

# STORM DRAINAGE SYSTEMS

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## 3.1 Overview

### 3.1.1 Introduction

In this chapter, guidelines are given for evaluating roadway features and design criteria as they relate to gutter and inlet hydraulics and storm drain design. Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grate and combination inlets are based on information contained in HEC-12 (USDOT, FHWA, 1984). Storm drain design is based on the use of the rational formula. For storm drain design, inlet and outlet control shall be checked for all pipes to determine where the flow control is and what the actual quantity of flow through the pipe will be (See Chapter 4 – Design of Culverts for inlet and outlet control nomographs and procedures).

### 3.1.2 Inlet Definition

There are three storm water inlet categories:

- curb opening inlets
- grated inlets
- combination inlets

In addition, 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.

### 3.1.3 Criteria

The following criteria shall be used for inlet design.

#### Design Frequencies

- Cross Drainage Facilities (Transport storm runoff under roadways) 100-year
- Storm Drains (Lateral Closed Systems) 25-year
- Inlets 10-year
- Outlet Protection 25-year

#### Spread Limits

- Catch basins shall be spaced so that the spread in the street for the 10-year design flow shall not exceed the following, as measured from the face of the curb:
  - (1) 8 feet if the street is classified as a Minor Collector or Major Thoroughfare;
  - (2) 16 feet at any given section, but in no case greater than 10 feet on one side of the street, if the street is classified as a Local Street.

## 3.2 Symbols And Definitions

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

**Table 3-1 Symbols And Definitions**

<u>Symbol</u>	<u>Definition</u>	<u>Units</u>
a	Gutter depression	in
A	Area of cross section	ft <sup>2</sup>
d or D	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
E <sub>o</sub>	Ratio of frontal flow to total gutter flow Q <sub>w</sub> /Q	-
g	Acceleration due to gravity (32.2 ft/s <sup>2</sup> )	ft/s <sup>2</sup>
h	Height of curb opening inlet	ft
H	Head loss	ft
K	Loss coefficient	-
L or L <sub>T</sub>	Length of curb opening inlet	ft
L	Pipe length	ft
n	Roughness coefficient in the modified Manning 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 <sub>i</sub>	Intercepted flow	cfs
Q <sub>s</sub>	Gutter capacity above the depressed section	cfs
S or S <sub>x</sub>	Cross Slope - Traverse slope	ft/ft
S or S <sub>L</sub>	Longitudinal slope of pavement	ft/ft
S <sub>f</sub>	Friction slope	ft/ft
S' <sub>w</sub>	Depression section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
T <sub>s</sub>	Spread above depressed section	ft
V	Velocity of flow	ft/s
W	Width of depression for curb opening inlets	ft
Z	T/d, reciprocal of the cross slope	-

### 3.3 Gutter Flow Calculations

#### 3.3.1 Formula

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

$$Q = [0.56 / n] S_x^{5/3} S^{1/2} T^{8/3} \quad (3.1)$$

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

### 3.3.2 Nomograph

A nomograph for solving Equation 3.1 is presented on the next page (Figure 3-1). Manning's  $n$  values for various pavement surfaces are presented in the table below.

### 3.3.3 Manning's $n$ Table

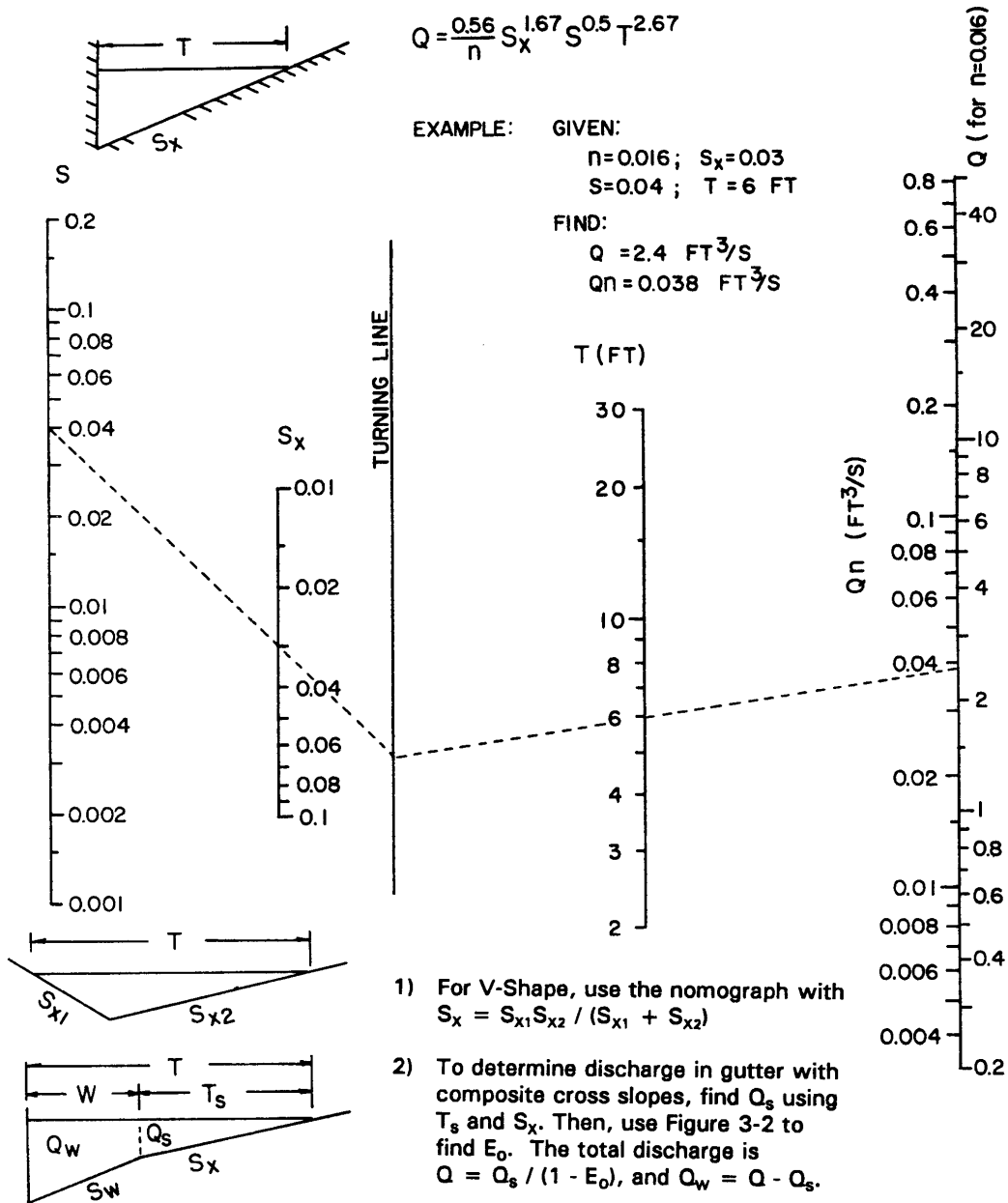
<u>Type of Gutter or Pavement</u>	<u>Range of Manning's <math>n</math></u>
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slopes, where sediment may accumulate, increase above values of $n$ by	0.002
<small>Note: Estimates are by the Federal Highway Administration  Reference: USDOT, FHWA, HDS-3 (1961).</small>	

### 3.3.4 Uniform Cross Slope

The nomograph in Figure 3-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

1. Determine input parameters, including longitudinal slope ( $S$ ), cross slope ( $S_x$ ), gutter flow ( $Q$ ), and Manning's  $n$ .
2. Draw a line between the  $S$  and  $S_x$  scales and note where it intersects the turning line.
3. Draw a line between the intersection point from Step 2 and the appropriate gutter flow



**Figure 3-1 Flow In Triangular Gutter Sections**

value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the

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product of Q and n.

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.

1. Determine input parameters, including longitudinal slope (S), cross slope ( $S_x$ ), spread (T), and Manning's n.
2. Draw a line between the S and  $S_x$  scales and note where it intersects the turning line.
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.
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.

### 3.3.5 Composite Gutter Sections

Figure 3-2 in combination with Figure 3-1 can be used to find the flow in a gutter with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

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

Condition 1: Find spread, given gutter flow.

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 ( $Q_s$ ).
2. Calculate the gutter flow in W ( $Q_w$ ), using the equation:  $Q_w = Q - Q_s$  (3.2)
3. Calculate the ratios  $Q_w/Q$  or  $E_o$  and  $S_w/S_x$  and use Figure 3-2 to find an appropriate value of W/T.
4. Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.
5. Find the spread above the depressed section ( $T_s$ ) by subtracting W from the value of T obtained in Step 4.

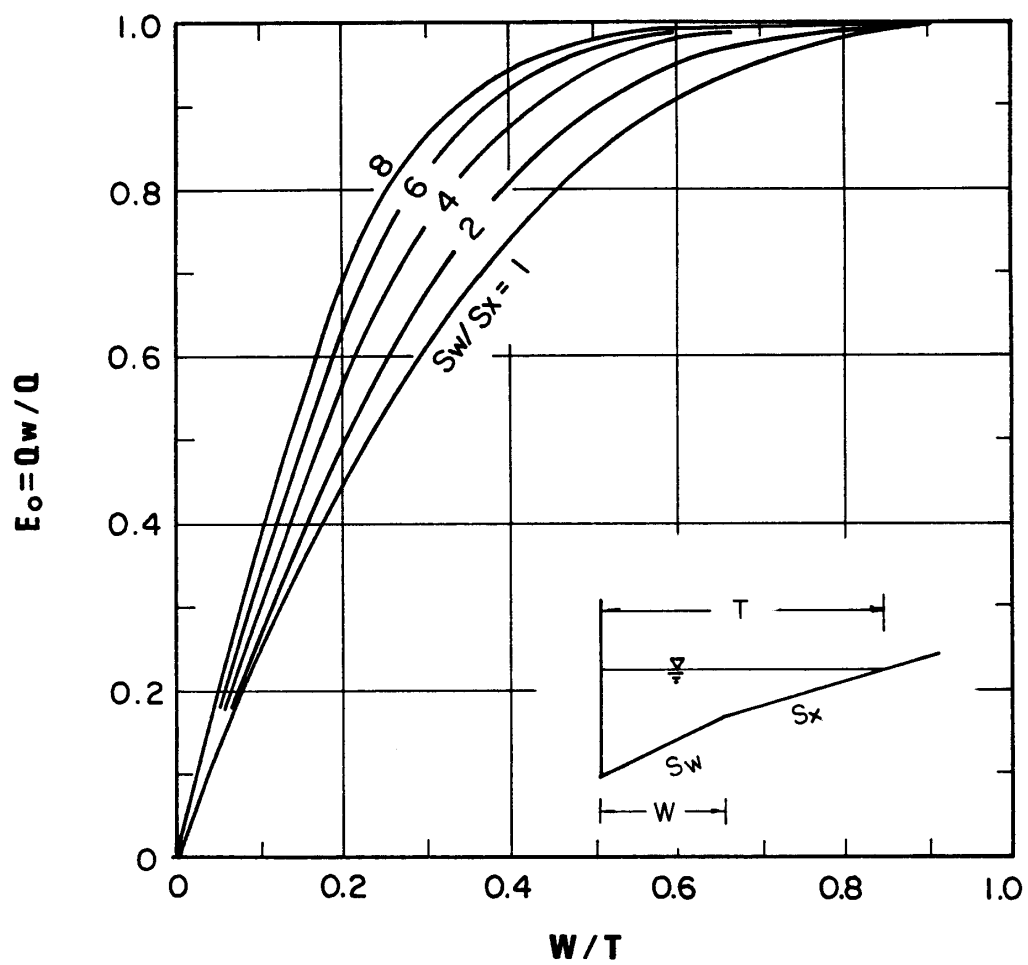
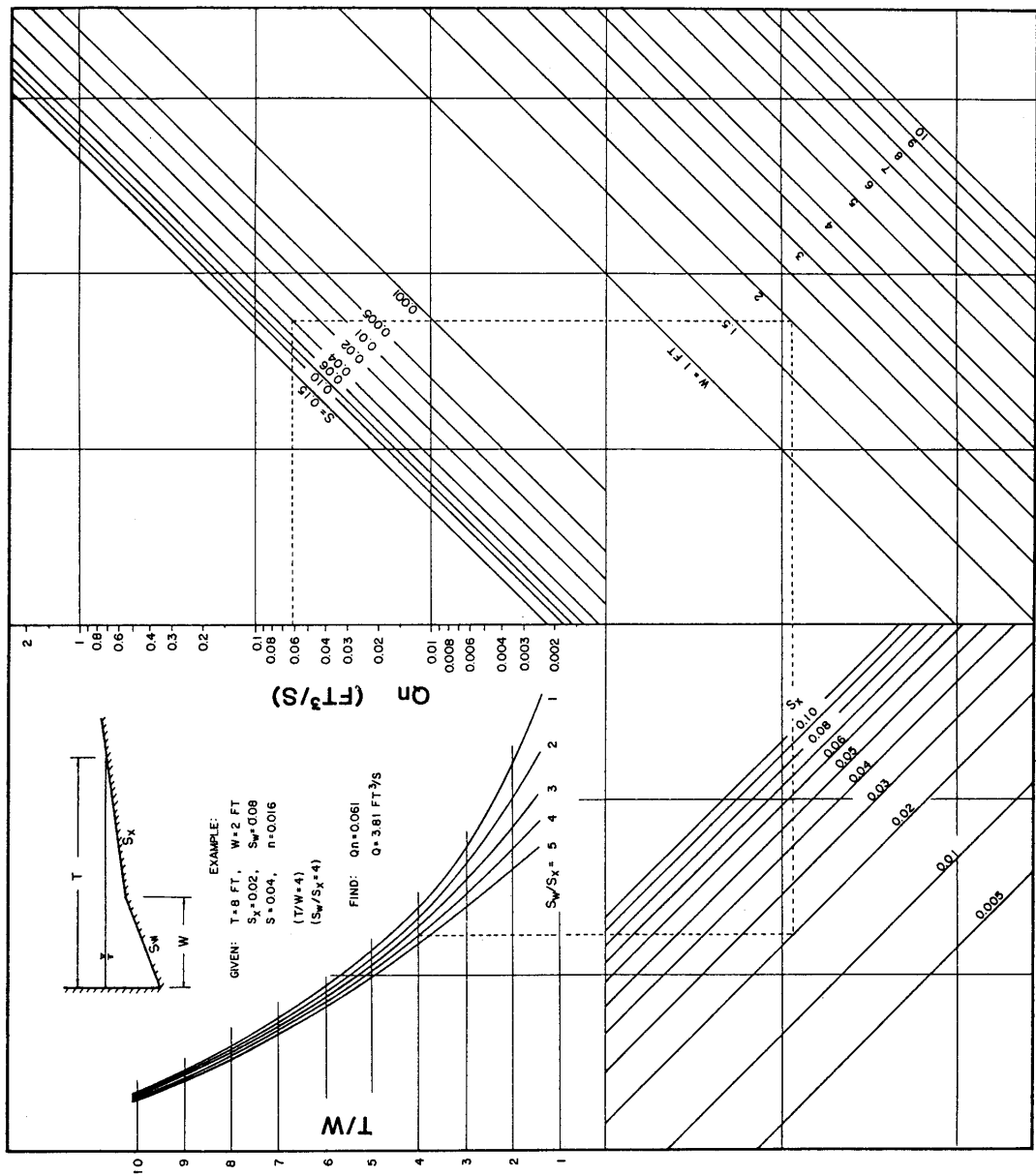


Figure 3-2 Ratio Of Frontal Flow To Total Gutter Flow



**Figure 3-3 Flow In Composite Gutter Sections**

6. Use the value of  $T_s$  from Step 5 along with Manning's  $n$ ,  $S$ , and  $S_x$  to find the actual value of  $Q_s$  from Figure 3-1.
7. Compare the value of  $Q_s$  from Step 6 to the trial value from Step 1. If values are not



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comparable, select a new value of  $Q_s$  and return to Step 1.

Condition 2: Find gutter flow, given spread.

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).
2. Use Figure 3-1 to determine the capacity of the gutter section above the depressed section ( $Q_s$ ). Use the procedure for uniform cross slopes (Condition 2), substituting  $T_s$  for T.
3. Calculate the ratios  $W/T$  and  $S_w/S_x$ , and, from Figure 3-2, find the appropriate value of  $E_o$  (the ratio of  $Q_w/Q$ ).
4. Calculate the total gutter flow using the equation:

$$Q = Q_s / (1 - E_o) \quad (3.3)$$

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 ( $Q_w/Q$ )

5. Calculate the gutter flow in width (W), using Equation 3.2.

### 3.3.6 Examples

#### Example 1

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

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

Solution: (1) From Figure 3-1,  $Qn = 0.03$   
 $Q = Qn/n = 0.03/0.015 = 2.0$  cfs

(2)  $T = 8 - 2 = 6$  ft  
 $(Qn)_2 = 0.014$  (Figure 3-1) (flow in 6 ft 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

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Find: Flow in the composite gutter

Solution: (1) Use Figure 3-1 to find the gutter section capacity above the depressed section.

$$Q_{sn} = 0.038$$

$$Q_s = 0.038/0.014 = 2.7 \text{ cfs}$$

(2) Calculate  $W/T = 1.5/6 = 0.25$  and

$$S_w/S_x = 0.0833/0.03 = 2.78$$

Use Figure 3-2 to find  $E_o = 0.64$

(3) Calculate the gutter flow using Equation 3.3

$$Q = 2.7/(1 - 0.64) = 7.5 \text{ cfs}$$

(4) Calculate the gutter flow in width,  $W$ , using Equation 3.2

$$Q_w = 7.5 - 2.7 = 4.8 \text{ cfs}$$

## 3.4 Grate Inlets Design

### 3.4.1 Types

Inlets are drainage structures utilized 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.

1. 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 of spacer bars to form slot openings.
2. Curb-Opening Inlets - These inlets are vertical openings in the curb covered by a top slab.
3. 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.

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

The design of grate inlets will be discussed in this section, curb inlet design in Section 3.5, and combination inlets in Section 3.6.

### 3.4.2 Grate Inlets On Grade

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low

velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small.

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

**Table 3-3** 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	Recticuline	12	16
8	P - 1-1/8	9	20

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

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

$$E_o = Q_w/Q = 1 - (1 - W/T)^{2.67} \quad (3.4)$$

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)

Figure 3-2 provides a graphical solution of  $E_o$  for either depressed gutter sections or straight cross slopes. The ratio of side flow,  $Q_s$ , to total gutter flow is:

$$Q_s/Q = 1 - Q_w/Q = 1 - E_o \quad (3.5)$$

The ratio of frontal flow intercepted to total frontal flow,  $R_f$ , is expressed:

$$R_f = 1 - 0.09 (V - V_o) \quad (3.6)$$

Where:  $V$  = velocity of flow in the gutter (ft/s) (using  $Q$  from Figure 3-1)  
 $V_o$  = gutter velocity where splash-over first occurs (ft/s) (from Figure 3-4)

This ratio is equivalent to frontal flow interception efficiency. Figure 3-4 provides a solution of equation 3.6 which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 3-4 is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow,  $R_s$ , or side flow interception efficiency, is expressed by:

$$R_s = 1 / [1 + (0.15V^{1.8}/S_x L^{2.3})] \quad (3.7)$$

Where: L = length of the grate (ft)

Figure 3-5 provides a solution to equation 3.7.

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

$$E = R_f E_o + R_s (1 - E_o) \quad (3.8)$$

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

$$Q_i = EQ = Q[R_f E_o + R_s (1 - E_o)] \quad (3.9)$$

The following example illustrates the use of this procedure.

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

Find: Interception capacity of:  
 (1) a curved vane grate, and  
 (2) a reticuline grate 2-ft long and 2-ft wide

Solution: From Figure 3-4 for Curved Vane Grate,  $R_f = 1.0$   
 From Figure 3-4 for Reticuline Grate,  $R_f = 1.0$   
 From Figure 3-5  $R_s = 0.1$  for both grates.  
 From Equation 3.9:  $Q_i = 3.0[1.0 \times 0.69 + 0.1(1 - 0.69)] = 2.2$  cfs

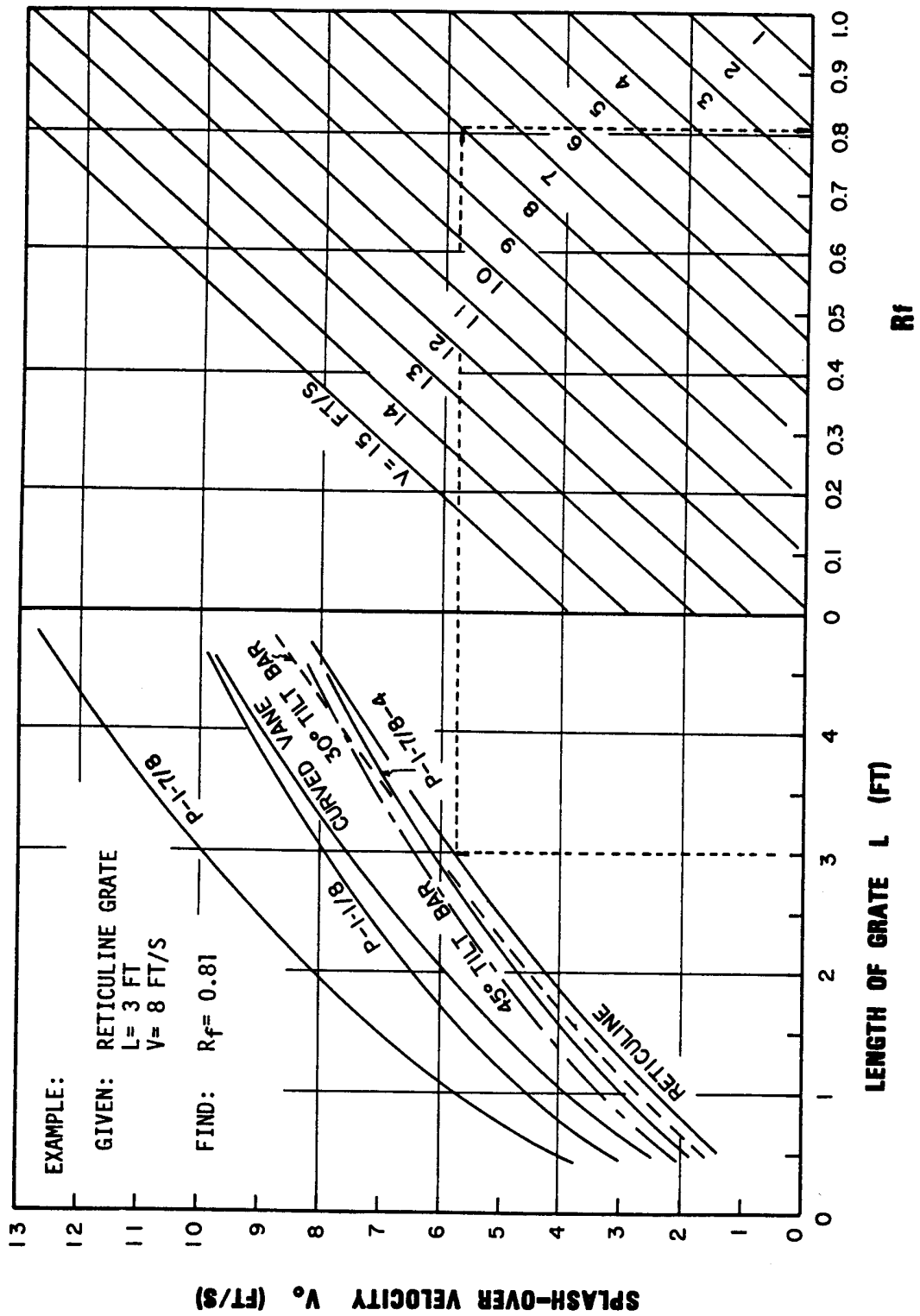
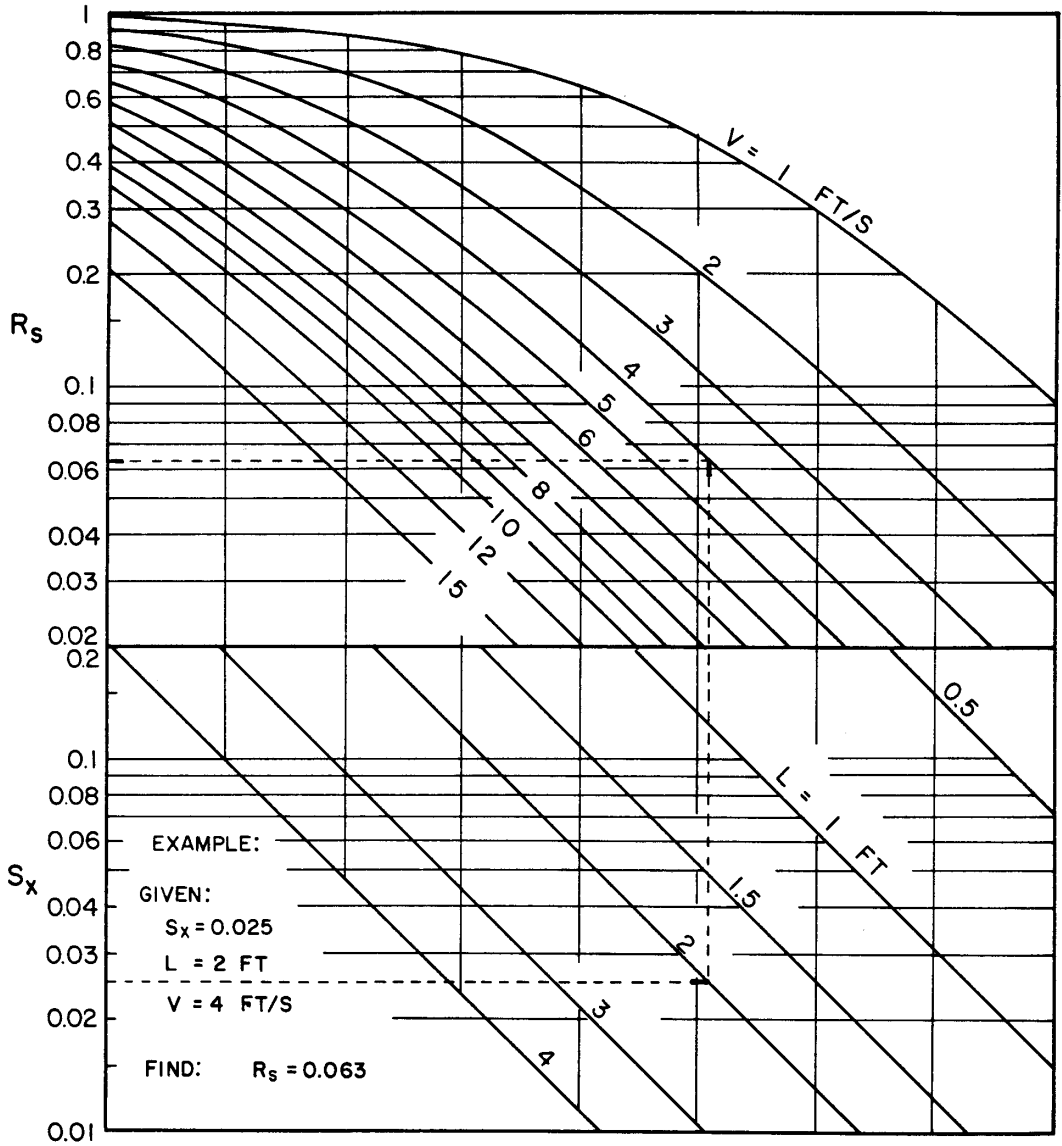


Figure 3-4 Grate Inlet Frontal Flow Interception Efficiency

Source: HEC-12



**Figure 3-5 Grate Inlet Side Flow Interception Efficiency**

Source: HEC-12

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

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### 3.4.3 Grate Inlets In Sag

A grate inlet in a sag operates as a weir up to a certain depth dependent on the bar configuration and size of the grate and as an orifice at greater depths. For standard gutter inlet grate, weir operation continues to a depth of about 0.4 foot 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 foot and about 1.4 feet, a transition from weir to orifice flow occurs.

The capacity of grate inlets operating as a weir is:

$$Q_i = CPd^{1.5} \quad (3.10)$$

Where:  $P$  = perimeter of grate excluding bar widths and side against the curb (ft)  
 $C$  = 3.0  
 $d$  = depth of water above grate (ft)

and as an orifice is:

$$Q_i = CA(2gd)^{0.5} \quad (3.11)$$

Where:  $C$  = 0.67 orifice coefficient  
 $A$  = clear opening area of the grate (ft<sup>2</sup>)  
 $g$  = 32.2 ft/s<sup>2</sup>

Figure 3-6 is a plot of equations 3.10 and 3.11 for various grate sizes. The effects of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

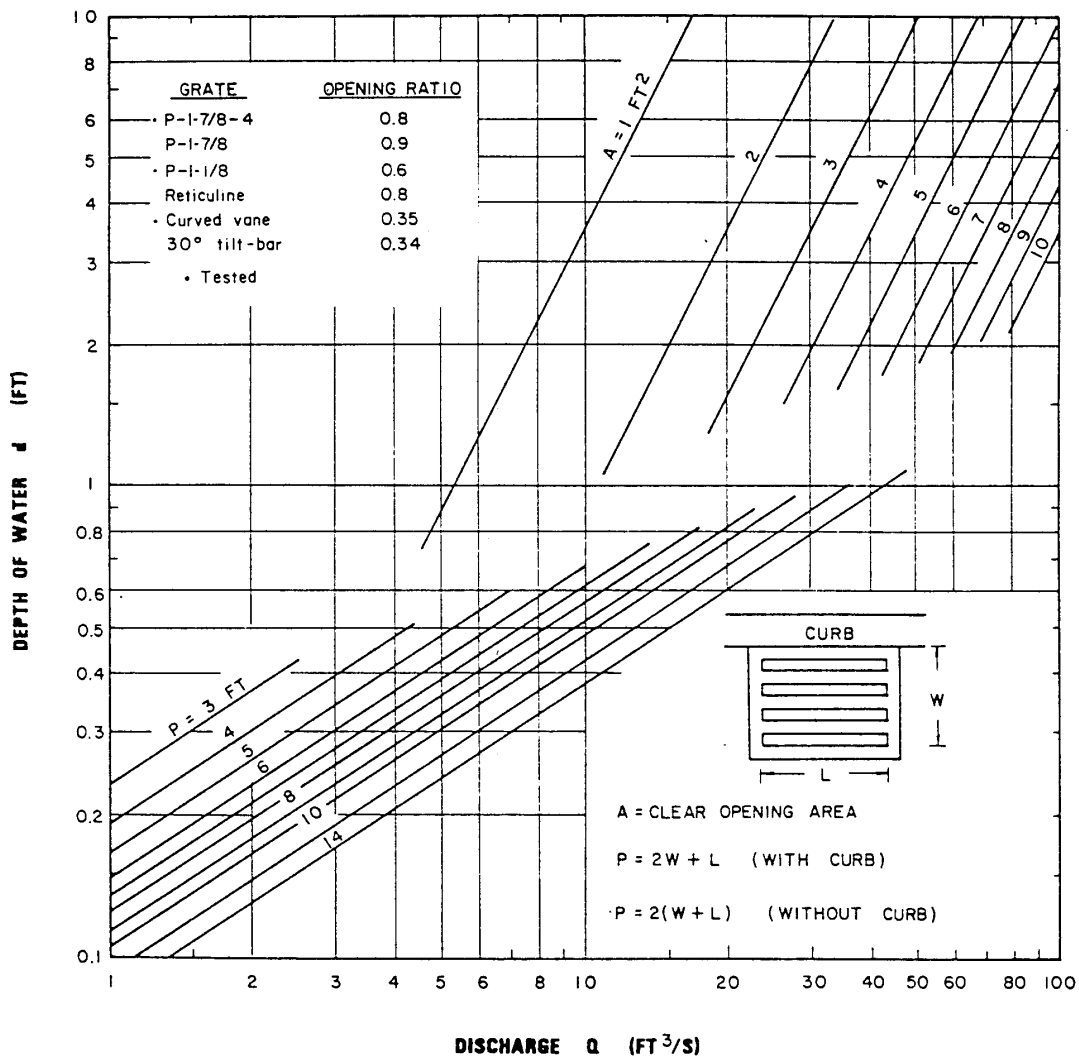
The following example illustrates the use of this figure.

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

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

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

Solution: From Figure 3-6, a grate must have a perimeter of 8 ft to intercept 8 cfs at a depth of 0.5 ft.



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

**Figure 3-6 Grate Inlet Capacity In Sump Conditions**

Source: HEC-12

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 percent 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 percent. For example if a 2-ft x 4-ft grate is clogged so that the effective width is 1.0-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 percent and the perimeter by 25 percent.



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Therefore, assuming 50 percent 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 percent clogged.

Assuming that the installation chosen to meet design conditions is a double 2 x 3 ft grate, for 50 percent clogged conditions:

$$P = 1 + 6 + 1 = 8 \text{ ft}$$

For 10-year flow:

$$d = 0.5 \text{ ft (from Figure 3-6)}$$

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

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

$$Q = 3.6 \text{ cfs, } T = 8.2 \text{ ft (10-year storm) (Figure 3-1)}$$

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

## 3.5 Curb Inlet Design

### 3.5.1 Curb Inlets On Grade

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

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope, is determined using Figure 3-7. The efficiency of curb-opening inlets shorter than the length required for total interception, is determined using Figure 3-8.

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

$$S_e = S_x + S'_w E_o \quad (3.12)$$

Where:  $E_o$  = ratio of flow in the depressed section to total gutter flow  
 $S'_w$  = cross slope of the gutter measured from the cross slope of the pavement,  
 $S_x$

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$$S'_w = (a/12W)$$

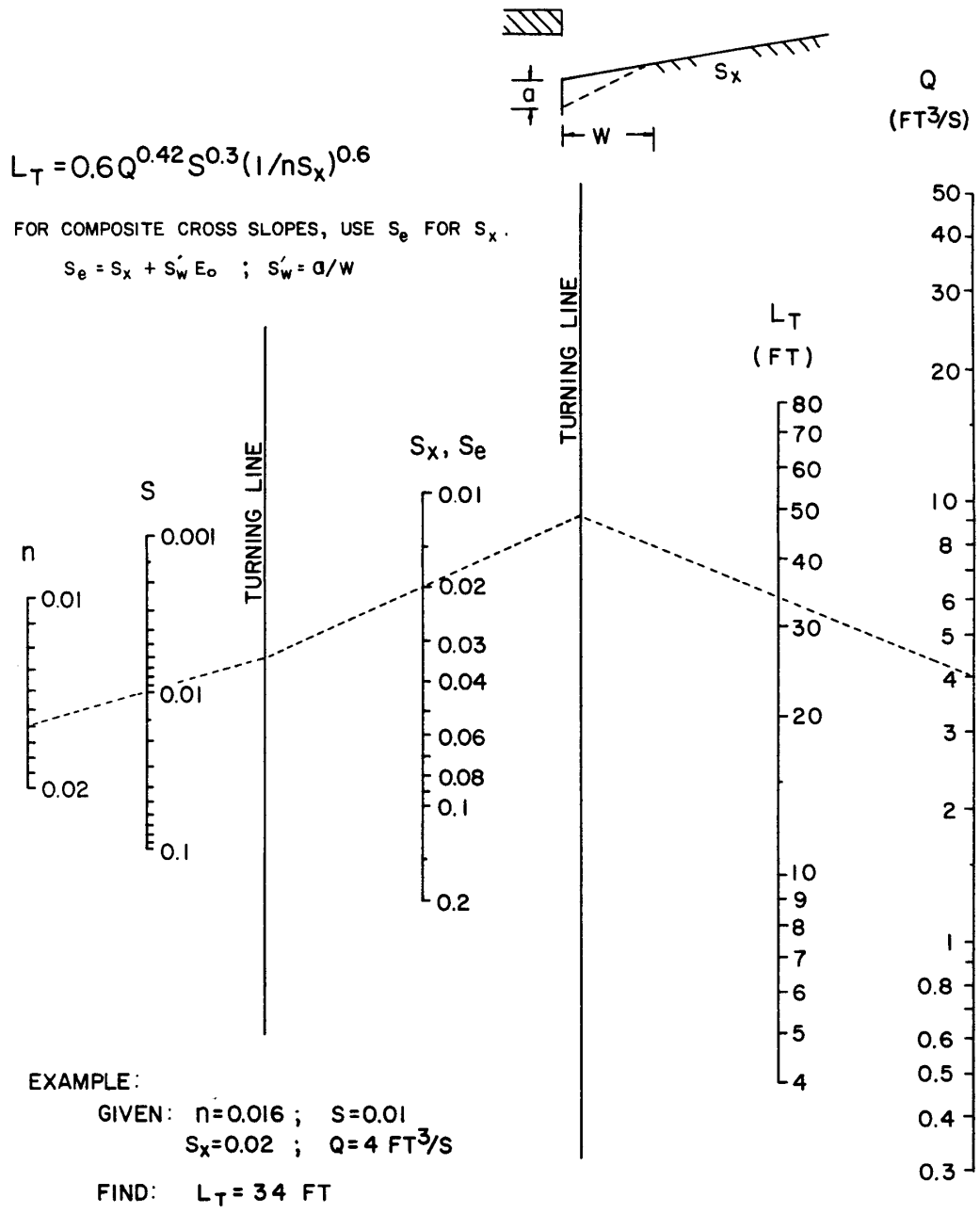
Where: a = gutter depression (in.)  
W = width of depressed gutter (ft)

It is apparent from examination of Figure 3-7 that the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

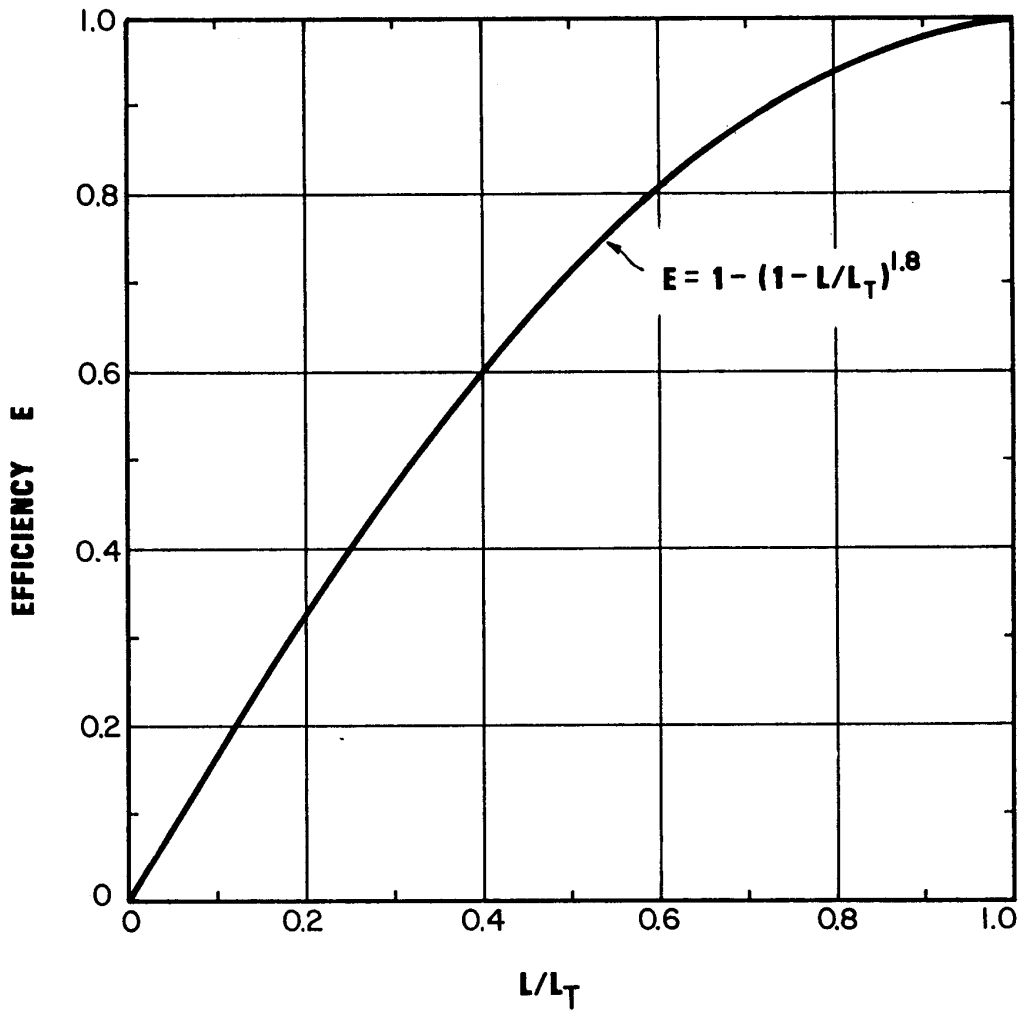
### 3.5.1.1 Design Steps

Steps for using Figures 3-7 and 3-8 in the design of curb inlets on grade are given below.

- (1) Determine the following input parameters:  
Cross slope =  $S_x$  (ft/ft)  
Longitudinal slope =  $S$  (ft/ft)  
Gutter flow rate =  $Q$  (cfs)  
Manning's  $n = n$   
Spread of water on pavement =  $T$  (ft) from Figure 3-1
- (2) Enter Figure 3-7 using the two vertical lines on the left side labeled  $n$  and  $S$ . Locate the value for Manning's  $n$  and longitudinal slope and draw a line connecting these points and extend this line to the first turning line.
- (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.
- (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  $L_T$ .
- (5) If the curb-opening inlet is shorter than the value obtained in step 4, Figure 3-8 can be used to calculate the efficiency. Enter the x-axis with the  $L/L_T$  ratio and draw a vertical line upward to the  $E$  curve. From the point of intersection, draw a line horizontally to the intersection with the y-axis and read the efficiency value.



**Figure 3-7 Curb-Opening And Slotted Drain Inlet Length For Total Interception**  
 Source: HEC-12



**Figure 3-8 Curb-Opening And Slotted Drain Inlet Interception Efficiency**

Source: HEC-12

### 3.5.1.2 Example

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

- Find: (1)  $Q_i$  for a 10-ft curb-opening inlet  
 (2)  $Q_i$  for a depressed 10-ft curb-opening inlet with  $a = 2$  in,  $W = 2$  ft,  
 $T = 8$  ft (Figure 3-1)

- Solution: (1) From Figure 3-7,  $L_T = 41$  ft,  $L/L_T = 10/41 = 0.24$   
 From Figure 3-8,  $E = 0.39$ ,  $Q_i = EQ = 0.39 \times 5 = 2$  cfs  
 (2)  $Q_n = 5.0 \times 0.016 = 0.08$  cfs  
 $S_w/S_x = (0.03 + 0.083)/0.03 = 3.77$   
 $T/W = 3.5$  (from Figure 3-3)  
 $T = 3.5 \times 2 = 7$  ft  
 $W/T = 2/7 = 0.29$  ft  
 $E_o = 0.72$  (from Figure 3-2)

$$\text{Therefore, } S_e = S_x + S'_w E_o = 0.03 + 0.083(0.72) = 0.09$$

$$\text{From Figure 3-7, } L_T = 23 \text{ ft, } L/L_T = 10/23 = 0.43$$

$$\text{From Figure 3-8, } E = 0.64, Q_i = 0.64 \times 5 = 3.2 \text{ cfs}$$

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

### 3.5.2 Curb Inlets In Sump

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

The capacity of curb-opening inlets in a sump location can be determined from Figure 3-9 which  
 accounts for the operation of the inlet as a weir and as an orifice at depths greater than 1.4h. This  
 figure is applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter  
 depression. The height (h) in the figure assumes a vertical orifice opening (see sketch on Figure  
 3-9). The weir portion of Figure 3-9 is valid for a depressed curb-opening inlet when  $d \leq (h +$   
 $a/12)$ .

The capacity of curb-opening inlets in a sump location with a vertical orifice opening but without  
 any depression can be determined from Figure 3-10. The capacity of curb-opening inlets in a  
 sump location with other than vertical orifice openings can be determined by using Figure 3-11.

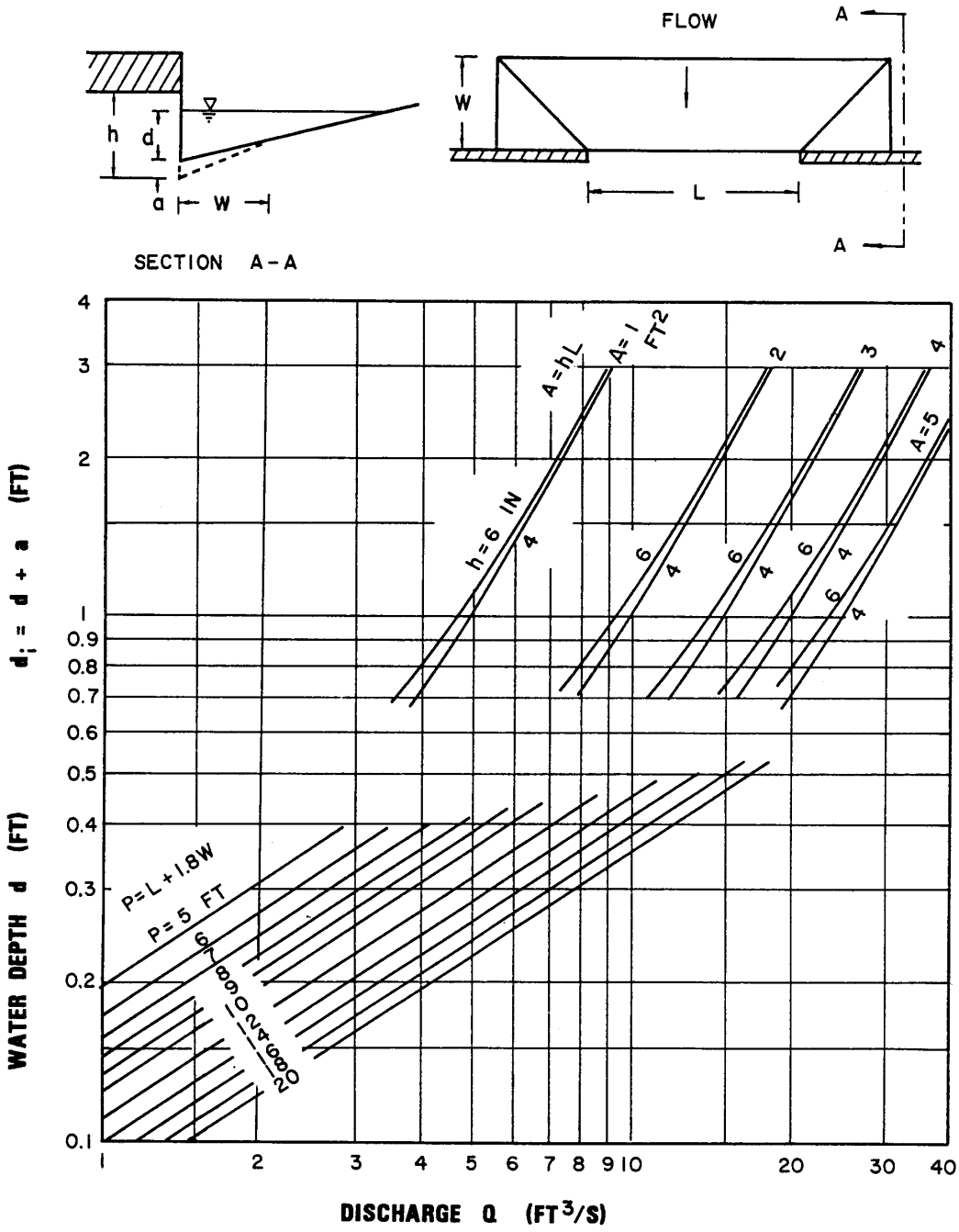


Figure 3-9 Depressed Curb-Opening Inlet Capacity In Sump Locations

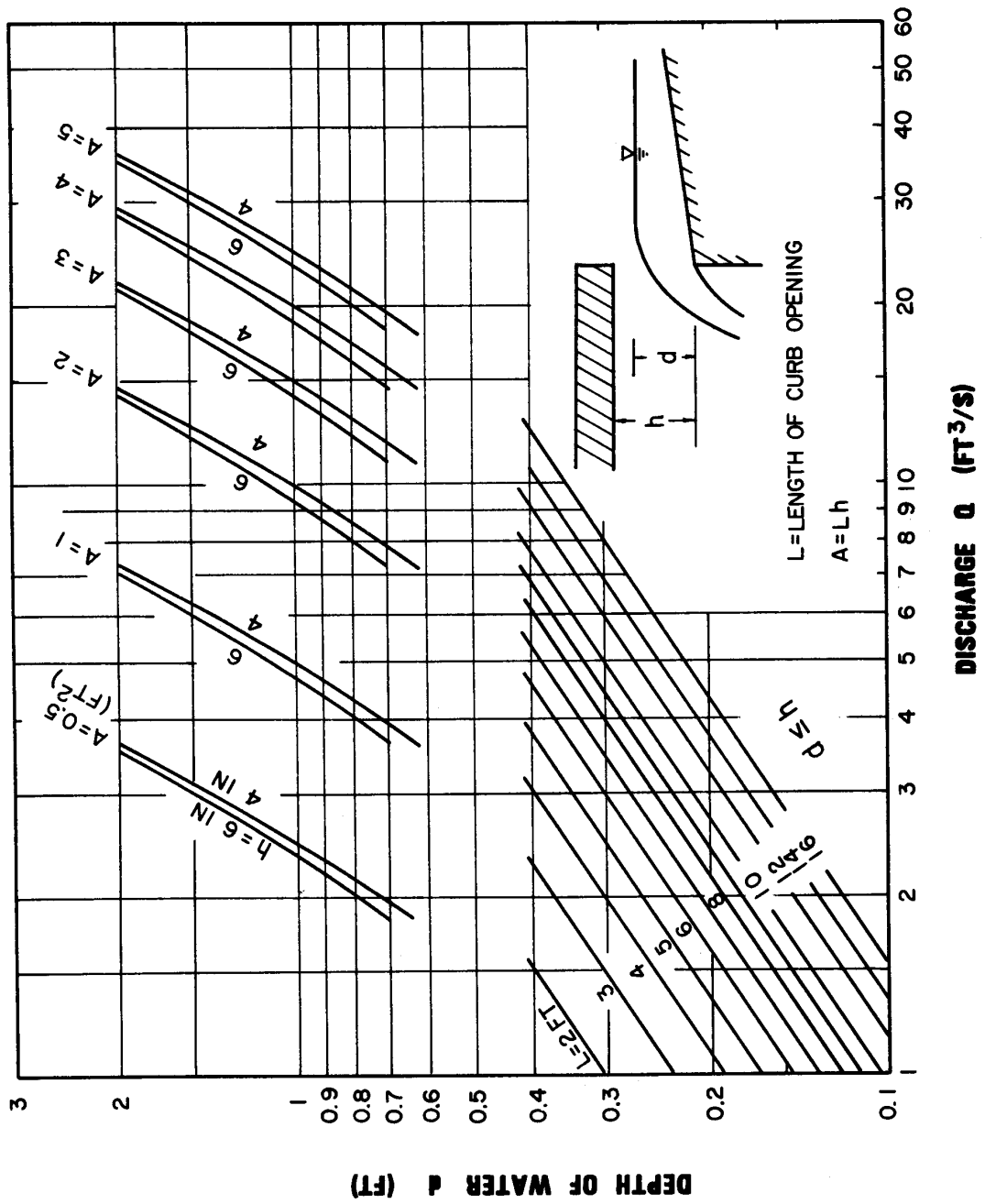
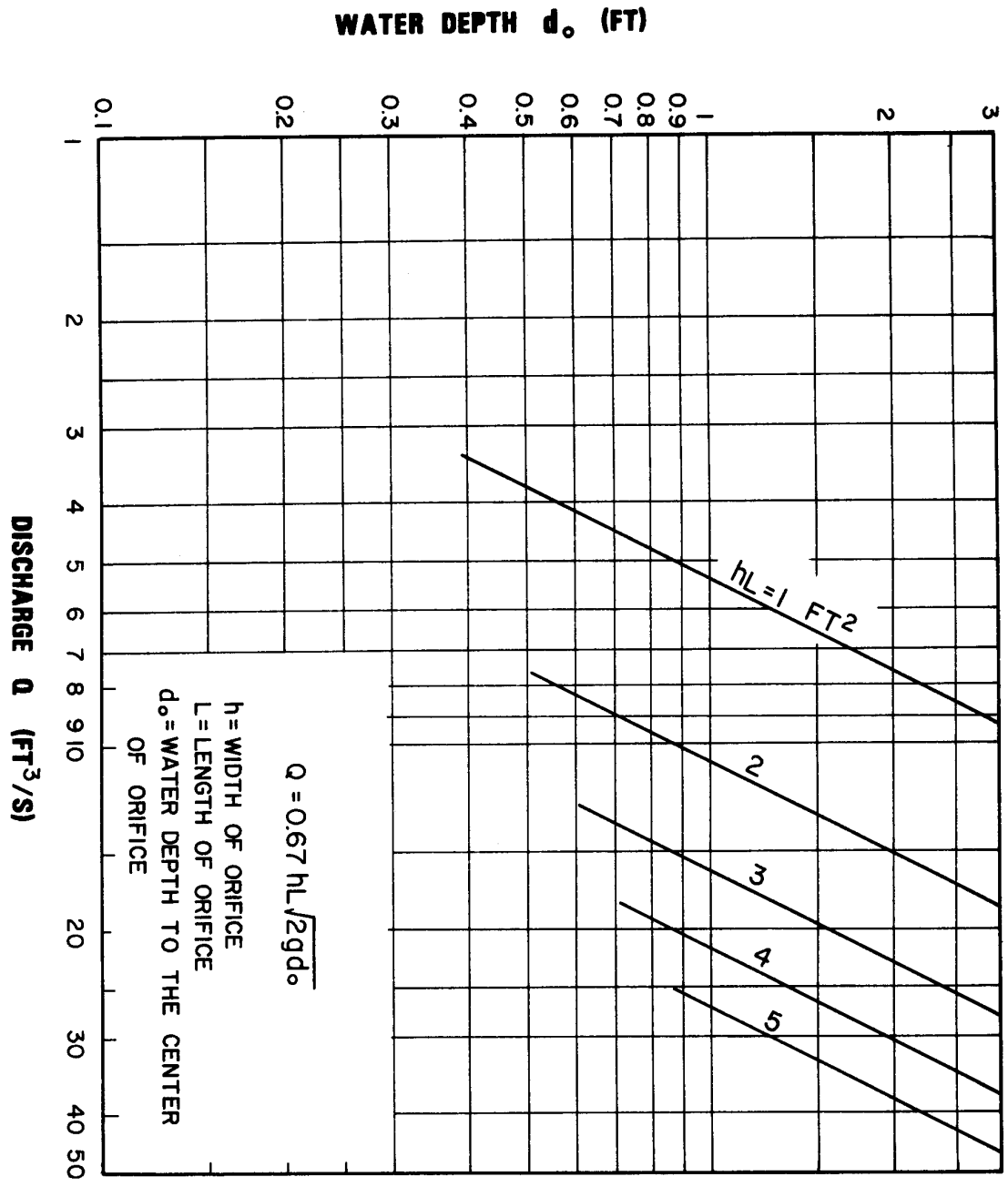


Figure 3-10 Curb-Opening Inlet Capacity In Sump Locations



**Figure 3-11 Curb-Opening Inlet Orifice Capacity For Inclined And Vertical Orifice Throats**

**3.5.2.1 Design Steps**

Steps for using Figures 3-9, 3-10, and 3-11 in the design of curb-opening inlets in sump locations are given below.

- (1) Determine the following input parameters:



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Cross slope =  $S_x$  (ft/ft)

Spread of water on pavement =  $T$  (ft) from Figure 3-1

Gutter flow rate =  $Q$  (cfs) or dimensions of curb-opening inlet [ $L$  (ft) and  $H$  (in)]

Dimensions of depression if any [ $a$  (in) and  $W$  (ft)]

- (2) To determine discharge given the other input parameters, select the appropriate Figure (3-9, 3-10, or 3-11 depending whether the inlet is in a depression and if the orifice opening is vertical).
- (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.
- (4) To determine the water depth given the discharge, use the procedure described in step 3 except you enter the figure at the value for the discharge on the x-axis.

### 3.5.2.2 Example

Given: Curb-opening inlet in a sump location

$$L = 5 \text{ ft}$$

$$h = 5 \text{ in}$$

- (1) Undepressed curb opening

$$S_x = 0.05 \text{ ft/ft}$$

$$T = 8 \text{ ft}$$

- (2) Depressed curb opening

$$S_x = 0.05 \text{ ft/ft}$$

$$a = 2 \text{ in}$$

$$W = 2 \text{ ft}$$

$$T = 8 \text{ ft}$$

Find: Discharge  $Q_i$

Solution: (1)  $d = TS_x = 8 \times 0.05 = 0.4 \text{ ft}$

$$d < h$$

From Figure 3-10,  $Q_i = 3.8 \text{ cfs}$

- (2)  $d = 0.4 \text{ ft}$

$$h + a/12 = (5 + 2/12)/12 = 0.43 \text{ ft}$$

since  $d < 0.43$  the weir portion of Figure 3-9 is applicable (lower portion of the Figure).

$$P = L + 1.8W = 5 + 3.6 = 8.6 \text{ ft}$$

From Figure 3-9,  $Q_i = 5 \text{ cfs}$

At  $d = 0.4 \text{ ft}$ , the depressed curb-opening inlet has about 30 percent more capacity than an inlet without depression.

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## 3.6 *Combination Inlets*

### 3.6.1 On Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of Figures 3-4, 3-5 and 3-6.

### 3.6.2 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 3-9, 3-10, and 3-11 for curb-opening inlets should be used for design.

## 3.7 *Storm Drains*

### 3.7.1 Introduction

After the tentative locations of inlets, drain pipes, and outfalls with tail-waters have been determined and the inlets sized, the next logical step is the computation of the rate of discharge to be carried by each drain pipe and the determination of the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from upstream of a line to 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 drain pipe serving that discharge is sized, and the process is repeated for the next run downstream. It should be recognized that the rate of discharge to be carried by any particular section of drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

For ordinary conditions, drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge. The Manning Formula is recommended for capacity calculations.

### 3.7.2 Design Criteria

The standard recommended maximum and minimum slopes for storm drains should conform to the following criteria:

1. The maximum hydraulic gradient should not produce a velocity that exceeds 15 feet per second.
2. The minimum desirable physical slope should be 0.5 percent or the slope which will produce a velocity of 3.0 feet per second when the storm sewer is flowing full, whichever is greater.

If the potential water surface elevation exceeds the gutter invert or grate elevation for the design flow, adjustments are needed in the system to reduce the elevation of the hydraulic grade line.

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### 3.7.3 Hydraulic Grade Lines

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

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

#### Design Procedure - Outlet Control

The head losses are calculated beginning from the control point to the first junction and the procedure is repeated for the next junction. The computation for an outlet control may be tabulated on Figure 3-12 using the following procedure:

1. Enter in Col. 1 the station for the junction immediately upstream of the outflow pipe. Hydraulic grade line computations begin at the outfall and are worked upstream taking each junction into consideration.
2. Enter in Col. 2 the outlet water surface elevation if the outlet will be submerged during the design storm or 0.8 diameter plus invert out elevation of the outflow pipe whichever is greater.
3. Enter in Col. 3 the diameter ( $D_o$ ) of the outflow pipe.
4. Enter in Col. 4 the design discharge ( $Q_o$ ) for the outflow pipe.
5. Enter in Col. 5 the length ( $L_o$ ) of the outflow pipe.
6. Enter in Col. 6 the friction slope ( $S_f$ ) in ft/ft of the outflow pipe. This can be determined by using the following formula:

$$S_f = (Q^2)/K \quad (3.13)$$

Where:  $S_f$  = friction slope

$$K = [1.486 AR^{2/3}]/n$$

7. Multiply the friction slope ( $S_f$ ) in Col. 6 by the length ( $L_o$ ) in Col. 5 and enter the friction loss ( $H_f$ ) in Col. 7. On curved alignments, calculate curve losses by using the formula  $H_c = 0.002 (\Delta)(V_o^2/2g)$ , where  $\Delta$  = angle of curvature in degrees and add to the friction loss.
8. Enter in col. 8 the velocity of the flow ( $V_o$ ) of the outflow pipe.
9. Enter in Col. 9 the contraction loss ( $H_o$ ) by using the formula:  $H_o = [0.25 (V_o^2)] / 2g$ , where  $g = 32.2 \text{ ft/s}^2$ .
10. Enter in Col. 10 the design discharge ( $Q_i$ ) for each pipe flowing into the junction. Neglect lateral pipes with inflows of less than ten percent of the mainline outflow. Inflow must be adjusted to the mainline outflow duration time before a comparison is made.

- 
11. Enter in Col. 11 the velocity of flow ( $V_i$ ) for each pipe flowing into the junction (for exception see Step 10).
  12. Enter in Col. 12 the product of  $Q_i \times V_i$  for each inflowing pipe. When several pipes inflow into a junction, the line producing the greatest  $Q_i \times V_i$  product is the one that should be used for expansion loss calculations.
  13. Enter in Col. 13 the controlling expansion loss ( $H_i$ ) using the formula:  
 $H_i = [0.35 (V_i^2)] / 2g$ .
  14. Enter in Col. 14 the angle of skew of each inflowing pipe to the outflow pipe (for exception, see Step 10).
  15. Enter in Col. 15 the greatest bend loss ( $H$ ) calculated by using the formula  $H = [KV_i^2] / 2g$  where  $K$  = the bend loss coefficient corresponding to the various angles of skew of the inflowing pipes.
  16. Enter in Col. 16 the total head loss ( $H_t$ ) by summing the values in Col. 9 ( $H_o$ ), Col. 13 ( $H_i$ ), and Col. 15 ( $H_\Delta$ ).
  17. If the junction incorporates adjusted surface inflow of ten percent or more of the mainline outflow, i.e., drop inlet, increase  $H_t$  by 30 percent and enter the adjusted  $H_t$  in Col. 17.
  18. If the junction incorporates full diameter inlet shaping, such as standard manholes, reduce the value of  $H_t$  by 50 percent and enter the adjusted value in Col. 18.
  19. Enter in Col. 19 the FINAL H, the sum of  $H_f$  and  $H_t$ , where  $H_t$  is the final adjusted value of the  $H_t$ .
  20. Enter in Col. 20 the sum of the elevation in Col. 2 and the Final H in Col. 19. This elevation is the potential water surface elevation for the junction under design conditions.
  21. Enter in Col. 21 the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Col. 20. If the potential water surface elevation exceeds one foot below ground elevation for the design flow, the top of the pipe or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the Hydraulic Grade Line (H.G.L.).
  22. Repeat the procedure starting with Step 1 for the next junction upstream.
  23. At last upstream entrance, add  $V_1^2/2g$  to get upstream water surface elevation.

HYDRAULIC GRADE LINE										PROJECT											
STATION	Outlet Water Surface Elev.	D <sub>o</sub>	Q <sub>o</sub>	L <sub>o</sub>	S <sub>f<sub>o</sub></sub> %	H <sub>f</sub>	JUNCTION LOSS								FINAL H	Inlet Water Surface Elev.	Rim Elev.				
							V <sub>o</sub>	H <sub>o</sub>	Q <sub>1</sub>	V <sub>1</sub>	Q <sub>1</sub> V <sub>1</sub>	$\frac{V_1^2}{2g}$	H <sub>1</sub>	ANGLE				H <sub>Δ</sub>	H <sub>1</sub>	1.3 H <sub>1</sub>	0.5 H <sub>1</sub>
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	

$H_1 = 0.35 \frac{V_1^2}{2g}$       $H_o = 0.25 \frac{V_o^2}{2g}$       $H_s = K \frac{V_1^2}{2g}$      90° K = 0.70     50° K = 0.47     20° K = 0.16  
 $H_t = H_o + H_1 + H_s$      80° K = 0.66     40° K = 0.38     15° K = 0.10  
 $H_t = H_o + H_1 + H_s$      70° K = 0.61     30° K = 0.28  
     60° K = 0.55     25° K = 0.22

Figure 3-12 Hydraulic Grade Line Computation Form

STORM DRAIN COMPUTATION SHEET

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

**Figure 3-13 Storm Sewer Computation Form**

**3.7.4 Minimum Grade**

All storm drains should be designed such that velocities of flow will not be less than 3.0 feet per second at design flow or lower, with a minimum slope of 0.5 percent. For very flat flow lines the general practice is to design components so that flow velocities will increase progressively

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throughout the length of the pipe system. Upper reaches of a storm drain system should have flatter slopes than slopes of lower reaches. Progressively increasing slopes keep solids moving toward the outlet and deters settling of particles due to steadily increasing flow streams.

The minimum slopes are calculated by the modified Manning formula:

$$S = [(nV)^2]/[2.208 R^{4/3}] \quad (3.14)$$

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 divided by wetted perimeter)

### 3.7.5 Design Procedures

The design of storm drain systems is generally divided into the following operations:

1. The first step is the determination of inlet location and spacing as outlined earlier in this chapter.
2. The second step is the preparation of a plan layout of the storm sewer drainage system establishing the following design data:
  - 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.
3. The design of the storm drain system is then accomplished by determining drainage areas, computing runoff by rational method, and computing the hydraulic capacity by Manning's equation.
4. The storm drain design computation sheet (Figure 3-13) can be used to summarize the hydrologic, hydraulic and design computations.
5. Examine all assumptions to determine if any adjustments are needed to the final design.

## References

U. S. Department of Transportation, Federal Highway Administration, 1984. Drainage of Highway Pavements. Hydraulic Engineering Circular No. 12.





