0.62$ ft or use $d_{50} = 8$ in

Other basin dimensions should be designed in accordance with details shown on Figure 4.5. Figure 4.10 is provided as a convenient form to organize and present the results of riprap basin designs.

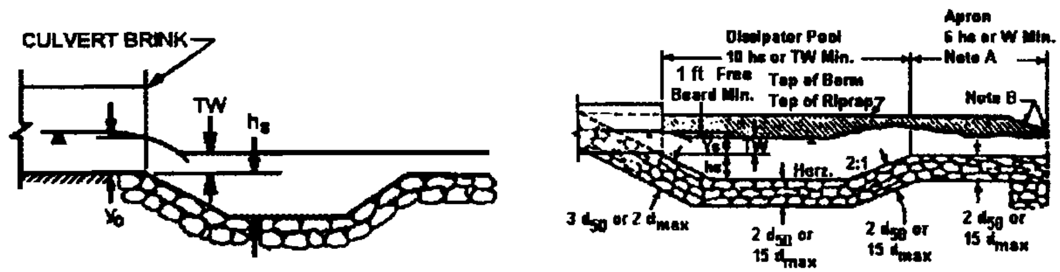
Note: When using the design procedure outlined in this section, it is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the 3 x d_{50} thickness of riprap on the approach and the 2 x d_{50} thickness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

RIPRAP BASIN

Project No. ______________________________

Designer ______________________________ Date __________

Reviewer ______________________________ Date __________



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

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

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

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

Figure 4.10 Riprap Basin Design Form

(Source: USDOT, FHWA, HEC-14, 1983)

4.6 Baffled Outlets

4.6.1 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 4.11. 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.

4.6.2 Design Procedure

The following design procedure is based on physical modeling studies summarized from the U.S. Department of Interior (1978). The dimensions of a baffled outlet as shown in Figure 4.11 should be calculated as follows:

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 = 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

g = Acceleration of gravity (32.2 ft/s)

Calculate the minimum basin width, W, in ft, using the following equation.

$$W/d = 2.88Fr^{0.566} \text{ or } W = 2.88dFr^{0.566} \qquad (4.1)$$

Where:

W = minimum basin width (ft)

d = depth of incoming flow (ft)

Fr = $v/(gd)^{0.5}$ = 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.

Calculate the other basin dimensions as shown in Figure 4.11, as a function of W. Construction drawings for selected widths are available from the U.S. Department of the Interior (1978).

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 W (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.

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.

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

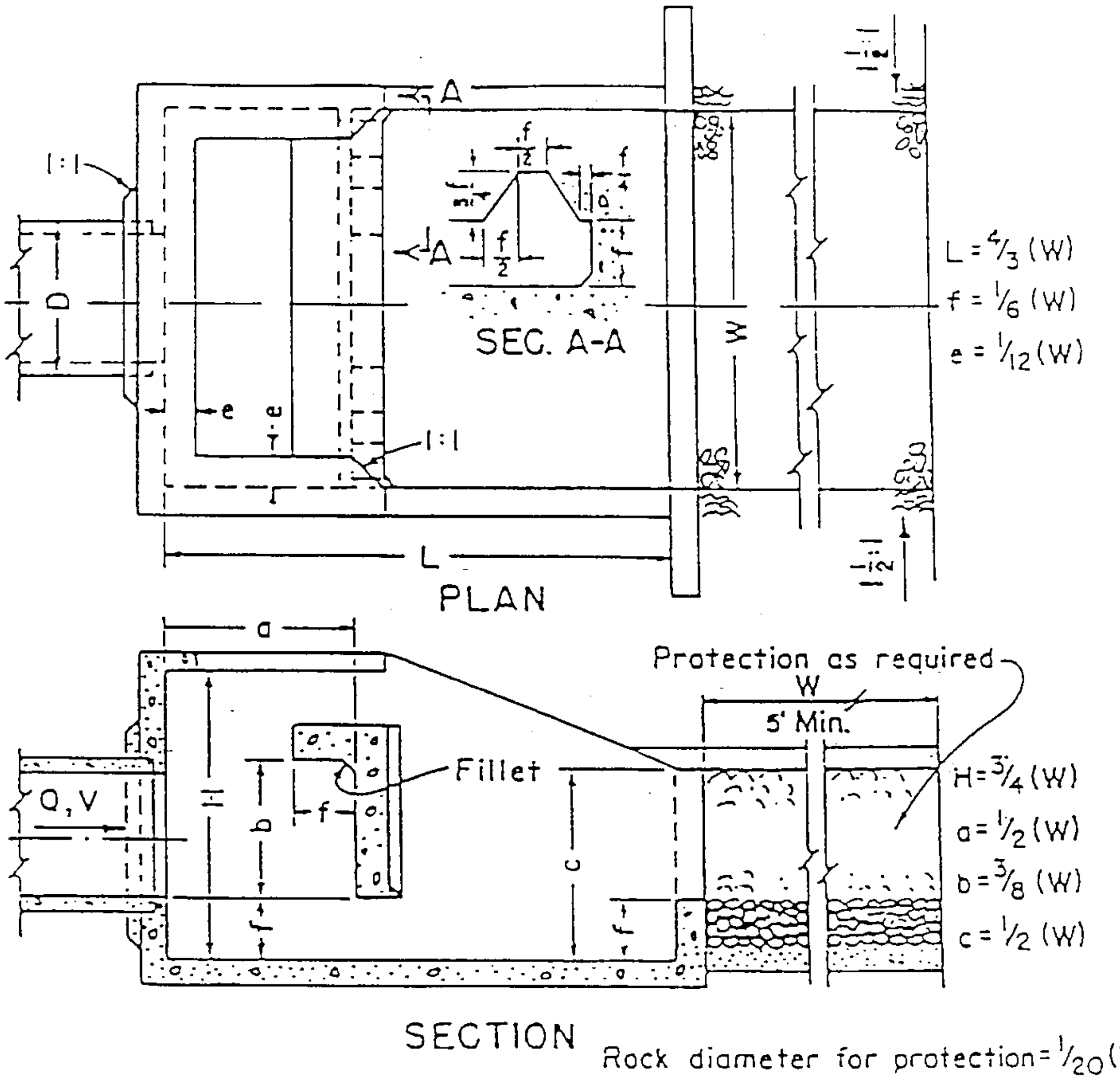


Figure 4.11 Schematic of Baffled Outlet

(Source: U.S. Dept. of the Interior, 1978)

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

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.

4.6.3 Example Design

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.

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

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

3. Compute the flow depth using the area from Step 2.
   $d = (A)^{0.5} = (4.8\ ft^2)^{0.5} = 2.12\ ft$

4. Compute the Froude number using the results from Steps 1 and 3.
   $Fr = v/(gd)^{0.5} = 31.1\ ft/sec/[(32.2\ ft/sec^2)(2.12\ ft)]^{0.5} = 3.8$

5. Determine the basin width using Equation 4.1 with the Froude number from Step 4.
   $W = 2.88\ dFr^{0.566} = 2.88\ (2.12)\ (3.8)^{0.566} = 13.0\ ft$ (minimum)
   Use 13 ft as the design width.

6. Compute the remaining basin dimensions (as shown in Figure 4.11):
   L = 4/3 (W) = 17.3 ft, use L = 17 ft, 4 in
   f = 1/6 (W) = 2.17 ft, use f = 2 ft, 2 in
   e = 1/12 (W) = 1.08 ft, use e = 1 ft, 1 in
   H = 3/4 (W) = 9.75 ft, use H = 9 ft, 9 in
   a = 1/2 (W) = 6.5 ft, use a = 6 ft, 6 in
   b = 3/8 (W) = 4.88 ft, use b = 4 ft, 11 in
   c = 1/2 (W) = 6.5 ft, use c = 6 ft, 6 in

   Baffle opening dimensions would be calculated as shown in Figure 4.11.

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 2 ft, 8 in below ground surface.

8. The riprap transition from the baffled outlet to the natural channel should be 13 ft long by 13 ft wide by 2 ft, 2 in deep (W x W x f). Median rock diameter should be of diameter W/20, or about 8 in.

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

4.7 Grade Control Structures

When channels are relocated through non-stable soils and stream gradients are increased, the stream bottom may degrade or dig itself deeper. This can cause bank instability, increased upstream scouring, and sloughing of natural slopes. The U.S. Soil Conservation Services (SCS) requires that streambed stability be maintained in any of its stream projects. This can be accomplished by grade stabilization structures; in essence a series of low-head weirs.

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

Specific construction requirements and techniques can be obtained from the SCS or other agencies upon request. The intent of this general discussion of grade stabilization structures is to promote consideration of such measures early in the planning process.

Source: US Army Corp of Engineers, Nashville District, *"Mitigating the Impacts of Stream Alterations"*, unkn.

Hydrology:

1.0 Hydrological Analysis
2.0 Downstream Assessment
3.0 Streambank Protection
4.0 Water Balance
5.0 Rainfall Tables
6.0 Hydrologic Soils Data

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1.0 Hydrological Analysis

1.1. Estimating Runoff

1.1.1. Introduction to Hydrologic Methods

Hydrology deals with estimating flow peaks, volumes, and time distributions of stormwater runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems and structural stormwater controls. In the hydrologic analysis of a development/redevelopment site, there are a number of variable factors that affect the nature of stormwater runoff from the site. Some of the factors that need to be considered include:

- Rainfall amount and storm distribution
- Drainage area size, shape, and orientation
- Ground cover and soil type
- Slopes of terrain and stream channel(s)
- Antecedent moisture condition
- Rainfall abstraction rates (Initial and continued)
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Characteristics of the local drainage system

There are a number of empirical hydrologic methods available to estimate runoff characteristics for a site or drainage subbasin; however, the following methods have been selected to support hydrologic site analysis for the design methods and procedures included in this Manual:

- Rational Method
- SCS Unit Hydrograph Method
- Snyder's Unit Hydrograph Method
- USGS & TXDOT Regression Equations
- iSWM Water Quality Protection Volume Calculation
- Water Balance Calculations

These methods were selected based upon a verification of their accuracy in duplicating local hydrologic estimates for a range of design storms throughout the state and the availability of equations, nomographs, and computer programs to support the methods.

Table 1.1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 1.2 provides some limitations on the use of several methods.

In general:

The Rational Method is recommended for small highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters.

The USGS (U.S. Geological Survey) and TXDOT (Texas Department of Transportation) regression equations are recommended for drainage areas with characteristics within the ranges given for the equations. These equations should be used with caution when there are significant storage areas within the drainage basin or where other drainage characteristics indicate general regression equations might not be appropriate.

Table 1.1 Applications of the Recommended Hydrologic Methods

Method	Technical Manual Section	Rational Method	SCS Method	Modified Rational	Snyder's Unit Hydrograph	USGS / TXDOT Equations	iSWM Water Quality Volume Calculation
Water Quality Protection Volume (WQ_v)	Section 1.2 of Water Quality						✓
Streambank Protection Volume (SP_v)	Section 3.0 of Hydrology		✓		✓		
Flood Mitigation Discharge (Q_f)	Section 1.3 of Criteria Manual		✓		✓	✓	
Storage Facilities	Section 2.0 of Hydraulics		✓	✓	✓		✓
Outlet Structures	Section 2.2 of Hydraulics		✓		✓		
Gutter Flow and Inlets	Section 1.2 of Hydraulics	✓					
Storm Drain Pipes	Section 1.1 of Hydraulics	✓	✓		✓		
Culverts	Section 3.3 of Hydraulics	✓	✓		✓	✓	
Bridges	Section 3.4 of Hydraulics		✓		✓		
Small Ditches	Section 3.2 of Hydraulics	✓	✓		✓		
Open Channels	Section 3.2 of Hydraulics		✓		✓	✓	
Energy Dissipation	Section 4.0 of Hydraulics		✓		✓		

Table 1.2 Constraints on Using Recommended Hydrologic Methods		
Method	**Size Limitations[1]**	**Comments**
Rational	0 – 100 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems.
Modified Rational[2]	0 – 200 acres	Method can be used for estimating runoff volumes for storage design.
Unit Hydrograph (SCS)[3]	Any Size	Method can be used for estimating peak flows and hydrographs for all design applications.
Unit Hydrograph (Snyder's)[4]	1 acre and larger	Method can be used for estimating peak flows and hydrographs for all design applications.
TXDOT Regression Equations	10 to 100 mi^2	Method can be used for estimating peak flows for rural design applications.
USGS Regression Equations	3 – 40 mi^2	Method can be used for estimating peak flows for urban design applications.
iSWM Water Quality Protection Volume Calculation	Limits set for each Structural Control	Method can be used for calculating the Water Quality Protection Volume (WQ_v).

[1] Size limitation refers to the drainage basin for the stormwater management facility (e.g., culvert, inlet).
[2] Where the Modified Rational Method is used for conceptualizing, the engineer is cautioned that the method could underestimate the storage volume.
[3] This refers to SCS routing methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology.
[4] This refers to the Snyder's methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology.

If other hydrologic methods are to be considered and used by a local review authority or design engineer, the method should first be calibrated to local conditions and tested for accuracy and reliability. If local stream gage data are available, these data can be used to develop peak discharges and hydrographs. The user is referred to standard hydrology textbooks for statistical procedures that can be used to estimate design flood events from stream gage data.

Note: It must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex and too little data are available on the factors influencing the rainfall-runoff relationship to expect exact solutions.

1.1.2. Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 1.3 will be used. These symbols were selected because of their wide use in technical publications. 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 1.3 Symbols and Definitions

Symbol	Definition	Units
A	Drainage area or area	acres or square feet
B_f	Baseflow	acre-feet
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
D	Time interval	hours
E	Evaporation	ft
E_t	Evapotranspiration	ft
F	Pond and swamp adjustment factor	-
G_h	Hydraulic gradient	-
I or i	Rainfall intensity	in/hr
I	Percent of impervious cover	%
I	Infiltration	acre-feet
I_a	Initial abstraction from total rainfall	in
k_h	Infiltration rate	ft/day
L	Flow length	ft
n	Manning roughness coefficient	-
NRCS	Natural Resources Conservation Service (formerly SCS)	-
O_f	Overflow	acre-feet
P	Accumulated rainfall	in
P_2	2-year, 24-hour rainfall	in
P_w	Wetted perimeter	ft
PF	Peaking factor	-
Q	Rate of runoff	cfs (or inches)
Q_i	Peak inflow discharge	cfs
Q_o	Peak outflow discharge	cfs
Q_p	Peak rate of discharge	cfs
Q_{wq}	Water Quality peak rate of discharge	cfs
q	Storm runoff during a time interval	in
q_u	Unit peak discharge	cfs (or cfs/mi²/inch)
R	Hydraulic radius	ft
R_o	Runoff	acre-feet
R_v	Runoff Coefficient	-
S	Ground slope	ft/ft or %
S	Potential maximum retention	in
S	Slope of hydraulic grade line	ft/ft
SCS	Soil Conservation Service (Now NRCS)	-
SP_v	Streambank Protection Volume	acre-feet
T	Channel top width	ft
T_L	Lag time	hours
T_p	Time to peak	hours
T_t	Travel time	hours
t	Time	min

Symbol	Definition	Units
t_c	Time of concentration	min
TIA	Total impervious area	%
V	Velocity	ft/s
V	Pond volume	acre-feet
V_d	Developed runoff volume	in
V_f	Flood control volume	acre-feet
V_r	Runoff volume	acre-feet
V_s	Storage volume	acre-feet
WQ_v	Water quality protection volume	acre-feet

1.1.3. Rainfall Estimation

The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

Duration (hours) – Length of time over which rainfall (storm event) occurs
Depth (inches) – Total amount of rainfall occurring during the storm duration
Intensity (inches per hour) – Depth divided by the duration

The Frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of *exceedance probability* or *return period*.

Exceedance Probability – Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically in years
Return Period – Average length of time between events, which have the same duration and volume

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedance probability of 0.01 and a return period of 100 years.

Rainfall intensities for the 16 counties which participate in the NCTCOG area (see Figure 1.1) are provided in *Section 5.0* and should be used for all hydrologic analysis within the given county. The values in these tables were derived in the following way:

- New IDF values for the 1-year through 500-year storm return periods were determined for the NCTCOG area on a county by county basis.
- All values were plotted and smoothed to ensure continuity. The values were smoothed by fitting an equation of the form:

  $$\mathbf{i = b/(t + d)^e} \qquad \textbf{(1.1)}$$

  where:
  i = rainfall intensity (inches per hour)
  t = rainfall duration (minutes)
  b, d and e = parameters found at the top of each of the tables in Section 5.0.
- The tabular values in *Section 5.0* Rainfall Tables were determined from the new IDF curves.

Figure 1.2 shows an example Intensity-Duration-Frequency (IDF) Curve for Dallas County, for seven storms (1-year – 100-year). These curves are plots of the tabular values. No values are given for times less than 5 minutes. The 500-year values are given for durations no less than 15 minutes.

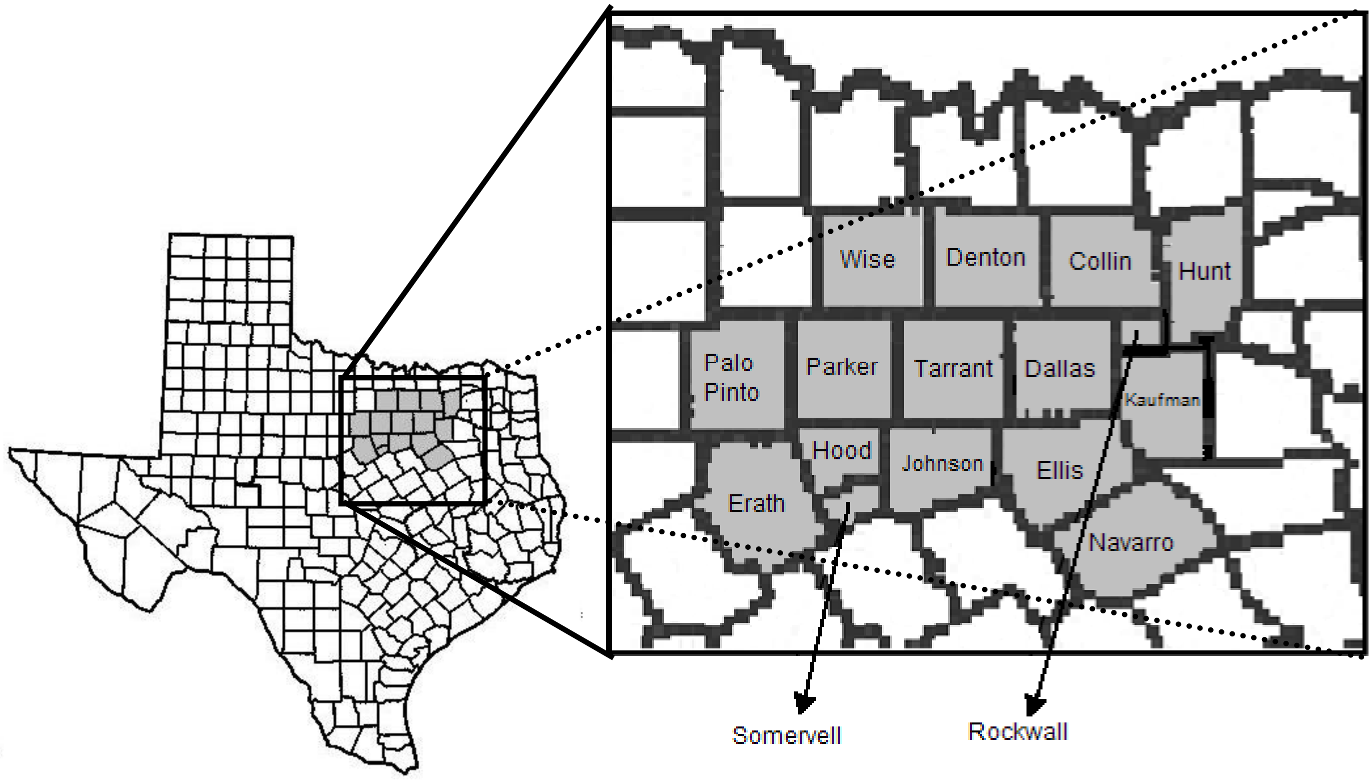


Figure 1.1 The 16 Counties Participating in NCTCOB

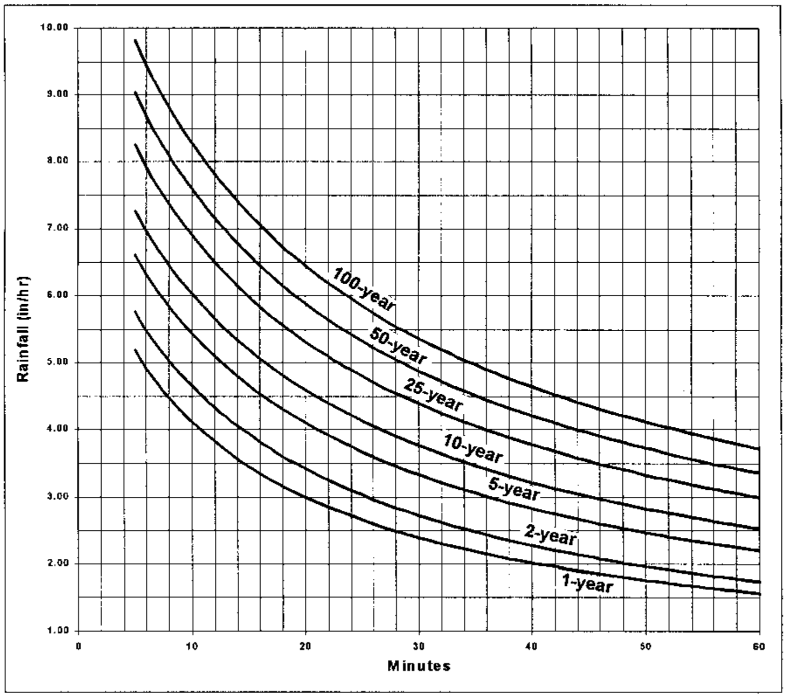


Figure 1.2 Example IDF Curve (Dallas County, Texas)

1.2. Rational Method

1.2.1. Introduction

An important formula for determining the peak runoff rate is the Rational Formula. It is characterized by:

- Consideration of the entire drainage area as a single unit
- Estimation of flow at the most downstream point only
- The assumption that rainfall is uniformly distributed over the drainage area and is constant over time

The Rational Formula adheres to the following assumptions:

- The predicted peak discharge has the same probability of occurrence (return period) as the rainfall intensity (I)
- The runoff coefficient (C) is constant during the storm event

When using the Rational Method some precautions should be considered:

- In determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility.
- Since the Rational Method uses a composite C and a single t_c value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), then the basin should be divided into sub-drainage basins.
- The charts, graphs, and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer should use sound engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate adjustments are appropriate.

1.2.2. Application

The Rational Method can be used to estimate stormwater runoff peak flows for the design of gutter flows, drainage inlets, storm drainpipe, culverts, and small ditches. It is most applicable to small, highly impervious areas. The recommended maximum drainage area that should be used with the Rational Method is 200 acres.

The Rational Method should not be used for storage design or any other application where a more detailed routing procedure is required. However, the Modified Rational method is used by some for design of small detention facilities, so the method has been included in *Section 1.5*. The normal use of the Modified Rational method significantly under predicts detention volumes, but the improved method in *Section 1.5* corrects this deficiency in the method and can be used for detention design for drainage areas up to 200 acres.

The Rational Method should not be used for calculating peak flows downstream of bridges, culverts, or storm sewers that may act as restrictions causing storage, which impacts the peak rate of discharge.

1.2.3. Equations

The Rational Formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and the mean rainfall intensity for a duration equal to the time of concentration, t_c (the time required for water to flow from the most remote point of the basin to the location being analyzed).

The Rational Formula is expressed as follows:

$$Q = CIA \tag{1.2}$$

where:

Q	=	maximum rate of runoff (cfs)
C	=	runoff coefficient representing a ratio of runoff to rainfall
I	=	average rainfall intensity for a duration equal to the t_c (in/hr)
A	=	drainage area contributing to the design location (acres)

The coefficients given in Table 1.6 are applicable for storms with return periods less than or equal to 10 years. Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin Engineers, 1969). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f. The Rational Formula now becomes:

$$Q = C_f CIA \tag{1.3}$$

The C_f values that can be used are listed in Table 1.4. The product of C_f times C shall not exceed 1.0.

Table 1.4 Frequency Factors for Rational Formula

Recurrence Interval (years)	C_f
10 or less	1.0
25	1.1
50	1.2
100	1.25

1.2.4. *Time of Concentration*

Use of the Rational Formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The time of concentration consists of an overland flow time to the point where the runoff is concentrated or enters a defined drainage feature (e.g., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Figure 1.3 can be used to estimate overland flow time. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter. In urban areas, the length of overland flow distance should realistically be no more than 50 – 100 feet.

Although there is no formula for the graph shown in Figure 1.3, the formula often used, which seems to match the nomograph very closely, is as follows:

$$T_c = 1.8(1.1 - C)(D)^{0.5}/(S)^{(1/3)} \tag{1.4}$$

where:

T_c	=	time of concentration (min)
C	=	average or composite runoff coefficient
D	=	distance from upper end of watershed to outlet (ft)
S	=	average slope along distance “D”, in percent (ft/100 ft)

Example: Given the following values, determine the time of concentration using (1) Equation 1.4, and (2) Figure 1.3: D = 250 ft, C = 0.7, S = 0.50% slope.

1. Figure 1.3 gives approximately 15 minutes.
2. $T_c = 1.8(1.1 – 0.7)(250)^{0.5}/(0.50)^{(1/3)} = 14.34$ min

Other methods and charts may be used to calculate overland flow time if approved by the local review authority.

Generally, the time of concentration for overland flow is only a part of the overall design problem. Often one encounters swale flow, confined channel flow, and closed conduit flow travel times that must be added as part of the overall time of concentration. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown in Figure 1.4. More guidance on travel time estimation is given in *Section 1.3.6*.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. For example, if the flow time in a channel is 15 minutes and the overland flow time from the ridge line to the channel is 10 minutes, then the total time of concentration is 25 minutes. Note that the time of concentration cannot be less than 5 minutes or that which is established by local standards.

Table 1.5 gives recommended minimum and maximum times of concentration based on land use categories. The minimum time of concentration should be used for the most upstream inlet (minimum inlet time). Computed downstream travel times will be added to determine times of concentration through the system. For anticipated future upstream development, the time of concentration should be no greater than the maximum.

Table 1.5 Times of Concentration

Land Use	Minimum (minutes)	Maximum (minutes)
Residential Development	15	30
Commercial and Industrial	10	25
Central Business District	10	15

Two common errors should be avoided when calculating time of concentration. First, in some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. Second, when designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 50 feet for impervious areas should be done only after careful consideration.

1.2.5. Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in in/hr for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration data given in the rainfall tables in *Section 5.0*.

1.2.6. Runoff Coefficient (C)

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 1.6 gives the recommended runoff coefficients for the Rational Method.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 1.6 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as rooftops, asphalt, and concrete streets and sidewalks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

It should be remembered that the Rational Method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area, then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

It may be that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site. This should be checked particularly in areas where the overland portion is grassy (yielding a long t_c) to avoid underestimating peak runoff.

1.2.7. Example Problem

Following is an example problem that illustrates the application of the Rational Method to estimate peak discharges.

Estimates of the maximum rate of runoff are needed at the inlet to a proposed culvert for a 25-year return period.

Site Data

From a topographic map of the City of Arlington (Tarrant County) and a field survey, the area of the drainage basin upstream from the point in question is found to be 23 acres. In addition, the following data were measured:

Average overland slope = 2.0%
Length of overland flow = 50 ft
Length of main basin channel = 2,250 ft
Slope of channel = .018 ft/ft = 1.8%
Roughness coefficient (n) of channel was estimated to be 0.090
From existing land use maps, land use for the drainage basin was estimated to be:
Residential (single family – ¼ acre lots) - 80%
Graded - sandy soil, 3% slope - 20%

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: Lawn - sandy soil, 2% slope

Overland Flow

A runoff coefficient (C) for the overland flow area is determined from Table 1.6 to be 0.10.

Table 1.6 Recommended Runoff Coefficient Values

Description of Area	Runoff Coefficients (C)
Lawns:	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Agricultural	0.30
Forest	0.15
Streams, Lakes, Water Surfaces	1.00
Business:	
Downtown areas	0.95
Neighborhood areas	0.70
Residential:	
Single Family (1/8 acre lots)	0.65
Single Family (1/4 acre lots)	0.60
Single Family (1/2 acre lots)	0.55
Single Family (1+ acre lots)	0.45
Multi-Family Units, (Light)	0.65
Multi-Family, (Heavy)	0.85
Commercial/Industrial:	
Light areas	0.70
Heavy areas	0.80
Parks, cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Streets:	
Asphalt and Concrete	0.95
Brick	0.85
Drives, walks, and roofs	0.95
Gravel areas	0.50
Graded or no plant cover:	
Sandy soil, flat, 0 - 5%	0.30
Sandy soil, flat, 5 - 10%	0.40
Clayey soil, flat, 0 - 5%	0.50
Clayey soil, average, 5 - 10%	0.60

Time of Concentration

From Figure 1.3 with an overland flow length of 50 ft, slope of 2% and a C of 0.10, the overland flow time is 10 min. Channel flow velocity is determined from Figure 1.4 to be 3.1 ft/s (n = 0.090, R = 1.62 (from channel dimensions) and S = .018). Therefore,

$$\text{Flow Time} = \frac{2{,}250 \text{ feet}}{(3.1 \text{ ft/s}) / (60 \text{ s/min})} = 12.1 \text{ minutes}$$

and $t_c = 10 + 12.1 = 22.1$ min (use 22 min)

Rainfall Intensity

From Table 5.15 in *Section 5.0*, using a duration equal to 22 minutes,

I_{25} (25-yr return period) = 5.41 in/hr

<u>Runoff Coefficient</u>

A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the values from Table 1.7.

Table 1.7 Weighted Runoff Coefficient

1	2	3	4
Land Use	**Percent of Total Land Area**	**Runoff Coefficient**	**Weighted Runoff Coefficient***
Residential (Single Family – ¼ acre lots)	0.80	0.60	0.48
Graded area	0.20	0.30	0.06
	Total Weighted Runoff Coefficient =		<u>0.54</u>
*Column 4 equals column 2 multiplied by column 3.			

Peak Runoff

The estimate of peak runoff for a 25-yr design storm for the given basin is:

$Q_{25} = C_f CIA = (1.10)(.54)(5.41)(23) = 73.9$ cfs

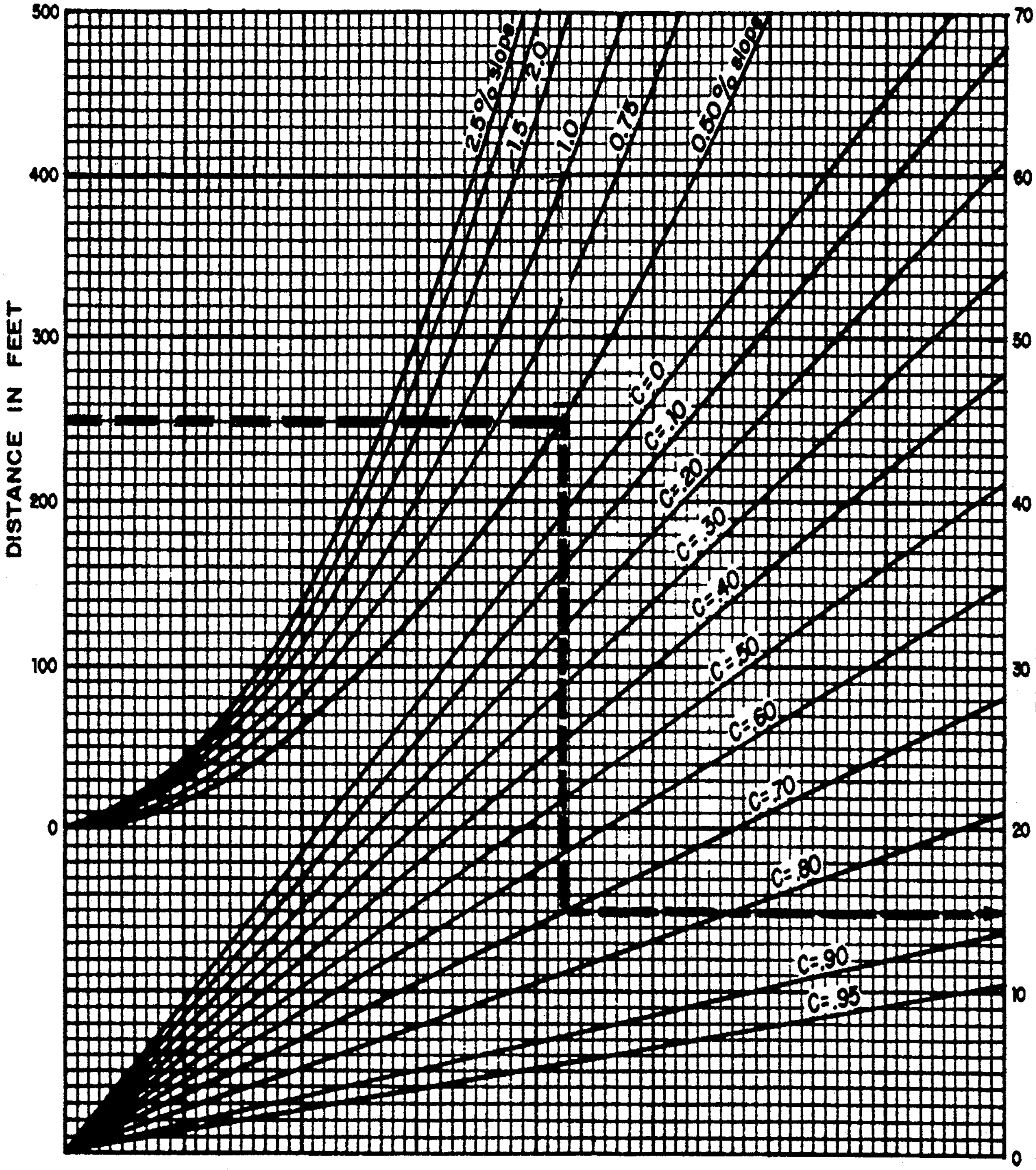
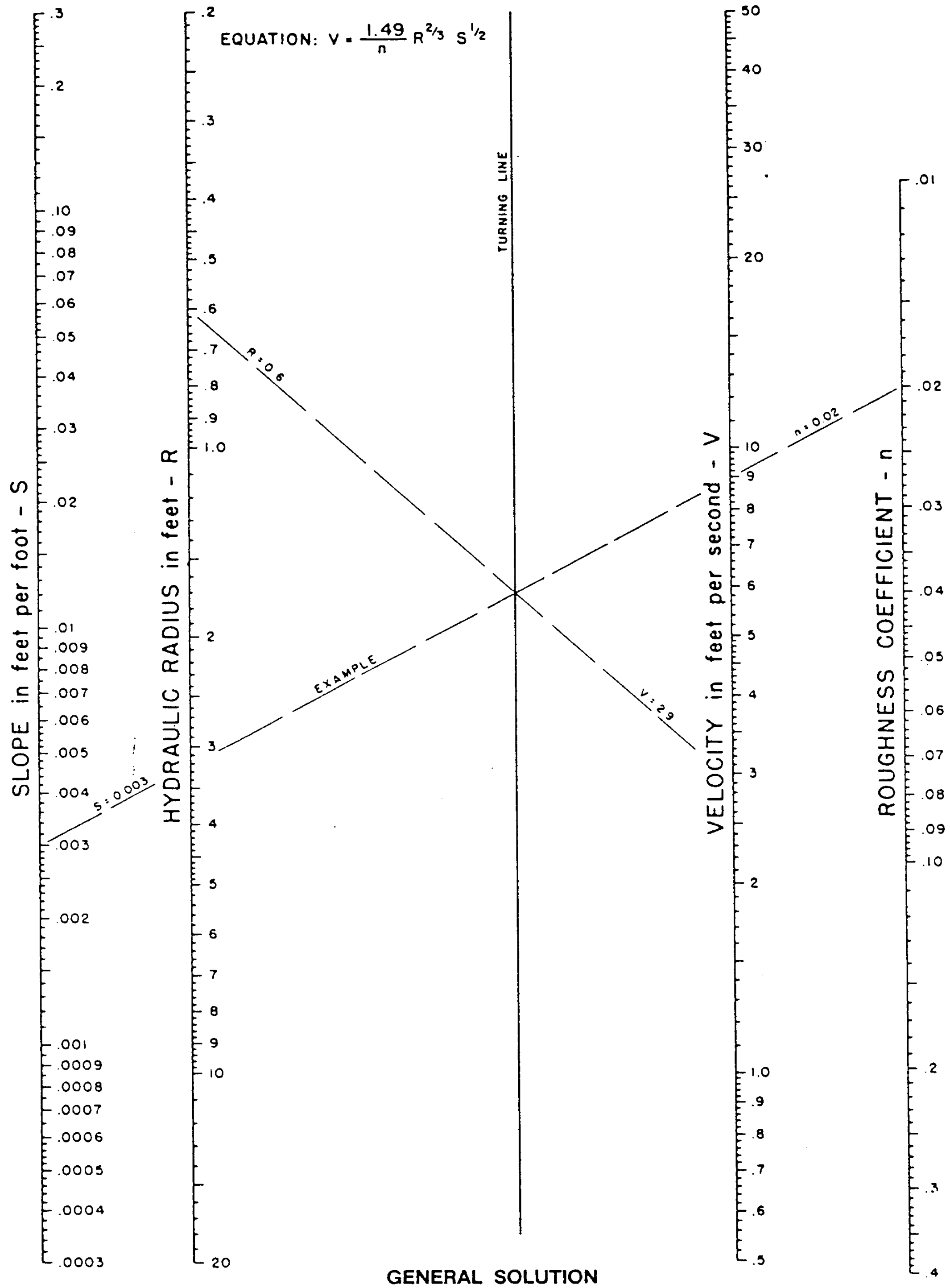


Figure 1.3 Rational Formula - Overland Time of Flow Nomograph
(Source: Airport Drainage, Federal Aviation Administration, 1965)



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

Figure 1.4 Manning's Equation Nomograph
(Source: USDOT, FHWA, HDS-3 (1961))

1.3. SCS Hydrological Method

1.3.1. Introduction

The Soil Conservation Service[1] (SCS) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the *SCS National Engineering Handbook, Section 4, Hydrology.*

A typical application of the SCS method includes the following basic steps:

1. Determination of curve numbers that represent different land uses within the drainage area.
2. Calculation of time of concentration to the study point.
3. Using the Type II rainfall distribution, total and excess rainfall amounts are determined. Note: See Figure 1.5 for the geographic boundaries for the different SCS rainfall distributions.
4. Using the unit hydrograph approach, the hydrograph of direct runoff from the drainage basin can be developed.

1.3.2. Application

The SCS method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The simplified method of *Section 1.3.7* can be used for drainage areas up to 2,000 acres. Thus, the SCS method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches, open channels, and energy dissipaters.

1.3.3. Equations and Concepts

The hydrograph of outflow from a drainage basin is the sum of the elemental hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical characteristics of the basin including shape, size, and slope are constant, the unit hydrograph approach assumes there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the runoff volume. Therefore, a storm that produces 2 inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the flow of the hydrograph would be one-half of the flow of the unit hydrograph.

The following discussion outlines the equations and basic concepts used in the SCS method.

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, combine hydrographs from different sub-basins as applicable, and/or route flows to points of interest.

Rainfall - The SCS method applicable to North Central Texas is based on a storm event that has a Type II time distribution. This distribution is used to distribute the 24-hour volume of rainfall for the different storm frequencies (Figure 1.5).

[1] The Soil Conservation Service is now known as the Natural Resources Conservation Service (NRCS)

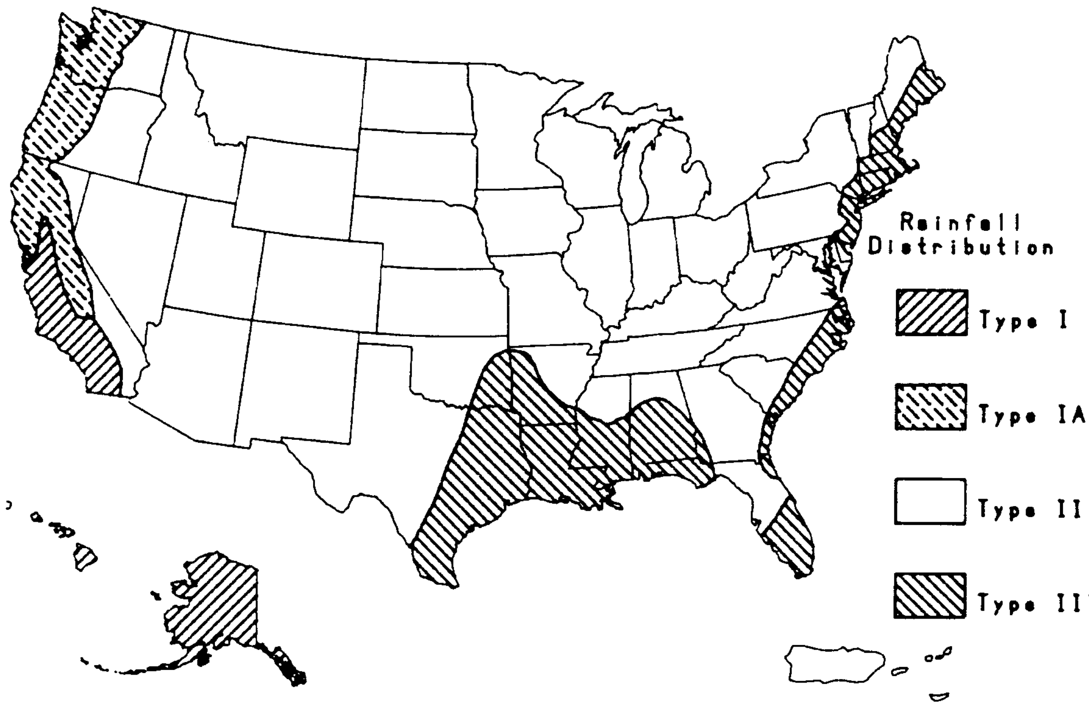


Figure 1.5 Approximate Geographic Boundaries for SCS Rainfall Distributions

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The following SCS runoff equation is used to estimate direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$Q = (P - I_a)^2 / [(P - I_a) + S] \quad (1.5)$$

where:

- Q = accumulated direct runoff (in)
- P = accumulated rainfall (potential maximum runoff) (in)
- I_a = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)
- S = 1000/CN - 10
  where:
  CN = SCS curve number

An empirical relationship used in the SCS method for estimating I_a is:

$$I_a = 0.2S \quad (1.6)$$

This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment. Table 1.11 provides values of I_a for a wide range of curve numbers (CN).

Substituting 0.2S for I_a in Equation 1.6, the equation becomes:

$$Q = (P - 0.2S)^2 / (P + 0.8S) \quad (1.7)$$

Figure 1.6 shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurred on a watershed with a curve number of 85.

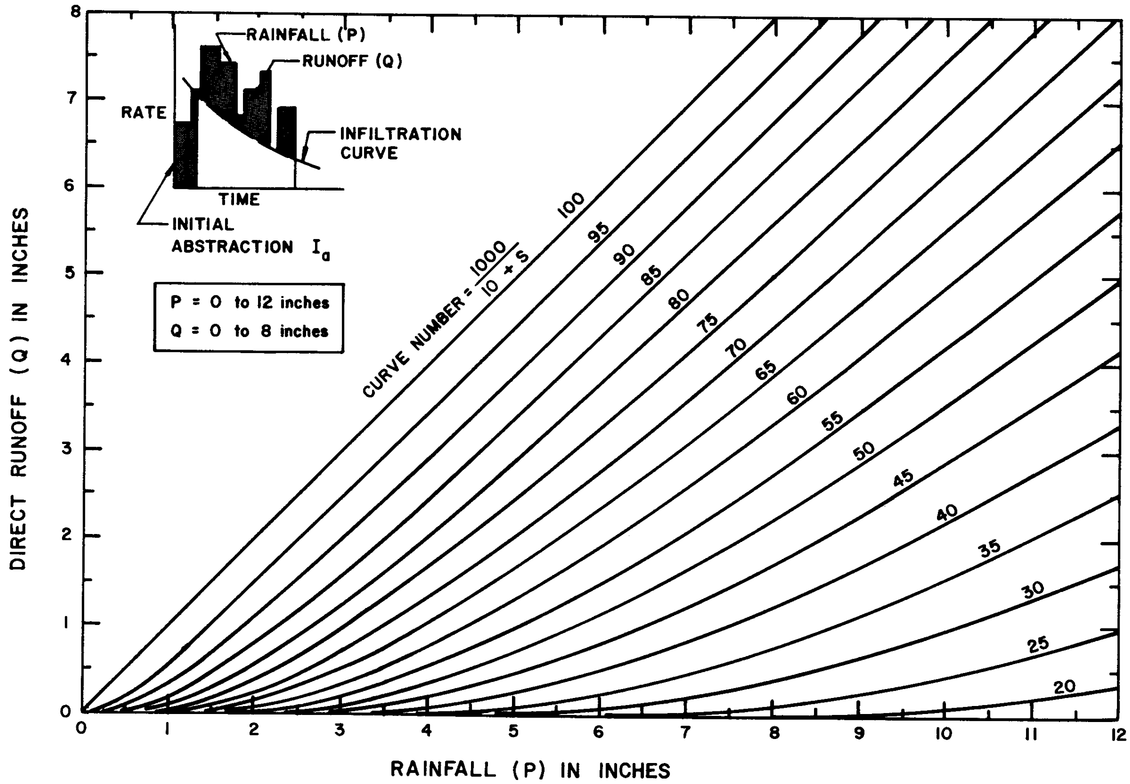


Figure 1.6 SCS Solution of the Runoff Equation

(Source: SCS, TR-55, Second Edition, June 1986)

Equation 1.7 can be rearranged so the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$\mathbf{CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}]} \qquad \textbf{(1.8)}$$

1.3.4. Runoff Factor (CN)

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the SCS has divided soils into four hydrologic soil groups.

Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

A list of soils throughout the State of Texas and their hydrologic classification can be found in the publication *Urban Hydrology for Small Watersheds, 2nd Edition, Technical Release Number 55, 1986*. Soil Survey maps can be obtained from local USDA Natural Resources Conservation Service offices for use in estimating soil type. *Section 6.0 - Hydrologic Soils Data* contains hydrologic soils classification data for North Central Texas. County specific data can be found on-line through NRCS at http://soils.usda.gov/ and/or www.nctcog.dst.tx.us/.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions. Average antecedent soil moisture conditions (AMC II) are recommended for most hydrologic analysis. Areas with high water table conditions may want to consider using AMC III antecedent soil moisture conditions. This should be considered a calibration parameter for modeling against real calibration data. Table 1.9 gives recommended curve number values for a range of different land uses.

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by using the weighted method as presented in Table 1.8.

Table 1.8 Composite Curve Number Calculation Example

Land Use	Percent of Total Land Area	Curve Number	Weighted Curve Number (% area x CN)
Residential 1/8 acre Soil Group B	0.80	0.85	0.68
Meadow Good condition Soil Group C	0.20	0.71	0.14
Total Weighted Curve Number = 0.68 + 0.14 = 0.82			

The different land uses within the basin should reflect a uniform hydrologic group represented by a single curve number. Any number of land uses can be included, but if their spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

1.3.5. Urban Modifications of the SCS Method

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 1.9 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system. It is possible for curve number values from urban areas to be reduced by not directly connecting impervious surfaces in the drainage system, but allowing runoff to flow as sheet flow over significant pervious areas.

The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

Connected Impervious Areas

The CNs provided in Table 1.9 for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

1. Pervious urban areas are equivalent to pasture in good hydrologic condition, and
2. Impervious areas have a CN of 98 and are directly connected to the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 1.9 are not applicable, use Figure 1.7 to compute a composite CN. For example, Table 1.9 gives a CN of 70 for a 1/2-acre lot in hydrologic soil group B, with an assumed impervious area of 25%. However, if the lot has 20% impervious area and a pervious area CN of 61, the composite CN obtained from Figure 1.7 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use Figure 1.8 if total impervious area is less than 30% or (2) use Figure 1.7 if the total impervious area is equal to or greater than 30%, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When the impervious area is less than 30%, obtain the composite CN by entering the right half of Figure 1.8 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20% total impervious area (75% of which is unconnected) and pervious CN of 61, the composite CN from Figure 1.8 is 66. If all of the impervious area is connected, the resulting CN (from Figure 1.7) would be 68.

1.3.6. Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another within a watershed, through the various components of the drainage system. Time of concentration (t_c) is computed by summing all the travel times for consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed. Following is a discussion of related procedures and equations (USDA, 1986).

<u>Travel Time</u>

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$T_t = L/3600V \qquad (1.9)$$

where:

- T_t = travel time (hr)
- L = flow length (ft)
- V = average velocity (ft/s)
- 3600 = conversion factor from seconds to hours

Table 1.9 Runoff Curve Numbers

Cover Description		Curve numbers for hydrologic soil groups[1]			
Cover type and hydrologic condition	*Average percent impervious area[2]*	**A**	**B**	**C**	**D**
Cultivated Land:					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land:					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow:					
Good condition		30	58	71	78
Wood or forest land:					
Thin stand, poor cover		45	66	77	83
Good cover		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town house)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (previous areas only, no vegetation)		77	86	91	94

Table 1.9 Runoff Curve Numbers					
Cover Description		**Curve numbers for hydrologic soil groups[1]**			
Cover type and hydrologic condition	*Average percent impervious area[2]*	**A**	**B**	**C**	**D**

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

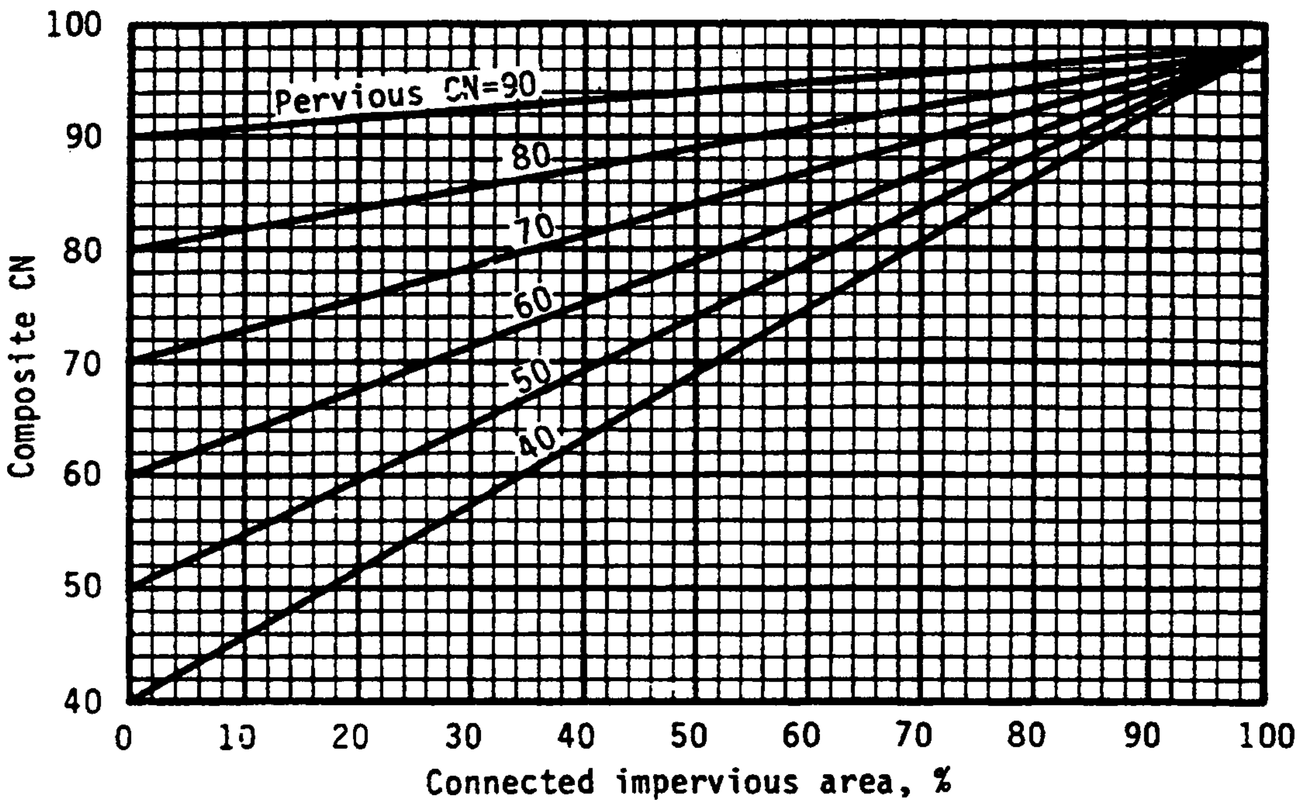


Figure 1.7 Composite CN with Connected Impervious Areas

(Source: SCS, TR-55, Second Edition, June 1986)

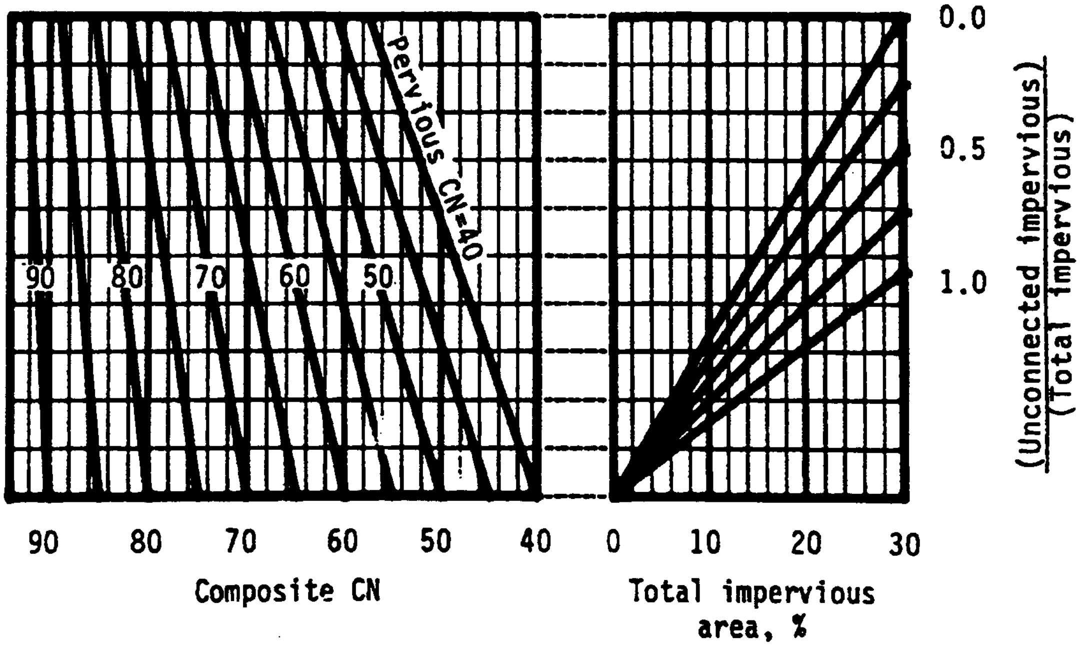


Figure 1.8 Composite CN with Unconnected Impervious Areas (Total Impervious Area Less Than 30%)

(Source: SCS, TR-55, Second Edition, June 1986)

Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_t = \frac{0.42\,(nL)^{0.8}}{60\,(P_2)^{0.5}(S)^{0.4}} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}(S)^{0.4}} \quad (1.10)$$

where:

T_t = travel time (hr)
n = Manning roughness coefficient (see Table 1.10)
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall (in)
S = land slope (ft/ft)

Table 1.10 Roughness Coefficients (Manning's n) for Sheet Flow

Surface Description	n[1]
Smooth surfaces	
(concrete, asphalt, gravel or bare soil)	0.011
Fallow	
(no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41

Range	
(natural)	0.13
Woods[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values are a composite of information by Engman (1986).

[2] Includes species such as bluestem grass, buffalo grass, grama grass, and native grass mixtures.

[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Source: SCS, TR-55, Second Edition, June 1986.

Shallow Concentrated Flow

After 50 to 100 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 1.9, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using Figure 1.9, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved $$V = 16.13(S)^{0.5} \quad (1.11)$$

Paved $$V = 20.33(S)^{0.5} \quad (1.12)$$

where:

V = average velocity (ft/s)
S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity using Figure 1.9 or Equations 1.11 or 1.12, use Equation 1.9 to estimate travel time for the shallow concentrated flow segment.

Open Channels

Velocity in channels should be calculated from the Manning equation. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by the local municipality, or where stream designations appear on United States Geological Survey (USGS) quadrangle sheets. Manning's Equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions.

Manning's Equation is

$$\mathbf{V = (1.49/n)\,(R)^{2/3}\,(S)^{1/2}} \qquad \mathbf{(1.13)}$$

where:

- V = average velocity (ft/s)
- R = hydraulic radius (ft) and is equal to A/P_w
- A = cross sectional flow area (ft^2)
- P_w = wetted perimeter (ft)
- S = slope of the hydraulic grade line (ft/ft)
- n = Manning's roughness coefficient for open channel flow

After average velocity is computed using Equation 1.13, T_t for the channel segment can be estimated using Equation 1.9.

Limitations

- Equations in this section should not be used for sheet flow longer than 50 feet for impervious surfaces.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate t_c.
- A culvert or bridge can act as detention structure if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert or bridge.

1.3.7. Simplified SCS Peak Runoff Rate Estimation

The following SCS procedures were taken from the SCS Technical Release 55 (USDA, 1986) which presents simplified procedures to calculate storm runoff volume and peak rate of discharges. For full description and compliance with methodology, please refer to SCS Technical Release 55 (USDA, 1986).

These procedures are applicable to small drainage areas (typically less than 2,000 acres) with homogeneous land uses, which can be described by a single CN value. The peak discharge equation is:

$$\mathbf{Q_p = q_u A Q F_p} \qquad \mathbf{(1.14)}$$

where:

- Q_p = peak discharge (cfs)
- q_u = unit peak discharge (cfs/mi²/in)
- A = drainage area (mi²)
- Q = runoff (in)
- F_p = pond and swamp adjustment factor

Fi

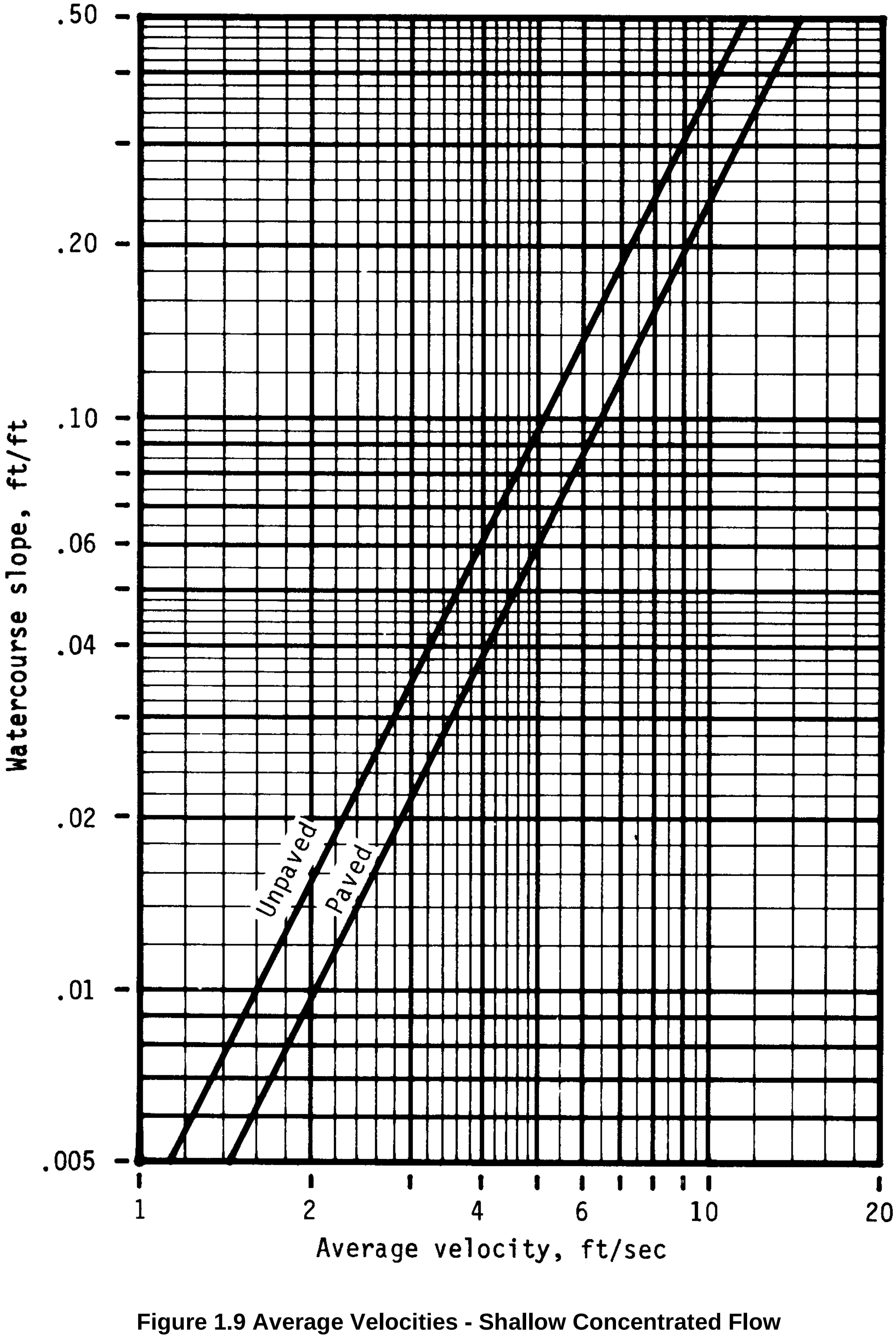


Figure 1.9 Average Velocities - Shallow Concentrated Flow

(Source: SCS, TR-55, Second Edition, June 1986)

Computations for the peak discharge method proceed as follows:

1. The 24-hour rainfall depth (P) is determined from the rainfall tables in *Section 5.0* for the selected location and return frequency.
2. The runoff curve number, CN, is estimated from Table 1.9 and direct runoff, Q, is calculated using Equation 1.7.
3. The CN value is used to determine the initial abstraction, I_a, from Table 1.11, and the ratio I_a/P is then computed (P = accumulated 24-hour rainfall).
4. The watershed time of concentration is computed using the procedures in *Section 1.3.6* and is used with the ratio I_a/P to obtain the unit peak discharge (q_u) from Figure 1.10 for the Type II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figures, either the limiting values or another peak discharge method should be used. Note: Figure 1.10 is based on a peaking factor of 484. If a peaking factor of 300 is needed, these figures are not applicable and the simplified SCS method should not be used. Peaking factors are discussed further in *Section 1.3.9*.
5. The pond and swamp adjustment factor, F_p, is estimated from below:

Pond and Swamp Areas (%*)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

*Percent of entire drainage basin

6. The peak runoff rate is computed using Equation 1.14.

1.3.8. Example Problem 1

Compute the flood mitigation storm peak discharge for a 50-acre watershed located in Fort Worth, which will be developed as follows:

1. Pasture / range land - good condition (hydrologic soil group D) = 10 ac
2. Pasture / range land - good condition (hydrologic soil group C) = 10 ac
3. 1/3 acre residential (hydrologic soil group D) = 20 ac
4. Industrial development (hydrological soil group C) = 10 ac

Other data include the following: Total impervious area = 18 acres, % of pond / swamp area = 0

Computations

1. Calculate rainfall excess:
   - The flood mitigation storm, 24-hour rainfall is 9.12 inches (.38 in/hr x 24 hours – From *Section 5.0*, Table 5.15).
   - Composite weighted runoff coefficient is:

Dev. #	Area	% Total	CN	Composite CN
1	10 ac.	0.20	80	18.2
2	10 ac.	0.20	74	14.8
3	20 ac.	0.40	86	34.4
4	10 ac.	0.20	91	18.2
Total	50 ac.	1.00		83

* from Equation 2.1.7 Q (flood mitigation storm) = 7.1 inches

Table 1.11 I_a Values for Runoff Curve Numbers

Curve Number	I_a (in)	Curve Number	I_a (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.74	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Source: SCS, TR-55, Second Edition, June 1986

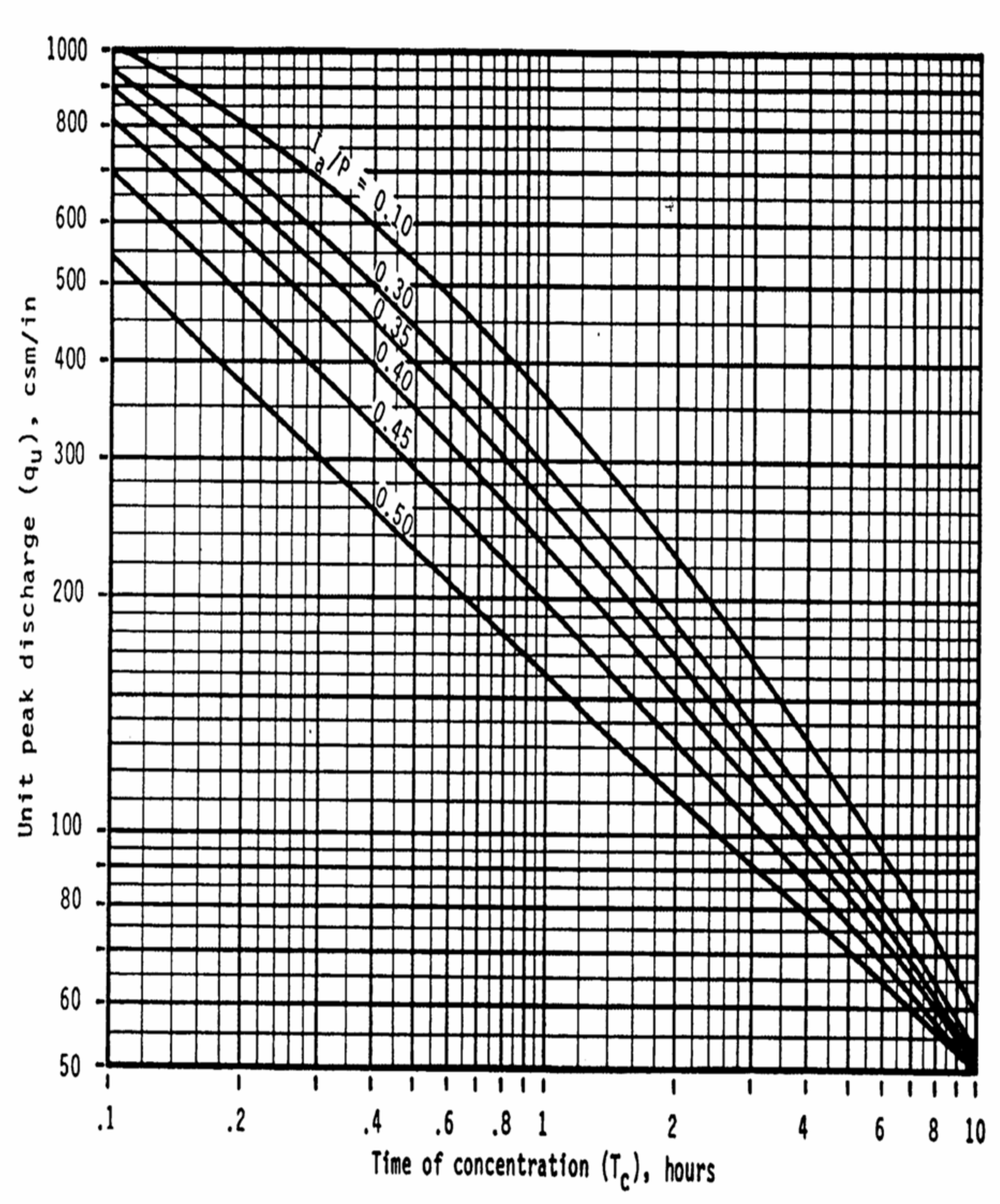


Figure 1.10 SCS Type II Unit Peak Discharge Graph

(Source: SCS, TR-55, Second Edition, June 1986)

2. Calculate time of concentration

The hydrologic flow path for this watershed = 1,890 ft

Segment	Type of Flow	Length (ft)	Slope (%)
1	Overland n = 0.24	40	2.0
2	Shallow channel (unpaved)	750	1.7
3	Main channel*	1100	0.50

* For the main channel, n = .06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 - Travel time from Equation 1.10 with P_2 = 3.36 inches (0.14 x 24 – *Section 5.0*, Table 5.15)

$T_t = [0.42(0.24 \text{ X } 40)^{0.8}] / [(3.36)^{0.5} (.020)^{0.4}]$ = 6.69 minutes

Segment 2 - Travel time from Figure 1.9 or Equation 1.10
V = 2.1 ft/sec (from Equation 1.11)
T_t = 750 / 60 (2.1) = 5.95 minutes

Segment 3 - Using Equation 1.13
$V = (1.49/.06)\,(1.43)^{0.67}\,(.005)^{0.5}$ = 2.23 ft/sec
T_t = 1100 / 60 (2.23) = 8.22 minutes

t_c = 6.69 + 5.95 + 8.22 = 20.86 minutes (.35 hours)

3. Calculate I_a/P for CN = 83 (Table 1.9), I_a = .410 (Table 1.11)

   I_a/P = (.410 / 9.12) = .05
   (Note: Use I_a/P = .10 to facilitate use of Figure 1.10.)

4. Unit discharge q_u (flood mitigation storm) from Figure 1.10 = 650 csm/in

5. Calculate peak discharge with F_p = 1 using Equation 1.14

   Q_{100} = 650 (50/640)(7.1)(1) = 360 cfs

1.3.9. Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The SCS has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for small drainage areas (less than 2,000 acres). The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, SCS has developed hydrograph procedures to be used to generate composite flood hydrographs. For the development of a hydrograph from a homogeneous developed drainage area and drainage areas that are not homogeneous, where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the SCS in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

The unit hydrograph equations used in the SCS method for generating hydrographs includes a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. SCS indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat (coastal) areas.

A value of 484 should be used for most areas of North Texas; however, there are flat areas where a lesser value may be appropriate.

The development of a runoff hydrograph from a watershed is a laborious process not normally done by hand calculation. For that reason, only an overview of the process is given here to assist the designer in reviewing and understanding the input and output from a typical computer program. There are choices of computational interval, storm length (if the 24-hour storm is not going to be used), and other "administrative" parameters, which are peculiar to each computer program.

The development of a runoff hydrograph for a watershed or one of many sub-basins within a more complex model involves the following steps:

1. Development or selection of a design storm hyetograph. Often the SCS 24-hour storm described in *Section 1.3.3* is used. This storm is recommended for use in North Central Texas.
2. Development of curve numbers and lag times for the watershed using the methods described in *Sections 1.3.4, 1.3.5, and 1.3.6*.
3. Development of a unit hydrograph using the standard (peaking factor of 484) dimensionless unit hydrograph. See discussion below.
4. Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation (Equation 1.8).
5. Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of rainfall (this is called "convolution").
6. Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

To assist the designer in using the SCS unit hydrograph approach with a peaking factor of 484, Figure 1.11 and Table 1.12 have been developed. The unit hydrograph with a peaking factor of 300 is shown in the figure for comparison purposes, but, typically, should not be used for areas in North Central Texas.

The procedure to develop a unit hydrograph from the dimensionless unit hydrograph in the table below is to multiply each time ratio value by the time-to-peak (T_p) and each value of q/q_u by q_u calculated as:

$$q_u = (PF*A) / (T_p) \qquad (1.15)$$

where:

q_u = unit hydrograph peak rate of discharge (cfs)
PF = peaking factor (484)
A = area (mi^2)
d = rainfall time increment (hr)
T_p = time to peak = $d/2 + 0.6\, t_c$ (hr)

For ease of spreadsheet calculations, the dimensionless unit hydrograph for 484 can be approximated by the equation:

$$\frac{q}{q_u} = \left(\frac{t}{T_p}\, e^{[1-(t/T_p)]} \right)^X \qquad (1.16)$$

where X is 3.79 for the PF=484 unit hydrograph.

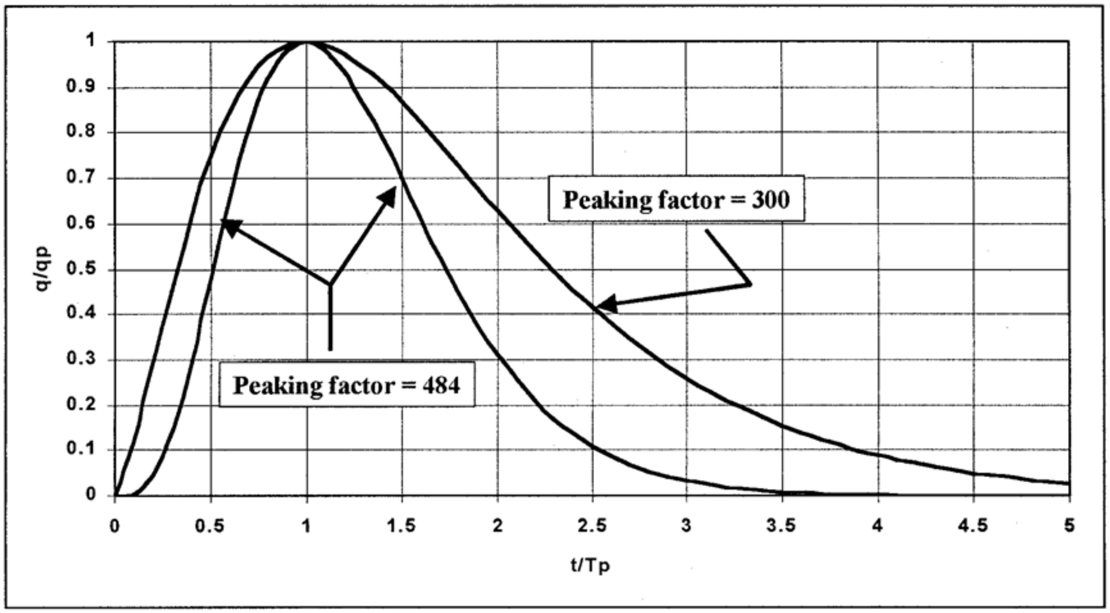


Figure 1.11 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

Table 1.12 Dimensionless Unit Hydrograph with Peaking Factor of 484

t/T_t	484	
	q/q_u	Q/Q_p
0.0	0.0	0.0
0.1	0.005	0.000
0.2	0.046	0.004
0.3	0.148	0.015
0.4	0.301	0.038
0.5	0.481	0.075
0.6	0.657	0.125
0.7	0.807	0.186
0.8	0.916	0.255
0.9	0.980	0.330
1.0	1.000	0.406
1.1	0.982	0.481
1.2	0.935	0.552
1.3	0.867	0.618
1.4	0.786	0.677
1.5	0.699	0.730
1.6	0.611	0.777
1.7	0.526	0.817
1.8	0.447	0.851
1.9	0.376	0.879
2.0	0.312	0.903
2.1	0.257	0.923
2.2	0.210	0.939
2.3	0.170	0.951
2.4	0.137	0.962
2.5	0.109	0.970
2.6	0.087	0.977
2.7	0.069	0.982
2.8	0.054	0.986
2.9	0.042	0.989
3.0	0.033	0.992
3.1	0.025	0.994
3.2	0.020	0.995
3.3	0.015	0.996
3.4	0.012	0.997
3.5	0.009	0.998
3.6	0.007	0.998
3.7	0.005	0.999
3.8	0.004	0.999
3.9	0.003	0.999
4.0	0.002	1.000

1.3.10. Example Problem 2

Compute the unit hydrograph for the 50-acre watershed in Example Problem 1 (*Section 1.3.8*).

Computations

1. Calculate T_p and time increment

   The time of concentration (t_c) is calculated to be 20.86 minutes for this watershed. If we assume a computer calculation time increment (d) of 3 minutes then:

   T_p = d/2 + 0.6t_c = 3/2 + 0.6 * 20.86 = 14.02 minutes (0.234 hrs)

2. Calculate q_{pu}

   q_u = PF*A/T_p = (484 * 50/640) / (0.234) = 162 cfs

3. Calculate unit hydrograph.

   Based on spreadsheet calculations using Equations 1.15 and 1.16, Table 1.13 has been derived.

Table 1.13 Example of Dimensionless Unit Hydrograph with Peaking Factor of 484

Time		484	
t/T_p	time (min)	q/q_u	Q
0	0	0	0.00
0.21	3	0.06	9.23
0.43	6.0	0.35	56.77
0.64	9.0	0.72	117.29
0.86	12.0	0.96	155.09
1.00	14.02	1.00	162.00
1.07	15.0	0.99	160.57
1.28	18.0	0.88	142.42
1.50	21.0	0.70	113.52
1.71	24.0	0.52	83.69
1.93	27.0	0.36	58.12
2.14	30.0	0.24	38.51
2.35	33.0	0.15	24.56
2.57	36.0	0.09	15.18
2.78	39.0	0.06	9.14
3.00	42.0	0.03	5.38
3.21	45.0	0.02	3.10
3.42	48.0	0.01	1.76
3.64	51.0	0.01	0.99
3.85	54.0	0.00	0.54
4.07	57.0	0.00	0.30
4.28	60.0	0.00	0.16
4.49	63.0	0.00	0.09
4.71	66.0	0.00	0.05
4.92	69.0	0.00	0.02
5.14	72.0	0.00	0.01
5.35	75.0	0.00	0.01
5.56	78.00	0.00	0.00

1.3.11. Hydrologic Stream Routing

Water requires a certain amount of time to travel down a stream or channel reach. A flood wave is attenuated by friction and channel storage as it passes through the reach. The process of computing the travel time and attenuation of water flowing in the reach is often called routing.

Hydrologic routing involves the balancing of inflow, outflow, and volume of storage through the use of the continuity equation. The relation between the outflow rate and storage in the system is also required.

Travel time and attenuation characteristics vary widely between different streams. The travel time is dependent on characteristics such as length, slope, friction, and flow depth. Attenuation is also dependent on friction, in addition to other characteristics such as channel storage. Many routing methods have been developed under different assumptions and for different stream types. Some of the routing methods include: kinematic wave, lag, modified Puls, Muskingum, Muskingum-Cunge 8-point section, and Muskingum-Cunge standard section.

The routing methods selected for use in North Central Texas are the Modified Puls and the Muskingum-Cunge methods (USACE, HEC-HMS, 2000 and Bedient and Huber, 1988).

1.4. Snyder's Unit Hydrograph Method

1.4.1. Introduction

Snyder's unit hydrograph method is the primary method utilized by the Corps of Engineers Fort Worth District for the majority of hydrologic studies in the region, and is also commonly used by consultants and other entities within the NCTCOG region. It is similar in nature to the SCS method, in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm.

1.4.2. Application

Snyder's unit hydrograph method may be used for drainage areas 100 acres or larger. This method, detailed in the U.S. Army Corps of Engineers Engineering Manual (EM 1110-2-1405), *Flood-Hydrograph Analysis and Computations* and The Bureau of Reclamation's "Flood Hydrology Manual, A Water Resources Technical Publication," utilizes the following equations:

$$t_p = C_t (L\ L_{ca})^{0.3} \quad (1.17)$$
$$t_r = t_p \div 5.5 \quad (1.18)$$
$$q_p = C_p 640 \div t_p \quad (1.19)$$
$$t_{pR} = t_p + 0.25(t_R - t_r) \quad (1.20)$$
$$q_{pR} = C_p 640 \div t_{pR} \quad (1.21)$$
$$q_{pR} = q_p\ t_p \div t_{pR} \quad (1.22)$$
$$Q_p = q_p\ A \quad (1.23)$$

The terms in the above equations are defined as:

t_r = The standard unit rainfall duration, in hours.

t_R = The unit rainfall duration in hours other than standard unit, t_r, adopted in specific study.

t_p = The lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours.

t_{pR} = The lag time from midpoint of unit rainfall duration, t_R, to peak of unit hydrograph in hours.

q_p = The peak rate of discharge of unit hydrograph for unit rainfall duration, t_r, in cfs/sq. mi.

q_{pR} = The peak rate of discharge in cfs/sq mi. of unit in hydrograph for unit rainfall duration, t_R.

Q_p = The peak rate of discharge of unit hydrograph in cfs.

A = The drainage area in square miles.

L_{ca} = The river mileage from the design point to the centroid of gravity of the drainage area.

L = The river mileage from the given station to the upstream limits of the drainage area.

C_t = Coefficient depending upon units and drainage basin characteristics.

C_p = Coefficient depending upon units and drainage basin characteristics.

The coefficient C_t is a regional coefficient for variations in slopes within the watershed. Typical values of C_t range from 0.4 to 2.3 and average about 1.1. The value of C_t for the East Fork Trinity River is 2.0. C_t for a watershed can be estimated if the lag time, t_p, stream length, L, and distance to the basin centroid, L_{ca}, are known. The coefficient C_p is the peaking coefficient, which typically ranges from 0.3 to 1.2 with an average value of 0.8, and is related to the flood wave and storage conditions of the watershed. The C_p value for the East Fork Trinity River is 0.69. Larger values of C_p are generally associated with smaller values of C_t. Typical values of C_p are listed in Table 1.14.

Table 1.14 Typical Values of Cp

Typical Drainage Area Characteristics	C_p
Undeveloped Areas w/ Storm Drains	
Flat Basin Slope (less than 0.50%)	0.55
Moderate Basin Slope (0.50% to 0.80%)	0.58
Steep Basin Slope (greater than 0.80%)	0.61
Moderately Developed Area	
Flat Basin Slope (less than 0.50%)	0.63
Moderate Basin Slope (0.50% to 0.80%)	0.66
Steep Basin Slope (greater than 0.80%)	0.69
Highly Developed/Commercial Area	
Flat Basin Slope (less than 0.50%)	0.70
Moderate Basin Slope (0.50% to 0.80%)	0.73
Steep Basin Slope (greater than 0.80%)	0.77

1.4.3. Urbanization Curves

To account for the effects of urbanization, another method was developed by the Corps of Engineers to adjust the t_p coefficient. Urbanization curves allow for the determination of t_p based on the percent urbanization and the type of soil in the study area. Urbanization curves for the Dallas-Fort Worth area were determined from the equation below:

$$t_p = 10^{[0.3833 \cdot \log_{10}(L \cdot L_{ca}/(S_{st})^{0.5}) + (\log_{10}(Ip)) - BW \cdot (\%Urb/100)]} \quad (1.24)$$

$$S_{st} = (el_{85\%} - el_{15\%})/(0.7 \cdot L) \quad (1.25)$$

where:

- t_p = The lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours.
- L_{ca} = The river mileage from the design point to the centroid of the drainage area.
- L = The river mileage from the design point to the upstream limits of the drainage area.
- S_{st} = The weighted slope of the flow path (ft/mi)
- Ip = The calibration point, defined as t_p where ($L \cdot L_{ca}/S_{st}$^.5) = 1 and urbanization = 0%.
- BW = The bandwidth, equal to the log of the width between each 20% urbanization line.
- %Urb = A value representative of the degree to which urbanization has occurred in the basin, in percent.
- $el_{85\%}$ = The elevation at a location 85% upstream of the given station.
- $el_{15\%}$ = The elevation at a location 15% upstream of the given station.

For the Dallas-Fort Worth area, the Ip values used are 0.94 for clay and 1.76 for sand. The bandwidth (BW) value for both of the soil types is 0.266. For a study area that is composed of both sand and clay, a weighted average of the two can be calculated by:

t_p weighted = % sand*t_p sand + % clay * t_p clay.

Design runoff may be determined for a given watershed by applying the intensity-duration-frequency relationships to the unit hydrograph by multiplying each ordinate of the unit hydrograph by the rainfall intensity.

1.4.4. Determination of Percent Urbanization and Percent Sand

The lag time, t_p, is the critical parameter in establishing the timing of the response of a watershed to rainfall. The degree of urbanization is an important variable that determines the value of the lag time. Thomas L. Nelson, Fort Worth District, USACE, defined the general relationship between the lag time, t_p, and the percent of Urbanization, %Urb, and presented a set of Urbanization Curves for the Dallas-Fort Worth area in 1970.

The soil type of a watershed also plays an important role in its response to rainfall. It was found that predominantly sandy soils responded differently to rainfall than predominantly clayey soils. Therefore, two sets of Urbanization Curves were developed to better define the lag time, one set for sandy soils and one set for clayey soils. A paper by Paul K. Rodman, Fort Worth District, USACE presented urbanization curves in 1977 for both “clay loam” and “clay” in the Fort Worth-Dallas area and other Texas locations.

To obtain consistency of computational results, it is necessary to have a logical and routine procedure for the determination of Percent Urbanization (%Urb) and Percent Sand/Clay (%Sand/%Clay). Procedures for their determination are presented below.

Percent Urbanization

Urbanization is defined as the percentage of the basin which has been developed and improved with channelization and/or a stormwater collection network. Urbanization of natural and agricultural land converts pervious soils to impervious surfaces. Disturbed soils exhibit a lower infiltration capacity than natural soils. This results in less infiltration which translates to an increased volume of runoff.

Natural flow paths in the watershed may be replaced with prismatic channels. Significant drainage infrastructure may be added in a development composed of streets and gutters, storm sewers, open channels, and other drainage elements. This alteration of the original drainage system changes the watershed's response to precipitation. The addition of drainage infrastructure along with the increase in imperviousness results in significantly increased peak discharges and a greater volume of runoff.

The determination of the percent urbanization (%Urb) as used in the Urbanization Curves defined by Equation 1.24 is somewhat subjective, but is related to the type and intensity of development. The U.S. Army Corps of Engineers (USACE) has worked over the years to define the relationship between the type of development and the degree of urbanization. The result of their effort is reflected in Table 1.15. These are provided for the user's consideration and guidance.

Other techniques to relate the impacts of urbanization on rainfall runoff have been used. Another such technique is presented in *Section 1.6* in the application of USGS regression equations to determine peak flows for urban basins.

Percent Sand/Clay

The Fort Worth District, USACE, evaluated methods for determining the percent sand in a watershed and concluded that the permeability rate method was the best method. The procedure was described in the referenced report as follows.

"The permeability rate method uses the range of permeabilities found in the table (Table 1.16) of physical and chemical properties in the SCS soil surveys for multiple soil classifications and assigns a percent sand to each of the seven ranges. A percent sand of 0 is given to any soil with a permeability less than 0.06 inches per hour which corresponds to the permeability of the Houston Blackland clay upon which the clay urban curves are based. Also, a percent sand of 100 is given to any soil with a rate of 0.6 to 2.0 inches per hour which corresponds to the Crosstell series soil upon which the sandy loam curves are based. The percent sand for the permeability ranges 0.06 to 0.2 inches, 0.2 to 0.6 inches, 2.0 to 6.0 inches, 6.0 to 10.0 inches, and greater than 20 inches are 33, 66, 133, 166, 200 percent sand, respectively. Each soil in the watershed is assigned a percent sand based upon its permeability and a weighted average is computed." (USACE, 1986)

Table 1.15 Percent Urbanization and Imperviousness Summary with Associated Land Use Categories

Land Use	Description	Percent Imperviousness	Percent Urbanization
Low Density Residential	Single family: ½ – 2 units per acre; average 1 unit per acre.	25	30
Medium Density Residential	Single family: 2 – 3½ units per acre; average 3 units per acre.	41	80
High Density Residential	Single family: greater than 3½ units per acre; average 4 units per acre.	47	90
Multifamily Residential	Row houses, apartments, townhouses, etc.	70	95
Mobile Home Parks	Single family: 5–8 units per acre.	20	40
Central Business District	Intensive, high-density commercial	95	95
Strip Commercial	Low-density commercial; average 3 units per acre.	90	90
Shopping Centers	Grocery stores, drug stores, malls, etc.	95	95
Institutional	Schools, churches, hospitals, etc.	40	50
Industrial	Industrial centers and parks; light and heavy industry.	90	95
Transportation	Major highways, railroads.	35	80
Communication	Microwave towers, etc.	35	50
Public Utilities	Transformer stations, transmission line right-of-way, sewage treatment facilities, water towers, and water treatment facilities.	60	70
Strip Settlement	Densities less than ½ – 2 units per acre; average 1 unit per 3 – 5 acres.	10	20
Parks and Developed Open Space	Parks, cemeteries, etc.	6	10
Developing	Land currently being developed.	15	20
Cropland		3	5
Grassland	Pasture, short grasses.	0	0
Woodlands, Forest		0	0
Water Bodies	Lakes, large ponds.	100	100
Barren Land	Bare exposed rock, strip mines, gravel pits.	0	0

Sources: Determination of Percent Urbanization/Imperviousness in Watersheds, May 1, 1986, U.S. Army Corps of Engineers
SCS, TR-55, Second Edition, June 1986

Table 1.16 Permeability Rating for the Determination of Percent Sand

Permeability (inches/hr)	Percent Sand Assignment (%)
< 0.06	0
0.06 to 0.20	33
0.20 to 0.60	66
0.60 to 2.00	100
2.00 to 6.00	133
6.00 to 20.00	166
> 20.00	200

The Houston Black soil series consists of moderately well-drained, deep, cyclic, clayey soils on wetlands. This series formed in alkaline, marine clay, and material weathered from shale. Land slopes range from 1 to 4 percent. The permeability is less than 0.06 inches per hour. This soil is the predominate series found in watersheds used to develop the Dallas-Fort Worth Clay Urbanization Curves. Therefore this soil has a percent sand of 8 for use with the urban curves. The Crosstell soil series consists of moderately well-drained, deep loamy soils on uplands that formed in shaley and clayey sediment containing thin strata of weakly cemented sandstone. Land slopes range from 1 to 6 percent. The permeability for this soil is in the range between 0.6 and 2.0 inches per hour. The Crosstell series is the major soil contained in watersheds used to derive the Dallas-Fort Worth Sandy Loam Urbanization Curves. This soil, therefore, has a percent sand of 100 for use with the urban curves.

Example: Procedure for the Determination of Percent Sand (%Sand).

Given the percent sand assignments below, determine the percent sand for Watershed B.

Watershed	Soil Type No.	Percent Sand	% of Area	% Sand * % Area
B	13	66	2.6	171.6
	23	33	39.7	1310.1
	32	133	31.4	4176.2
	51	33	1.7	56.1
	64	133	17.9	2380.7
	85	33	6.7	221.1
			100	8315.8

Weighted %Sand = 8315.8/100 = 83.2%

There is the possibility of computing greater than 100 percent sand for areas that are very sandy. Soil disturbances during development (urbanization) usually diminish the natural permeability of the soil. Often there is no data reflecting the permeability rate for an urban soil. Therefore, care should be used in applying this method. The percent sand assignment should be that of the controlling sublayer of the soil profile. Consideration should also be given to other factors affecting the initial and time rates of rainfall abstractions. For example, well-vegetated clayey soils may respond hydrologically more like a sandy soil. Urban lands are usually taken one step down (lower percent sand) from soil types shown in the SCS soil report. The engineer should evaluate all factors bearing on the soil response and determine whether there is a need to make adjustments.

Loss Rates

Several loss rate methodologies, as shown in Table 1.17, are acceptable for use with the Snyder's Unit Hydrograph Method including:

- Block and Uniform
- Holtan
- SCS Curve Number
- Green and Ampt
- Exponential

Block and uniform loss rates developed by the Corps of Engineers during the development of the urbanization curves are listed by clay and sand categories. Losses for a specific basin are determined by a weighting procedure. Adjustments to these values are allowed based on historic storm reproductions.

Table 1.17 Hydrologic Loss Rates

Frequency	Losses			
	Clay		Sand	
	Block (in)	Uniform (in/hr)	Block (in)	Uniform (in/hr)
2-year	1.5	0.20	2.1	0.26
5-year	1.3	0.16	1.8	0.21
10-year	1.12	0.14	1.5	0.18
25-year	0.95	0.12	1.3	0.15
50-year	0.84	0.1	1.1	0.13
100-year	0.75	0.07	0.9	0.10

Stream Routing

The Modified Puls and Muskingum-Cunge are acceptable routing methods. See *Section 1.3.11*, for an explanation of routing methods and references for further information.

1.5. Modified Rational Method

1.5.1. Introduction

For drainage areas of *less than 200 acres*, a modification of the Rational Method can be used for the estimation or design of storage volumes for detention calculations.

The Modified Rational Method uses the peak flow calculating capability of the Rational Method paired with assumptions about the inflow and outflow hydrographs to compute an approximation of storage volumes for simple detention calculations. There are many variations on the approach.

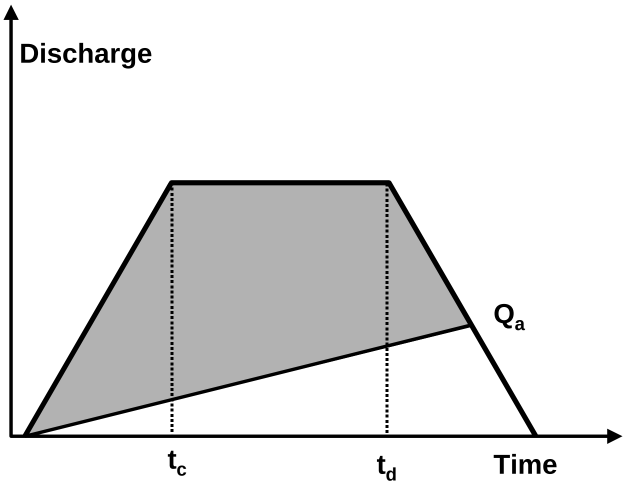


Figure 1.12 Modified Rational Definitions

illustrates one application. The rising and falling limbs of the inflow hydrograph have a duration equal to the time of concentration (t_c). An allowable target outflow is set (Q_a) based on pre-development conditions. The storm duration is t_d, and is varied until the storage volume (shaded gray area) is maximized. It is normally an iterative process done by hand or on a spreadsheet. Downstream analysis is not possible with this method as only approximate graphical routing takes place.

1.5.2. Design Equations

The design of detention using the Modified Rational Method is presented as a non-iterative approach suitable for spreadsheet calculation (Debo & Reese, 2003).

The allowable release rate can be determined from:

$$\mathbf{Q_a = C_a\ i\ A} \qquad \textbf{(1.26)}$$

where:

Q_a = allowable release rate (cfs)

C_a = predevelopment Rational Method runoff coefficient

i = rainfall intensity for the corresponding time of concentration (in/hr)

A = area (acres)

The critical duration of storm, the time value to determine rainfall intensity, at which the storage volume is maximized, is:

$$T_d = \sqrt{\frac{2CAab}{Q_a}} - b \qquad \textbf{(1.27)}$$

where:

T_d = critical storm duration (min)

Q_a = allowable release rate (cfs)

C = developed condition Rational Method runoff coefficient

A = area (acres)

a, b = rainfall factors dependent on location and return period taken from Table 1.18

The required storage volume, in cubic feet can be obtained from the equations below:

$$V_{preliminary} = 60\ [CAa - (2CabAQ_a)^{1/2} + (Q_a/2)\ (b-t_c)] \qquad (1.28a)$$

$$V_{max} = V_{preliminary} * P_{180}/P_{td} \qquad (1.28b)$$

where:

$V_{preliminary}$ = preliminary required storage (ft^3)
V_{max} = required storage (ft^3)
t_c = time of concentration for the developed condition (min)
P_{180} = 3-hour (180-minute) storm depth (in)
P_{td} = storm depth for the critical duration (in)
all other variables are as defined above

The equations above include the use of an adjustment factor to the calculated storage volume to account for under sizing. The factor (P_{180}/P_{td}) is the ratio of the 3-hour storm depth for the return frequency divided by the rainfall depth for the critical duration calculated in Equation 1.27.

The Modified Rational Method also often under sizes storage facilities in flat and more sandy areas where the target discharge may be set too large, resulting in an oversized orifice. In these locations modifications to the C factor or time of concentration should be considered in the design of the orifice.

Table 1.18 Rainfall Factors "a" and "b" for the Modified Rational Method (1-year through 100-year return periods)

County		Return Interval						
		1	2	5	10	25	50	100
Collin	a	101.14	129.51	177.49	209.08	250.52	283.13	320.81
	b	14.214	16.634	20.174	21.668	22.821	23.455	24.502
Dallas	a	99.8	128.85	178.58	210.73	253.77	288.56	327.75
	b	14.114	16.624	20.352	21.785	23.03	23.866	24.893
Denton	a	97.258	124.47	173.1	205.74	248.54	283.99	325.18
	b	13.788	16.121	19.754	21.358	22.615	23.508	24.822
Ellis	a	101.94	129.3	181.43	214.61	259.34	295.76	336.3
	b	14.511	16.697	20.792	22.384	23.744	24.681	25.818
Erath	a	90.53	113.9	159.31	189.97	228.79	260.81	298.07
	b	13.32	14.99	18.439	19.981	20.955	21.65	22.712
Grayson	a	100.87	128.89	175.74	208.17	250.17	285.35	325.63
	b	14.086	16.567	20.006	21.751	22.993	24.027	25.322
Hood	a	93.351	117.38	163	194.75	235.56	269.71	309.25
	b	13.654	15.308	18.65	20.281	21.438	22.299	23.508
Hunt	a	107.65	131.48	178.92	209.36	249.71	282.05	318.9
	b	15.348	16.855	20.456	21.855	22.995	23.713	24.744
Johnson	a	94.751	120.21	168.39	198.98	240.45	275.19	313.38
	b	13.414	15.543	19.272	20.676	21.847	22.804	23.875
Kaufman	a	104.54	132.07	183.2	216.62	260.03	295.03	334.63
	b	14.637	16.912	20.837	22.424	23.65	24.42	25.496
Navarro	a	108.66	132.42	185.55	221.63	268.93	306.83	350.06
	b	15.326	16.758	20.945	22.903	24.437	25.402	26.665
Palo Pinto	a	91.031	115.97	164.22	196.59	242.51	281.03	326
	b	13.127	15.264	19.05	20.714	22.468	23.769	25.388
Parker	a	95.164	118.64	166.17	198.53	242.46	279.34	321.89
	b	13.848	15.396	18.999	20.608	22.048	23.123	24.527
Rockwall	a	107.9	131.23	179.89	212.63	254.36	287.68	325.96
	b	15.671	16.882	20.467	22.064	23.178	23.891	24.906
Somervell	a	92.245	116.25	162.12	193.36	232.22	265.8	303.15
	b	13.091	14.967	18.503	20.102	21.066	22.001	23.039
Tarrant	a	95.835	121.96	170.81	203.93	247.1	282.6	322.07
	b	13.425	15.704	19.435	21.09	22.366	23.302	24.388
Wise	a	93.326	118.05	165.95	200.22	247.21	287.89	334.11
	b	13.491	15.315	18.974	20.889	22.662	24.112	25.784

1.5.3. Example Problem

A 5-acre site is to be developed in Dallas. Based on site and local information, it is determined that streambank protection is not required and that limiting the 25-year and flood mitigation storm is also not required. The local government has determined that the development must detain the 2-year and 10-year storms. Rainfall values are taken from *Section 5.0*. The following key information is obtained:

- Area = 5 acres
- Slope is about 5%
- Pre-development t_c = 21 minutes and C factor = 0.22
- Post-development t_c = 10 minutes and C factor = 0.80

Steps	**2 - year**	**10 - year**
t_c (min)	21	21
i (in/hr)	3.35	4.79
Q_a (Equation 1.26) (cfs)	3.69	5.27
a (from Table 1.18)	128.85	210.73
b (from Table 1.18)	16.624	21.785
V_{pre} (Equation 1.28a) (ft^3)	16,570	26,042
P_{180} (in)	2.28	3.60
T_d (Equation 1.27) (min)	51.52	61.69
P_{td} (in)	1.65	2.66
V_{max} (Equation 1.28b) (ft^3)	22,897	35,245

1.6. USGS and TxDOT Regression Methods

1.6.1. Introduction

Regional regression equations are the most commonly accepted method for establishing peak flows at larger ungauged sites (or sites with insufficient data for a statistical derivation of the flood versus frequency relation). Regression equations have been developed to relate peak flow at a specified return period to the physiography, hydrology, and meteorology of the watershed.

Regression analyses use stream gauge data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar watershed, channel, and meteorological characteristics; they are often termed hydrologically homogeneous geographic areas. For this manual, the USGS regression equations are used to determine peak flows in urban drainage areas, and the TXDOT regression equations are used to determine peak flows in rural drainage areas. It may be difficult to choose the proper set of regression equations when the design site lies on or near the hydrologic boundaries of relevant studies. Another problem occurs when the watershed is partly or totally within an area subject to mixed population floods.

The following suggestions should be considered when using regression equations:

- Conduct a field visit to compare and assess the watershed characteristics for comparison with other watersheds.

- Collect all available historical flood data.
- Use the gathered data to interpret any discharge values.

1.6.2. USGS Equations for Urban Basins

Regression equations developed by the USGS for urban streams in Dallas-Fort Worth are for estimating peak discharges (QT) having recurrence intervals (T) that range from 2 to 100 years. The explanatory basin variables used in the equations are drainage area (DA), in square miles, and an urbanization index (UI), which is evaluated as described in the report by Land and others (U.S.G.S., 1982).

The urbanization index is an attempt to more accurately quantify the degree of urbanization by incorporating the factors of storm sewers, curbs and gutters, and channel rectifications. The index is developed by considering these alterations in the upper, middle, and lower third of the drainage basin. Values are assigned to each factor in each one-third of the basin on the basis of the percentage of the subbasin containing that factor. Each factor carries an equal weight regardless of location within the subbasin. The values of each factor vary from 1 to 4, based on the degree of development. The sum of the 9 factors can vary from 9 to 36 and is the value of the urbanization index.

The factor values and corresponding percentages of the subbasin affected are listed below:

Percent	Value
0 – 24	1
25 – 49	2
50 – 74	3
75 – 100	4

The following example is given to illustrate the determination of the urbanization index.

Urbanization Index Factors

Sub area	Storm Sewers	Curbs and Gutters	Channel Rectifications	Total
Upper	4	4	2	10
Middle	3	4	1	8
Lower	3	4	1	8
Urbanization Index				**26**

Source: Techniques for Estimating the Magnitude and Frequency of Floods in the Dallas-Fort Worth Metropolitan Area, Texas, U.S. Geological Survey, Water Resources Investigation 82-18

1.6.3. Application of USGS Equations

The USGS regression equations were developed from peak-discharge records from drainage areas in the Dallas-Fort Worth area ranging from 1.25 to 66.4 square miles with results considered applicable to drainage areas between 3 and 40 square miles having urbanization indexes between 12 and 33. The standard errors of estimate of the regression equations are about 30 percent. As such, the USGS regression method should only be used for calculating peak discharge in urban drainage areas as described.

The USGS method can be used for several design applications, including storm drain systems, culverts, small drainage ditches and open channels, and energy dissipaters.

For a complete description of the USGS regression equations presented below, consult the USGS publication *Techniques for estimating the magnitude and frequency of floods in the Dallas-Fort Worth metropolitan area, Texas: U.S. Geological Survey Water-Resources Investigations Report 82-18, 55 p*. Table 1.19 gives the USGS regression equations for urban streams in the Dallas-Fort Worth area.

1.6.4. Peak Discharge Limitations for Urban Basins

Following are the limitations of the variables within the peak discharge equations. These equations should not be used on drainage areas which have physical characteristics outside the limits listed below:

Physical Characteristics	Minimum	Maximum	Units
A - Drainage Area	3	40	mi^2
UI – Urbanization Index	12	33	

Table 1.19 USGS Peak Flow Regression Equations for Dallas-Fort Worth Urban Areas

Frequency	Equations
2-year	$Q_2 = 42.83(A)^{0.704}(UI)^{0.836}$
5-year	$Q_5 = 82.92(A)^{0.724}(UI)^{0.751}$
10-year	$Q_{10} = 120.7(A)^{0.735}(UI)^{0.697}$
25-year	$Q_{25} = 184.8(A)^{0.745}(UI)^{0.632}$
50-year	$Q_{50} = 246.4(A)^{0.752}(UI)^{0.587}$
100-year	$Q_{100} = 362.1(A)^{0.752}(UI)^{0.510}$
For these equations: A = drainage area in mi^2, UI = urbanization index Source: USGS, 1982	

1.6.5. TxDOT Equations for Rural (or Undeveloped) Basins

The Texas Department of Transportation (TxDOT) has a regression method for estimating peak discharges for rural basins. For a complete discussion of the development of these equations consult Chapter 5, Section 11 of the TxDOT Hydraulic Design Manual, available online at http://manuals.dot.state.tx.us/docs/colbridg/forms/hyd.pdf or the reference USGS, 1997.

1.6.6. Rural (or Undeveloped) Basin Application

Equation 1.29 applies to rural, uncontrolled watersheds. Figure 1.13 presents the geographic extents of each region. Note that most of the NCTCOG region lies within Region 7, with small portions of Region 3 and 4. Table 1.20 presents the coefficients and limits of applicability for Regions 3, 4, and 7. Generally, use this equation to compare with the results of other methods, check existing structures, or where it is not practicable to use any other method, keeping in mind the importance of the facility being designed.

$$Q_T = aA^bSH^cSL^d \tag{1.29}$$

where:

- Q_T = T-year discharge (cfs)
- A = contributing drainage area (sq. mi.)
- SH = basin-shape factor defined as the ratio of main channel length squared to contributing drainage area (sq. mi./sq. mi.)
- SL = mean channel slope defined as the ratio of headwater elevation of longest channel minus main channel elevation at site to main channel length (ft./mi.). Note: This differs from previous rural regression equations in which slope was defined between points 10 and 85 percent of the distance along the main channel from the outfall to the basin divide.
- a, b, c, d = multiple linear regression coefficients dependent on region number and frequency.

The equations to be used for Regions 3, 4, and 7 are found in Table 1.20.

Regions 3, 4, and 7 have two sets of coefficients. For these regions, if the drainage area is between 10 and 100 sq. mi., determine a weighted discharge (Q_w) as shown in Equation 1.30.

$$Q_w = (2 - \log(A/z))Q_1 + (\log(A/z)-1)Q_2 \quad (1.30)$$

where:

- Q_w = weighted discharge (cfs)
- A = contributing drainage area (sq. mi.)
- z = 1.0 for English measurements units
- Q_1 = discharge based on regression coefficients for A < 32 sq. mi. (cfs)
- Q_2 = discharge based on regression coefficients for A ≥ 32 sq. mi. (cfs)

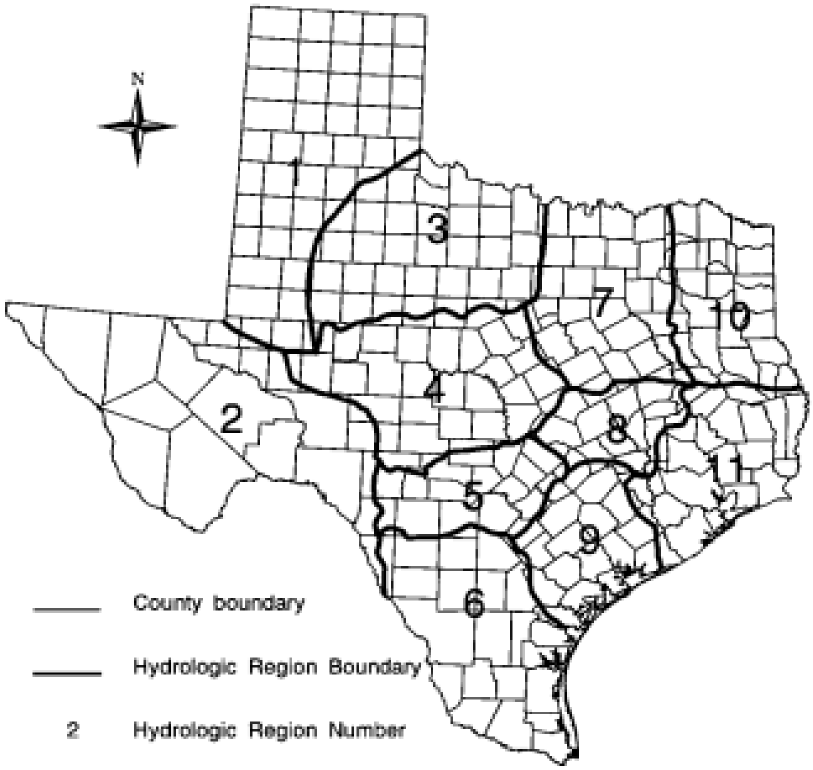


Figure 1.13 Hydrologic Regions for Statewide Rural Regression Equations
Source: TXDOT, 2002

Table 1.20 Regression Equations for Estimation of Peak-Streamflow Frequency for Hydrologic Regions of Texas

[yr, year; A, contributing drainage area in square miles; SH, basin shape factor – ration of length of longest mapped channel (stream length) squared to contributing drainage area (dimensionless); SL, stream slope in feet per mile – ration of change in elevation of (1) longest mapped channel from site (or station) to headwaters to (2) length of longest mapped channel]

Hydrologic region and recurrence interval	Weighted least-squares regression equation for corresponding recurrence interval	Range of indicated independent variables in corresponding region (units as noted)
Region 3 (sites with contributing drainage area less than 32 square miles)[2]		

Table 1.20 Regression Equations for Estimation of Peak-Streamflow Frequency for Hydrologic Regions of Texas

[yr, year; A, contributing drainage area in square miles; SH, basin shape factor – ration of length of longest mapped channel (stream length) squared to contributing drainage area (dimensionless); SL, stream slope in feet per mile – ration of change in elevation of (1) longest mapped channel from site (or station) to headwaters to (2) length of longest mapped channel]

Hydrologic region and recurrence interval	Weighted least-squares regression equation for corresponding recurrence interval	Range of indicated independent variables in corresponding region (units as noted)
2 yr	$Q_2 = 119 A^{.592}$	A: 0.10 to 97.0
5 yr	$Q_5 = 252 A^{.629}$	
10 yr	$Q_{10} = 373 A^{.652}$	SH: 0.16 to 9.32
25 yr	$Q_{25} = 566 A^{.679}$	
50 yr	$Q_{50} = 743 A^{.698}$	SL: 10.7 to 105
100 yr	$Q_{100} = 948 A^{.715}$	
Region 3 (sites with contributing drainage area greater than 32 square miles)[2]		
2 yr	$Q_2 = 8.05 A^{.668} SL^{.659} SH^{.189}$	A: 11.8 to 14,635
5 yr	$Q_5 = 4.20 A^{.626} SL^{.574}$	
10 yr	$Q_{10} = 91.9 A^{.579} SL^{.537}$	SH: 1.71 to 75.0
25 yr	$Q_{25} = 233 A^{.523} SL^{.476}$	
50 yr	$Q_{50} = 448 A^{.484} SL^{.425}$	SL: 4.81 to 36.3
100 yr	$Q_{100} = 835 A^{.447} SL^{.372}$	
Region 4 (sites with contributing drainage area less than 32 square miles)[2]		
2 yr	$Q_2 = 97.1 A^{.626}$	A: 0.19 to 81.1
5 yr	$Q_5 = 196 A^{.650} SH^{.257}$	
10 yr	$Q_{10} = 293 A^{.697} SH^{.281}$	SH: 0.05 to 6.52
25 yr	$Q_{25} = 455 A^{.741} SH^{.311}$	
50 yr	$Q_{50} = 53 A^{.927} SL^{.558} SH^{.333}$	SL: 13.5 to 226
100 yr	$Q_{100} = 51 A^{.968} SL^{.627} SH^{.353}$	
Region 4 (sites with contributing drainage area greater than 32 square miles)[2]		
2 yr	$Q_2 = 0.00660 A^{1.29} SL^{2.09}$	A: 12 to 19,819
5 yr	$Q_5 = 0.0212 A^{1.24} SL^{2.18}$	
10 yr	$Q_{10} = 0.0467 A^{1.20} SL^{2.18}$	SH: 0.49 to 19.7
25 yr	$Q_{25} = 0.102 A^{1.16} SL^{2.18}$	
50 yr	$Q_{50} = 0.166 A^{1.13} SL^{2.19}$	SL: 3.52 to 36.1
100 yr	$Q_{100} = 0.252 A^{1.11} SL^{2.19}$	
Region 7 (sites with contributing drainage area less than 32 square miles)[2]		
2 yr	$Q_2 = 832 A^{.568} SL^{-.285}$	A: 0.20 to 78.7
5 yr	$Q_5 = 584 A^{.610}$	
10 yr	$Q_{10} = 831 A^{.592}$	SH: 0.037 to 36.6
25 yr	$Q_{25} = 1196 A^{.576}$	
50 yr	$Q_{50} = 1505 A^{.566}$	SL: 7.25 to 116
100 yr	$Q_{100} = 1842 A^{.558}$	
Region 7 (sites with contributing drainage area greater than 32 square miles)[2]		

Table 1.20 Regression Equations for Estimation of Peak-Streamflow Frequency for Hydrologic Regions of Texas		
[yr, year; A, contributing drainage area in square miles; SH, basin shape factor – ration of length of longest mapped channel (stream length) squared to contributing drainage area (dimensionless); SL, stream slope in feet per mile – ration of change in elevation of (1) longest mapped channel from site (or station) to headwaters to (2) length of longest mapped channel]		
Hydrologic region and recurrence interval	**Weighted least-squares regression equation for corresponding recurrence interval**	**Range of indicated independent variables in corresponding region (units as noted)**
2 yr	$Q_2 = 129\ A^{.578}\ SL^{.364}$	A: 13 to 2,615
5 yr	$Q_5 = 133\ A^{.605}\ SL^{.578}$	
10 yr	$Q_{10} = 178\ A^{.644}\ SL^{.699}\ SH^{-.239}$	SH: 1.66 to 36.6
25 yr	$Q_{25} = 219\ A^{.651}\ SL^{.776}\ SH^{-.267}$	
50 yr	$Q_{50} = 261\ A^{.653}\ SL^{.817}\ SH^{-.291}$	SL: 3.85 to 31.9
100 yr	$Q_{100} = 313\ A^{.654}\ SL^{.849}\ SH^{-.316}$	
1. Source: U.S.G.S., 1997, pp. 62-65.		
2. Use Equation 1.29 to calculate a weighted discharge for streams with contributing drainage area falling within the arrange of 10 to 100 square miles.		

1.6.7. Example Problem

For the 100-year storm, calculate the peak discharge for a rural drainage area located in Region 7 on Timber Creek near Collinsville, Texas.

- Drainage Area = 38.8 mi^2
- Main Channel Slope = 13.13 ft/mi
- Main Channel Length= 14.24 mi.
- Shape Factor = $(\text{channel miles})^2$ divided by Area = 5.23

Peak Discharge Calculations

The 100-year storm Rural Peak Discharge determination for Region 7 will necessitate the use of Equation 1.30 because the drainage area is in the range of 10-100 square miles. The first step is to determine the discharge based on regression coefficients for areas greater than 32 square miles and less than 32 square miles. Table 1.20 provides the regression equations for Region 7 as follows;

For contributing drainage area less than 32 square miles,

$$\begin{aligned} Q_1 &= 1842\ A^{.558} \\ &= 1842(38.8)^{.558} \\ &= 14{,}186 \text{ cfs} \end{aligned}$$

For contributing drainage area greater than 32 square miles,

$$\begin{aligned} Q_2 &= 313\ A^{.654}\ SL^{.849}\ SH^{-.316} \\ &= 313(38.8)^{.654}\ (13.13)^{.849}\ (5.23)^{-.316} \\ &= 18{,}072 \text{ cfs} \end{aligned}$$

Equation 1.30 is then used to determine the 100-year storm Rural Peak Discharge.

$$\begin{aligned} Q_{100} &= (2-\log(A))Q_1 + (\log(A)-1)Q_2 \\ &= (2 - \log(38.8))14{,}186 + (\log(38.8)-1)\ 18{,}072 \text{ cfs} \\ &= 16{,}474 \text{ cfs} \end{aligned}$$

1.7. References

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2.0 Downstream Assessment

2.1. Introduction

The downstream impacts of development must be carefully evaluated. The purpose of the downstream assessment is to protect downstream properties from increased flooding and downstream channels from increased erosion potential due to upstream development. The importance of the downstream assessment is particularly evident for larger sites or developments that have the potential to dramatically impact downstream areas. The cumulative effect of smaller sites, however, can be just as dramatic.

The assessment should extend from the outfall of a proposed development to a point downstream where the discharge from a proposed development no longer has a significant impact on the receiving stream or storm drainage system. The assessment should be a part of the concept, preliminary, and final iSWM plans, and should include the following properties:

- Hydrologic analysis of the pre- and post-development on-site conditions
- Drainage path which defines extent of the analysis.
- Capacity analysis of all existing constraint points along the drainage path, such as existing floodplain developments, underground storm drainage systems culverts, bridges, tributary confluences, or channels
- Offsite undeveloped areas are considered as “full build-out” for both the pre- and post-development analyses
- Evaluation of peak discharges and velocities for three (3) 24-hour storm events
    - “Streambank Protection” Storm
    - “Conveyance” storm
    - “Flood Mitigation” storm
- Separate analysis for each major outfall from the proposed development

Once the analysis is complete, the designer should ask the following three questions at each determined junction downstream:

- Are the post-development discharges greater than the pre-development discharges?
- Are the post-development velocities greater than the pre-development velocities?
- Are the post-development velocities greater than the velocities allowed for the receiving system?

These questions should be answered for each of the three storm events. The answers to these questions will determine the necessity, type, and size of non-structural and structural controls to be placed on-site or downstream of the proposed development.

2.2. Downstream Hydrologic Assessment

Common practice requires the designer to control peak flow at the outlet of a site such that post-development peak discharge equals pre-development peak discharge. It has been shown that in certain cases this does not always provide effective water quantity control downstream from the site and may actually exacerbate flooding problems downstream. The reasons for this have to do with (1) the timing of the flow peaks, and (2) the total increase in volume of runoff.

Due to a site’s location within a watershed, there may be very little reason for requiring flood control from a particular site. In certain circumstances where detention is in place or a master drainage plan has been adopted, a development may receive or plan to receive less that ultimate developed flow conditions from upstream. This might be considered in the detention needed and its influence on the downstream assessment. Any consideration in such an event would be with the approval of the local authority. This section outlines a suggested procedure for determining the impacts of post-development stormwater peak flows and volumes that a community may require as part of a developer's stormwater management site plan.

2.3. Reasons for Downstream Problems

Flow Timing

If water quantity control (detention) structures are indiscriminately placed in a watershed and changes to the flow timing are not considered, the structural control may actually increase the peak discharge downstream. The reason for this may be seen in Figure 2.1. The peak flow from the site is reduced appropriately, but the timing of the flow is such that the combined detained peak flow (the larger dashed triangle) is actually higher than if no detention were required.

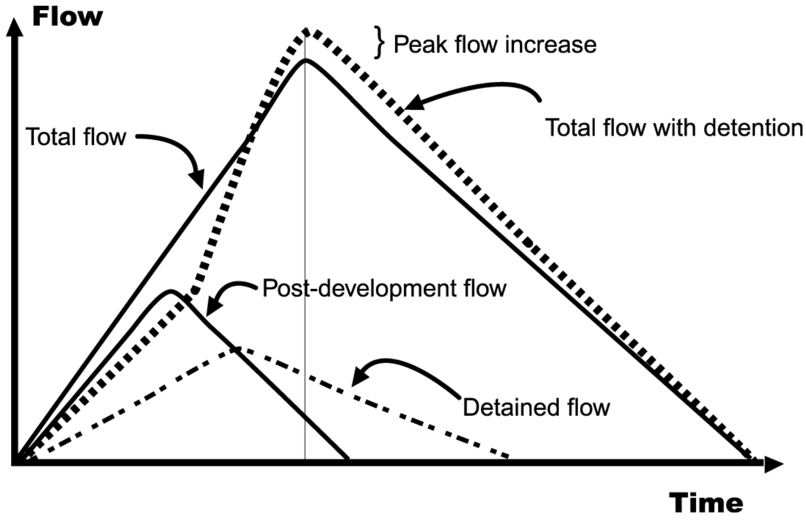


Figure 2.1 Detention Timing Example

In this case, the shifting of flows to a later time brought about by the detention pond actually makes the downstream flooding worse than if the post-development flows were not detained. This is most likely to happen if detention is placed on tributaries towards the bottom of the watershed, holding back peak flows and adding them as the peak from the upper reaches of the watershed arrives.

Increased Volume

An important impact of new development is an increase in the total runoff volume of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with downstream tributaries to increase the downstream peak flows.

Figure 2.2 illustrates this concept. The figure shows the pre- and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. In this case, the detention volume would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

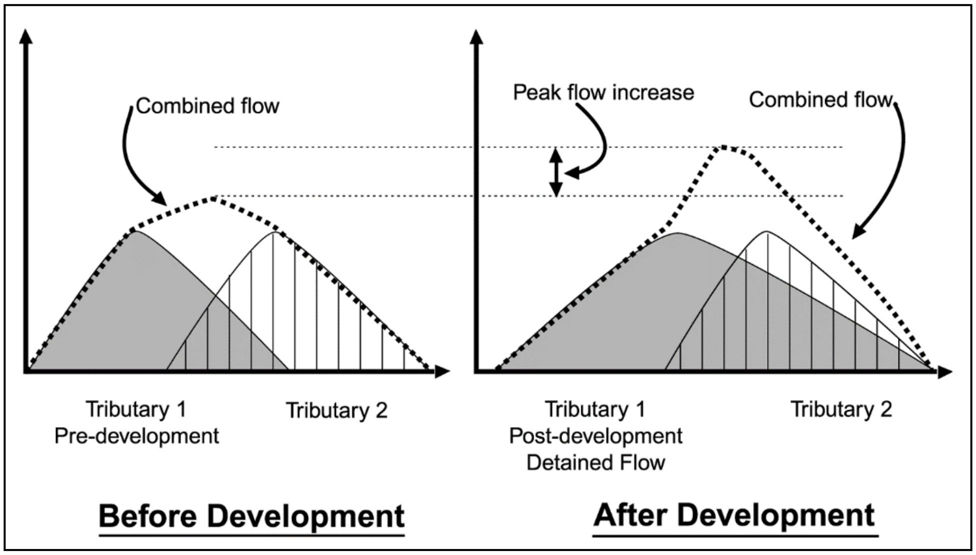


Figure 2.2 Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph

2.4. Methods for Downstream Evaluation

The downstream assessment is a tool by which the impacts of development on stormwater peak flows and velocities are evaluated downstream. The assessment extends from an outfall of a development to a point downstream, determined by one of two methods:

- *Zone of Influence* – Point downstream where the discharge from a proposed development no longer has a significant impact upon the receiving stream or storm drainage system
- *Adequate Outfall* – Location of acceptable outfall that does not create adverse flooding or erosion conditions downstream

These methods recognize the fact that a structural control providing detention has a "zone of influence" downstream where its effectiveness can be felt. Beyond this zone of influence the stormwater effects of a structural control become relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, a general rule of thumb is that the zone of influence can be considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. This is known as the *10% Rule*. As an example, if a structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

Typical steps in a downstream assessment include:

1. Determine the outfall location of the site and the pre- and post-development site conditions.
2. Using a topographic map determine a preliminary lower limit of the zone of influence (approximately 10% point).
3. Using a hydrologic model determine the pre-development peak flows and velocities at each junction beginning at the development outfall and ending at the next junction beyond the 10%

point. Undeveloped off-site areas are modeled as "full build-out" for both the pre- and post-development analyses. The discharges and velocities are evaluated for three storms:

- "Streambank Protection" storm
- "Conveyance" storm
- "Flood Mitigation" storm

4. Change the land use on the site to post-development conditions and rerun the model.
5. Compare the pre- and post-development peak discharges and velocities at the downstream end of the model. If the post-developed flows are higher than the pre-developed flows for the same frequency event, or the post-developed velocities are higher than the allowable velocity of the downstream receiving system, extend the model downstream. Repeat steps 3 and 4 until the post-development flows are less than the pre-developed flows, and the post-developed velocities are below the allowable velocity. Allowable velocities are given in *Table 3.2 of the Hydraulics Technical Manual.*
6. If shown that no peak flow increases occur downstream, and post-developed velocities are allowable, then the control of the flood protection volume (Q_f) can be waived by the local authority. The developer saves the cost of sizing a detention basin for flood control. In this case the downstream assessment saved the construction of an unnecessary structural control facility that would have been detrimental to the watershed flooding problems. In some communities this situation may result in a fee being paid to the local government in lieu of detention. That fee would go toward alleviating downstream flooding or making channel or other conveyance improvements.
7. If peak discharges are increased due to development, or if downstream velocities are erosive, one of the following options are required.
    - Document that existing downstream conveyance is adequate to convey post-developed stormwater discharges (Option 1 for Streambank Protection and Flood Control)
    - Work with the local government to reduce the flow elevation and/or velocity through channel or flow conveyance structure improvements downstream. (Option 2 for Streambank Protection and Flood Control)
    - Design an on-site structural control facility such that the post-development flows do not increase the peak flows, and the velocities are not erosive, at the outlet and the determined junction locations.

Even if the results of the downstream assessment indicate that no downstream flood or erosion protection is required, the water quality steps of the *integrated* Design Approach will still need to be addressed.

2.5. Example Problem

Figure 2.3 illustrates the concept of the ten-percent rule for two sites in a watershed.

Discussion
Site A is a development of 10 acres, all draining to a wet Extended Detention (ED) stormwater pond. The flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked "80 acres." The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The assumption is that if there is no peak flow increase or erosive velocities at the 80-acre point then the same will be true through the next stream reach downstream through the 10% point (100 acres) to the 120-acre point. The designer constructs a simple HEC-1 model of the 80-acre areas using single, "full build-out" condition sub-watersheds for each tributary. Key detention structures existing in other tributaries must be modeled. An approximate curve number is used since the *actual* peak flow is not key for initial analysis;

only the increase or decrease is important. The accuracy in curve number determination is not as significant as an accurate estimate of the time of concentration. Since flooding is an issue downstream, the pond is designed (through several iterations) until the peak flow does not increase, and velocities are not erosive, at junction points downstream to the 80-acre point.

Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate the time of concentration for input into the TR-20 model of the watershed. The model shows a detention facility, in this case, will actually <u>increase</u> the peak flow in the stream.

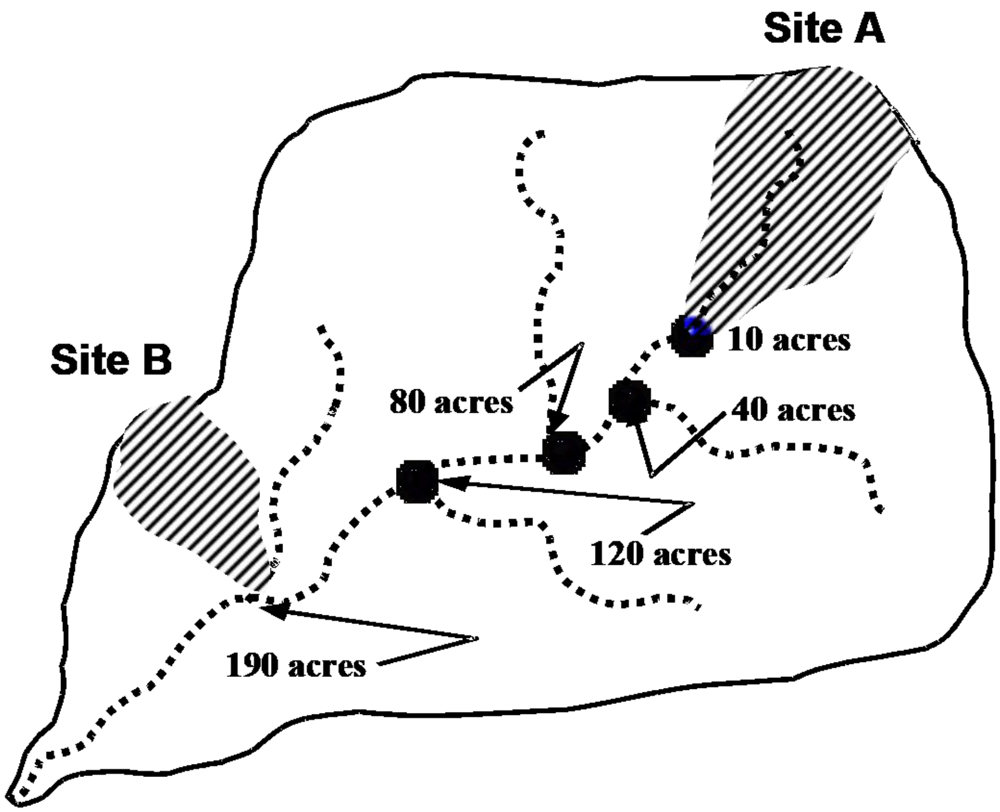


Figure 2.3 Example of the Ten-Percent Rule

3.0 Streambank Protection Volume Estimation

3.1. Streambank Protection Volume Calculation

The Simplified SCS Peak Runoff Rate Estimation approach (see *Section 1.3.7*) can be used for estimation of the Streambank Protection Volume (SP_v) for storage facility design.

This method should not be used for standard detention design calculations. See the modified rational method in *Section 1.5* for preliminary detention calculations without formal routing or the SCS Hydrologic Method in *Section 1.3.*

For SP_v estimation, using Figure 1.10, the unit peak discharge (q_U) can be determined based on I_a/P and time of concentration (t_C). Knowing q_U and T (extended detention time, typically 24 hours), the q_O/q_i ratio (peak outflow discharge/peak inflow discharge) can be estimated from Figure 3.1.

Using the following equation from TR-55 for a Type II rainfall distribution, V_S/V_r can be calculated.

Note: Figure 3.2 can also be used to estimate V_S/V_r.

$$V_S/V_r = 0.682 - 1.43\,(q_O/q_i) + 1.64\,(q_O/q_i)^2 - 0.804\,(q_O/q_i)^3 \quad (3.1)$$

where:

V_S = required storage volume (acre-feet)
V_r = runoff volume (acre-feet)
q_O = peak outflow discharge (cfs)
q_i = peak inflow discharge (cfs)

The required storage volume can then be calculated by:

$$V_S = \frac{(V_S/V_r)(Q_d)(A)}{12} \quad (3.2)$$

where:

V_S and V_r are defined above
Q_d = the developed runoff for the design storm (inches)
A = total drainage area (acres)

While the TR-55 short-cut method reports to incorporate multiple stage structures, experience has shown that an additional 10-15% storage is required when multiple levels of extended detention are provided inclusive with the 25-year storm.

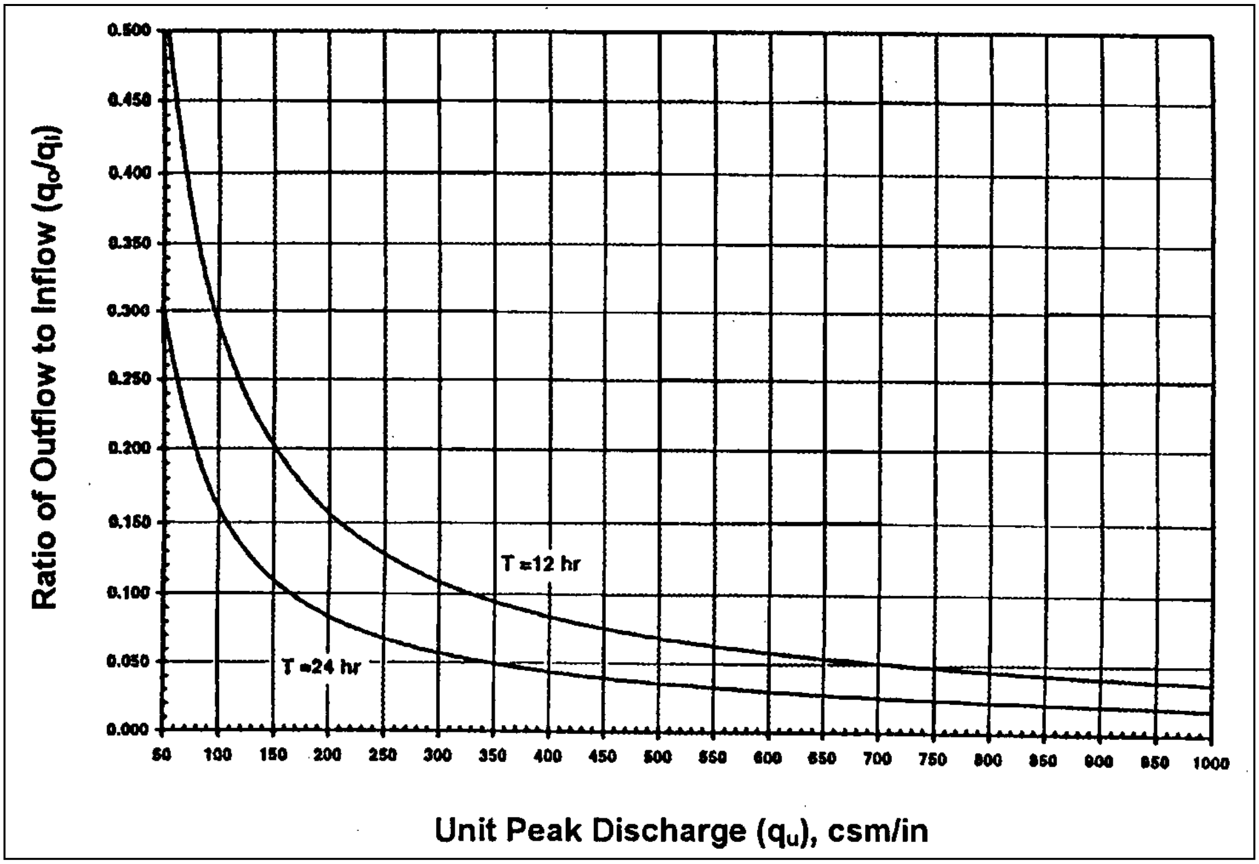


Figure 3.1 Detention Time vs. Discharge Ratios

(Source: MDE, 1998)

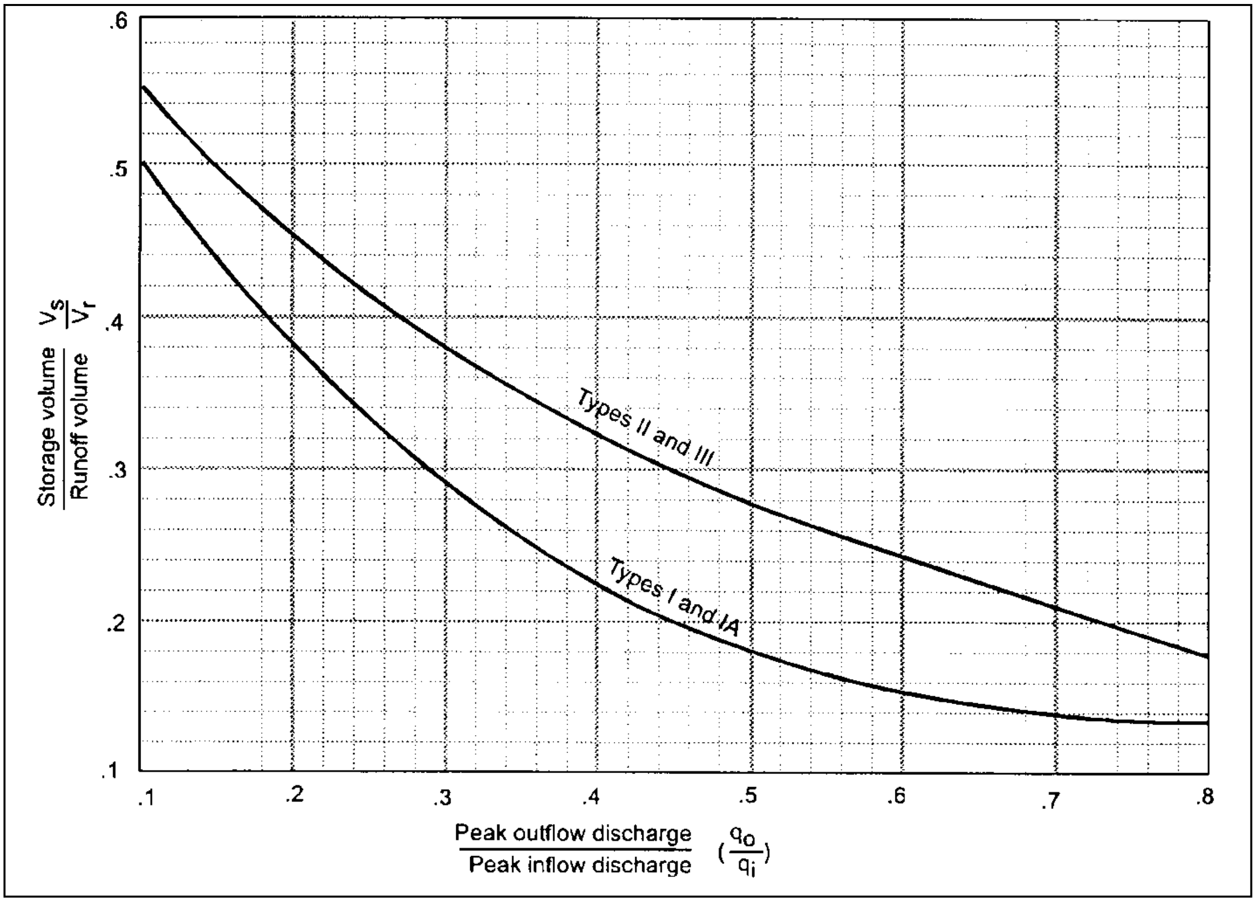


Figure 3.2 Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III

(Source: TR-55, 1986)

3.2. Example Problem

Compute the Streambank Protection Volume (SP_v) for the 50-acre watershed in *Section 1.3.8 Example Problem One.*

Computations

1. Calculate rainfall excess:
   - The 1-year, 24 hour rainfall is 2.64 inches (0.11 in/hr x 24 hours – From Table 5.16).
   - Composite area-weighted Curve Number is 83.
   - From Equation 2.1.7, Q_d (1-year developed) = 1.2 inches

2. Calculate time of concentration
   t_c = 20.86 minutes (.35 hours)

3. Calculate I_a/P for CN = 83; I_a = .410 (Table 1.11)
   I_a/P = (.410 / 2.64) = .155 (Note: Use straight-line interpolation to facilitate use of Figure 1.10)

4. Find unit discharge q_u:
   From Figure 1.10 for I_a/P = .155 and t_c = .35 hr
   q_u = 600 csm/in

5. Find discharge ratio q_O/q_I:
   From Figure 3.1 for q_u = 600 csm/in and T = 24 hr
   $q_O/q_I = 0.03$

6. Calculate streambank protection volume ($SP_v = V_S$)
   For a Type II rainfall distribution,
   $V_S/V_r = 0.682 – 1.43 (q_O/q_I) + 1.64 (q_O/q_I)^2 – 0.804 (q_O/q_I)^3$
   $V_S/V_r = 0.682 – 1.43 (0.03) + 1.64 (0.03) – 0.804 (0.03) = 0.64$

Therefore, streambank protection volume with Q_d (1-year developed) = 1.2 inches, from Step 1, is

$SP_v = V_S = (V_S/V_r)(Q_d)(A)/12 = (0.64)(1.2)(50)/12 = 3.20$ acre-feet

4.0 Water Balance

4.1. Introduction

Water balance calculations can help determine if a drainage area is large enough, or has the right characteristics, to support a permanent pool of water during average or extreme conditions. When in doubt, a water balance calculation may be advisable for retention pond and wetland design.

The details of a rigorous water balance are beyond the scope of this manual. However, a simplified procedure is described herein to provide an estimate of pool viability and point to the need for more rigorous analysis. Water balance can also be used to help establish planting zones in a wetland design.

4.2. Basic Equations

Water balance is defined as the change in volume of the permanent pool resulting from the total inflow minus the total outflow (actual or potential):

$$\Delta V = \Sigma I - \Sigma O \qquad (4.1)$$

where:

Δ = "change in"

V = pond volume (ac-ft)

Σ = "sum of"

I = Inflows (ac-ft)

O = Outflows (ac-ft)

The inflows consist of rainfall, runoff, and baseflow into the pond. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the pond or wetland. Equation 4.1 can be changed to reflect these factors.

$$\Delta V = P + R_o + B_f - I - E - E_t - O_f \qquad (4.2)$$

where:

V = volume (ac-ft)

P = precipitation (ac-ft) = (Rainfall in Inches times area in acres divided by 12)

R_o = runoff (ac-ft)

B_f = baseflow (ac-ft)

I = infiltration (ac-ft) (Use Equation 4.4)

E = evaporation (ac-ft) (Surface evaporation in feet times surface area)

E_t = evapotranspiration (ac-ft)

O_f = overflow (ac-ft)

Δ = "change in" (+ gain; - loss)

Rainfall (P) – Monthly rainfall values can be obtained from National Weather Service climatology data at:

http://www.srh.noaa.gov/fwd/ntexclima.html

Monthly values are commonly used for calculations of values over a season. Rainfall is then the direct amount that falls on the pond surface for the period in question. When multiplied by the pond surface area (in acres) and divided by 12, it becomes acre-feet of volume. Table 4.1 shows monthly rainfall rates for the Dallas-Fort Worth area based on a 30-year period of record at Dallas-Fort Worth International Airport.

Runoff (R_o) – Runoff is equivalent to the rainfall for the period times the "efficiency" of the watershed, which is equal to the ratio of runoff to rainfall. In lieu of gage information, Q/P can be estimated one of several ways. The best method would be to perform long-term simulation modeling using rainfall records and a watershed model. Two other methods have been proposed.

Equation 1.1 of the Water Quality Technical Manual gives the volumetric coefficient (R_v) of the drainage area. If it can be assumed that the average storm producing runoff has a similar ratio, then the R_v value can serve as the ratio of rainfall to runoff. Not all storms produce runoff in an urban setting. Typical initial losses (often called "initial abstractions") are normally taken between 0.1 and 0.2 inches. When compared to the rainfall records in Texas, this is equivalent of about a 10% runoff volume loss. Thus a factor of 0.9 should be applied to the calculated R_v value to account for storms producing no runoff. Equation 4.3 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

$$R_o = 0.9(P/12)R_vA \qquad (4.3)$$

where:

- P = precipitation (in)
- R_o = runoff volume (acre-ft)
- R_v = volumetric runoff coefficient [see *Equation 1.1 of the Water Quality Technical Manual*]
- A = Area in acres

Table 4.1 Monthly Precipitation Values

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Precipitation(in)	1.90	2.37	3.06	3.20	5.15	3.23	2.12	2.03	2.42	4.11	2.57	2.57
Annual Precipitation (in) 34.73												

Source: National Weather Service, 2002

Baseflow (B_f) – Most stormwater ponds and wetlands have little, if any, baseflow, as they are rarely placed across perennial streams. If so placed, baseflow must be estimated from observation or through theoretical estimates. Methods of estimation and baseflow separation can be found in most hydrology textbooks. Consideration may also have to be given to irrigation return flow in certain areas.

Infiltration (I) – Infiltration is a very complex subject and cannot be covered in detail here. The amount of infiltration depends on soils, water table depth, rock layers, surface disturbance, the presence or absence of a liner in the pond, and other factors. The infiltration rate is governed by the Darcy equation as:

$$I = Ak_hG_h \qquad (4.4)$$

where:

- I = infiltration (ac-ft/day)
- A = cross sectional area through which the water infiltrates (ac)
- K_h = saturated hydraulic conductivity or infiltration rate (ft/day)
- G_h = hydraulic gradient = pressure head/distance

G_h can be set equal to 1.0 for pond bottoms and 0.5 for pond sides steeper than about 4:1. Infiltration rate can be established through testing, though not always accurately. As a first cut estimate Table 4.2 can be used.

Table 4.2 Saturated Hydraulic Conductivity		
Material	**Hydraulic Conductivity**	
	in/hr	**ft/day**
ASTM Crushed Stone No. 3	50,000	100,000
ASTM Crushed Stone No. 4	40,000	80,000
ASTM Crushed Stone No. 5	25,000	50,000
ASTM Crushed Stone No. 6	15,000	30,000
Sand	8.27	16.54
Loamy sand	2.41	4.82
Sandy loam	1.02	2.04
Loam	0.52	1.04
Silt loam	0.27	0.54
Sandy clay loam	0.17	0.34
Clay loam	0.09	0.18
Silty clay loam	0.06	0.12
Sandy clay	0.05	0.10
Silty clay	0.04	0.08
Clay	0.02	0.04
Source: Ferguson and Debo, "On-Site Stormwater Management," 1990		

Evaporation (E) – Evaporation is from an open lake water surface. Evaporation rates are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, water purity, and shape and depth of the pond. It is estimated or measured in a number of ways, which can be found in most hydrology textbooks. Pan evaporation methods are also used. A pan coefficient of 0.7 is commonly used to convert the higher pan value to the lower lake values.

Table 4.3 gives pan evaporation rate distributions for a typical 12-month period based on pan evaporation information for Grapevine, Texas. Figure 4.1 depicts a map of annual free water surface (FWS) evaporation averages for Texas based on a National Oceanic and Atmospheric Administration (NOAA) assessment done in 1982. FWS evaporation differs from lake evaporation for larger and deeper lakes, but can be used as an estimate of it for the type of structural stormwater ponds and wetlands being designed in Texas. Total annual values can be estimated from this map and distributed according to Table 4.3.

Table 4.3 Evaporation Monthly Distribution - Grapevine, Texas as a % of Annual Total

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
3.1%	4.0%	7.2%	8.7%	10.3%	12.4%	14.5%	13.9%	9.8%	7.4%	4.9%	3.9%

Evapotranspiration (E_t). Evapotranspiration consists of the combination of evaporation and transpiration by plants. The estimation of E_t for crops in Texas is well documented and has become standard practice. However, for wetlands the estimating methods are not documented, nor are there consistent studies to assist the designer in estimating the demand wetland plants would put on water volumes. Literature values for various places in the United States vary around the free water surface lake evaporation values. Estimating E_t only becomes important when wetlands are being designed and emergent vegetation covers a significant portion of the pond surface. In these cases conservative estimates of lake evaporation should be compared to crop-based E_t estimates and a decision made. Crop-based E_t estimates can be obtained from typical hydrology textbooks or from the web site mentioned above.

Overflow (O_f) – Overflow is considered as excess runoff, and in water balance design is either not considered, since the concern is for average values of precipitation, or is considered lost for all volumes above the maximum pond storage. Obviously, for long-term simulations of rainfall-runoff, large storms would play an important part in pond design.

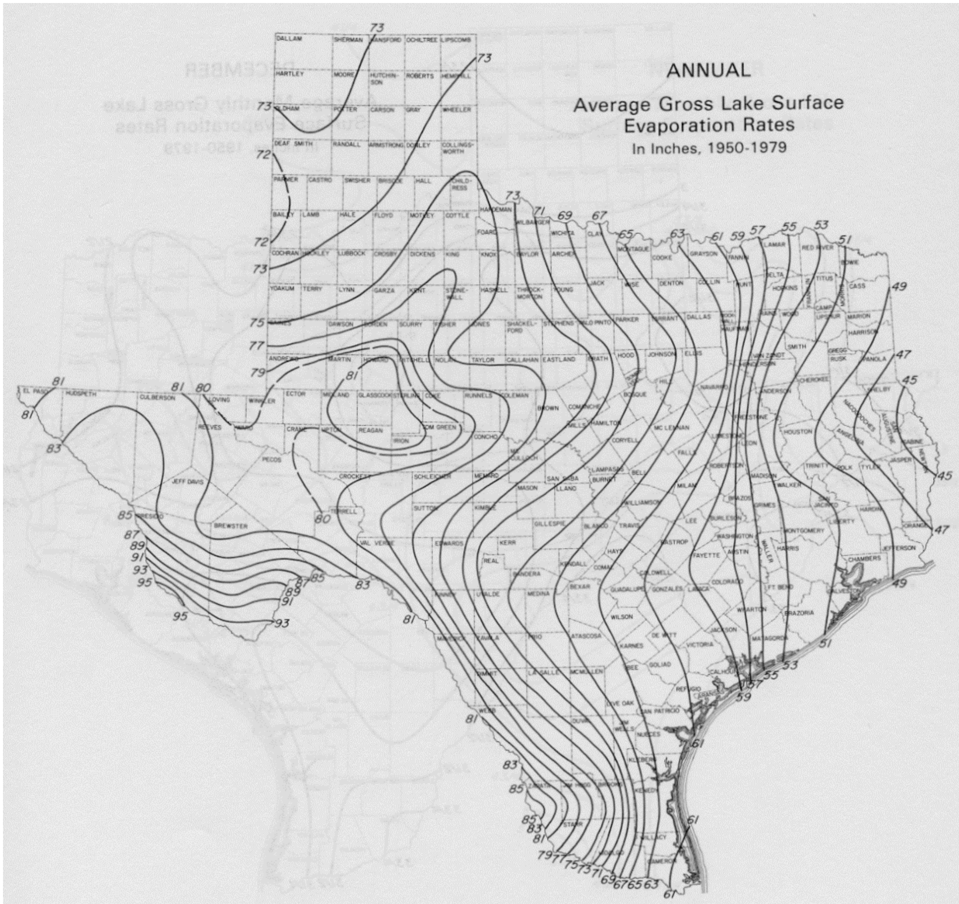


Figure 4.1 Average Annual Free Water Surface Evaporation (in inches)

(Source: NOAA, 1982)

4.3. Example Problem

A 26-acre site in North Dallas is being developed along with an estimated 0.5-acre surface area pond. There is no baseflow. The desired pond volume to the overflow point is 2 acre-feet. Will the site be able to support the pond volume? From the basic site data, we find the site is 75% impervious with clay loam soil.

- From *Equation 1.1 of the Water Quality Technical Manual*, R_v = 0.05 + 0.009 (75) = 0.73. With the correction factor of 0.9 the watershed efficiency is 0.65.
- The annual lake evaporation from Figure 4.1 is about 64 inches.
- For a clay loam the infiltration rate is I = 0.18 ft/day (Table 4.2).
- From a grading plan, it is known that about 10% of the total pond area is sloped greater than 1:4.
- Monthly rainfall for Dallas was found from a Web site similar to the one provided above.

Table 4.4 shows summary calculations for this site for each month of the year.

Table 4.4 Summary Information for the North Dallas Site

Drainage Area (Acres)	26
Pond Surface (Acres)	0.5
Volume at Overflow (Ac-Ft)	2
Watershed Efficiency	0.65
Annual Rainfall	34.73
Infiltration Rate (In/Day)	0.18 Clay Loam
% Pond Bottom Flat (Acres)	90
% Pond Bottom > 1:4 (Acres)	10
Annual Lake Evaporation (in)	64 Assume Pond Starts Full

1	Months of Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2	Days Per Month	31	28	31	30	31	30	31	31	30	31	30	31
3	Monthly Precipitation	1.9	2.37	3.06	3.2	5.15	3.23	2.12	2.03	2.42	4.11	2.57	2.57
4	Evaporation - % of Yr	3.1	4	7.2	8.7	10.3	12.4	14.5	13.9	9.8	7.4	4.9	3.9
5	Runoff (Ac-Ft)	2.68	3.34	4.31	4.51	7.25	4.55	2.99	2.86	3.41	5.79	3.62	3.62
6	Precipitation (Ac-Ft)	0.08	0.10	0.13	0.13	0.21	0.13	0.09	0.08	0.10	0.17	0.11	0.11
7	Evaporation (Ac-Ft)	0.08	0.11	0.19	0.23	0.27	0.33	0.39	0.37	0.26	0.20	0.13	0.10
8	Infiltration (Ac-Ft)	2.65	2.39	2.65	2.57	2.65	2.57	2.65	2.65	2.57	2.65	2.57	2.65
9													
10	Balance (Ac-Ft)	0.02	0.94	1.59	1.84	4.54	1.79	0.04	-0.08	0.68	3.11	1.03	0.97
11	Running Balance (Ac-Ft)	2.00	2.00	2.00	2.00	2.00	2.00	2.00	1.92	2.00	2.00	2.00	2.00

Explanation of Table line number:

1. Months of year
2. Days per month
3. Monthly precipitation
4. Distribution of evaporation by month
5. In the example, watershed efficiency of 0.65 times the rainfall and area (in acres) and converted to acre-feet. The Watershed efficiency must be determined for each watershed.
6. Precipitation volume directly into pond equals precipitation depth times pond surface area divided by 12 to convert to acre-feet

7. Evaporation equals the monthly percentage of the annual gross lake evaporation in inches converted to acre-feet
8. Infiltration equals infiltration rate times 90% of the surface area plus infiltration rate times 0.5 (banks greater than 1:4) times 10% of the pond area converted to acre-feet
10. Lines 5 and 6 minus lines 7 and 8
11. Accumulated total from line 10 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design. Each pond has a unique volume at which overflows occur and it would be used for line 11. The pond volume in January should be set equal to the expected end-of-year volume.

It can be seen that, for this example, the pond has potential to maintain a wet pond in all months. Had the soil been a sandy clay loam with an infiltration rate of 0.34 inches per day, the pond would have been dry most months of the year. Excessive infiltration rates may be remedied in a number of ways including compacting the pond bottom, placing a liner of clay or geosynthetics, and changing the pond geometry to decrease surface area.

Climatic data for North Texas, as that in Figure 4.2, can be obtained from the following web site: http://www.srh.noaa.gov/fwd/CLIMO/dfw/normals/dfwann.html.

DFW Annual Summary of Normal, Means, and Extremes

Rain (in.)

		POR	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	YEAR
Normal		30	1.90	2.37	3.06	3.20	5.15	3.23	2.12	2.03	2.42	4.11	2.57	2.57	34.73
Monthly Maximum		45	5.07	7.40	6.69	12.19	13.66	8.75	11.13	6.85	9.52	14.18	6.23	8.75	14.18
Year of Occurrence			1998	1997	1995	1957	1982	1989	1973	1970	1964	1981	1964	1991	Oct 1981
Minimum Monthly		45	T	0.15	0.10	0.11	0.95	0.40	0	T	0.09	T	0.20	0.17	0
Year of Occurrence			1986	1963	1972	1987	1996	1964	1993	1980	1984	1975	1970	1981	Jul 1993
Max in 24 hours		45	3.15	4.06	4.39	4.55	5.34	3.15	3.76	4.05	4.76	5.91	2.83	422	5.91
Year of Occurrence			1998	1965	1977	1957	1989	1989	1975	1976	1965	1959	1964	1991	Oct 1959
Number of Days with…	Precipitation > Tr.	30	6.7	6.3	7.3	7.6	8.7	6.4	4.7	4.6	7.1	6.2	6.0	6.5	78.1
	Precipitation > 0.99	30	0.3	05	0.7	1.2	1.4	0.9	0.7	0.8	1.1	1.4	0.6	0.4	10.0

Figure 4.2 Dallas/Fort Worth Precipitation Information

4.4. References

Federal Highway Administration, 1983. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14.

Federal Highway Administration, 1967. Use of Riprap for Bank Protection. Hydraulic Engineering Circular No. 11.

Searcy, James K., 1967. Use of Riprap for Bank Protection. Federal Highway Administration.

USDA, SCS, 1975. Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas, College Park, Maryland.

U.S. Department of Interior, Bureau of Reclamation, 1978. Design of Small Canal Structures.

5.0 Rainfall Tables

5.1. Methodology

Rainfall tables are based on the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 for the counties within the jurisdiction of NCTCOG. NOAA Atlas 14 is produced by the NOAA's National Weather Service, Hydrometeorological Design Studies Center and may be considered as the national standard for precipitation frequency estimates. NOAA Atlas 14 is published in volumes for different geographic areas of the US. The final version of NOAA Atlas 14 Volume 11 for Texas was released on September 2018 and has been peer-reviewed extensively. Volume 11 provides precipitation frequency estimates, upper and lower bounds for 90% confidence intervals for durations of 5-minute through 60-day and recurrence intervals of 1, 2, 5, 10, 25, 50, 100, 200, 500 and 1,000-year for the State of Texas.

Precipitation frequency estimates are computed using regional frequency analysis based on L-moment statistics calculated from annual maxima series (AMS). NOAA Atlas 14 employs a regionalization approach wherein the L-moment statistics are calculated by grouping stations within a 60-mile radius. This results in 700 to 1,800 years of data for daily durations and 200 to 700 years for hourly durations. Several distribution functions were examined and ultimately the generalized extreme value (GEV) distribution was adopted for fall stations and durations. The upper and lower 90% confidence intervals are based on a Monte-Carlo simulation approach.

Gridded precipitation frequency estimates at a spatial resolution of 30-arc second are also available for all durations and recurrence intervals discussed above. The gridded frequency estimates are generated from grids of mean annual maxima which are derived from at-stations mean annual maxima using the PRISM interpolation methodology (http://www.prism.oregonstate.edu/). The precipitation frequency grids are the basis of the NOAA Precipitation Frequency Data Server (PFDS) for retrieval of precipitation frequency estimates by co-ordinates.

Based on the review of the NOAA Atlas 14 methodology, it is apparent that a robust and standardized technique has been adopted for the development of precipitation frequency estimates. The methods and results have also been extensively peer reviewed. In addition, Atlas 14 also uses a long period of data for the development of frequency estimates (average record length of approximately 60 years) and more recent data, as available.

Rainfall tables have been generated at the center of each county within the jurisdiction of the North Central Texas Council of Governments (NCTCOG). AMS based precipitation frequency grids for all available recurrence intervals and durations were first downloaded from the NOAA PFDS[1]. Subsequently, at the centroid of each county, the value associated with each duration and recurrence interval was extracted from the respective AMS frequency grids in ArcGIS. The compiled precipitation frequencies for each county in a tabular format are provided below. Note that the estimates for 1-yr recurrence interval are based on the frequency analysis of partial duration series (PDS).

While precipitation frequency estimates at the centroid of a county is a reasonable approach for summarizing the NOAA Atlas 14 data, these estimates may not be representative of smaller areas such as a metropolitan area or census block. For such cases, precipitation frequency estimates may be directly downloaded from the NOAA PFDS[2] by specifying the desired geographic co-ordinates.

5.2. References

Sanja Perica, Sandra Pavlovic, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Orlan Wilhite (2018). *NOAA Atlas 14 Volume 11 Version 2, Precipitation-Frequency Atlas of the United States, Texas*. NOAA, National Weather Service, Silver Spring, MD.

[1] https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_gis.html
[2] https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html

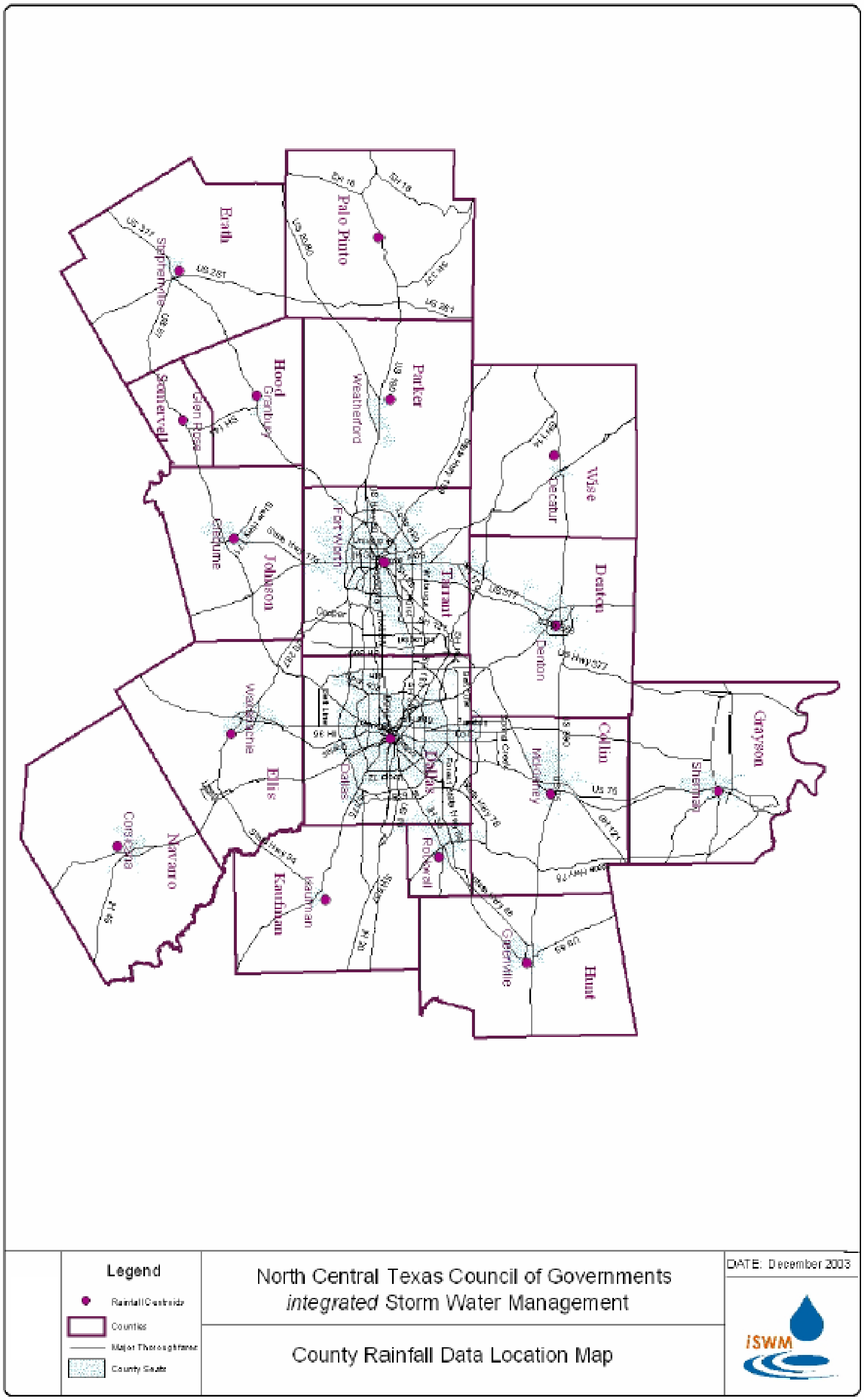


Figure 5.1 County Rainfall Data Location Map

Table 5.1 AMS-Based Precipitation Frequency Estimates for Collin County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.430	0.462	0.588	0.683	0.806	0.897	0.987	1.077	1.198	1.289
10-min	0.687	0.739	0.943	1.095	1.293	1.441	1.585	1.725	1.904	2.034
15-min	0.858	0.921	1.171	1.358	1.601	1.779	1.956	2.133	2.369	2.547
30-min	1.195	1.282	1.625	1.881	2.212	2.453	2.693	2.942	3.278	3.537
60-min	1.554	1.670	2.128	2.470	2.915	3.240	3.569	3.915	4.390	4.763
2-hr	1.898	2.056	2.666	3.130	3.749	4.215	4.696	5.206	5.915	6.478
3-hr	2.100	2.287	3.001	3.550	4.292	4.861	5.456	6.088	6.968	7.671
6-hr	2.482	2.719	3.615	4.311	5.263	6.005	6.788	7.623	8.791	9.729
12-hr	2.938	3.224	4.302	5.138	6.281	7.171	8.113	9.121	10.539	11.682
24-hr	3.456	3.790	5.054	6.032	7.364	8.395	9.487	10.666	12.334	13.684
48-hr	4.032	4.412	5.855	6.968	8.477	9.634	10.864	12.210	14.135	15.707
3-day	4.407	4.816	6.376	7.578	9.207	10.453	11.778	13.237	15.328	17.040
4-day	4.670	5.105	6.757	8.032	9.767	11.100	12.519	14.078	16.311	18.138
7-day	5.221	5.714	7.568	9.011	10.992	12.537	14.187	15.975	18.518	20.585
10-day	5.708	6.243	8.259	9.827	11.982	13.669	15.461	17.387	20.103	22.297
20-day	7.470	8.096	10.503	12.339	14.795	16.656	18.585	20.636	23.488	25.762
30-day	8.964	9.660	12.384	14.433	17.116	19.099	21.115	23.237	26.151	28.446
45-day	11.030	11.840	15.054	17.445	20.525	22.765	24.986	27.265	30.320	32.668
60-day	12.866	13.782	17.444	20.151	23.603	26.093	28.517	30.945	34.131	36.520

Table 5.2 AMS-Based Precipitation Frequency Estimates for Dallas County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.428	0.462	0.595	0.695	0.824	0.919	1.014	1.111	1.239	1.338
10-min	0.684	0.738	0.952	1.112	1.320	1.474	1.626	1.776	1.971	2.115
15-min	0.856	0.923	1.188	1.385	1.641	1.828	2.014	2.203	2.456	2.650
30-min	1.199	1.291	1.657	1.929	2.280	2.535	2.790	3.054	3.412	3.690
60-min	1.560	1.684	2.169	2.532	3.004	3.348	3.696	4.063	4.567	4.963
2-hr	1.903	2.068	2.702	3.185	3.827	4.310	4.809	5.339	6.072	6.656
3-hr	2.103	2.295	3.027	3.590	4.351	4.935	5.545	6.193	7.097	7.818
6-hr	2.466	2.707	3.613	4.318	5.286	6.043	6.847	7.705	8.913	9.885
12-hr	2.865	3.155	4.240	5.090	6.264	7.190	8.182	9.255	10.781	12.020
24-hr	3.316	3.657	4.930	5.929	7.312	8.401	9.577	10.864	12.713	14.229
48-hr	3.857	4.248	5.714	6.858	8.433	9.658	10.984	12.462	14.611	16.390
3-day	4.215	4.636	6.222	7.455	9.148	10.460	11.878	13.465	15.780	17.700
4-day	4.466	4.910	6.580	7.881	9.667	11.053	12.551	14.220	16.646	18.653
7-day	4.995	5.486	7.327	8.765	10.749	12.303	13.975	15.805	18.428	20.576
10-day	5.460	5.986	7.962	9.504	11.625	13.287	15.063	16.987	19.722	21.945
20-day	7.127	7.725	10.033	11.790	14.130	15.885	17.720	19.723	22.565	24.871
30-day	8.532	9.189	11.776	13.713	16.233	18.064	19.944	21.994	24.883	27.215
45-day	10.454	11.221	14.270	16.534	19.442	21.541	23.635	25.820	28.787	31.097
60-day	12.156	13.029	16.509	19.087	22.385	24.777	27.106	29.424	32.455	34.719

Table 5.3 AMS-Based Precipitation Frequency Estimates for Denton County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.419	0.452	0.584	0.682	0.808	0.901	0.992	1.085	1.207	1.299
10-min	0.671	0.725	0.936	1.094	1.298	1.448	1.595	1.737	1.920	2.055
15-min	0.836	0.902	1.162	1.356	1.606	1.788	1.967	2.148	2.387	2.568
30-min	1.162	1.252	1.610	1.875	2.217	2.463	2.708	2.959	3.297	3.557
60-min	1.508	1.627	2.100	2.452	2.907	3.237	3.569	3.915	4.388	4.757
2-hr	1.842	2.000	2.612	3.076	3.690	4.149	4.621	5.121	5.814	6.363
3-hr	2.040	2.223	2.927	3.465	4.189	4.741	5.315	5.927	6.779	7.460
6-hr	2.407	2.635	3.502	4.172	5.085	5.791	6.536	7.335	8.458	9.363
12-hr	2.833	3.106	4.143	4.944	6.033	6.872	7.760	8.723	10.088	11.194
24-hr	3.310	3.630	4.847	5.785	7.056	8.030	9.062	10.189	11.796	13.105
48-hr	3.838	4.206	5.606	6.684	8.143	9.258	10.440	11.732	13.573	15.074
3-day	4.180	4.580	6.102	7.275	8.867	10.088	11.382	12.794	14.806	16.444
4-day	4.420	4.848	6.465	7.718	9.429	10.753	12.162	13.696	15.880	17.659
7-day	4.922	5.413	7.241	8.677	10.672	12.253	13.954	15.795	18.416	20.548
10-day	5.362	5.901	7.896	9.470	11.665	13.416	15.302	17.335	20.222	22.567
20-day	6.930	7.566	9.970	11.833	14.377	16.347	18.442	20.713	23.939	26.559
30-day	8.254	8.968	11.704	13.798	16.614	18.752	20.998	23.431	26.872	29.657
45-day	10.090	10.933	14.181	16.657	19.966	22.475	25.066	27.801	31.580	34.576
60-day	11.723	12.687	16.403	19.233	23.012	25.891	28.826	31.841	35.918	39.080

Table 5.4 AMS-Based Precipitation Frequency Estimates for Ellis County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.429	0.465	0.607	0.714	0.856	0.963	1.072	1.186	1.341	1.462
10-min	0.685	0.744	0.971	1.143	1.372	1.546	1.721	1.896	2.130	2.307
15-min	0.859	0.932	1.215	1.429	1.709	1.917	2.128	2.349	2.652	2.891
30-min	1.202	1.302	1.691	1.984	2.368	2.651	2.940	3.250	3.680	4.024
60-min	1.559	1.691	2.205	2.595	3.111	3.497	3.897	4.329	4.937	5.427
2-hr	1.887	2.065	2.738	3.259	3.971	4.522	5.108	5.744	6.647	7.381
3-hr	2.074	2.283	3.060	3.671	4.520	5.192	5.917	6.703	7.824	8.739
6-hr	2.420	2.682	3.642	4.408	5.488	6.359	7.312	8.353	9.852	11.082
12-hr	2.806	3.119	4.264	5.179	6.474	7.520	8.674	9.954	11.819	13.366
24-hr	3.255	3.619	4.955	6.019	7.518	8.717	10.045	11.541	13.750	15.599
48-hr	3.831	4.242	5.772	6.973	8.639	9.936	11.371	13.034	15.534	17.655
3-day	4.213	4.651	6.296	7.578	9.341	10.698	12.192	13.934	16.557	18.787
4-day	4.476	4.933	6.655	7.995	9.835	11.252	12.803	14.592	17.263	19.519
7-day	5.020	5.517	7.386	8.842	10.842	12.397	14.075	15.937	18.635	20.862
10-day	5.497	6.024	8.009	9.551	11.661	13.301	15.048	16.949	19.656	21.860
20-day	7.189	7.772	10.042	11.757	14.017	15.686	17.421	19.320	22.014	24.201
30-day	8.622	9.255	11.777	13.646	16.039	17.739	19.469	21.368	24.050	26.217
45-day	10.615	11.354	14.332	16.520	19.280	21.224	23.140	25.146	27.864	29.978
60-day	12.389	13.236	16.653	19.159	22.310	24.542	26.687	28.818	31.592	33.651

Table 5.5 AMS-Based Precipitation Frequency Estimates for Erath County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.417	0.451	0.585	0.687	0.821	0.923	1.024	1.124	1.255	1.354
10-min	0.666	0.722	0.937	1.101	1.317	1.483	1.645	1.799	1.995	2.136
15-min	0.834	0.902	1.166	1.366	1.631	1.832	2.031	2.228	2.484	2.675
30-min	1.165	1.258	1.620	1.894	2.255	2.528	2.801	3.075	3.439	3.716
60-min	1.509	1.634	2.118	2.485	2.971	3.335	3.706	4.088	4.605	5.008
2-hr	1.827	1.997	2.652	3.151	3.820	4.326	4.853	5.416	6.202	6.833
3-hr	2.007	2.209	2.980	3.571	4.369	4.977	5.618	6.315	7.303	8.104
6-hr	2.326	2.581	3.547	4.295	5.315	6.101	6.941	7.864	9.189	10.274
12-hr	2.644	2.951	4.099	4.997	6.237	7.210	8.257	9.407	11.060	12.416
24-hr	3.016	3.371	4.695	5.733	7.173	8.312	9.536	10.868	12.768	14.317
48-hr	3.545	3.936	5.407	6.552	8.125	9.358	10.666	12.064	14.028	15.609
3-day	3.897	4.307	5.858	7.059	8.698	9.973	11.316	12.743	14.735	16.329
4-day	4.135	4.558	6.166	7.407	9.090	10.390	11.754	13.205	15.231	16.853
7-day	4.621	5.069	6.790	8.105	9.868	11.205	12.601	14.101	16.205	17.898
10-day	5.035	5.504	7.319	8.697	10.529	11.901	13.328	14.869	17.036	18.781
20-day	6.415	6.951	9.046	10.624	12.695	14.225	15.791	17.461	19.780	21.624
30-day	7.568	8.163	10.499	12.251	14.535	16.213	17.912	19.696	22.140	24.060
45-day	9.180	9.867	12.581	14.606	17.230	19.149	21.063	23.033	25.681	27.724
60-day	10.615	11.388	14.452	16.732	19.673	21.822	23.941	26.084	28.924	31.080

Table 5.6 AMS-Based Precipitation Frequency Estimates for Hood County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.399	0.434	0.569	0.671	0.805	0.905	1.005	1.108	1.245	1.351
10-min	0.641	0.697	0.915	1.079	1.295	1.457	1.618	1.776	1.983	2.137
15-min	0.792	0.862	1.136	1.342	1.611	1.810	2.007	2.206	2.468	2.665
30-min	1.096	1.191	1.563	1.842	2.205	2.474	2.742	3.018	3.389	3.675
60-min	1.424	1.547	2.030	2.393	2.870	3.223	3.582	3.960	4.477	4.885
2-hr	1.750	1.911	2.530	3.002	3.636	4.119	4.618	5.142	5.866	6.439
3-hr	1.945	2.131	2.839	3.386	4.127	4.701	5.300	5.929	6.800	7.490
6-hr	2.298	2.531	3.406	4.086	5.018	5.747	6.517	7.332	8.469	9.377
12-hr	2.678	2.962	4.038	4.870	6.003	6.877	7.804	8.811	10.242	11.405
24-hr	3.105	3.441	4.722	5.706	7.037	8.050	9.128	10.324	12.048	13.466
48-hr	3.606	3.983	5.425	6.534	8.030	9.166	10.379	11.737	13.708	15.339
3-day	3.944	4.346	5.877	7.056	8.650	9.864	11.161	12.608	14.704	16.435
4-day	4.193	4.614	6.220	7.454	9.124	10.395	11.749	13.250	15.416	17.197
7-day	4.734	5.195	6.958	8.311	10.135	11.524	12.988	14.584	16.850	18.691
10-day	5.186	5.678	7.562	9.006	10.946	12.422	13.967	15.632	17.974	19.862
20-day	6.599	7.161	9.330	10.982	13.183	14.845	16.564	18.391	20.930	22.953
30-day	7.764	8.387	10.805	12.637	15.065	16.887	18.755	20.722	23.433	25.574
45-day	9.387	10.122	12.989	15.153	18.000	20.126	22.279	24.509	27.536	29.892
60-day	10.825	11.669	14.971	17.455	20.710	23.134	25.566	28.052	31.387	33.953

Table 5.7 AMS-Based Precipitation Frequency Estimates for Hunt County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.443	0.474	0.596	0.687	0.806	0.894	0.981	1.069	1.187	1.278
10-min	0.708	0.757	0.954	1.101	1.292	1.435	1.574	1.710	1.886	2.016
15-min	0.885	0.946	1.188	1.368	1.603	1.775	1.946	2.120	2.351	2.528
30-min	1.237	1.321	1.653	1.901	2.222	2.457	2.691	2.936	3.267	3.525
60-min	1.617	1.731	2.177	2.511	2.945	3.261	3.583	3.924	4.394	4.767
2-hr	1.996	2.153	2.763	3.224	3.830	4.278	4.740	5.239	5.939	6.501
3-hr	2.222	2.409	3.134	3.684	4.414	4.959	5.526	6.142	7.012	7.715
6-hr	2.637	2.877	3.799	4.504	5.450	6.167	6.919	7.739	8.902	9.846
12-hr	3.102	3.394	4.505	5.363	6.528	7.426	8.373	9.395	10.837	12.002
24-hr	3.620	3.963	5.262	6.267	7.638	8.700	9.826	11.042	12.761	14.152
48-hr	4.197	4.581	6.046	7.171	8.690	9.842	11.070	12.443	14.432	16.075
3-day	4.585	4.994	6.565	7.767	9.380	10.591	11.885	13.346	15.483	17.259
4-day	4.875	5.306	6.962	8.229	9.931	11.210	12.574	14.105	16.333	18.178
7-day	5.512	5.995	7.838	9.254	11.165	12.624	14.161	15.831	18.198	20.118
10-day	6.064	6.585	8.576	10.103	12.162	13.736	15.380	17.129	19.569	21.519
20-day	7.938	8.537	10.888	12.651	14.954	16.649	18.365	20.165	22.626	24.558
30-day	9.511	10.173	12.819	14.775	17.268	19.052	20.816	22.644	25.108	27.013
45-day	11.700	12.469	15.582	17.860	20.717	22.729	24.667	26.615	29.170	31.084
60-day	13.645	14.516	18.058	20.639	23.847	26.094	28.217	30.293	32.953	34.889

Table 5.8 AMS-Based Precipitation Frequency Estimates for Johnson County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.406	0.442	0.583	0.691	0.836	0.947	1.062	1.182	1.347	1.477
10-min	0.649	0.708	0.934	1.107	1.341	1.521	1.705	1.893	2.146	2.342
15-min	0.809	0.882	1.162	1.376	1.663	1.884	2.111	2.348	2.672	2.928
30-min	1.125	1.226	1.612	1.908	2.304	2.605	2.917	3.247	3.702	4.063
60-min	1.466	1.599	2.108	2.498	3.023	3.423	3.840	4.286	4.908	5.406
2-hr	1.806	1.978	2.631	3.137	3.826	4.361	4.927	5.533	6.386	7.073
3-hr	2.010	2.208	2.954	3.535	4.335	4.962	5.629	6.346	7.358	8.175
6-hr	2.382	2.627	3.540	4.256	5.247	6.030	6.869	7.775	9.059	10.101
12-hr	2.797	3.089	4.182	5.037	6.217	7.144	8.139	9.222	10.768	12.028
24-hr	3.258	3.599	4.883	5.882	7.255	8.323	9.469	10.725	12.526	14.001
48-hr	3.774	4.165	5.645	6.790	8.350	9.551	10.835	12.252	14.291	15.965
3-day	4.117	4.538	6.136	7.370	9.045	10.330	11.699	13.211	15.386	17.171
4-day	4.368	4.811	6.491	7.788	9.550	10.902	12.342	13.924	16.193	18.049
7-day	4.911	5.397	7.238	8.661	10.601	12.099	13.688	15.411	17.857	19.842
10-day	5.378	5.896	7.861	9.379	11.442	13.033	14.715	16.531	19.098	21.175
20-day	6.938	7.520	9.781	11.492	13.755	15.428	17.182	19.134	21.939	24.242
30-day	8.236	8.877	11.402	13.288	15.737	17.498	19.329	21.392	24.376	26.839
45-day	10.011	10.777	13.797	16.054	18.983	21.109	23.276	25.610	28.865	31.468
60-day	11.576	12.468	15.972	18.601	22.034	24.576	27.122	29.728	33.228	35.921

Table 5.9 AMS-Based Precipitation Frequency Estimates for Kaufman County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.437	0.471	0.603	0.701	0.829	0.923	1.017	1.113	1.241	1.339
10-min	0.698	0.751	0.964	1.122	1.328	1.482	1.631	1.778	1.968	2.108
15-min	0.875	0.942	1.203	1.398	1.650	1.835	2.019	2.205	2.455	2.646
30-min	1.229	1.320	1.681	1.949	2.295	2.544	2.794	3.057	3.415	3.696
60-min	1.602	1.725	2.209	2.570	3.039	3.380	3.727	4.099	4.617	5.030
2-hr	1.955	2.124	2.770	3.265	3.929	4.432	4.959	5.529	6.333	6.982
3-hr	2.159	2.360	3.117	3.704	4.509	5.135	5.801	6.523	7.546	8.376
6-hr	2.532	2.788	3.738	4.489	5.536	6.372	7.277	8.261	9.669	10.820
12-hr	2.938	3.251	4.394	5.306	6.593	7.632	8.771	10.023	11.833	13.324
24-hr	3.409	3.775	5.119	6.190	7.702	8.919	10.258	11.742	13.901	15.691
48-hr	4.011	4.423	5.964	7.171	8.842	10.150	11.576	13.179	15.531	17.490
3-day	4.408	4.848	6.506	7.795	9.562	10.925	12.404	14.071	16.516	18.553
4-day	4.681	5.142	6.882	8.234	10.084	11.513	13.056	14.784	17.302	19.390
7-day	5.245	5.751	7.652	9.134	11.172	12.764	14.469	16.327	18.979	21.143
10-day	5.739	6.278	8.308	9.888	12.054	13.747	15.544	17.474	20.195	22.391
20-day	7.473	8.080	10.439	12.223	14.579	16.329	18.138	20.089	22.825	25.024
30-day	8.947	9.609	12.240	14.193	16.698	18.489	20.299	22.251	24.968	27.136
45-day	11.027	11.786	14.849	17.096	19.927	21.921	23.872	25.885	28.582	30.653
60-day	12.889	13.742	17.202	19.729	22.886	25.110	27.224	29.302	31.979	33.939

Table 5.10 AMS-Based Precipitation Frequency Estimates for Navarro County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.431	0.469	0.611	0.720	0.865	0.977	1.091	1.208	1.366	1.489
10-min	0.688	0.748	0.977	1.152	1.387	1.569	1.752	1.931	2.165	2.339
15-min	0.864	0.937	1.219	1.434	1.722	1.941	2.164	2.393	2.700	2.937
30-min	1.215	1.315	1.702	1.996	2.389	2.685	2.990	3.312	3.756	4.107
60-min	1.575	1.710	2.230	2.627	3.159	3.562	3.984	4.442	5.089	5.613
2-hr	1.887	2.076	2.784	3.336	4.095	4.686	5.327	6.046	7.098	7.974
3-hr	2.061	2.287	3.122	3.782	4.706	5.436	6.243	7.162	8.525	9.671
6-hr	2.389	2.677	3.724	4.562	5.752	6.710	7.783	9.014	10.859	12.422
12-hr	2.780	3.119	4.344	5.333	6.745	7.894	9.187	10.665	12.876	14.747
24-hr	3.243	3.630	5.033	6.164	7.777	9.088	10.558	12.229	14.718	16.816
48-hr	3.832	4.266	5.868	7.138	8.917	10.328	11.889	13.661	16.290	18.498
3-day	4.222	4.683	6.404	7.753	9.620	11.077	12.674	14.490	17.178	19.433
4-day	4.491	4.970	6.762	8.163	10.098	11.606	13.249	15.102	17.823	20.093
7-day	5.050	5.563	7.480	8.979	11.048	12.668	14.415	16.335	19.098	21.367
10-day	5.539	6.077	8.097	9.670	11.832	13.520	15.321	17.273	20.045	22.298
20-day	7.270	7.862	10.165	11.905	14.200	15.899	17.658	19.564	22.247	24.410
30-day	8.736	9.373	11.917	13.798	16.200	17.905	19.626	21.487	24.083	26.158
45-day	10.780	11.500	14.427	16.562	19.219	21.059	22.850	24.716	27.229	29.169
60-day	12.601	13.401	16.685	19.062	21.981	23.982	25.874	27.762	30.213	32.027

Table 5.11 AMS-Based Precipitation Frequency Estimates for Palo Pinto County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.392	0.426	0.558	0.656	0.785	0.878	0.974	1.075	1.217	1.329
10-min	0.631	0.685	0.898	1.058	1.265	1.418	1.572	1.729	1.942	2.106
15-min	0.779	0.845	1.106	1.300	1.553	1.736	1.922	2.121	2.399	2.620
30-min	1.069	1.159	1.513	1.777	2.117	2.361	2.611	2.884	3.269	3.580
60-min	1.376	1.493	1.957	2.303	2.753	3.080	3.416	3.783	4.303	4.724
2-hr	1.672	1.826	2.419	2.870	3.474	3.928	4.401	4.909	5.620	6.191
3-hr	1.846	2.024	2.702	3.225	3.935	4.484	5.059	5.668	6.516	7.192
6-hr	2.162	2.385	3.216	3.869	4.773	5.492	6.254	7.057	8.174	9.065
12-hr	2.504	2.776	3.773	4.568	5.689	6.597	7.581	8.633	10.121	11.326
24-hr	2.897	3.220	4.398	5.340	6.676	7.761	8.949	10.240	12.092	13.609
48-hr	3.399	3.767	5.132	6.208	7.706	8.893	10.185	11.613	13.681	15.388
3-day	3.732	4.127	5.604	6.759	8.352	9.596	10.943	12.444	14.624	16.429
4-day	3.965	4.378	5.932	7.143	8.807	10.100	11.498	13.054	15.315	17.187
7-day	4.450	4.902	6.605	7.927	9.738	11.140	12.647	14.317	16.732	18.722
10-day	4.862	5.344	7.167	8.581	10.511	12.002	13.599	15.360	17.896	19.979
20-day	6.202	6.774	8.946	10.623	12.902	14.657	16.517	18.538	21.411	23.744
30-day	7.318	7.963	10.422	12.315	14.880	16.851	18.924	21.152	24.286	26.811
45-day	8.881	9.633	12.513	14.720	17.694	19.969	22.337	24.846	28.330	31.105
60-day	10.273	11.121	14.379	16.869	20.208	22.754	25.382	28.136	31.923	34.912

Table 5.12 AMS-Based Precipitation Frequency Estimates for Parker County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.395	0.430	0.565	0.668	0.806	0.911	1.018	1.126	1.270	1.382
10-min	0.635	0.691	0.909	1.075	1.297	1.467	1.639	1.807	2.029	2.195
15-min	0.785	0.854	1.122	1.325	1.597	1.804	2.015	2.228	2.513	2.732
30-min	1.080	1.175	1.542	1.821	2.192	2.472	2.758	3.050	3.445	3.752
60-min	1.399	1.523	2.002	2.367	2.854	3.225	3.604	3.994	4.523	4.934
2-hr	1.720	1.878	2.475	2.937	3.567	4.062	4.572	5.091	5.791	6.332
3-hr	1.915	2.094	2.766	3.291	4.016	4.596	5.198	5.805	6.621	7.250
6-hr	2.272	2.493	3.311	3.956	4.859	5.591	6.357	7.129	8.168	8.971
12-hr	2.670	2.943	3.953	4.750	5.859	6.754	7.697	8.662	9.978	11.007
24-hr	3.122	3.449	4.672	5.631	6.956	8.009	9.122	10.289	11.906	13.191
48-hr	3.655	4.030	5.442	6.539	8.043	9.215	10.461	11.803	13.699	15.234
3-day	3.997	4.402	5.939	7.128	8.744	9.989	11.313	12.760	14.826	16.511
4-day	4.235	4.664	6.288	7.544	9.252	10.567	11.966	13.498	15.686	17.473
7-day	4.726	5.203	7.008	8.407	10.318	11.798	13.373	15.084	17.519	19.499
10-day	5.144	5.660	7.607	9.117	11.179	12.778	14.478	16.320	18.934	21.057
20-day	6.524	7.129	9.448	11.225	13.616	15.427	17.343	19.451	22.469	24.936
30-day	7.681	8.364	10.996	13.000	15.675	17.679	19.789	22.121	25.465	28.204
45-day	9.318	10.135	13.271	15.669	18.884	21.320	23.870	26.627	30.516	33.656
60-day	10.782	11.726	15.333	18.104	21.848	24.726	27.726	30.896	35.296	38.796

Table 5.13 AMS-Based Precipitation Frequency Estimates for Rockwall County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.438	0.470	0.596	0.690	0.812	0.900	0.988	1.078	1.199	1.291
10-min	0.700	0.752	0.954	1.106	1.301	1.446	1.587	1.725	1.905	2.038
15-min	0.875	0.938	1.188	1.374	1.613	1.788	1.960	2.137	2.373	2.553
30-min	1.223	1.310	1.653	1.908	2.235	2.471	2.706	2.953	3.289	3.552
60-min	1.593	1.709	2.168	2.510	2.951	3.271	3.594	3.938	4.415	4.792
2-hr	1.950	2.108	2.722	3.187	3.803	4.263	4.738	5.244	5.949	6.512
3-hr	2.160	2.348	3.067	3.618	4.358	4.922	5.511	6.138	7.013	7.713
6-hr	2.546	2.785	3.690	4.390	5.346	6.088	6.872	7.712	8.892	9.842
12-hr	2.983	3.273	4.363	5.212	6.376	7.284	8.253	9.307	10.807	12.029
24-hr	3.476	3.817	5.099	6.096	7.464	8.527	9.669	10.930	12.747	14.242
48-hr	4.049	4.436	5.903	7.036	8.576	9.753	11.019	12.444	14.527	16.259
3-day	4.426	4.842	6.426	7.645	9.297	10.551	11.899	13.423	15.655	17.513
4-day	4.693	5.133	6.804	8.092	9.842	11.177	12.610	14.218	16.559	18.500
7-day	5.258	5.749	7.602	9.042	11.017	12.553	14.194	15.979	18.524	20.598
10-day	5.754	6.284	8.282	9.835	11.965	13.629	15.392	17.278	19.928	22.061
20-day	7.520	8.127	10.478	12.263	14.632	16.412	18.245	20.186	22.875	25.010
30-day	9.016	9.685	12.326	14.298	16.851	18.709	20.581	22.547	25.238	27.349
45-day	11.094	11.870	14.979	17.272	20.187	22.272	24.313	26.392	29.156	31.259
60-day	12.945	13.822	17.363	19.959	23.226	25.546	27.770	29.966	32.812	34.912

Table 5.14 AMS-Based Precipitation Frequency Estimates for Somervell County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.409	0.443	0.578	0.679	0.812	0.910	1.009	1.110	1.246	1.351
10-min	0.656	0.711	0.928	1.091	1.305	1.466	1.624	1.780	1.984	2.136
15-min	0.813	0.882	1.150	1.352	1.618	1.817	2.015	2.213	2.472	2.668
30-min	1.127	1.220	1.586	1.861	2.220	2.487	2.754	3.028	3.398	3.684
60-min	1.462	1.585	2.067	2.429	2.903	3.252	3.608	3.986	4.507	4.920
2-hr	1.796	1.959	2.589	3.069	3.711	4.196	4.701	5.239	5.990	6.591
3-hr	1.993	2.184	2.913	3.474	4.233	4.816	5.429	6.084	7.003	7.741
6-hr	2.338	2.579	3.487	4.192	5.155	5.905	6.700	7.558	8.770	9.752
12-hr	2.681	2.974	4.086	4.945	6.116	7.017	7.979	9.036	10.553	11.795
24-hr	3.065	3.409	4.718	5.728	7.097	8.144	9.263	10.508	12.308	13.793
48-hr	3.545	3.927	5.374	6.494	8.021	9.197	10.457	11.853	13.868	15.527
3-day	3.878	4.282	5.810	6.994	8.612	9.862	11.199	12.670	14.785	16.519
4-day	4.132	4.553	6.151	7.387	9.072	10.372	11.758	13.272	15.437	17.204
7-day	4.700	5.159	6.909	8.255	10.075	11.468	12.933	14.512	16.736	18.530
10-day	5.175	5.662	7.528	8.956	10.873	12.332	13.852	15.476	17.744	19.561
20-day	6.652	7.198	9.320	10.924	13.040	14.613	16.229	17.952	20.345	22.251
30-day	7.860	8.457	10.804	12.562	14.851	16.528	18.230	20.036	22.527	24.499
45-day	9.515	10.210	12.972	15.024	17.666	19.581	21.484	23.445	26.083	28.118
60-day	10.970	11.763	14.927	17.268	20.259	22.420	24.527	26.638	29.409	31.489

Table 5.15 AMS-Based Precipitation Frequency Estimates for Tarrant County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.405	0.439	0.572	0.673	0.807	0.909	1.012	1.117	1.261	1.371
10-min	0.649	0.704	0.919	1.081	1.297	1.462	1.627	1.791	2.008	2.171
15-min	0.807	0.875	1.137	1.336	1.601	1.801	2.004	2.212	2.493	2.711
30-min	1.117	1.210	1.570	1.842	2.203	2.475	2.752	3.041	3.436	3.746
60-min	1.449	1.572	2.047	2.408	2.888	3.248	3.619	4.011	4.553	4.983
2-hr	1.773	1.935	2.555	3.031	3.671	4.159	4.669	5.215	5.977	6.588
3-hr	1.966	2.154	2.871	3.423	4.173	4.752	5.361	6.013	6.930	7.668
6-hr	2.323	2.558	3.446	4.135	5.077	5.810	6.586	7.419	8.591	9.536
12-hr	2.733	3.016	4.089	4.919	6.051	6.928	7.852	8.840	10.224	11.336
24-hr	3.191	3.522	4.776	5.747	7.069	8.096	9.175	10.321	11.921	13.201
48-hr	3.692	4.063	5.447	6.532	8.033	9.225	10.488	11.816	13.663	15.136
3-day	4.018	4.417	5.901	7.067	8.686	9.976	11.348	12.792	14.805	16.411
4-day	4.258	4.680	6.252	7.488	9.204	10.573	12.030	13.569	15.716	17.433
7-day	4.771	5.249	7.037	8.438	10.375	11.908	13.541	15.281	17.726	19.693
10-day	5.216	5.736	7.688	9.211	11.306	12.950	14.699	16.573	19.213	21.343
20-day	6.739	7.340	9.640	11.404	13.780	15.588	17.492	19.564	22.505	24.891
30-day	8.012	8.684	11.283	13.257	15.883	17.846	19.898	22.138	25.318	27.899
45-day	9.755	10.569	13.712	16.103	19.289	21.690	24.175	26.822	30.508	33.449
60-day	11.294	12.248	15.917	18.720	22.477	25.346	28.296	31.362	35.554	38.840

Table 5.16 AMS-Based Precipitation Frequency Estimates for Wise County (inches)

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.404	0.437	0.569	0.667	0.795	0.889	0.983	1.078	1.206	1.303
10-min	0.648	0.702	0.914	1.072	1.278	1.431	1.581	1.729	1.922	2.066
15-min	0.804	0.870	1.131	1.326	1.578	1.763	1.946	2.133	2.384	2.576
30-min	1.111	1.201	1.560	1.826	2.170	2.418	2.667	2.925	3.274	3.546
60-min	1.438	1.557	2.027	2.377	2.832	3.165	3.500	3.850	4.327	4.700
2-hr	1.762	1.916	2.508	2.959	3.561	4.018	4.485	4.970	5.633	6.151
3-hr	1.955	2.131	2.803	3.321	4.023	4.568	5.133	5.715	6.509	7.130
6-hr	2.303	2.523	3.346	3.990	4.879	5.587	6.328	7.090	8.129	8.943
12-hr	2.682	2.954	3.963	4.757	5.863	6.753	7.691	8.654	9.969	11.000
24-hr	3.105	3.432	4.645	5.602	6.937	8.011	9.148	10.326	11.948	13.228
48-hr	3.592	3.969	5.376	6.481	8.013	9.231	10.531	11.912	13.850	15.407
3-day	3.920	4.330	5.864	7.066	8.726	10.036	11.437	12.950	15.094	16.834
4-day	4.168	4.604	6.233	7.510	9.277	10.670	12.165	13.787	16.096	17.976
7-day	4.714	5.207	7.040	8.482	10.487	12.076	13.790	15.655	18.322	20.501
10-day	5.173	5.709	7.701	9.269	11.450	13.181	15.049	17.084	19.994	22.373
20-day	6.619	7.255	9.641	11.504	14.067	16.074	18.224	20.567	23.911	26.639
30-day	7.814	8.533	11.244	13.348	16.229	18.468	20.852	23.439	27.114	30.101
45-day	9.485	10.339	13.557	16.059	19.488	22.167	25.003	28.028	32.270	35.679
60-day	10.968	11.949	15.634	18.506	22.455	25.565	28.841	32.283	37.055	40.850

6.0 Hydrologic Soils Data

6.1. Electronic Soil Maps

Electronic soils data in the Soil Survey Geographic (SSURGO) Database can be obtained free of charge from the National Resource Conservation Service (NRCS) at http://soildatamart.nrcs.usda.gov. The data is downloadable by county and includes extensive soil information, including hydrologic soil groups. The data is intended to be imported into a geographic information system (GIS) to allow for site-specific soil analysis of soil characteristics for storm design. All soil survey results can also be accessed online at http://websoilsurvey.nrcs.usda.gov/app/. Maps can be created and printed from this site without the use of a GIS. An example SSURGO Soil Map for West Tarrant County is shown in Figure 6.1.

6.2. Online Web Soil Survey

Following is a procedure for using the NRCS online Web Soil Survey.

1. Go to http://websoilsurvey.nrcs.usda.gov/app/
2. Click **Start WSS**
3. Define your Area of Interest by drawing a box around your site location.
4. Click the **Soil Map** tab
5. Click **Save or Print** in the upper right hand corner. A pdf will open in a new window that you can either print or save. It will show the area of interest along with a legend and the appropriate map units.

6.3. Downloading Soil Surveys

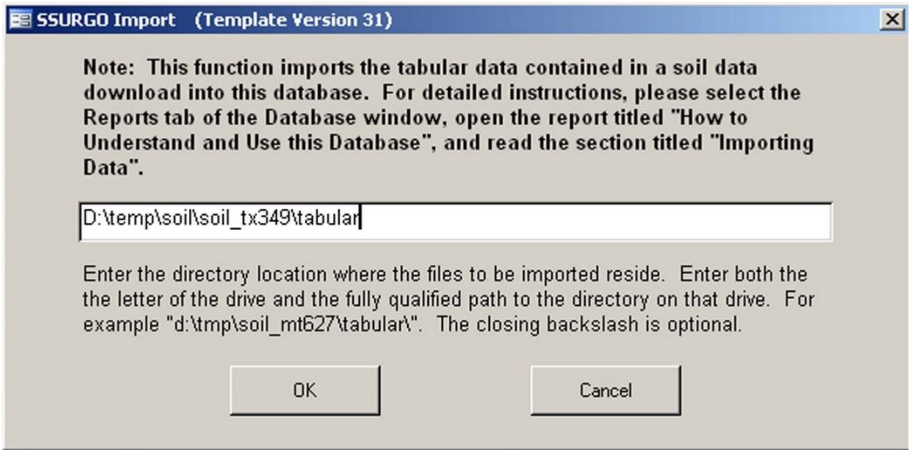
Following is a procedure for downloading data from the NRCS web site and importing it into ArcGIS.

Downloading SSURGO Soil Data into ArcInfo 9.x

1. Go to http://soildatamart.nrcs.usda.gov
2. Click **Select State**
3. Select State (**Texas)**
4. Select **County** of interest
5. Click **Select Survey Area**
6. Click **Download Data**
7. Enter your e-mail address in the provided form space
8. Click **Submit Request**
9. You will receive an immediate message acknowledging your request and a follow-up e-mail once your request has been processed.
10. The file(s) will be provided in compressed ZIP format, requiring the use of WinZip to extract.
11. Extract the files to a destination directory of your choice. The extracted files contain a spatial sub-folder, a tabular sub-folder, and a zip file containing the SSURGO MS Access template file.

Importing raw tabular soil data into Microsoft Access

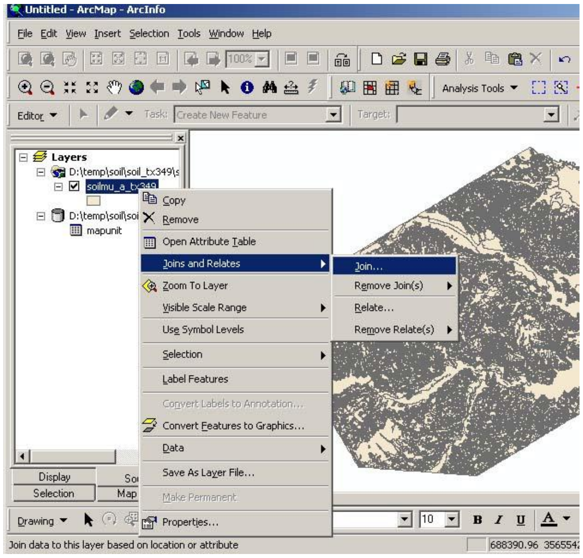
1. Extract the soildb_US_2002.zip file into the same destination folder by using the "extract to here" command in WinZip. This will extract the template database.
2. Open the template database and input the path name to the tabular data. This will build the SSURGO database and allow the creation of reports and queries.

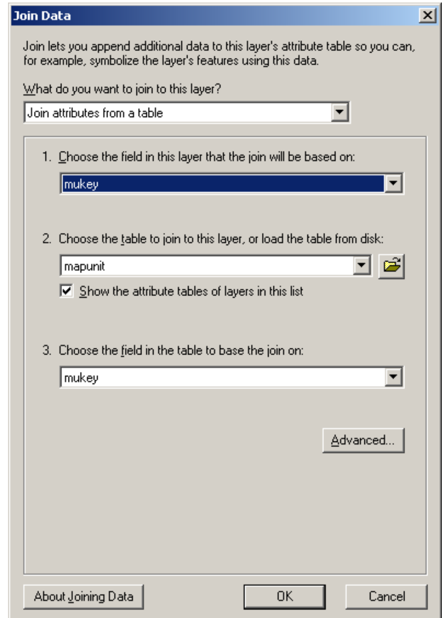


3. Once the data is imported into the database, a report can be run. With the soil reports dialog box up, press the **Select All** option and generate the report. Note: Regardless of what report you wish to run, all reports are simultaneously created. The selected report is displayed on the screen.
4. All the reports are now complete, and the tables can now be added directly into ArcGIS.

Joining tables to shapefiles in ArcGIS

1. Open **ArcGIS** and add the **Soils** shapefile.
2. Add the "**mapunit**" report to ArcGIS by navigating directly to the MS Access database and opening it (via the add data dialog box). Note: mapunit is only a commonly used example, containing full soil names and prime farmland information.
3. Now that the table is added to the Table of Contents, it is ready to be joined to the existing soils shapefile.
4. Right click on the **soils** shapefile and select **join.**





5. Under the **Join Data** dialog box, select the **mukey** field in Dropdown Box 1 and select **mapunit** in Dropdown Box 2.

6. Now that your shapefile is joined with the appropriate information, the next step is to export the shapefile into a new shapefile with the joins saved permanently. Right click on the **soils** shapefile and choose **Data > Export** and **Save** your file.

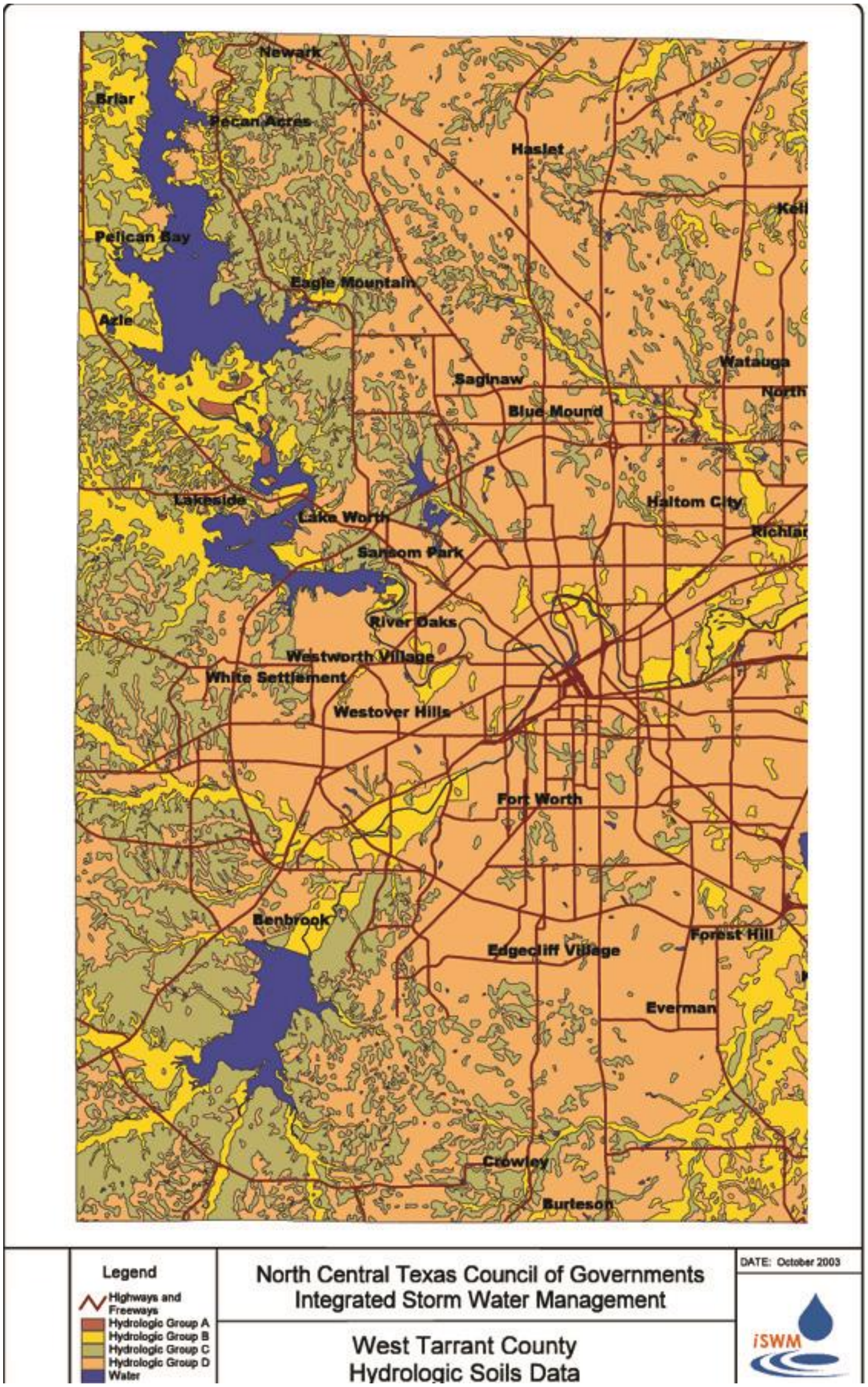


Figure 6.1 Example SSURGO Soil Map - West Tarrant County