Omaha Regional Stormwater Design Manual

Storm Drainage System

Chapter 3

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City of Omaha Environmental Quality Control Division
www.omahastormwater.org

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3.1 Overview

3.1.1 Introduction

Every urban area has two distinct drainage systems, the minor system and the major system. To provide for orderly urban growth, reduce costs to taxpayers, and obviate loss of life and property damage, both systems must be planned and properly engineered.

In this chapter, guidelines are given for evaluating and designing storm drainage system, such as pipes, inlets, curbs and gutters. The 10-year frequency storm is to be used for design of the minor drainage system. The 100-year frequency storm is to be used for the major drainage system. If downstream drainage facilities are undersized for the design flow, a detention structure may be needed to reduce the possibility of flooding. Storm sewer systems shall be designed and constructed in accordance with Omaha Public Works Department Standard Specifications.

3.1.2 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, Definitions And Units

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	Length of curb opening inlet	ft.
L_T	Length of curb opening inlet required for total interception of gutter flow	
P	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_i	Intercepted flow	cfs
Q_s	Gutter capacity above the depressed section	cfs
R	Hydraulic radius	ft.
S or S_x	Cross slope - Traverse slope	ft./ft.
S or S_L	Longitudinal slope of pavement	ft./ft.
S_f	Friction slope	ft./ft.
S_w	Gutter Slope	-
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	-

3.1.3 Concept Definitions

Definitions of concepts important in storm drain analysis and design used in this chapter are presented below.

Bypass — Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade. Inlets may be designed to allow a certain amount of bypass for one design storm and larger or smaller amounts for other design storms. The spread for lower catch basins must consider a reasonable calculated bypass flow from upper facilities.

Curb-Opening Inlet — A drainage inlet consisting of an opening in the roadway curb.

Cross-Over — Flow received by an inlet that has crossed over the crown of the road from the opposite side of the inlet. This is a common condition in residential streets where flow depth is allowed to reach the top of curb and can overtop the crown of the road.

Cross-Pan —A valley gutter, or continuation of a street gutter flow line across an intersecting street.

Drop Inlet — A drainage inlet with a horizontal or nearly horizontal opening.

Equivalent Cross Slope —
An imaginary continuous cross slope having conveyance capacity equal to that of the given compound cross slope.

Flanking Inlets — Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets are to intercept debris as the slope decreases and to act in relief of the inlet at the low point.

Frontal Flow — The portion of the flow which passes over the upstream side of a grate.

Grate Inlet — A drainage inlet composed of a grate in a gutter, parking lot, alley or area drain.

Gutter — That portion of the roadway section adjacent to the curb which is utilized to convey storm runoff water. It may include a portion or all of a traveled lane, shoulder or parking lane, and a limited width adjacent to the curb may be of different materials and have a different cross slope.

Hydraulic Grade Line — The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run.

Inlet Efficiency — The ratio of flow intercepted by an inlet to total flow in the gutter.

Pressure Head — Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

Scupper — A vertical hole through a bridge deck for the purpose of deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.

Side-Flow Interception — Flow which is intercepted along the side of a grate inlet, as opposed to frontal interception.

Slotted Drain Inlet — A drainage inlet composed of a continuous slot built into the top of a pipe which serves to intercept, collect and transport the flow.

Splash-Over — Portion of the frontal flow at a grate which skips or splashes over the grate and is not intercepted.

Spread — The width of flow measured laterally from the roadway curb.

Velocity Head — Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water.

For a more complete discussion of these concepts and others related to storm drain design, the reader is referred to: Drainage of Highway Pavements, Federal Highway Administration, Hydraulic Engineering Circular No. 12, March 1984.

3.2 Pavement Drainage

3.2.1 Introduction

There are many details to consider in the design and specification of storm drain systems. ASCE Manuals of Engineering Practice (1960, 1982, 1983) as well as other trade and vendor publications provide construction and specification details beyond the scope of this text. During the design phase, the system drainage area is defined and preliminary drainage routes are identified based on hydrologic analyses. Integration of the system with environmental features and neighborhood amenities should be assessed, and the location of quantity and quality control structures is determined.

The hydrologic analyses should include defining drainage areas for each inlet or ditch start, developing flow estimates for design frequencies throughout the system, and development of flow and spread calculations to determine permissible maximum spread.

Typical design factors to be considered during gutter, inlet, and pavement drainage calculations include:

- Return period
- Spread
- Storm drain location
- Inlet types and spacing
- Longitudinal slope
- Cross slope
- Curb and gutter sections
- Roadside and median channels
- Bridge decks
- Shoulder
- Median/Median barriers

3.2.2 Return Period

The design storm return period for pavement drainage should be consistent with the frequency selected for other components of the drainage system. The major considerations for selecting a design frequency are roadway classification, roadway speed, hazards, and pedestrian traffic.

3.2.3 Street Inundation

For multi-lane curb and gutter or guttered roadways with no parking, it is not practical to avoid travel lane encroachment by flooding when grades are flat. Allowable maximum street encroachment by stormwater as listed in the following tables.

Table 3-2 Allowable Maximum Street Encroachment by Minor (10-Year) Storms

Street Classification	Maximum Encroachment
Local	No curb overtopping. Flow may cover crown of street.*
Collector	No curb overtopping. Flow may cover all lanes of roadway, but not cover crown.*
Arterial	No curb overtopping. Flow spread must leave at least one lane free of water in each direction.
Freeway	No encroachment allowed on any traffic lane.

* Where no curbing exists, encroachment shall not extend over property lines except at drainage easements.

Storm drainage system construction is required to commence, and continue downstream from locations where the 10-year design storm would cause the allowable maximum encroachment of a street cross-section.

Table 3-3 Allowable Maximum Street Encroachment by Minor (100-Year) Storms

Street Classification	Maximum Encroachment
Local and Collector	The depth of water over the gutter flowline shall not exceed a depth that would result in water spreading beyond right-of-way width or easement boundaries. The maximum allowable depth at the gutter is 18 in.
Arterials and Freeway	Maximum depth of water at the street crown shall not exceed 6 in. to allow the operation of emergency vehicles. The depth of the water over the gutter flowline shall not result in water spreading beyond right-of-way or easement boundaries.

Table 3-4 Allowable Maximum Cross-Street Flow

Street Classification	Major Storm Design Runoff	Major Storm Design Runoff
Local	6 in. depth at crown. Where cross-pans allowed, depth shall not exceed 6 in.	The depth of water over the gutter flowline shall not exceed 18 in. maximum, or result in water spreading beyond right-of-way or easement boundaries.
Collector	Where cross-pans allowed, depth of flow shall not exceed 6 in.	The depth of water over the gutter flowline shall not exceed 18 in. maximum, or result in water spreading beyond right-of-way or easement boundaries.
Arterial	None	6 in. or less over crown, but not more than an 18 in. depth at the gutter. The cross-flow depth shall not result in water spreading beyond right-of-way or easement boundaries.
Freeway	None	6 in. or less over crown, but not more than an 18 in. depth at the gutter. The cross-flow depth shall not result in water spreading beyond right-of-way or easement boundaries.

3.2.4 Longitudinal Slope

To provide for drainage, and to avoid unacceptable stormwater spread into traffic lanes, curb and gutter grades shall not be less than 0.5 percent without approval from the Public Works Department.

3.2.5 Cross Slope

Roadway cross slopes are determined by the City of Omaha standard roadway sections. Drainage from median areas should not cross traveled lanes. Median shoulders should generally be sloped to drain away from the pavement. Narrow, raised medians are not subject to these provisions.

3.2.6 Curb and Gutter

Curb and gutter installation shall be designed in accordance with the most current Omaha Standard Plates and Specifications.

3.2.7 Roadside and Median Channels

Curbed highway sections are relatively inefficient at conveying water. The area tributary to the gutter section should be kept to a minimum to reduce the hazard from water on the pavement. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted by channels or storm sewer, and routed away from the highway pavement.

3.2.8 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. It is often less efficient, because cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets on scuppers have a higher potential for clogging by debris. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

Scuppers are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. For situations where traffic under the bridge or environmental concerns prevent the use of scuppers, grated bridge drains should be used.

Drainage from bridges must be collected before overflowing onto unprotected stream banks and properly conveyed to outlet safely into the channel to prevent streambank erosion. Proposed conveyance pipe entrances into channels shall meet the design criteria in Section 3.7.

3.2.9 Median/Barriers

Weep holes are often used to prevent ponding of water against barriers (especially on super elevated curves). In order to minimize flow across traveled lanes, it is preferable to collect the water into a subsurface system connected to the main storm drain system.

3.3 Gutter Flow Calculations

3.3.1 General

Flow capacities of the City of Omaha standard 25 ft. residential street cross-section are provided in Figure 3-1. Flow capacities of City of Omaha non-residential standard street cross-sections of various widths are provided in Figures 3-2a, b, c, d, e, f, g, h, and i. For non-standard applications, refer to Sections 3.3.2 through 3.3.7.

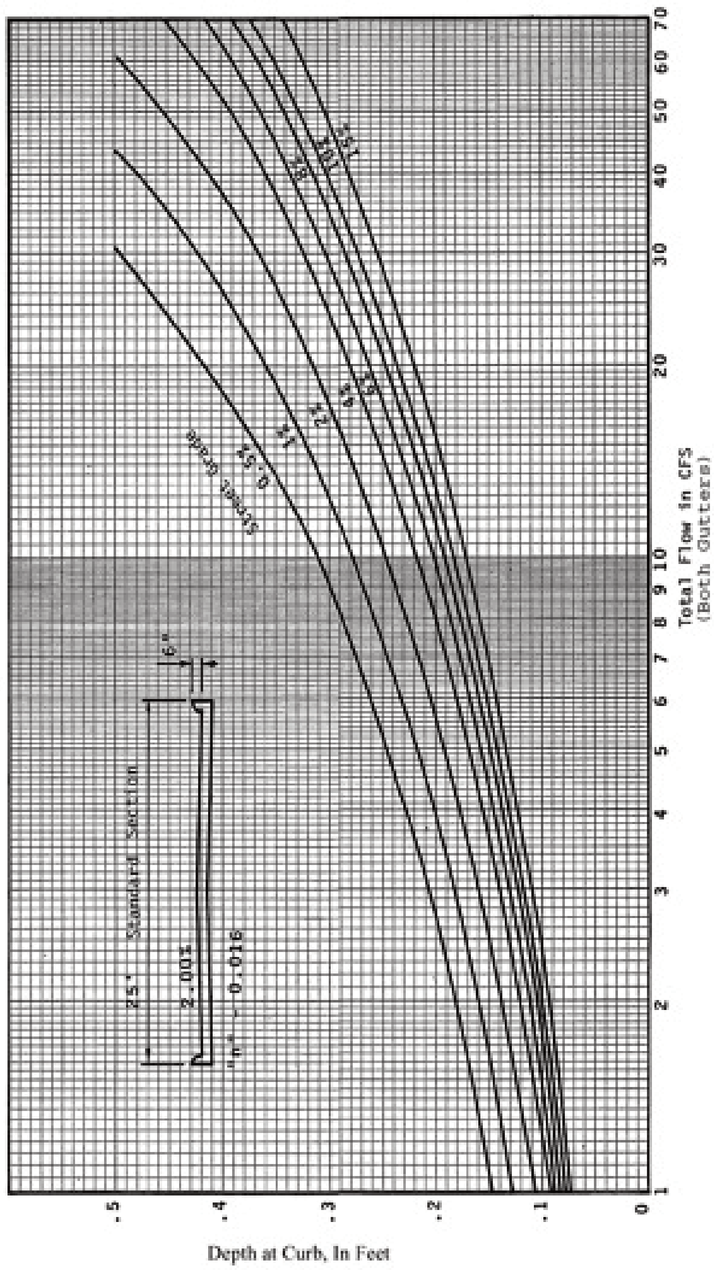


Figure 3-1 Flow In Residential Streets

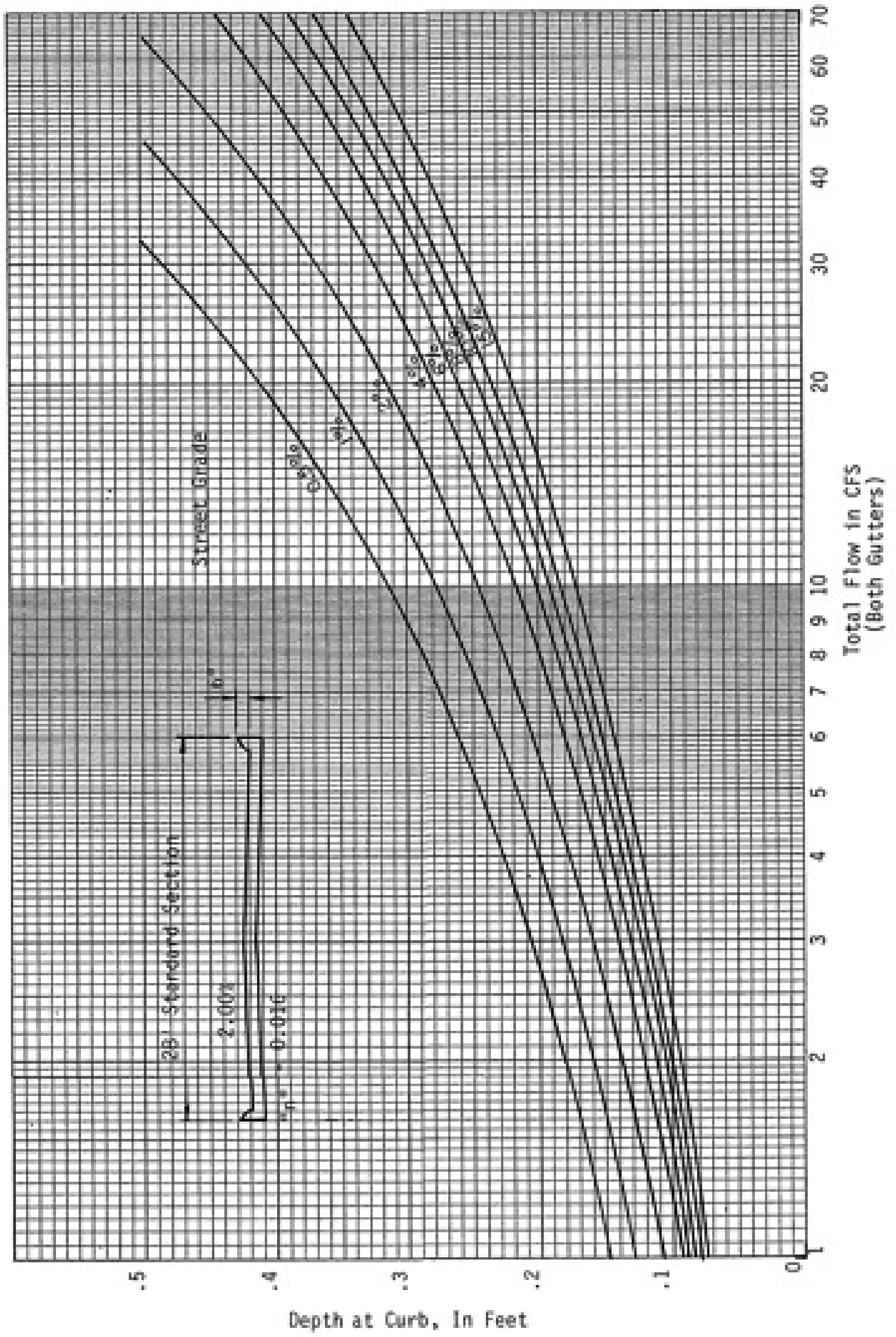


Figure 3-2a Flow In Street

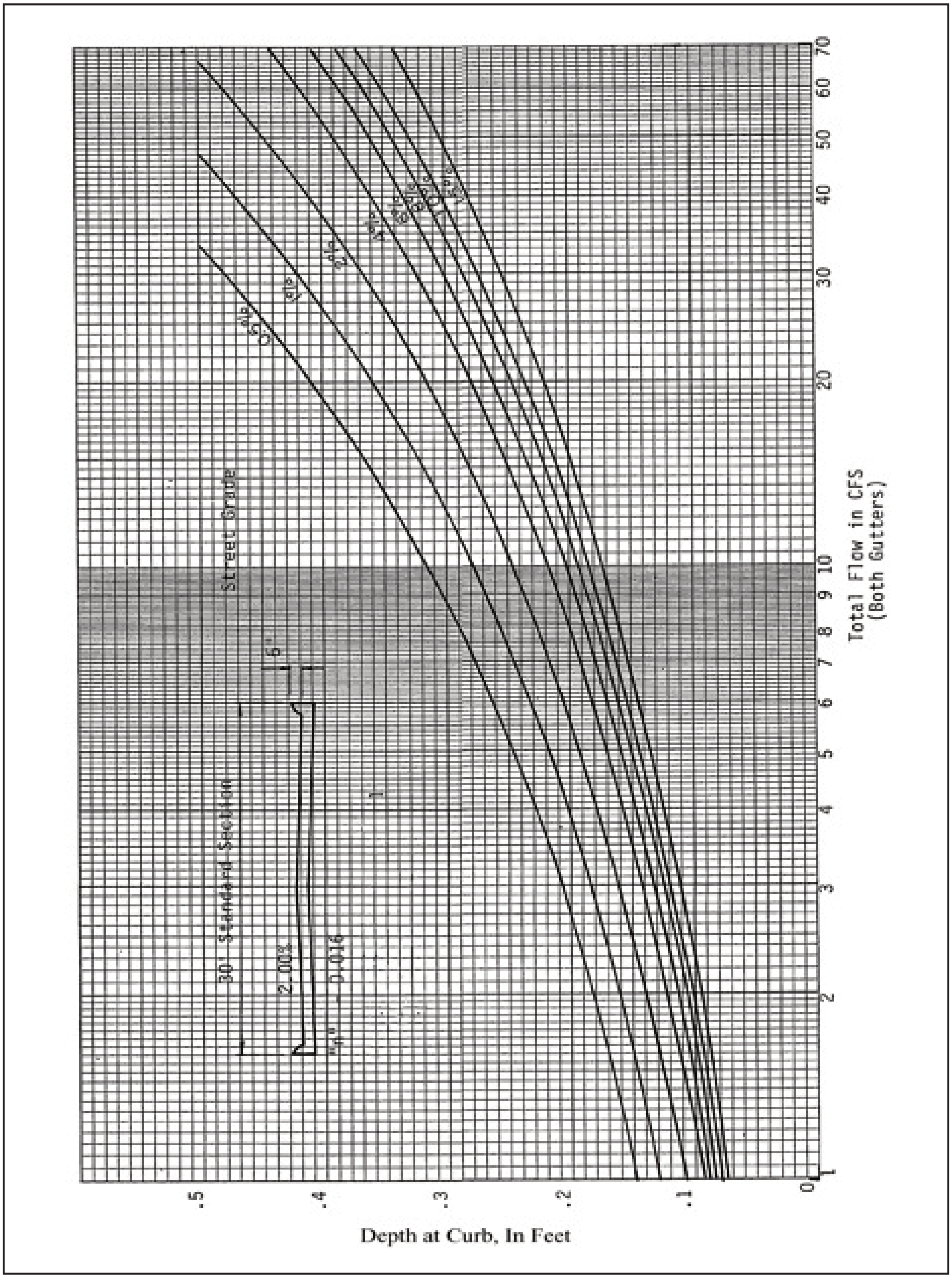


Figure 3-2b Flow In Street

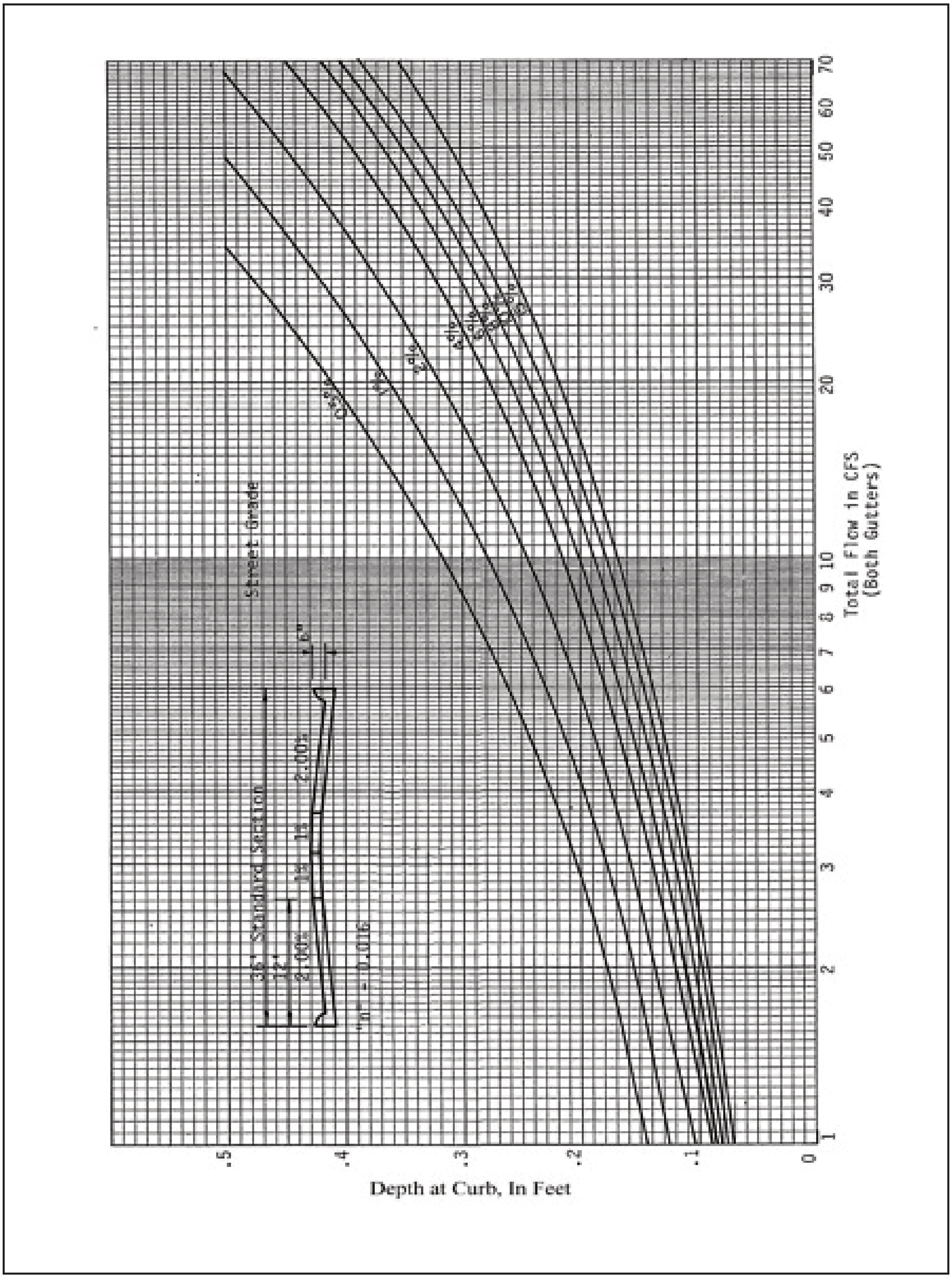


Figure 3-2c Flow In Street

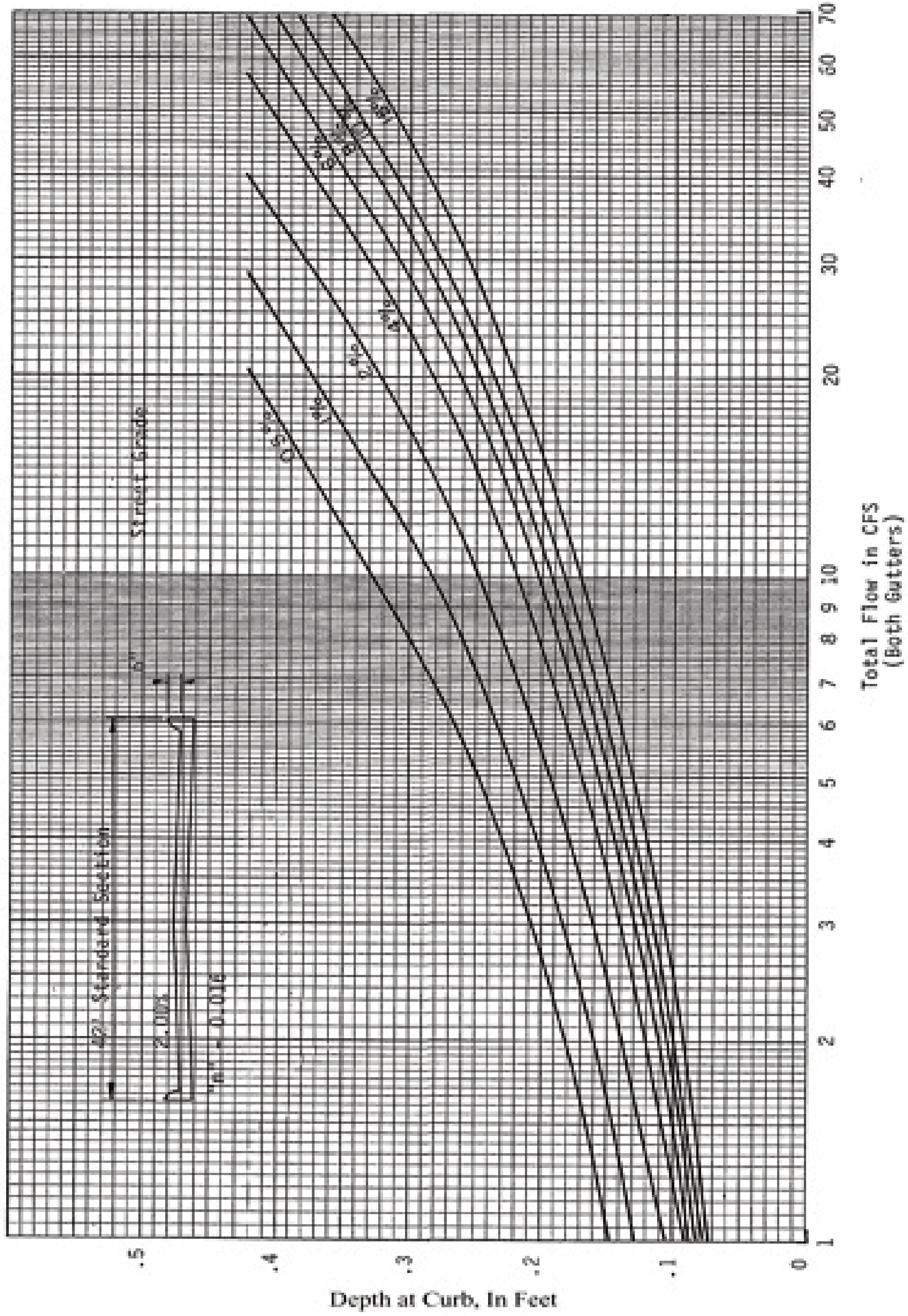


Figure 3-2d Flow In Street

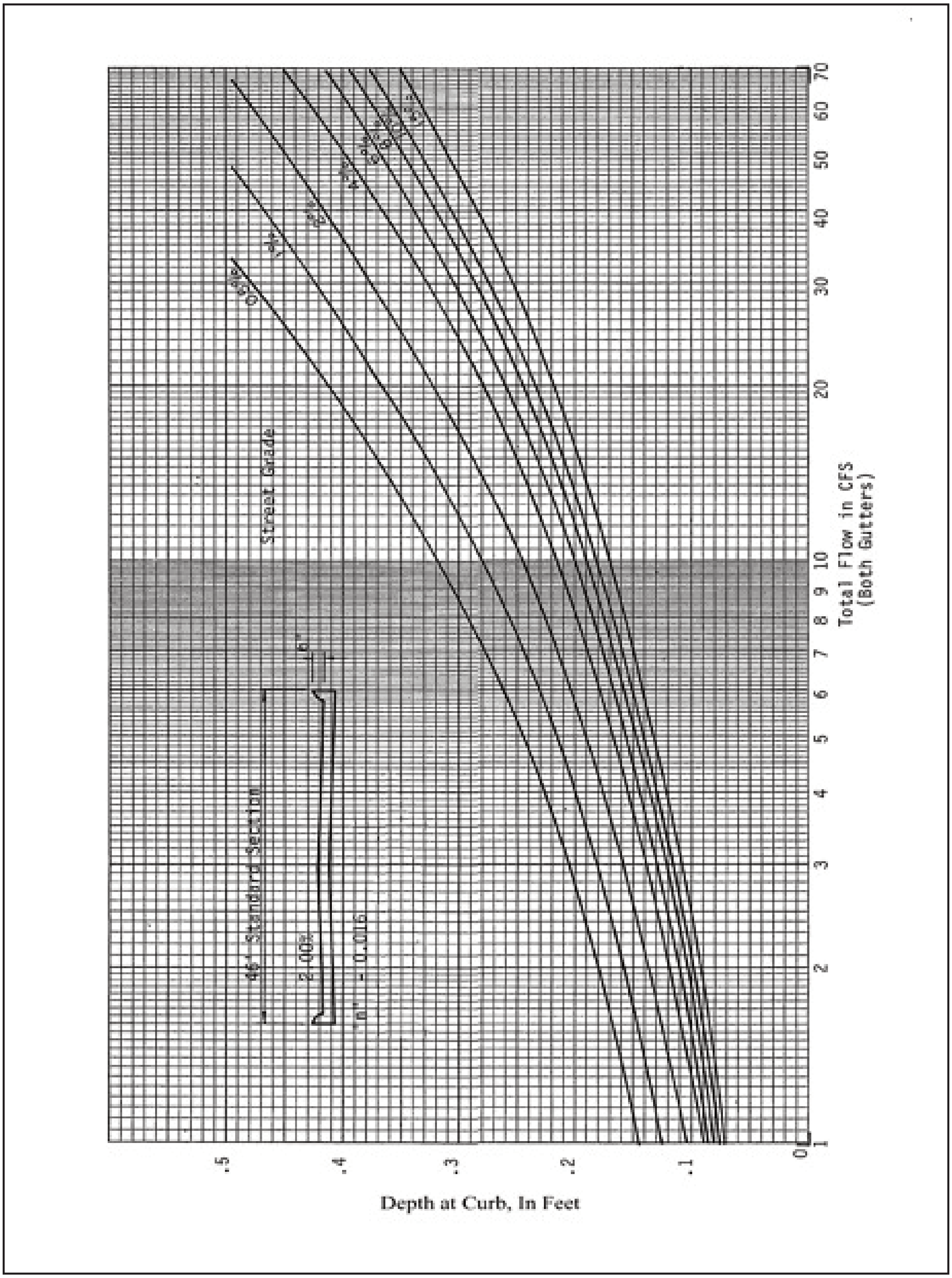


Figure 3-2e Flow In Street

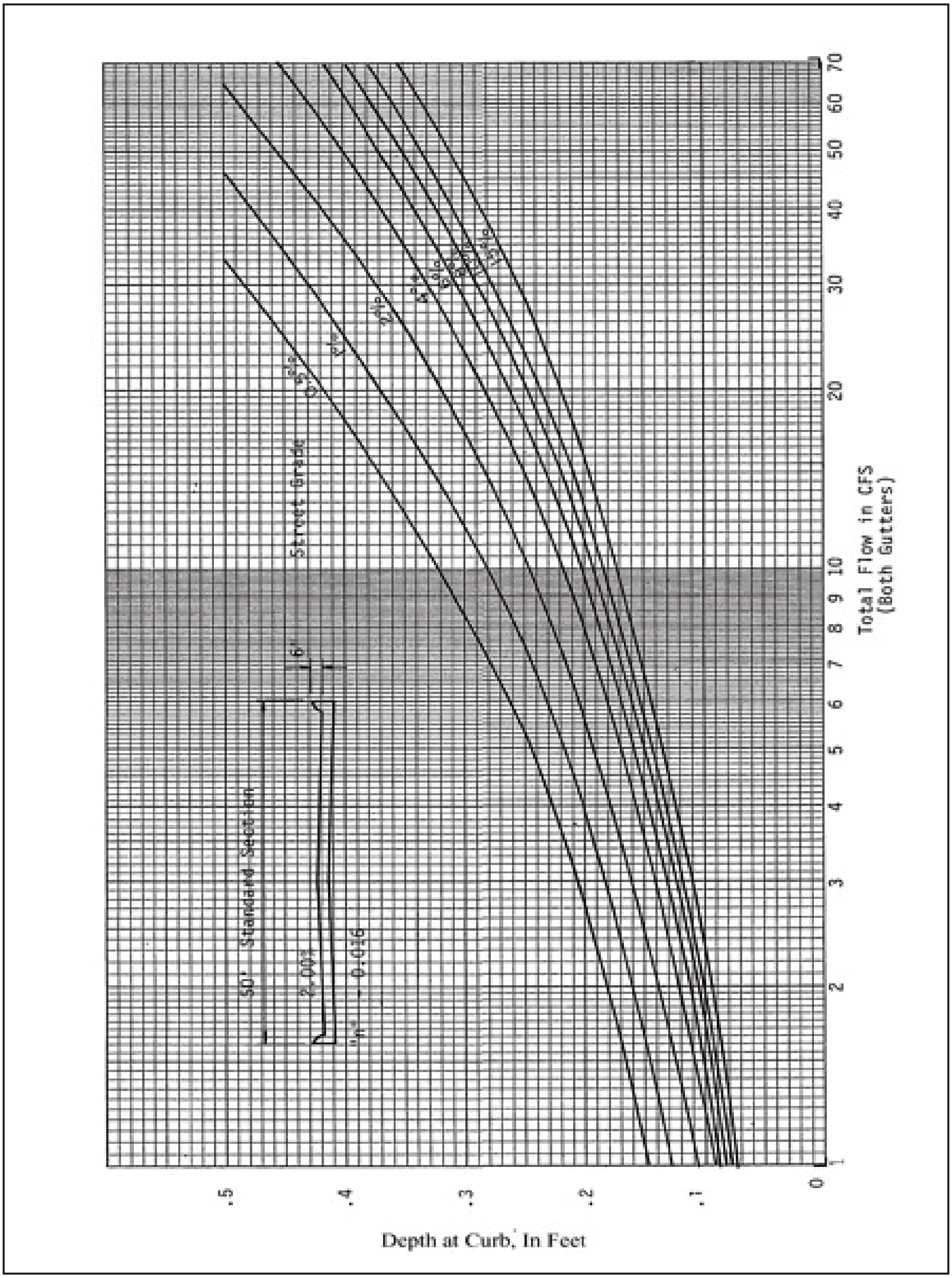


Figure 3-2f Flow In Street

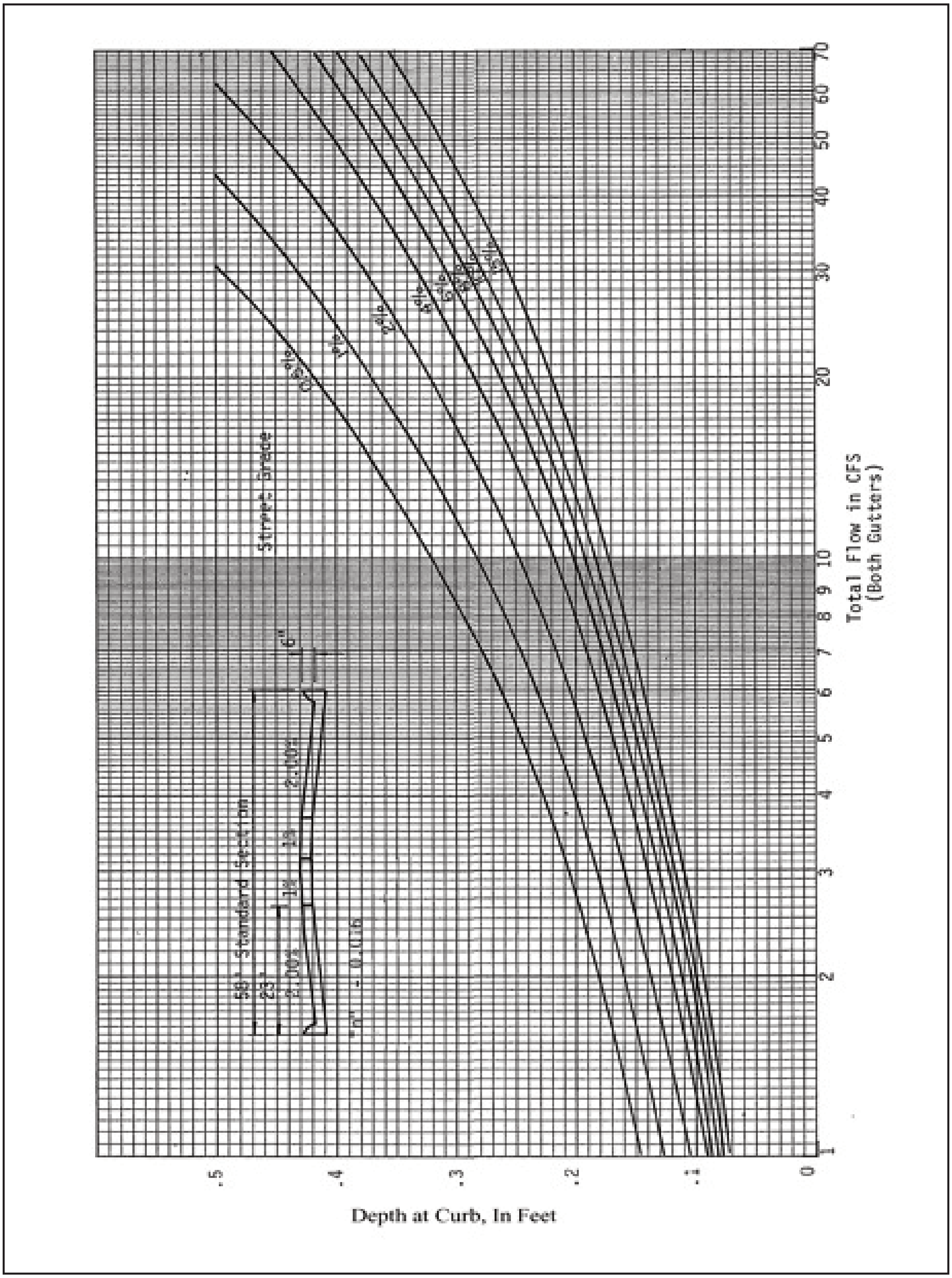


Figure 3-2g Flow In Street

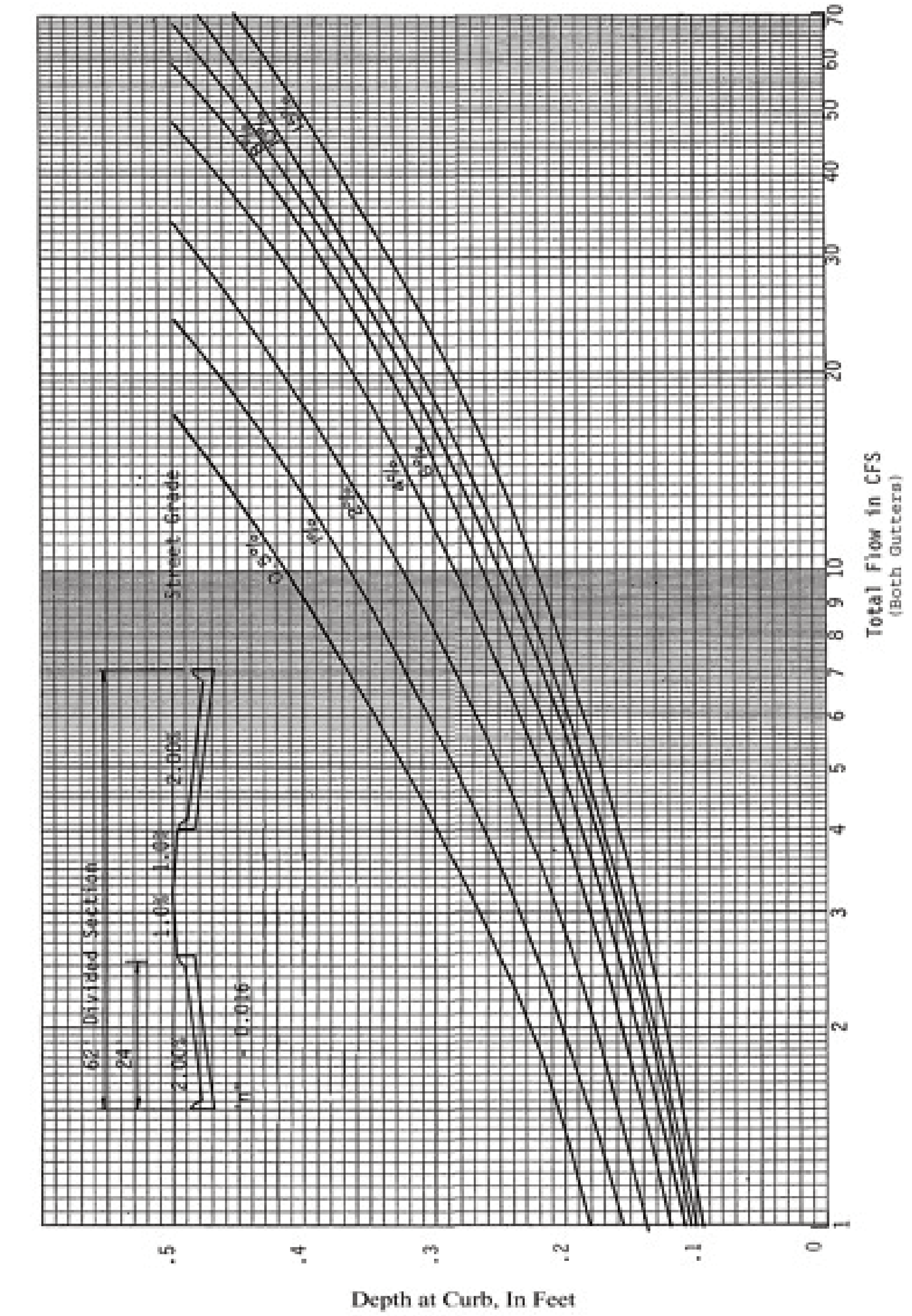


Figure 3-2h Flow In Street

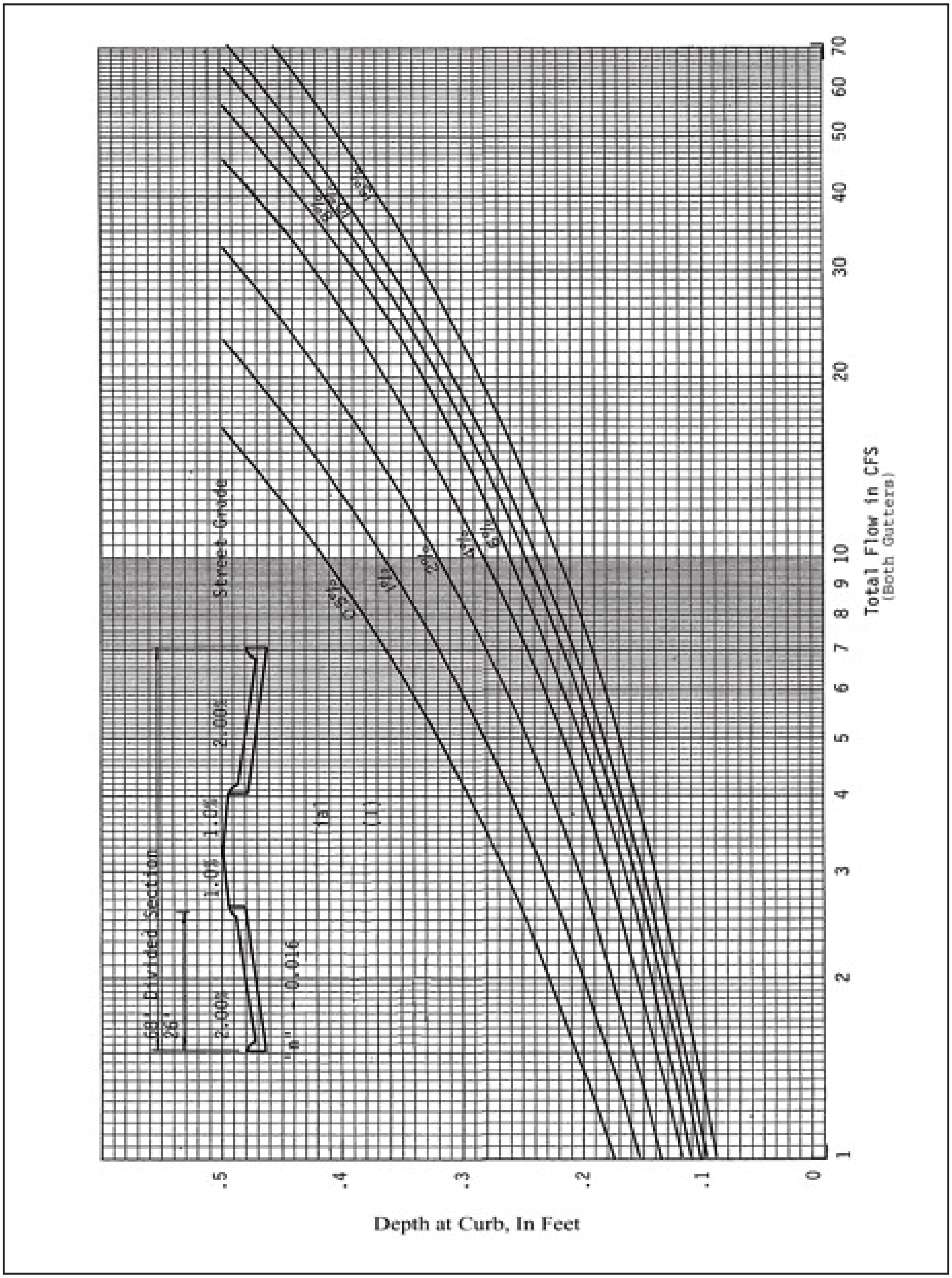


Figure 3-2i Flow In Street

3.3.2 Formula

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

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

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.)

3.3.3 Nomograph

A nomograph for solving Equation 3.1 is presented Figure 3-3.

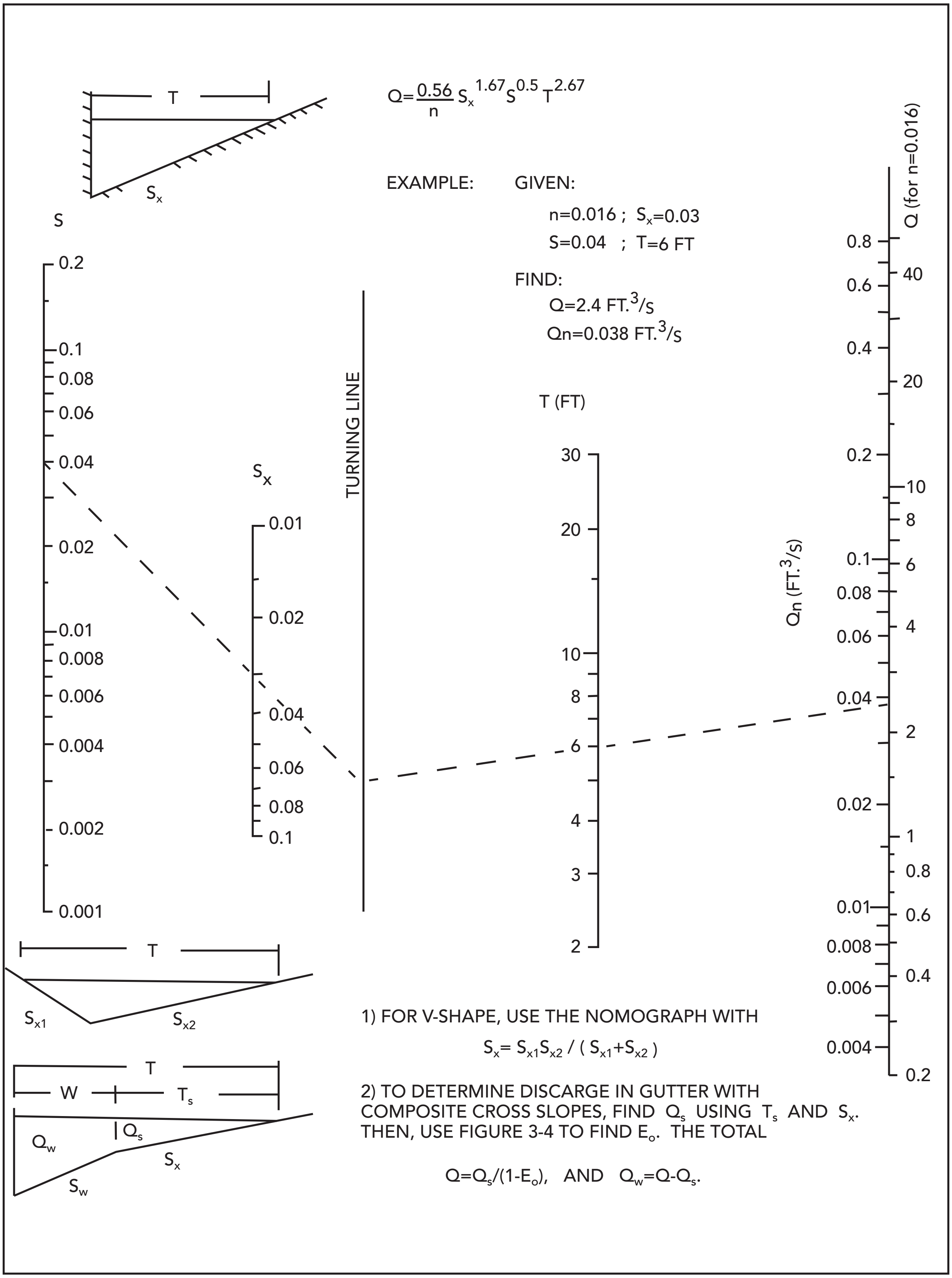


Figure 3-3 Flow In Triangular Gutter Sections Source: AASHTO Model Drainage Manual, 1991

3.3.4 Manning's n Table

Manning's n values for various pavement surfaces are presented in Table 3-5.

Table 3-5 Manning's n Values for Street and Pavement Gutters

Type Of Gutter Or Pavement		Range of Manning's n
Concrete gutter, troweled finish:		0.012
Asphalt pavement:	Smooth Texture	0.013
	Rough Texture	0.016
Concrete gutter with asphalt pavement:	Smooth	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

Note: Estimates are by the Federal Highway Administration
Reference: USDOT, FHWA, HDS-3 (1961).

3.3.5 Uniform Cross Slope

The nomograph in Figure 3-3 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 turning line 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 and the right scale on the capacity line. If the Manning's n is not 0.016, multiply Q and n from Step 1 and use the left scale on the capacity scale.
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 turning line intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q (from the right side of the scale) or Q_n (from the left side of the scale) 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 Manning's n (Q_n) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

3.3.6 Composite Gutter Sections

Figure 3-4 in combination with Figure 3-3 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. Figure 3-4 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. 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 the gutter capacity above the depressed section (Q_s).

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

$$Q_W = Q - Q_S \qquad \textbf{(3.2)}$$

3. Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 3-4 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.

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 3-3.

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.

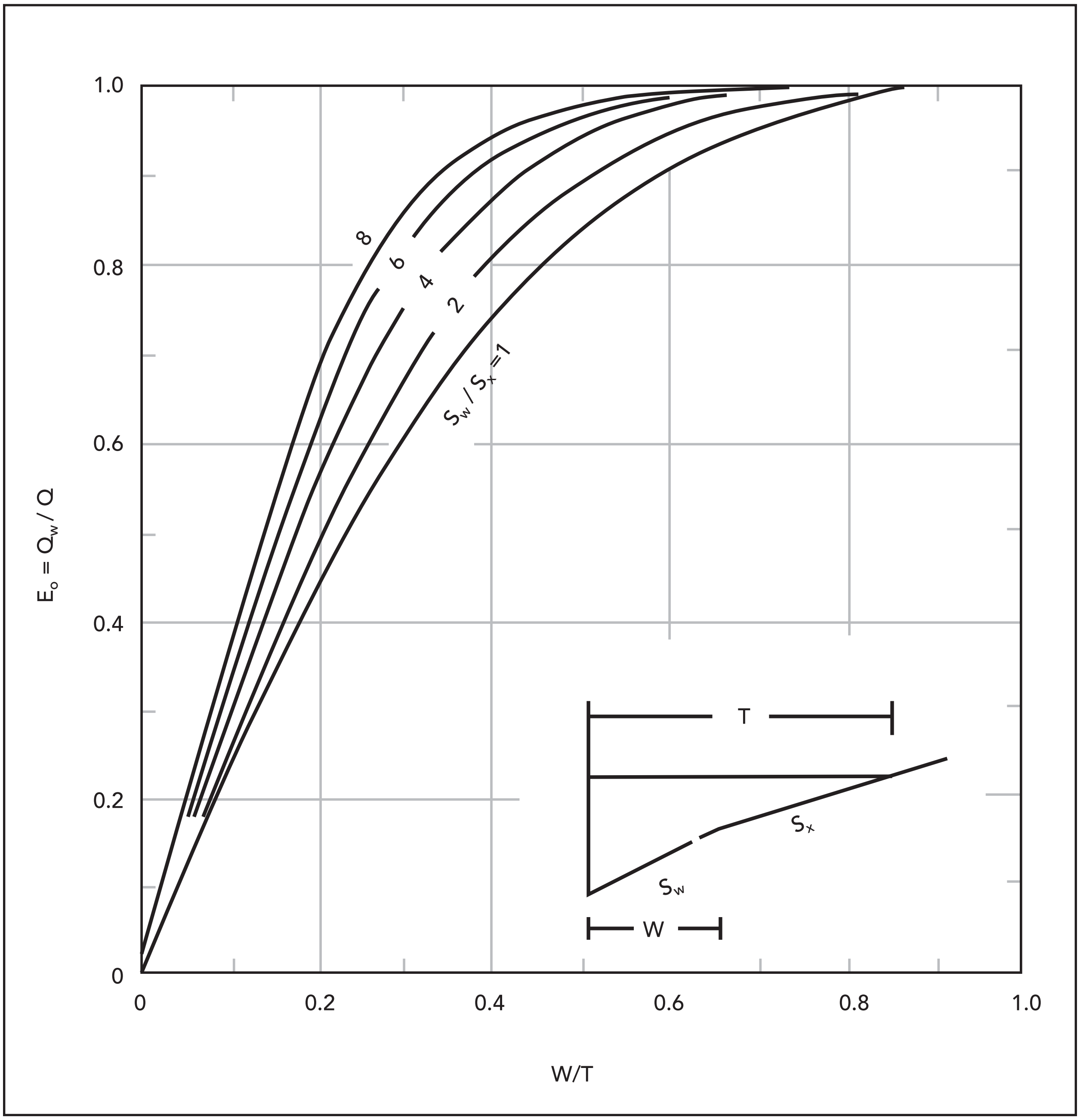


Figure 3-4 Ratio of Frontal Flow to Total Gutter Flow

Source: AASHTO Model Drainage Manual, 1991

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 3-3 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 3-4, find the appropriate value of E_o (the ratio of Q_w/Q).

4. Calculate the total gutter flow using the equation:

$$Q = Q_s / (1 - E_o) \tag{3.3}$$

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.

3.3.7 Examples

Example 1

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

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

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

(2) $T_s = 8 - 2 = 6$ ft.
$(Q_n)_2$ = 0.014 (Figure 3-3) (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.
$T_s = 6 - 1.5 = 4.5$ ft.
S_x = 0.03 ft./ft.
S_w = 0.0833 ft./ftn = 0.014
S = 0.04 ft./ft.
W = 1.5 ft.

Find: Flow in the composite gutter

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

$Q_s n = 0.017$

$Q_s = 0.017/0.014 = 1.2$ cfs

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

$S_w/S_x = 0.0833/0.03 = 2.78$

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

(3) Calculate the gutter flow using Equation 3.3:

$Q = 1.2/(1 - 0.64) = 3.3$ cfs

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

$Q_w = 3.3 - 1.2 = 2.1$ cfs

3.4 Stormwater Inlets

3.4.1 Overview

The primary aim of drainage design is to limit the amount of water flowing along the gutters or ponding at the sags to quantities which will not interfere with the passage of traffic for the design frequency. This is accomplished by placing inlets at such points and at such intervals to intercept flows and control spread. In this section, guidelines are given for designing roadway features 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 are based on information contained in HEC-22. Storm drain design is based on the use of the rational formula.

Drainage inlets are located to limit the depth or spread on traffic lanes to allowable limits for the design storm. Grates should safely accommodate bicycle and pedestrian traffic where appropriate.

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.

Inlets at vertical curve sags in the roadway grade should also be capable of limiting the spread to allowable limits. The width of water spread on the pavement at sag locations should not be greater than the width of spread encountered on continuous grades (see Section 3.2.3). Inlets should be located so that concentrated flow and heavy sheet flow will not cross traffic lanes, and should be located just upgrade of pedestrian crossings and locations where the pavement slope reverses.

The first (elevationally highest) inlet in a storm drainage system should be located where the water depth first reaches the crown of a two-lane street (usually 3 in. at the gutter or flowline). If the street is wider than two lanes, the first inlet in the system should be located where the clear street width is reduced by stormwater spread to one clear traffic lane in each direction.

Inlets used for the drainage of paved or unpaved surfaces can be divided into three major classes. These classes are:

- **Grate Inlets —** These include inlets consisting of an opening covered by one or more grates, and slotted inlets consisting of a pipe cut along the longitudinal axis with a grate of spacer bar to form slot openings.

- **Curb-Opening Inlets —** These inlets are vertical openings in the curb covered by a top slab.

- **Combination Inlets —** These have both vertical opening(s) and horizontal grate opening(s).

3.4.2 Criteria

Storm sewer inlets and pipes for are to be designed to limit the spread and depth of stormwater in roadways in accordance with the Criteria outlined in Section 3.2.

Inlets

- Curb-opening and combination inlets are generally used in the public street system.
- Grate inlets may be used for parking lot drains, area drains, etc.
- Inlets must be placed at the low or sag points in the street grade.
- In addition, a flanking inlet should be placed in the approach on each side of an inlet location at a low point in the street grade.

The location of the first (elevationally highest) inlet shall be determined by a trial and error process based to limit the depth of flow and spread in the roadway to the criteria outlined in Section 3.2. Subsequent inlets, downstream from other inlets, shall be located at or before points where the water depth again reaches the allowed maximum design depth and spread. Usually, inlets shall also be placed at the ends of radii and/or before crosswalks at intersections. Inlets which the study shows are needed at locations other than at intersections shall generally be centered between lot lines. Inlets shall be installed at the upper end of all storm drain lines and at low points in the street grades.

In a sag vertical curve on major streets and arterials three inlets are desirable. One at the low point and one on each side of this point. These flanking inlets should be 0.2 ft. higher than that the low point. These additional inlets furnish added capacity to collect flow bypassing upstream inlets, limit ponding and sediment deposition at the low point, and provide a safety factor in the event the low point inlet becomes clogged.

It may be necessary at some locations to use more than one inlet to pick up the contributing flow and limit stormwater spread to acceptable width.

Concrete valley gutters may be used across roadways at T-intersections of local roadways, if the calculated depth of flow for the minor system design flow in the curb and gutter section immediately upstream is less than the street crown height, and if there is no existing or proposed storm drain conduit extended to the intersection. The pavement cross-slope on the “uphill” lane of the minor street approach to the valley gutter shall be reduced at a gradual rate from 2% to 1% to allow drainage of the “uphill” gutter flow line through the return. No valley gutters shall be used across collector or arterial roadways.

Curb and gutter grades that are equal to pavement slopes shall not fall below 0.5 percent without approval from the Public Works Department.

3.4.3 Manholes

Manholes shall be installed at the upper end of all storm drain lines and at all changes in grade or size. In general, all pipe segments shall be designed and installed to straight alignment. Manholes shall be installed at changes in alignment unless otherwise approved by the Public Works Department. The recommended maximum manhole spacing for storm drain lines is 600 ft. Greater spacing of manholes in large-diameter storm drains may be used in design if approved by the Public Works Department. The minimum diameter manhole permitted in the City of Omaha is 54 in. Care should be taken to insure the diameter of the manhole is adequate to accommodate all entering and exiting pipes. The designer should utilize supplier’s recommendations and layout the geometrics of the pipes and manhole to verify the diameter is adequate. Manholes with diameters of 120 in. or larger must be cast-in-place. The crowns of all storm drain pipes entering and leaving a junction structure shall be at the same elevation. Drop manholes will not be permitted without prior approval from the Public Works Department. Laterals from a storm drain inlet to the main storm drain line may be tapped directly into the main storm drain line if the diameter of the lateral does not exceed one-half the diameter of the pipe being tapped, and if the lateral length does not exceed 50 ft. Unless directly tapped, the crown of the lateral pipe shall match the crown of the main storm drainpipe. Storm drain manholes, shall be constructed in accordance with the most current City Standard Plates and Specifications.

3.4.4 Grate Inlets

The capacity of a grate inlet depends upon its geometry and the cross slope, longitudinal slope, total 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 inlet grates and curb openings. 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 ft. long, intercepted flow is small. Inlet interception capacity has been investigated by agencies and manufacturers of grates. For inlet efficiency data for various sizes and shapes of grates, refer to Hydraulic Engineering Circular No. 12 (or No.22) Federal Highway Administration and inlet grate capacity charts published by the manufacturer(s) of specified grates.

The capacity of City of Omaha grate inlets can be determined by utilizing the following formula and nomographs as developed by the Neenah Foundry and published in their Inlet Grate Capacities booklet (1986). This method is used for City of Omaha grate inlets with the depth of flow being adjusted to compensate for the gutter depression at the grate.

$$Q_i = K d^{5/3} \tag{3.4}$$

Q_i = flow intercepted by inlet, cfs
K = unique coefficient which describes the geometry of a specific grate
d = depth of flow at curb line, ft.

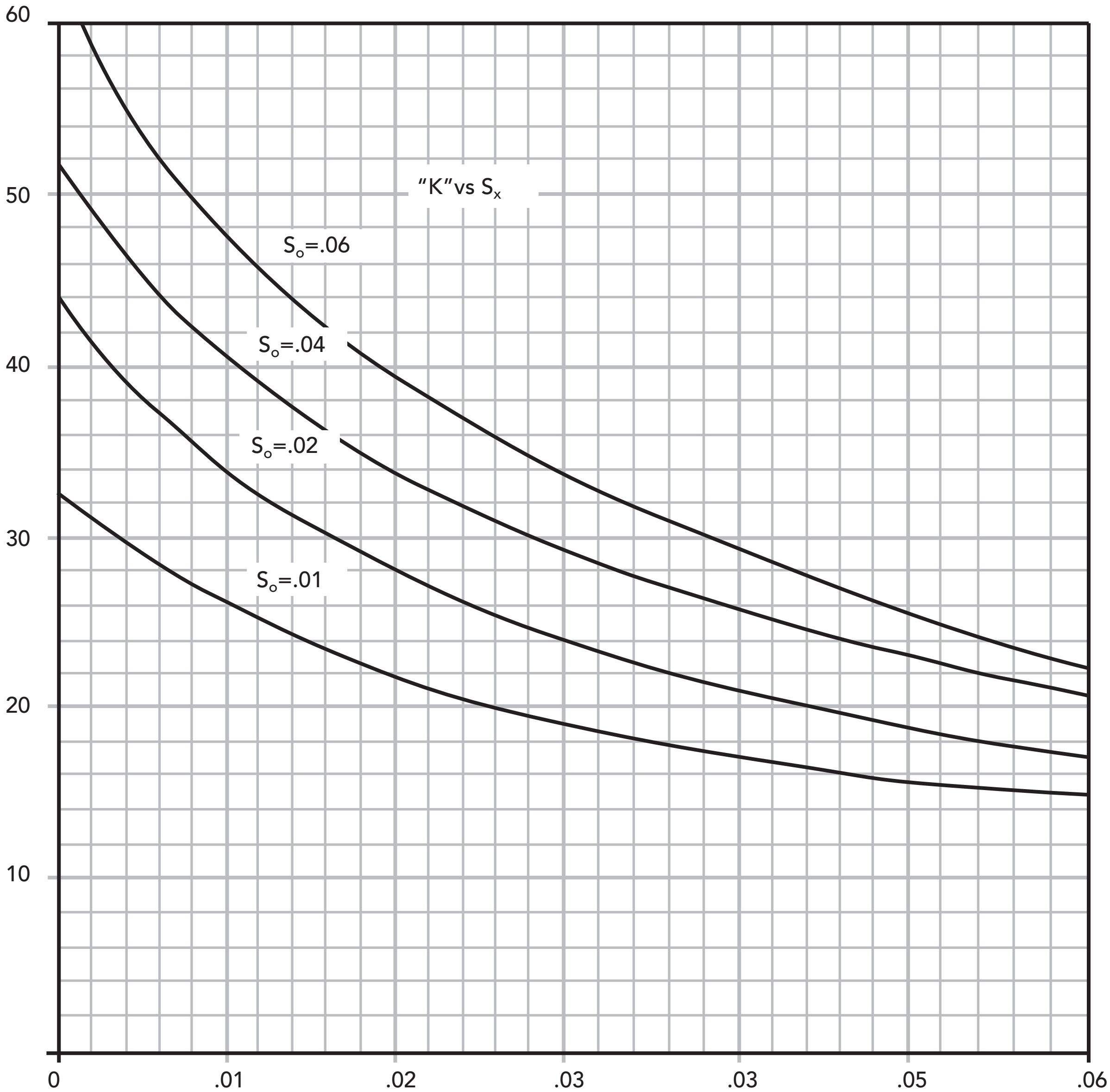
Example 4a

Given: Q = 1.5 cfs; pavement cross slope, S_x = ¼ in. per ft. = 2.08%;
Longitudinal slope, S = 2%

Find: Discharge intercepted by type "A-A" grate inlet, Q_i.

Solution:

1. Figure 3-3, lay a straight edge on S_x = 2.08 and S = 0.02; mark intersection on turning line
2. Lay straight edge on turning point marked in Step 1 and the given discharge, 1.5 cfs. Read T = 7.2.
3. From Figure 3-3, $S_x = d/T$, $d = S_x (T) = 0.0208(7.2) = 0.15$ ft.
4. Figure 3-5, lay a straight edge on S_x = 0.0208; mark intersection of straight edge S_o = 0.02; read to the left to find "K" = 27.5
5. Figure 3-5, for d = 0.15; $d^{5/3}$ = 0.0423
6. $Q_i = Kd^{5/3}$ – 27.5 (0.0423) = 1.16 cfs
7. Overflow = 1.5 cfs – 1.16 cfs = 0.34 cfs (to second grate)
8. Figure 3-3, T = 4.2, depth of flow at curb; S_x (T) = 0.09 ft.
9. Figure 3-5, for d = 0.09; $d^{5/3}$ = 0.0181
10. $Q_i = Kd^{5/3}$ = 27.5 (0.0181) = 0.50 cfs; 100% interception



$Q_i = Kd^{5/3}$

d = Depth of flow at gutter, ft.

S_X = Transverse Gutter Slope

S_o = Longitudinal Gutter Slope

"K" = Grate Inlet Coefficient

(from Neenah *Inlet Grate Capacities* booklet)

d	$d^{5/3}$	d	$d^{5/3}$
0.05	0.0068	0.30	0.1344
0.10	0.0215	0.35	0.1738
0.15	0.0423	0.40	0.2172
0.20	0.0684	0.45	0.2643
0.25	0.0992	0.50	0.3150

Note: For multiple grates, calculate interception for first grate; then depth of flow of overflow; then interception of second grate.

Figure 3-5 Grate Capacity Nomograph for City of Omaha Standard Plates No. 701 and 702 Gates

3.4.5 Curb-Opening Inlets

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-opening inlets are relatively free of clogging tendencies and offer little interference to traffic operation. Curb-opening inlets are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists. If possible, avoid placing curb inlets on intersection radii. They are difficult to construct. The capacity of the inlet depends upon the length of opening and depth of low at the opening. This depth in turn depends upon the amount of depression of the gutter at the inlet, the cross slope, longitudinal slope and roughness of the gutter.

Capacity of Curb-Opening Inlets on a Continuous Grade

The operation of a curb-opening inlet on a grade is usually described in terms of the ratio of the flow intercepted Qi to the approach flow Q. See Figure 3-6. The length of a depressed curb-opening inlet required for total interception of gutter flow on a pavement section with straight cross slope is expressed by:

$$L_t = 0.6Q^{0.42}S_o^{0.3}\left(\frac{1}{nS_x + naE_o/W}\right)^{0.6} \qquad \textbf{(3.5)}$$

Where:

L_t = curb-opening length for 100% interception, ft.
Q = flow in gutter at inlet, cfs
n = Manning roughness coefficient for pavement
S_x = cross slope of pavement, ft./ft.
E_o = ratio of flow in depressed gutter to total gutter flow, Figure 3-4.
a = gutter depression, in. (gutter depression is 5 in. for Omaha Standard Plate No. 721 curb inlets);
W = width of gutter depression (W is 2.75' for Omaha Standard Plate 721 curb inlets)
S_o = longitudinal gutter slope, ft./ft.

The interception of a standard length depressed curb-opening inlet is expressed by:

$$Q_i = Qe \qquad \textbf{(3.6)}$$

Where:

Q_i = flow intercepted by standard length inlet, ft.
Q = flow in gutter at inlet, cfs
e = % interception = $1- (1- L/L_T)^{1.8}$
L = length of City of Omaha inlet 10 ft.

The length of inlet required for a specific interception is expressed by:

$$e = 1 - (1 - L/L_T)^{1.8} \qquad \textbf{(3.7)}$$

Where:

L_i = length of inlet required to intercept a specific % of the flow in the gutter, ft.
L_t = curb-opening length required for 100% interception, ft.
e = efficiency of inlet or % interception.

The information above is summary of the methodology developed by the FHWA and is described in depth in HEC -22.

Example 5

Find: Interception for City of Omaha Std. Plate 721 Inlets for a (1) 4-ft Opening, and (2) a 10-ft. Opening

Given: Q = 2.4 cfs
S = 0.04
S_x = 0.03
a = 5 in.
W = 2.75 ft.

Solution: (1)

1. From Figure 3-3 find T (Spread above inlet)

   $T = 6$ ft.

2. From Figure 3-4 find E_o, with $W/T = (2.75/6) = 0.46$, $S_w/S_x = 1$
   (No depressed gutter upstream of inlet)

   $E_o = 0.80$

3. Calculate the curb opening length for 100% interception using Eq. 3.5

   $L_t = 0.6\ (2.4\text{cfs})^{0.42}\ (.04)^{0.3}\ [1\ /\ (0.016\ \text{x}.03 + 0.016\ \text{x}\ (5\ \text{in.}/12)\ \text{x}\ 0.80\ /\ 2.75\ \text{ft.})^{0.6} = 12.3$ ft.

4. Calculate the efficiency of the curb inlet using Equation 3.7

   $e = 1 - (1 - 4\ \text{ft.}\ /\ 12.3\ \text{ft.})^{1.8} = 0.51$

5. Calculate the intercepted flow using Eq. 3.6

   $Q_i = 2.4$ cfs. x 0.51 = 1.2 cfs.

Solution: (2)

1. L_t is calculated the same as in Part 1 of this example

2. Calculate the efficiency of the curb inlet using Equation 3.7

   $e = 1 - (1 - 10\ \text{ft.}\ /\ 12.3\ \text{ft.})^{1.8} = 0.95$

3. Calculate the intercepted flow using Eq. 3.6

   $Q_i = 2.4$ cfs. x 0.95 = 2.3 cfs.

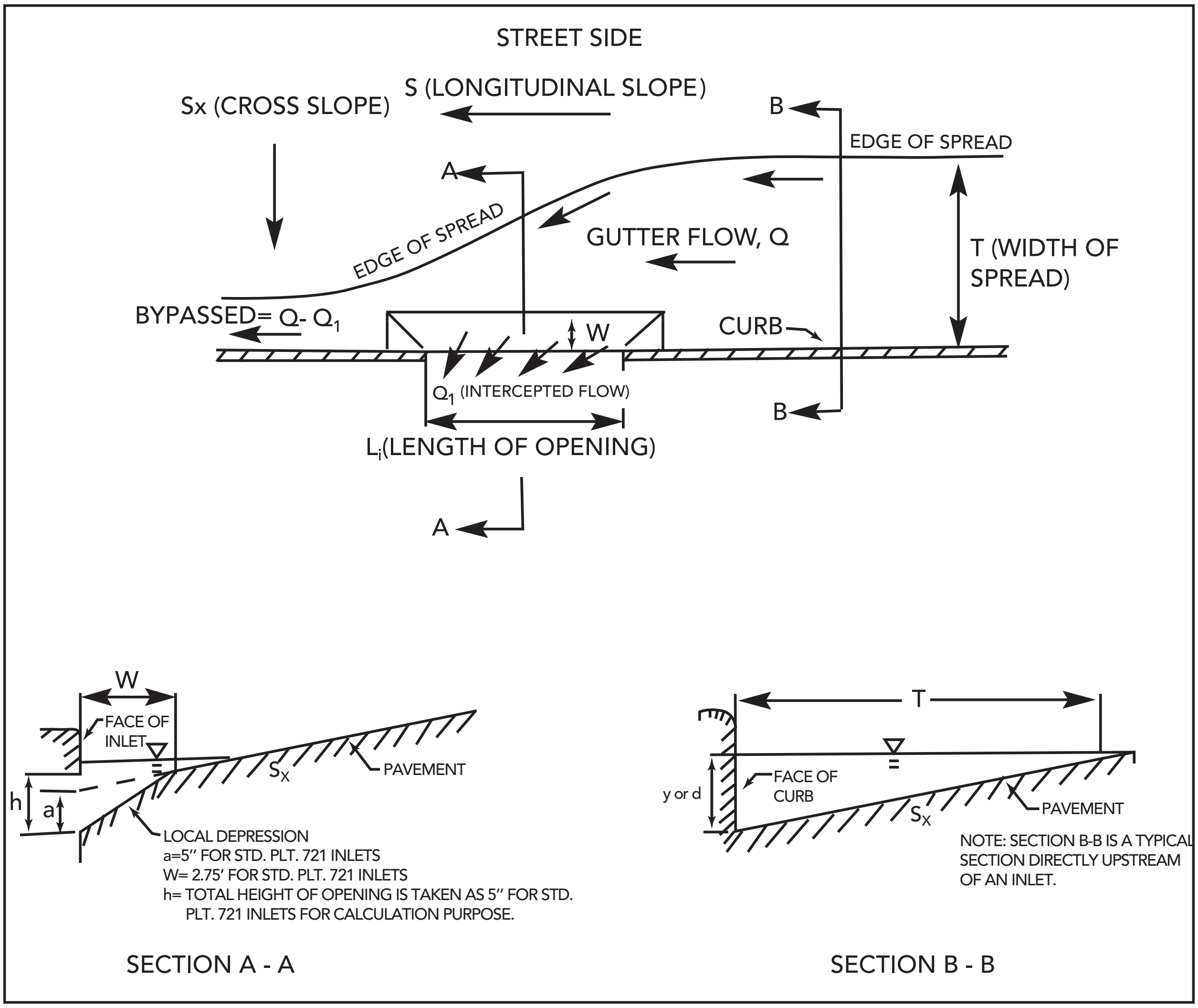


Figure 3-6 Graphical Definition of Symbols

Figure 3-7 may be used to determine curb-opening inlet capacity. The following information is needed:

- L = length;
- a = depth of gutter depression, if any, at the inlet (gutter depression is 5 in. for Omaha Standard Plate No. 721 curb inlets);
- Q_a = design discharge in the gutter (include bypass from upstream inlet and crossover from the other side of the street);
- y = depth of flow in normal gutter for the particular longitudinal slopes and street cross section above the inlet in question, which may be determined from Figures 3-1, 3-2, or 3-3.

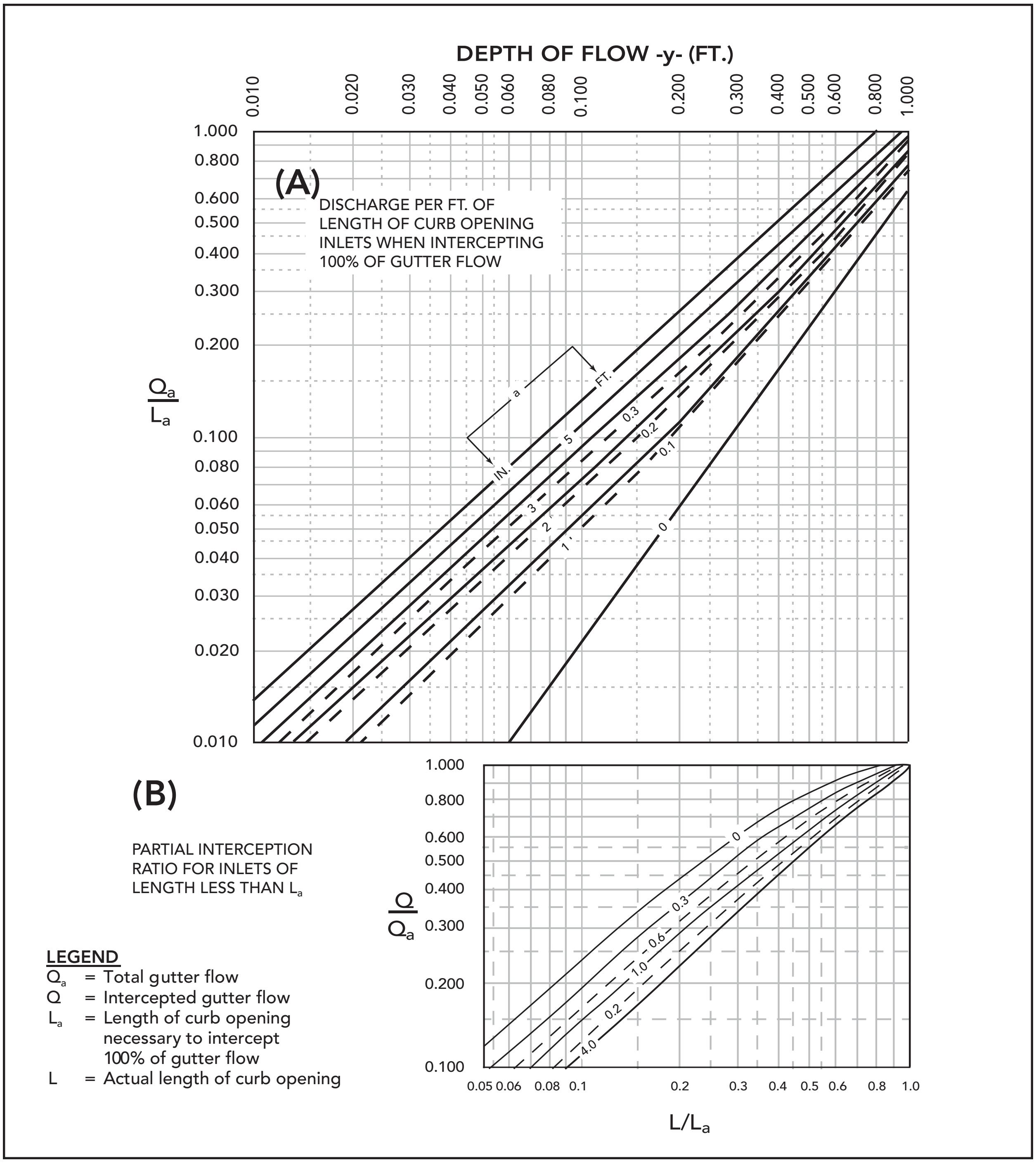


Figure 3-7 Capacity Nomograph for Curb Opening Inlets on Continuous Grade

The procedure is as follows:

1. Enter Chart A in Figure 3-7 with depth of flow, y and gutter depression at the inlet, a, and determine Q_a/L_a, the interception per ft. of inlet opening if the inlet were intercepting 100% of the gutter flow. Determine length of inlet, La, required to intercept 100% of the gutter flow:

$$L_a = Q_a/(Q_a/L_a) \quad \textbf{(3.10)}$$

L_a = length of inlet required for 100% interception, ft.
Q_a = total gutter flow, cfs.

2. Compute the ratio, L/L_a where L equals the actual length of the inlet in question.
3. Enter Chart B in Figure 3-7 with L/L_a and a/y. Determine the ratio, Q/Q_a, the proportion of the total gutter flow intercepted by the inlet in question.
4. Flow intercepted, Q, is the ratio, Q/Q_a, times the total gutter flow, Q_a.
5. Flow carried over to the next inlet equals Q_a minus Q.

3.4.6 Capacity of Curb-Opening Inlets in a Low Point or Sump

The capacity of a curb-opening inlet in a sag or low pointe depends on water depth at the curb, the curb-opening length and the height of the curb opening. 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. When flow depths are in the transition region the designer shall calculate the flow by both methods and utilize the most conservative result.

The equation for the interception capacity of a depressed curb-opening inlet operating as a weir is:

$$Q_i = C_W (L + 1.8W)d^{1.5} \quad \textbf{(3.11a)}$$

Where:

C_W = 2.3
L = length of curb opening, ft.
W = width of depression, ft.
d = depth of water at curb measured from the normal cross slope gutter flow line, ft.

See Figure 3-8 for a definition sketch.

The weir equation for curb-opening inlets without depression becomes:

$$Q_i = C_W L d^{1.5} \quad \textbf{(3.11b)}$$

The depth limitation for operation as a weir becomes: $d \leq h$.

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 × height of curb opening. The interception capacity can be computed by:

$$Q_i = C_o A[2g(d_i - h/2)]^{0.5} \quad \textbf{(3.12)}$$

Where:

C_o = orifice coefficient (0.67)
h = height of curb-opening orifice, ft.
A = clear area of opening, $ft.^2$
d_i = depth at lip of curb opening, ft.

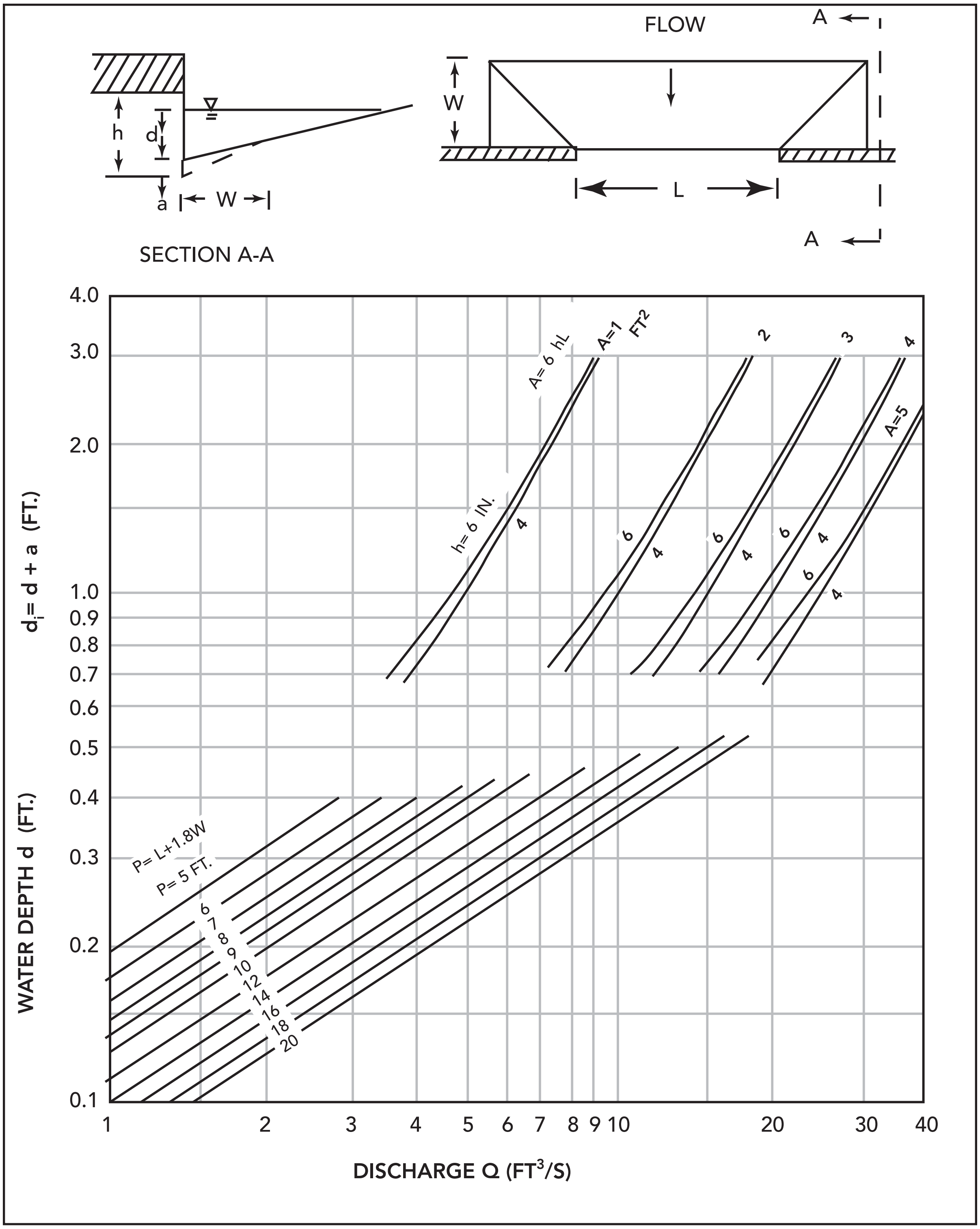


Figure 3-8 Depressed Curb-Opening Inlet Capacity in Sump Locations

The weir equations use an effective weir length and coefficient that is representative of the line of gutter transition to the depression. The user should be cautioned not to use the depth from the water surface to the depressed inlet throat, but to the undepressed depth (or more specifically, the depth at the beginning of the transition). Otherwise, the capacity for weir flow will be overestimated.

Note: Equation 3.12 is applicable to depressed and undepressed curb-opening inlets, and the depth at the inlet includes any gutter depression.

Example Problem 6

The following Example illustrates the use of this procedure:

Given: Curb-opening inlet in a sump location:

L = 5 ft. h = 5 in.

(1) Undepressed curb opening:

$S_x = 0.05$ T = 8 ft.

(2) Depressed curb opening:

$S_x = 0.05$ W = 2 ft.
a = 2 in. T = 8 ft.

Find: Q_i

Solution: (1) d = TSs = (8)(0.05) = 0.4 ft. d < h; therefore, weir controls
$Q_i = C_W Ld1.5 = (2.3)(5)(0.4)1.5 = 2.9$ ft.^{3}/s.

(2) d = 0.4 ft. < (1.4 h) = 0.6; therefore, weir controls
P = L + 1.8W = 5 + 1.8(2) = 8.6 ft.
$Q_i = (2.3)(8.6)(0.4)1.5 = 5$ ft.^{3}/s. (Figure 3-8)

At d = 0.4 ft., the depressed curb-opening inlet has about 70 percent more capacity than an inlet without depression. In practice, the flow rate would be known and the depth at the curb would be unknown.

3.4.7 Flared End Sections

Capacities for flared end sections shall be determined using the procedures provided in Chapter 4 - Design of Culverts.

3.4.8 Inlet Design Computation Form

The inlet design computation sheet shown in (Figure 3-9) shall be used to summarize inlet computations on roadways. Computational sheets with different formats but substantially the same information may be used but will require approval from the Public Works Department. Descriptions of the columns are provided below:

Column 1 – Inlet identification shown on drawings and drainage map.
Column 2 – Identification of the direct runoff drainage area contributing to that inlet; this area should be delineated on the drainage basin map.

Column 3 – Area of direct runoff contributing to the inlet as shown on the drainage map.
Column 4 – Runoff Coefficient.
Column 5 – Time of Concentration.
Column 6 – Rainfall Intensity for the Design event at the time of concentration to the inlet.
Column 7 – Flow received from the inlet by direct runoff from the direct drainage area impacting that inlet.
Column 8 – Flow received by the inlet that has bypassed an upstream inlet.
Column 9 – This column is to account for any additional flow received by the inlet that has crossed over the crown from the opposite side of the road upstream of the inlet. In certain cases common on Omaha residential streets, where flow exceeds the height of the crown, the flow may need to be "balanced" across the roadway section. In those cases adjust the total Gutter flow accordingly and provide a description in the remarks section. In all cases where cross over is present or balancing is necessary, a description of how the designer arrived at the Total Gutter Flow should be added to the remarks section.
Column 10 – Total flow the inlet receives, it is the sum of the direct runoff at the inlet, bypass flows from upstream inlets, and cross-over flows (if present) from the opposite side of the roadway crown.
Column 11 – Longitudinal slope of the roadway. In cases where the inlet is in a low point of a sag curve, report "Sump" in this Column.
Column 12 – Cross slope of the roadway at the inlet section. This does not include the inlet depression.
Column 13 – Depth of flow in the curb immediately upstream of the inlet.
Column 14 – Height of the curb immediately upstream of the inlet.
Column 15 – Spread of flow across the roadway section. In cases where flow exceeds the crown of a roadway (such as residential street), report the road width to the crown and include a description in the remarks section.
Column 16 – Inlet Type: Report the type of inlet (eg, Type I, Type IV, Type A).
Column 17 – Use this column to report the length of the inlet.
Column 18 – Efficiency of the inlet.
Column 19 – Flow Intercepted by the inlet.
Column 20 – Flow bypassing the inlet.
Column 21 – Remarks: Add any clarifying information describing the inlet situation and flows received. This column would be used to discuss adjustments to the total Q in cases where the flow may need to be balanced across the roadway (residential streets), flow cross over the crown from the opposite side of the roadway, or any other information pertinent to the design and placement of the inlet. If additional space is need, add a key note and supplemental pages.

CURB INLET DESIGN COMPUTATIONS
HEC 12 METHOD

DATE:
CALCULATED BY:

PROJECT
PROJECT NO.

LOCATION		DISCHARGE							DESIGN STORM=10 YR						INLET CAPTURE					REMARKS
INLET ID.	DRAINAGE BASIN ID.	AREA (A) ACRES	RUNOFF COEFFICENT (C)	TIME OF CONCENTRATION (T_c) MIN.	INTESITY (I) IN./HR.	DIRECT RUNOFF CFS	PREVIOUS BY-PASS CFS	CROSS OVER CFS	TOTAL GUTTER FLOW(Q) CFS	PAVING SLOPE (S) FT./FT.	CROSS SLOPE (S_X) FT./FT.	DEPTH (d) FT.	CURB HEIGHT FT.	GUTTER SPREAD FT.	INLET TYPE	OPENING LENGTH FT.	EFFICIENCY	INTERCEPTED (Q_1) CFS	BY-PASSED ($Q-Q_1$) CFS	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

COLUMN9. CROSS OVER. THE COLUMN MAY UTILIZED TO ACCOUNT FOR ADDITIONAL FLOWS RECEIVED BY THE INLET THATCROSSED OVER FROM THE OPPOSITE SIDE OF THE CROWN
COUMN13-DEPTH (d) AT GUTTERUPSTREAM OF THAT.

NOTES:

SHEET ______ OF ______

Figure 3-9 Inlet Design Computation Form

3.5 Storm Drains

3.5.1 Introduction

After the tentative location of inlets has 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 carry this discharge. The procedure is carried out for each section of pipe starting at the most upstream inlet and proceeding 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. In other words, the inlets are designed to assure that the full pipe capacity is utilized. 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 but will not be placed under pressure head. Pressurized design may be acceptable in certain circumstances **but will require advance approval from the Public Works Department.**

The Manning Formula, and a Manning n value of 0.013, is recommended for concrete pipe capacity calculations. The minimum storm drain pipe diameter to be used in the public system is 15 in.

3.5.2 Design Criteria

Storm sewer shall be analyzed for the design event using Manning Pipe flow formula. Additionally, in order to determine if design flows can be accommodated by the storm drains system without causing flooding, or causing flows to exit the system at unacceptable locations, the designer shall determine the hydraulic gradient. The following design criteria are applicable when evaluating the elevations along the design hydraulic grade line (HGL):

- The hydraulic grade line shall be below the hydraulic grade line of any entering nonpressurized system and at least 0.75 ft. below the intake lip of any affected inlet or any manhole cover.

- The energy grade line shall not rise above the intake lip of any affected inlet, any manhole cover or the finished grade over any entering nonpressurized system.

- The maximum desirable pipe velocity is 20 ft. per second. Flows exceeding 20 ft. per second may be permitted if the pipes, sturcutres, and joints are suitable for the design velocity.

- The minimum physical slope shall be the slope which will produce a minimum velocity of 3.0 ft. per second when the storm drain is flowing full. A minimum physical slope of 0.5 percent is desirable. Flatter slopes, especially for large-diameter storm drains, may be used in design if scour velocity is maintained and if approved by the Public Works Department. For very flat flow lines the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system.

Location and Alignment

In new subdivisions the storm drain system shall be located in accordance with the standards of the Subdivision Regulations. When construction of a storm drain system is necessary in the older parts of the City, the proposed location shall be acceptable to, or determined by the Public Works Department. No structures may be placed over a public storm or sanitary sewer system. Typically the top of the storm drain pipes will align at every junction location (ie; inlets, MH's, JB's,etc.)

Depth of Cover

The desired depth of cover above a storm drainpipe shall be 2 to 3 ft., with 1.5 ft. being the absolute minimum at an inlet location. Depth of cover greater than 3 ft. shall be avoided due to the possibility of the storm drain blocking access of sanitary sewer service lines to the main sanitary sewer lines.

Material and Joints

Storm drain pipe joints, and construction shall conform with the requirements of the City of Omaha Standard Specifications. Pressurized pipe installations will require water tight joints; soil tight joints will not be acceptable on any pressurized sewer installations.

Bar Grates on End Sections

An open pipe inlet from an open channel (similar to a culvert inlet) into a closed pipe storm drain shall be designed and constructed with flared end sections with a bar grate. No bar grate is required on the end section of a pipe outlet into an open channel unless directed by the Public Works Department.

3.5.3 General Design Approach

The design of storm drain systems is generally divided into the operations listed below. Supporting documentation shall be submitted with development plans for review:

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 drain system establishing the following design data:

    a. Location of proposed storm drains with proposed and existing facilities such as sanitary sewer, water, gas, power and communication lines. See Figure 3-10. Designer should make use of the block out details for the type of inlet being used and identify how the inlet block out detail will match the proposed pavement at curb returns and sag locations that require flanking inlets.
    b. Direction of flow.
    c. Location of manholes. Review MH lid locations to identify possible conflicts with curbs or other proposed or existing infrastructure. MH rims shall be located to not impact proposed curbs or curb ramp locations. If needed an existing MH shall be reconstructed to rotate the MH lid to a non impacting location.
    d. Location of major storm overflow routes.
    e. Location of existing facilities such as water, gas, or underground cables.
    f. Location of the trunk sewer is to be outside of the paving, typically located at the offset required to be installed at the offsets shown in Standard Plate 3-721 for the size of pipe being installed. Designer should identify all underground infrastructure before defining the location of the trunk sewer.

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 (shown in Figure 3-11) shall be used as described in Section 3.5.5 to summarize the system design computations. Computational sheets with different formats but substantially the same information may be used but require approval from the Public Works Department.

5. Hydraulic grade line computations, as described in Section 3.5.6, shall be used to determine the storm drain system hydraulic profile. The hydraulic grade line profile shall be provided on the storm drain system plans for the minor design storm.

3.5.4 Capacity

Storm drain capacity for reinforced concrete pipe can be determined using Figure 3-12 or information from Table 3-6. For non-standard applications, hydraulic capacity can be determined using the information provided below.

Formulas for Gravity and Pressure Flow

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

$$V = [1.486\, R^{2/3} S^{1/2}]/n \qquad \textbf{(3.14)}$$

Where:
- V = mean velocity of flow (ft./s.)
- R = the hydraulic radius (ft.) - 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:

$$Q = [1.486\, AR^{2/3} S^{1/2}]/n \qquad \textbf{(3.15)}$$

Where:
- Q = rate of flow (cfs)
- A = cross sectional area of flow (ft.^{2})

For pipes flowing full, the above equations become:

$$V = [0.590\, D^{2/3} S^{1/2}]/n \qquad \textbf{(3.16)}$$

$$Q = [0.463\, D^{8/3} S^{1/2}]/n \qquad \textbf{(3.17)}$$

Where:
- D = diameter of pipe (ft.)

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

$$H_f = [2.87\, n^2 V^2 L]/[S^{4/3}] \qquad \textbf{(3.18)}$$

$$H_f = [29\, n^2 L V^2]/[(R^{4/3})(2g)] \qquad \textbf{(3.19)}$$

Where:
- H_f = total head loss due to friction (ft.)
- 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./s.^{2}

3.5.4.1 Hydraulic Jump

The analysis of the hydraulic jump inside storm sewers is approximate, because of the lack of data for circular, elliptical, or arch sections. The jump can be approximately located by intersecting the energy grade line of the supercritical and subcritical flow reaches. The primary concerns are whether the pipe can withstand the forces which may separate the joint or damage the pipe wall, and whether the jump will affect the hydraulic characteristics. The effect on pipe capacity can be determined by evaluating the energy grade line, taking into account the energy lost by the jump. In general, for Froude numbers less than 2.0, the loss of energy is less than 10 percent. French (1985) provides semi-empirical procedures to evaluate the hydraulic jump in circular and other non-rectangular channel sections. "Hydraulic Analysis of Broken Back Culverts", Nebraska Department of Roads, January 1998 provides guidance for analysis of hydraulic jump in pipes.

3.5.4.2 Street Right-of-way and Overland Swale Drainage

For storm runoff events which exceed the required 10-year frequency design storm for storm drainage inlets and pipes, street right of ways and overflow swales or channels need to be planned and designed to safely convey the excess runoff. Hydraulic analysis and design of these overflow drainage routes should be done in accordance with the applicable provisions of Chapter 5 – Open Channels.

Table 3-6 Values of 1.486/n x A x R^b for Circular Concrete and Corrugated Metal Pipe

Source: ACPA, Design Data 4, Hydraulic Capacity of Sewers, Table III

TABLE III: Circular Concrete and Corrugated Metal Pipe

Pipe Dia, (in.)	A Area (Square ft.)	R Hydraulic Radius (ft.)	Values of 1.486/n x A x R $^{2/3}$					
			Concrete Pipe			Corrugated Metal Pipe		
			n=0.011	n=0.012	n=0.013	2-2/3'' x 1/2'' n=0.024	3'' x 1'' n=0.027	6'' x 2'' n=0.033
8	0.349	0.167	14.3	13.1	12.1	6.5		
10	0.545	0.208	25.8	23.6	21.8	11.8		
12	0.785	0.250	42.1	38.6	35.7	19.3		
15	1.227	0.312	76.5	70.1	64.7	35.0		
18	1.767	0.375	124.2	113.8	105.1	56.9		
21	2.405	0.437	187.1	171.5	158.3	85.7		
24	3.142	0.500	267.4	245.1	226.2	122.5		
27	3.976	0.562	365.8	335.3	309.6	167.7		
30	4.909	0.625	484.7	444.3	410.1	222.2		
33	5.940	0.688	623.6	573.7	529.6	286.9		
36	7.069	0.750	788	722	666	361	321	
42	9.621	0.875	1189	1090	1006	545	484	
48	12.566	1.000	1698	1556	1436	778	692	
54	15.904	1.125	2325	2131	1967	1065	947	
60	19.635	1.250	3077	2821	2604	1410	1254	1026
66	23.758	1.375	3967	3636	3357	1818	1616	1323
72	28.274	1.500	5004	4587	4234	2293	2039	1668
78	33.183	1.625	6195	5679	5242	2839	2524	2065
84	38.485	1.750	7549	6920	6388	3460	3075	2517
90	44.179	1.875	9078	8321	7681		3698	3026
96	50.266	2.000	10776	9878	9119		4390	3592
102	56.745	2.125	12671	11615	10722			4224
108	63.617	2.250	14756	13526	12486			4919
114	70.882	2.375	17044	15624	14422			5682
120	78.540	2.500	19544	17915	16537			6515
126	86.590	2.625	22255	20397	18829			7417
132	95.030	2.750	25200	23104	21327			8401
138	103.870	2.875	28372	26009	24011			9459
144	113.100	3.000	31780	29133	26894			10594
150	122.720	3.125						11810
156	132.730	3.250						13115
162	143.140	3.375						14504
168	153.940	3.500						16160
174	165.130	3.625						17551
180	176.710	3.750						19212

3.5.5 Design Procedure

The procedure for storm drain system design using the example calculation form of Figure 3-11 is outlined below. Other similar calculation forms, that include substantially the same information, may be used for drain system design. In addition, the hydraulic grade line design procedure described in Section 3.5.6.3 must be performed to demonstrate the storm drain system is adequately designed.

Column 1 – Pipe identification number, location, or other identification.
Column 2 – Structure or description of upstream end point of pipe.
Column 3 – Structure or description of down stream end point of pipe.
Column 4 – Identify the area discharging immediately at the upstream end of the pipe (eg. the area upstream of a curb inlet connected to the pipe - may not necessarily be the total area contibuting to the pipe.)
Column 5 – The length of overland flow in ft. runoff travels before reaching the upstream end point – of the pipe.
Column 6 – Used to identify the watercourse type.
Column 7 – The representative slope of the upstream watercourse (ft./ft.)
Column 8 – Overland flow velocity based on Figure 2-1 (ft./s.)
Column 9 – Time of concentration used to calculate the direct runoff at the upstream end of pipe (min.)
Column 10 – Intensity of rainfall contributing to the direct runoff taken from Figure 2-2 (in./hr.)
Column 11 – The area in acres contributing to the direct runoff (ac.)
Column 12 – Coefficient of runoff for Rational Method, see Table 2-3 and Table 2-4.
Column 13 – Coefficient of runoff for Rational Method, see Table 2-3 and Table 2-4.
Column 14 – The total flow in the system used to determine the travel time in the pipe (cfs.)
Column 15 – Pipe internal Diameter, and description (e.g. 18" RCP.)
Column 16 – Minimum pipe slope needed to convey the System Q (ft./ft.)
Column 17 – Design slope of the pipe (ft./ft.)
Column 18 – Design velocity of the pipe (ft./s.)
Column 19 – Full flow capacity of the pipe (cfs.)
Column 20 – Length of Pipe (ft.)
Column 21 – System (pipe) travel time (minutes.)
Column 22 – The cumulative time from the longest upstream path (minutes.)
Column 23 – System intensity in in./hr.
Column 24 – The weighted average of the runoff coefficient for all areas contributing flow to the pipe.
Column 25 – The total area contributing to the pipe (ac.)
Column 26 – The total flow from all contributing areas that the pipe conveys (cfs).
Column 27 – Remarks.

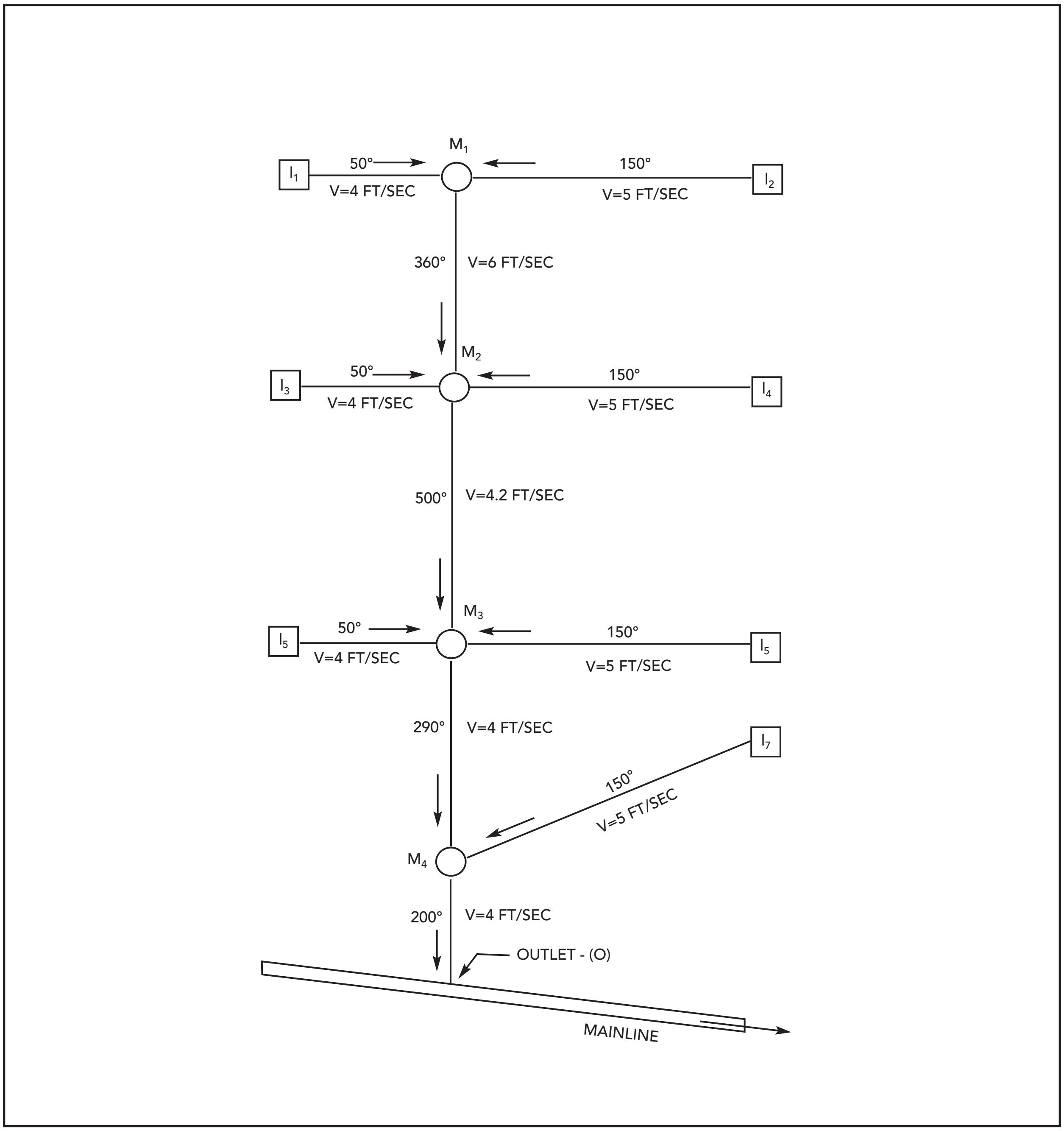


Figure 3-10 Hypothetical Storm Drain System Layout

STORM DRAIN COMPUTATION FORM
BY THE RATIONAL METHOD AND MANNING'S PIPE FLOW EQUATION

DATE:
CALCULATED BY:

PROJECT
PROJECT NO.
DESIGN STORM:

CONVEYANCE			DIRECT RUNOFF										SYSTEM DESIGN									TOTAL RUNOFF					REMARKS
PIPE ID.	FROM	TO	SUB BASIN ID.	O.F.L. FT.	W.C. TYPE	S %	V FPS	T_1 MIN.	I IN./HR.	A AC.	C	q	SYSTEM Q CFS	PIPE SIZE (IN.)	PIPE TYPE	SLOPE MIN. %	SLOPE DESIGN %	V DES. FPS	PIPE FULL CAP. CFS	PIPE LENGTH FT.	TIME IN PIPE MIN.	T_C MIN.	I IN./HR.	COMP C	TOTAL A AC.	DES Q CFS	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)		(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)

WATERCOURSE LEGEND.
FOR-FOREST NBG-NEAR BARE GROUND FAL-FALLOW
GRA=GRASS GWW-GRASS WATERWAY SGF-SHALLOW GUITAR FLOW

NOTES:

SHEET
_____ OF _____

Figure 3-11 Storm Drain Computation Form

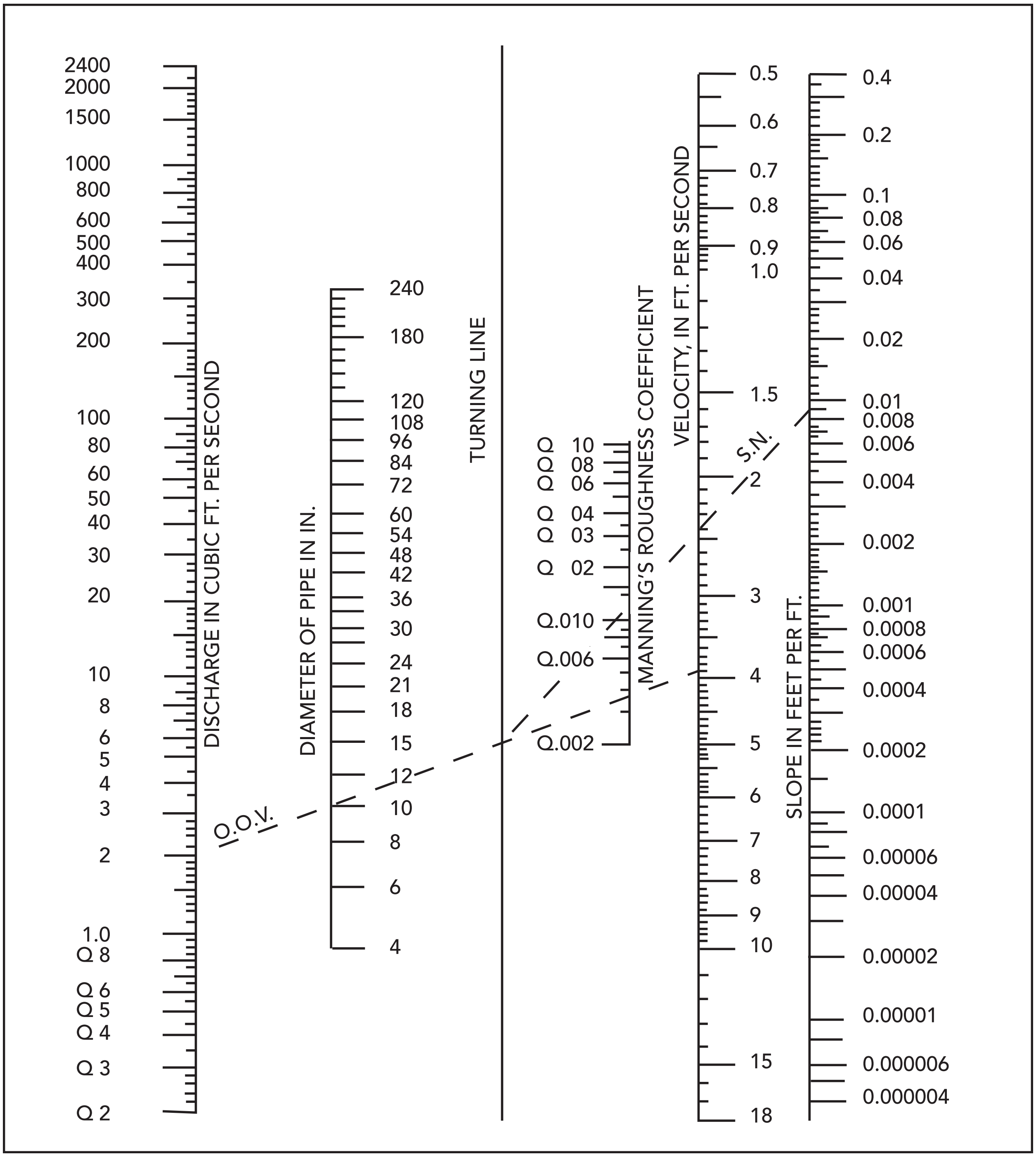


Figure 3-12 Nomograph for Solution of Manning's Formula in Storm Drains

3.5.6 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 (potential) head, velocity head and pressure head. 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. 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 3-13 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 pipe circumference. **Under gravity full flow, the HGL coincides with the crown of the pipe**.

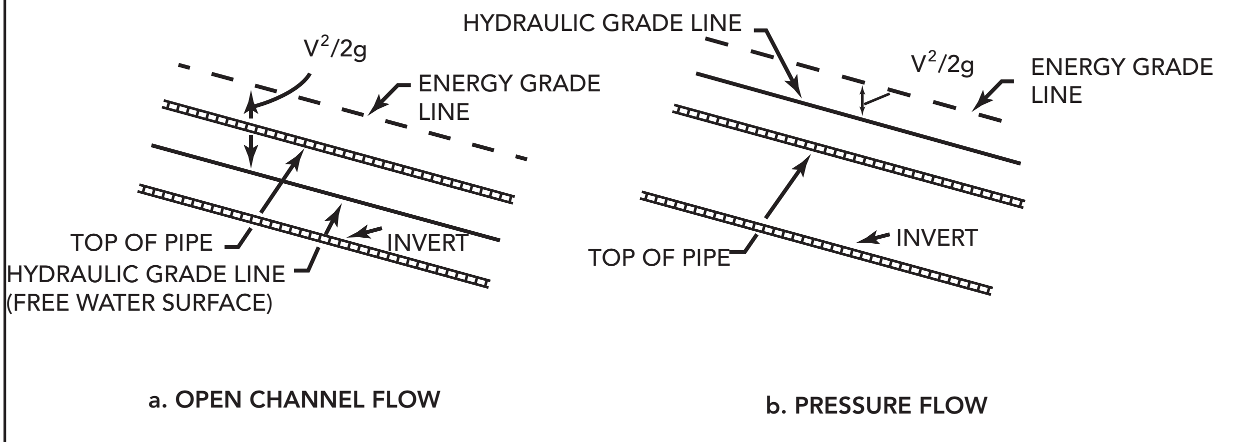


Figure 3-13 Hydraulic and energy grade lines in pipe flow

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.

Methods for determining energy losses in a storm drain are presented in Section 3.5.6.2. A detailed procedure for evaluating the energy grade line and the hydraulic grade line for storm drainage systems is presented in Section 3.5.6.3.

3.5.6.1 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 that is either existing or proposed for the purpose of conveying the stormwater away from the highway. 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 3-7 provides a comparison of discharge frequencies for coincidental occurrence for a 10- and 100-year design storm.

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 hectares and the storm drainage system has a drainage area of 2 hectares, the ratio of receiving area to storm drainage area is 200 to 2 which equals 100 to 1. From Table 3-7 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 3-7 Frequencies for Coincidental Occurrence

Area Ratio	Frequencies for Coincidental Occurrence			
	10-Year Design		100-Year Design	
	Main Stream	Tributary	Main Stream	Tributary
10,000 to 1	1 10	10 1	2 100	100 2
1,000 to 1	2 10	10 2	10 100	100 10
100 to 1	5 10	10 5	25 100	100 25
10 to 1	10 10	10 10	50 100	100 50
1 to 1	10 10	10 10	100 100	100 100

Note: Estimates are by the Federal Highway Administration
Reference: USDOT, FHWA, HDS-3 (1961).

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. The designer should consult the governing agency (e.g. NRD, USACE) for the outfall location to determine the criteria to be met in completing the outfall design.

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 cannot 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.

3.5.6.2 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 sections present relationships for estimating typical energy losses in storm drainage systems.

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:

$$H_f = S_f \, L \tag{3.20}$$

Where:

H_f = Friction loss, (ft.)
S_f = Friction slope, (ft./ft.)
L = Length of pipe, (ft.)

The friction slope in Equation 3.20 is also the slope of the hydraulic gradient for a particular pipe run. As indicated by Equation 3.20, the friction loss is simply the hydraulic gradient multiplied by the length of the run. Since this design procedure assumes steady uniform flow (see Section 3.5.6) in open channel flow, the friction slope will match the pipe slope for part full flow. Pipe friction losses for full flow in a circular pipe can be determined by combining Equation 3.20 with Equation 3.17 as follows:

$$S_f = (H_f / L) = [(Q\, n) / (K_Q\, D^{2.67})]^2 \tag{3.21}$$

Where:

K_Q = (0.46 in English units)

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:

$$H_o = 1.0\, [(V_o^2 / 2g) - (V_d^2 / 2g)] \tag{3.22}$$

Where:

V_o = Average outlet velocity
V_d = Channel velocity downstream of outlet in the direction of the pipe flow
g = Acceleration due to gravity, (32.2 ft./s.^{2})

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.

Inlet and Access Hole Losses: Approximate Method

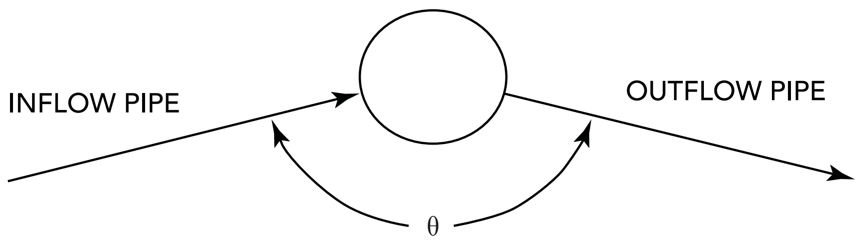
A more complex situation exists where an access hole or inlet exists at the junction between inflow and outflow pipes. The **Approximate method** is the simplest and appropriate only for preliminary design estimates. Approximate method application recognizes that initial layout of a storm drain system begins at the upstream end of the system. The designer must estimate sizes and establish preliminary elevations as the design progresses downstream. The Approximate method estimates losses across an access hole by multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 3.23. Table 3-8 tabulates typical coefficients (K_{ah}) applicable for use in this method.

$$H_{ah} = K_{ah} (V_o2 / 2g) \quad \textbf{(3.23)}$$

The Approximate method estimates 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.

Table 3-8 Head Loss Coefficients

Structure Configuration	K_{ah}
Inlet — straight run, square edge	0.50
Inlet — angle through 90^{o}	1.50
Access Hole — straight run	Min ~ 1.50
Access Hole — angle through 90^{o}	K_{ah}
90^{o}	1.00
120	0.85
135	0.75
157.5	0.45



However, this is a preliminary estimate only and should not be used when making energy grade line (EGL) calculations. Instead, FHWA has determined that the method outlined below, is most suitable and accurate to calculate the losses across an access hole when establishing the energy grade line.

FHWA Inlet and Access Hole Energy Loss

For many years, FHWA has been developing and refining more complex approaches for estimating losses in access holes and inlets. Various methodologies have been advanced for evaluating losses at access holes and other flow junctions. Notable methods included:

Corrective Coefficient Energy-Loss Method — based on a FHWA research report by Chang and Kilgore.

Composite Energy Loss Method — developed in the research report “Energy Losses through Junction Manholes” (FHWA-RD-94-080, November 1994)

Since their development and publication, researchers and practitioners in the transportation community have encountered several limitations in these methods (also including the Approximate method). These limitations include:

- Using or developing a single coefficient multiplied by an outlet velocity head limits the representation of very different hydraulic conditions within access holes
- Difficulties with producing reasonable results on some surcharged systems and/or systems with supercritical flows leaving the access hole
- Under prediction of calculated versus observed access hole flow depths, particularly in the Composite Energy Loss Method
- Methods are relatively complex and require iterative solutions
- Limitations in these methods result in problematic solutions in some situations
- Inconsistent application within FHWA and private vendor software tools for designing storm drains

To address these issues, FHWA supported research and laboratory efforts to improve methodologies. The resulting approach classifies access holes and their hydraulic conditions in a manner analogous to inlet control and full flow for culverts.

Given this characterization, the method applies equations in appropriate forms for the given classification. In addition to avoiding the limitations described above, this method has the following benefits:

1. Uses hydraulically sound fundamentals for key computations (inlet control and full flow analogies) as a foundation for extrapolating the method beyond laboratory data
2. Incorporates approach to handle surcharged systems with the full flow component of the method
3. Avoids problems associated with supercritical flows in outlet pipe by using a culvert inlet control analogy
4. Provides equivalent or better performance in predicting access hole water depth and inflow energy grade line on the extensive FHWA laboratory data set
5. Direct (non-iterative) and simple computational procedure that can be verified manually The method follows three fundamental steps (with terms as defined in Figure 3-14):

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

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

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

These three fundamental steps are described in the following sections.

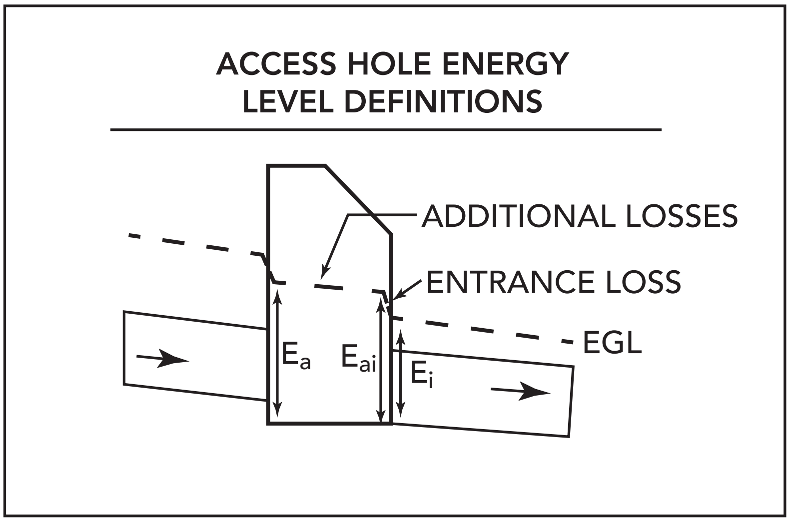


Figure 3-14 Definition sketch for FHWA access hole method

STEP 1: Initial Access Hole Energy Level

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

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

2. Inlet control (submerged) condition – considered to possibly occur if the opening in the access hole structure to the outlet pipe is limiting and the resulting water depth in the access hole is sufficiently high that flow through the opening is treated as an orifice.

3. Inlet control (unsubmerged) condition – considered to possibly occur if the flow control is also limited by the opening, but the resulting water level in the access hole requires treating the opening as a weir.

Energy, Velocity, Pressure, and Potential Heads

The method addresses one of the weaknesses of other methodologies, which is the large reliance on outflow pipe velocity. The full flow computation uses velocity head, but full flow only applies when the outflow pipe is flowing full. The two inlet control estimates depend only on discharge and pipe diameter. This is important because velocity is not a reliable parameter for the following reasons:

1. In cases where supercritical flow occurs in the outflow pipe, flow in the outflow pipe (and the corresponding velocity head) are defined by the upstream condition at the access hole rather than the velocity head determining upstream conditions.

2. In the laboratory setting used to derive most coefficients and methods, velocity is not directly measured. It is calculated from depth and the continuity relationship. Small errors in depth measurement can result in large variations in velocity head.

3. Velocities produced in laboratory experiments are the result of localized hydraulic conditions, which are not necessarily representative of the velocities calculated based on equilibrium pipe hydraulics in storm drain computations.

The situation may be exasperated when seeking to obtain values for other elements of total outflow pipe energy head (E_i), such as outflow pipe depth (potential head) and pressure head. E_i can be described as the sum of the potential, pressure, and velocity head components:

$$E_i = y + (P / \gamma) + (V^2 / 2g) \tag{3.24}$$

Where:

y	=	Outflow pipe depth (potential head), (ft.)
(P / γ)	=	Outflow pipe pressure head, (ft.)
$(V^2 / 2g)$	=	Outflow pipe velocity head, (ft.)

Solving for Equation 3.24 may be problematic for certain conditions (e.g., where P cannot be assumed to equal atmospheric pressure). Fortunately, E_i can also be determined by subtracting the outflow pipe invert elevation (Z_i) from the outflow pipe energy gradeline (EGL_i) (both known values) at that location:

$$E_i = EGL_i - Z_i \tag{3.25}$$

Algebraic manipulation of Equations 3.24 and 3.25 allows derivation of any problematic values, especially in a full flow condition.

Additionally, knowing E_i serves as a check on the method. There are rare, generally very low flow circumstances, where some of the elements may yield access hole energy levels less than the outflow pipe energy head. In such cases, the access energy levels should be made equal to the outflow pipe energy head.

Initial Access Hole Energy Level

The initial estimate of energy level is taken as the maximum of the three values:

$$E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu}) \tag{3.26}$$

Where:

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

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

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

$$E_{aio} = E_i + H_i \tag{3.27}$$

Where:

H_i	=	Entrance loss assuming outlet control, calculated using Equation 7-16

$$H_i = K_i\,(V^2/2g) \qquad \textbf{(3.28)}$$

Where:

K_i = Entrance loss coefficient = 0.2, dimensionless

As described earlier, using the concept of outflow pipe energy head (E_i) and Equation 3.27 allows estimation of energy level directly without considering the water surface within the access hole. Defining a one-dimensional velocity head in a location where highly turbulent multi-directional flow may exist presents a challenge.

Estimated Energy Level for Inlet Control: Submerged

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

$$DI = Q / [A\,(gD_o)^{0.5}] \qquad \textbf{(3.29)}$$

Where:

A = Area of outflow pipe, (ft.^{2})
D_o = Diameter of outflow pipe, (ft.)

The submerged inlet control condition uses an orifice analogy to estimate the energy level (E_{ais}) (Equation 3.30). Derivation of Equation 3.30 used data with discharge intensities less than or equal to 1.6.

$$E_{ais} = D_o\,(DI)^2 \qquad \textbf{(3.30)}$$

Estimated Energy Level for Inlet Control: Unsubmerged

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

$$E_{aiu} = 1.6\,D_o\,(DI)^{0.67} \qquad \textbf{(3.31)}$$

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

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

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

Revised Access Hole Energy Level

The revised access hole energy level (E_a) equals the initial estimate (E_{ai}) modified by each of the three factors covered in this section as shown below:

$$E_a = E_{ai} + H_B + H_\theta + H_P \qquad (3.32)$$

Where:

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

Ea represents the level of the energy gradeline in the access hole. **However, if E_a is calculated to be less than the outflow pipe energy head (E_i), then E_a should be set equal to E_i.**

Designers may also wish to know the water depth in the access hole (ya). A conservative approach would be to use E_a as ya for design purposes.

Traditional approaches to energy losses typically attempt to estimate all losses based on a single velocity head, with the limitations described earlier. The method estimates these additional energy losses as a function of the total energy losses computed between the access hole and the outflow pipe.

Additional Energy Loss: Benching

Benching tends to direct flow through the access hole, resulting in a reduction in energy losses. Figure 3-15 illustrates some typical bench configurations.

For the situation where benching occurs in the access hole, the additional benching energy loss is:

$$H_B = C_B (E_{ai} - E_i) \qquad (3.33)$$

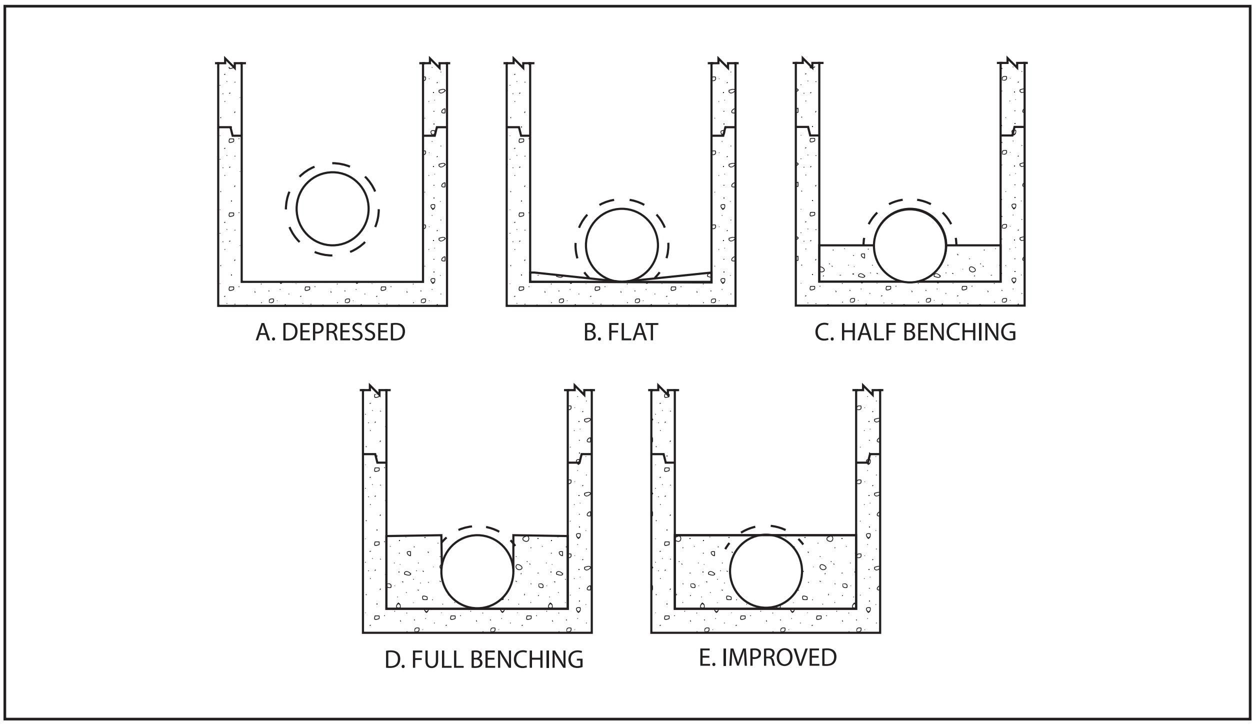


Figure 3-15 Access hole benching methods

Where:

C_B = Energy loss coefficient for benching, from Table 3-9. A negative value indicates water depth will be reduced rather than increased.

Table 3-9 Values for the Coefficient, C_B

Floor Configuration	Bench Submerged*	Bench Unsubmerged*
Flat (level)	-0.05	-0.05
Depressed	0.0	0.0
Half Benched	-0.05	-0.85
Full Benched	-0.25	-0.93
Improved	-0.60	-0.98

*A bench submerged condition has the properties of $(E_{ai}/D_o)>2.5$ and bench unsubmerged condition has the properties of $(E_{ai}/D_o)<1.0$. Linear inter- polation between the two values is used for intermediate values. Additional Energy Loss: Angled Inflow

Additional Energy Loss: Angled Inflow

The effect of skewed inflows entering the structure is addressed considering momentum vectors. To maintain simplicity, the contribution of all inflows contributing to structure and with a hydraulic connection (i.e., not plunging) are resolved into a single flow weighted angle (θ w):

$$\theta_w = \Sigma\ (Q_J\ \theta_J)\ /\ \Sigma Q_J \tag{3.34}$$

Where:

Q_J = Contributing flow from inflow pipe, (ft.³/s.)
θ_J = Angle measured from the outlet pipe (180 degrees is a straight pipe)

Figure 7-7 illustrates the orientation of the pipe inflow angle measurement. The angle for each of the non-plunging inflow pipes is referenced to the outlet pipe, so that the angle is not greater than 180 degrees. A straight pipe angle is 180 degrees. The summation only includes non- plunging flows as indicated by the subscript j. If all flows are plunging, θw is set to 180 degrees.

An angled inflow coefficient ($C\theta$) is then calculated as follows:

$$C_\theta = 4.5\ (\Sigma Q_J\ \ /\ Q_o)\ \cos\ (\theta_w\ /\ 2) \tag{3.35}$$

Where:

Q_o = flow in outflow pipe, (ft.³/s.)

The angled inflow coefficient approaches zero as θw approaches 180 degrees and the relative inflow approaches zero.

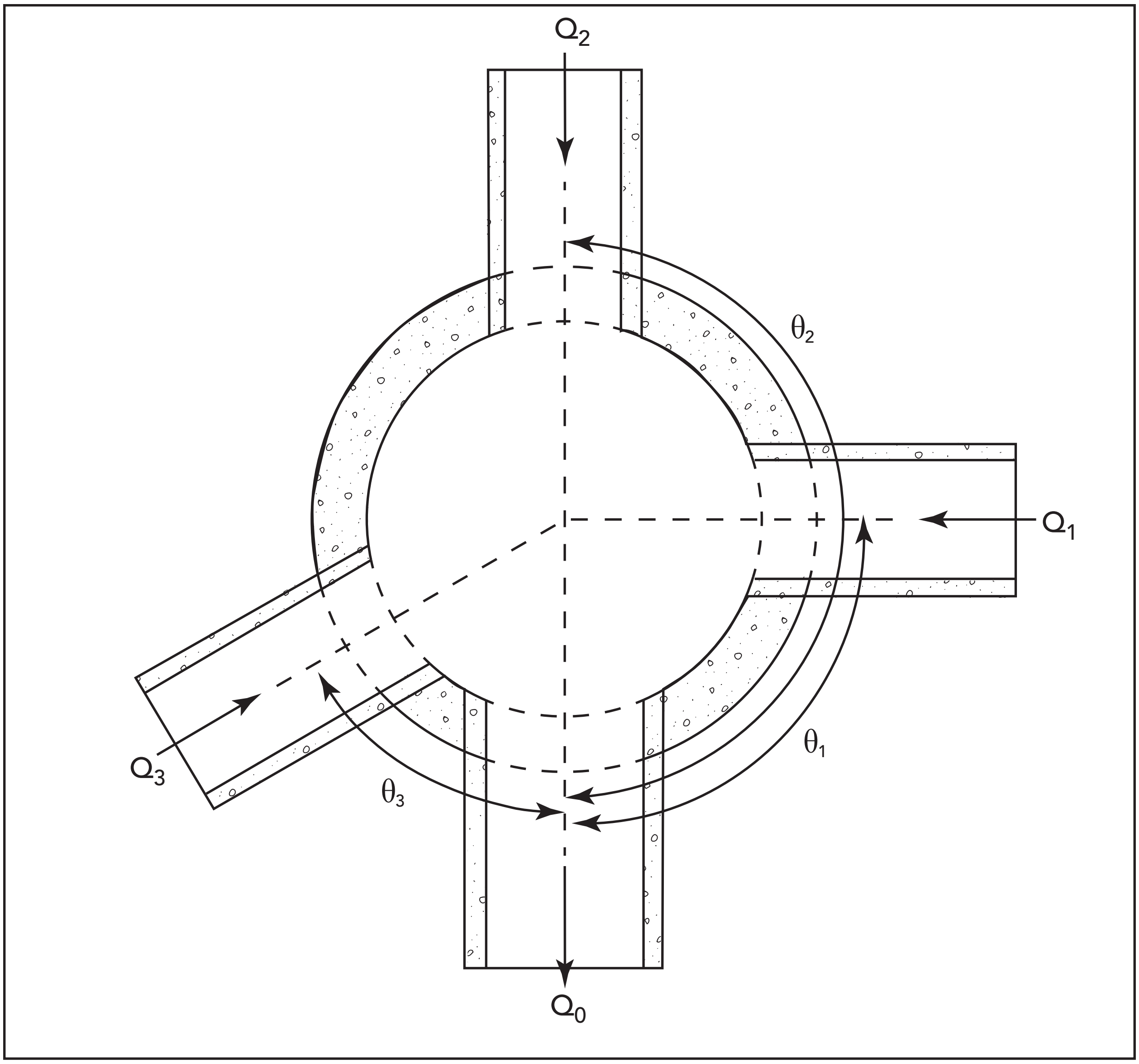


Figure 3-16 Access hole angled inflow definition.

The additional angle inflow energy loss is:

$$H_{\theta} = C_{\theta}(E_{ai} - E_i) \quad \textbf{(3.36)}$$

Additional Energy Loss: Plunging Inflow

Plunging inflow is defined as inflow (pipe or inlet) where the invert of the pipe (z_k) is greater than the estimated structure water depth (approximated by E_{ai}). The value of z_k is the difference between the access hole invert elevation and the inflow pipe invert elevation.

The method defines a relative plunge height (h_k) for a plunging pipe (denoted by the subscript $_k$) as:

$$h_k = (z_k - E_{ai}) / D_o \qquad \textbf{(3.37)}$$

This relative plunge height allows determination of the plunging flow coefficient (CP):

$$C_P = \Sigma(Q_k h_k) / Q_o \qquad \textbf{(3.38)}$$

As the proportion of plunging flows approaches zero, CP also approaches zero. Equation 3.37 and Equation 3.38 are limited to conditions where $z_k < 10D_o$. If $z_k > 10D_o$ it should be set to $10D_o$.

The additional plunging inflow energy loss is given by:

$$H_P = C_P (E_{ai} - E_i) \qquad \textbf{(3.39)}$$

Alternative Approach for finding Additional Energy Loss

The method allows determination of the incremental benching (H_B), inflow angle (H_θ), and plunging energy (H_P) terms. However, incremental losses can be small - possibly even small enough to be "lost in the rounding." An alternative means of computing Ha would be to algebraically rearrange Equations 3.33, 3.36, and 3.39 to yield:

$$H_a = (C_B + C_\theta + C_P)(E_{ai} - E_i) \qquad \textbf{(3.40)}$$

Note that the value of H_a should always be positive. If Equation 3.40 yields a negative value, H_a should be set equal to zero. By rearranging Equations 3.32 and 3.40, the access hole energy level (E_a) is:

$$E_a = E_{ai} + H_a \qquad \textbf{(3.41)}$$

As always, care should be taken such that E_a is never less than E_i. If this occurs, use the higher energy level.

Access Hole Energy Gradeline

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

$$EGL_a = E_a + Z_a \qquad \textbf{(3.42)}$$

As described earlier, the potentially highly turbulent nature of flow within the access hole makes determination of water depth problematic. However, it is not an unreasonable assumption to use the EGLa as a comparison elevation to check for potential surcharging of the system. Research has shown that determining velocity head within the access hole is very difficult - even in controlled laboratory conditions.

STEP 3: Inflow Pipe Exit Losses

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

Non-Plunging Inflow Pipe

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

$$EGL_o = EGL_a + H_o \tag{3.43}$$

Where:

H_o = Inflow pipe exit loss, calculated using Equation 3.44

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

$$H_o = K_o\,(V^2/2g) \tag{3.44}$$

Where:

K_o = Exit loss coefficient = 0.4, dimensionless

Plunging Inflow Pipe

The second case is for an inflow pipe in a plunging condition. For pipes that are plunging, the inflow pipe energy gradeline (EGL_o) is taken as the energy gradeline calculated from the inflow pipe hydraulics. Logically, EGL_o is independent of access hole water depth and losses. Determining the energy gradeline for the outlet of a pipe has already been described in Section 3.5.6.1.

Continuing Computations Upstream

For either the nonplunging or plunging cases, the resulting energy gradeline is used to continue computations upstream to the next access hole. The three step procedure of estimating: (1) entrance losses, (2) additional losses, and (3) exit losses is repeated at each access hole.

3.5.6.3 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 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 they can better interpret the output from computer generated storm drain designs. Figure 3-17 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 Figures 3-18a and 3-18b 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

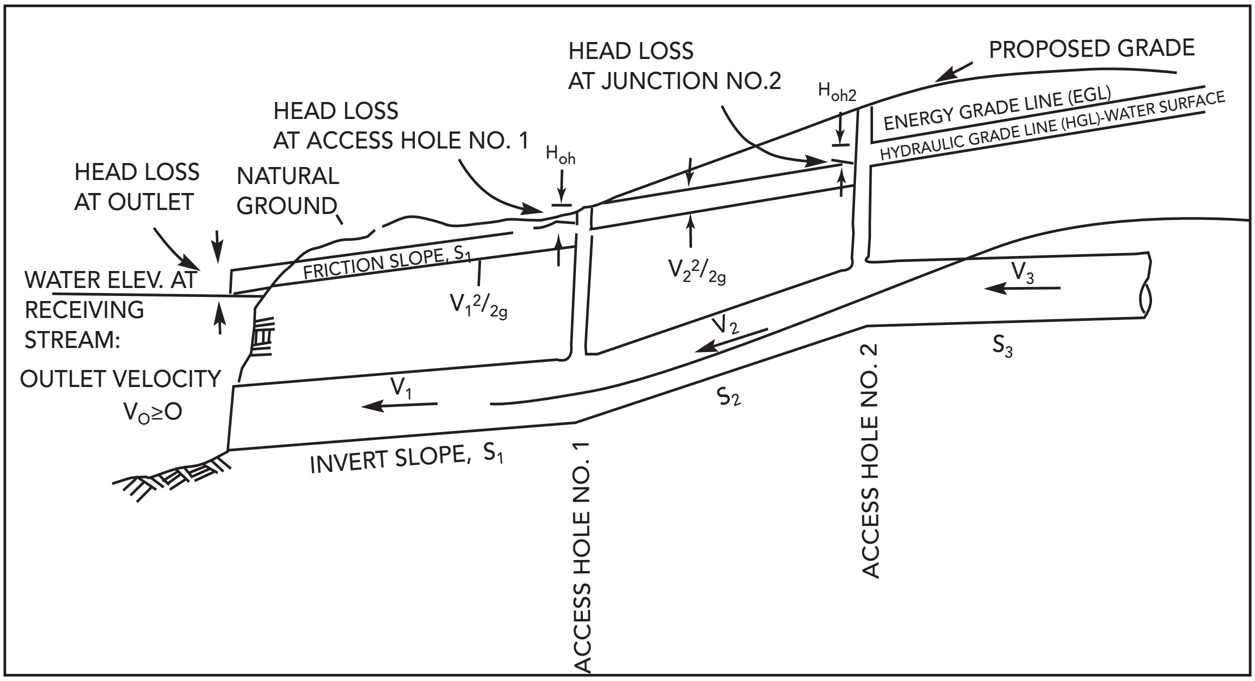


Figure 3-17 Energy and hydraulic grade line illustration.

sheet (lines may be skipped on the form for clarity). Table A (Figure 3-18) is used to calculate the HGL and EGL elevations while Table B (Figure 3-18) 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. The description of the computation procedures uses a column number followed by a letter A or B to indicate the appropriate table to use (for example, Table A, Column 1 would be designated as "Column 1A").

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.

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 EGL in Column 9A and the HGL in Column 10A of the first line on the computation sheet.

Note: The notation used in this procedure will be that one line in Table A will be devoted to each access hole or inlet and it's respective outflow conduit. The conduit downstream or "outfall" EGL_o and HGL_o values will be placed in Columns 9A and 10A, respectively. The conduit's upstream or "entrance" EGL_i and HGL_i will be placed in Columns 13A and 14A, respectively. The EGL_a in the access hole will be placed in Column 16A.

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 tailwater (the hydraulic grade line is either the downstream tailwater elevation or the average of critical depth and the height of the storm drain conduit, (dc + D)/2, whichever is greater), the tailwater energy gradeline, the top of conduit (TOC) elevation at the outfall end, and the ground surface elevation at the outfall end of the conduit. Place these values in Columns 1A, 9A, 10A, 17A, and 18A respectively. Also add the structure number in Column 1B.

Step 3 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 4 Full Flow Assumption: Determine the EGL and HGL at the inside face of D/S end of the conduit.

Notes:

(1) This is the outflow conduit of the structure identified in Step 3. The procedure assumes that while there may be several inflow conduits entering a structure, there is typically only a single conduit system as the outflow pipe.

(2) If the full flow assumption conditions apply at the conduit outfall the designer must later check the Hydraulic Gradeline, HGL_i, at the conduit inlet to confirm the conduit does not become unsealed and revert to partial flow conditions (see Step 14) within that section of the storm drain system.

The information required depends on the flow conditions at both structure and conduit. This requires consideration of several alternative cases:

Case A: If the TW elevation at the conduit outfall is greater than the TOC (submerged conduit outlet elevation), the pipe is assumed to be in a subcritical, full flow condition.

(i) Enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in column 8A.

(ii) The ***exit loss*** (H_o) is calculated (Equation 3.22) based on the velocity head for the conduit flow condition (Column 8A). Place the H_o value in Column 2B.

(iiii) The EGL_o (at inside face of conduit outfall) will be the EGL from line 2 Column 9A plus the exit loss (Column 2B). Place in Column 9A.

(iv) The HGL_o (at inside face of conduit outfall) will be the EGL_o (Column 9A) minus the velocity head (Column 8A). Place in Column 10A.

(v) Continue with Step 11.

Case B: If the energy gradeline elevation (EGL_a) in the access hole is greater than the TOC (submerged conduit outlet elevation), the pipe is assumed to be in a subcritical, full flow condition.

(i) Enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in column 8A.

(ii) The exit loss (H_o) will equal to 0.4 times the velocity head for the conduit flow condition (Column 8A) (Equation 3.44). Place the Ho value in Column 2B.

(iii) The EGL_o will be the EGL_a (Column 14A from the previous structure plus the exit loss (Column 2B). Place in Column 9A.

(iv) The HGL_o (at inside face of conduit outfall) will be the EGL_o (Column 9A) minus the velocity head (Column 8A). Place in Column 10A.

(v) Continue with Step 11.

ENERGY GRADE LINE COMPUTATION SHEET – TABLE A

COMPUTED BY:
CHECKED BY:
PAGE:

DATE:
DATE:
OF:

ROUTE:
SECTION:
COUNTY:

INITIAL TAILWATER ELEV.:

STRUCT. ID.	D (FT.,M)	Q (CFS, CMS)	L (FT., M)	V (FT./S., M/S.)	d (FT,M)	d_o (FT,M)	$V^2/2g$ (FT., M)	EGL_o (FT., M)	HGL_o (FT., M)	S_f (FT./FT., M/M)	TOTAL PIPE LOSS (TABLE B) (FT./M)	EGL (FT., M)	HGL_i (FT., M)	E_a (TABLE B) (FT., M)	EGL_a (FT., M)	U/S TOC (FT.,M)	SURFACE ELEV. (FT.,M)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)

Figure 3-18 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:

INITIAL TAILWATER ELEV.:

	EXIT	PIPE LOSSES (FT.,M)						STRUCTURE LOSSES (FT., M)								
STRUCT. ID.	H_o	H_f	H_b	H_c	H_e	H_j	TOTAL	E_i	$y+(P/\gamma)$	DI	E_{ai}	C_B	C_θ	C_P	H_a	E_a
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)

Figure 3-18 Energy grade line computation sheet - Table B

Step 5 Partial Flow Assumption: Use the hydraulic elements graph in Figure 3-19 with the ratio of part full to full flow (values from the Preliminary Storm Drain Computation Form) to compute the velocity and depth of flow (normal depth) in the conduit. Enter these values in Column 5A and 6A. Compute the velocity head (V2/2g) and place in Column 8A.

Note: 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, the calculations in the preliminary form may be helpful. Actual flow velocities need to be used in the EGL/HGL calculations.

Step 6 Compute critical depth for the conduit using Figure 3-20. If the conduit is not circular, see HDS-5 for additional charts. Enter this value in Column 7A.

Note: The values for d_c cannot be greater than the rise or diameter of the conduit.

Step 7 Still trying to determine the EGL_o and HGL_o at the downstream end of the conduit. The information required depends on flow condition and regime at both structure and conduit. This requires consideration of several alternatives (Note: BOC is the conduit downstream invert elevation):

Case A: If the TW elevation at the conduit outfall OR if the access hole energy gradeline (EGL_a) is less than the conduit invert elevation (i.e., a plunging pipe):

(i) The exit loss (H_o) does not affect the conduit hydraulics. Place a zero (0) value for the H_O value in Column 2B.

(ii) The EGL_o will be the normal depth elevation (Column 6A plus BOC) plus the velocity head for the conduit flow condition (Column 8A). Place the EGL_o in Column 9A.

(iii) The HGL_o will be the normal depth elevation (Column 6A plus BOC). Place the HGL_o in Column 10A.

Case B: If the TW elevation at the conduit outfall OR if the access hole energy gradeline (EGL_a) is equal to or less than conduit critical depth elevation(i.e., Column 7A plus BOC):

(i) The ***exit loss*** (H_o) does not affect the conduit hydraulics. Place a zero (0) value for the H_o value in Column 2B.

(ii) The EGL_o will be the normal depth elevation (Column 6A plus BOC) plus the velocity head for the conduit flow condition (Column 8A). Place the EGL_o in Column 9A.

(iii) The HGL_o will be the normal depth elevation (Column 6A plus BOC). Place the HGL_o in Column 10A.

Case C: If the TW elevation at the conduit outfall is greater than conduit critical depth elevation (i.e., Column 7A plus BOC) but less than or equal to the conduit normal depth elevation (i.e., Column 6A plus BOC):

(i) The ***exit loss*** (H_o) is calculated (Equation 7-4) based on the velocity head for the conduit flow condition (Column 8A). Place the H_o value in Column 2B.

(ii) The EGL_o will be the greater of: a) TW elevation plus the exit loss (Column 2B); or b) normal depth elevation (Column 6A plus BOC) plus the velocity head for the conduit flow condition (Column 8A). Place the EGL_o in Column 9A.

(iii) The HGL_o (at inside face of conduit outfall) will be the EGL_o (Column 9A) minus the velocity head (Column 8A). Place in Column 10A.

Case D: If the access hole energy gradeline (EGL_a) is greater than conduit critical depth elevation (i.e., Column 7A plus BOC) but less than or equal to the conduit normal depth elevation (i.e., Column 6A plus BOC):

(i) The exit loss (H_o) will equal to 0.4 times the velocity head (Equation 7-32) for the conduit flow condition (Column 8A). Place the H_o value in Column 2B.

(ii) The EGL_o will be the greater of: a) the access hole energy gradeline (EGL_a) (Column 8A) plus the exit loss (Column 2B); or b) normal depth elevation (Column 6A plus BOC) plus the velocity head for the conduit flow condition (Column 8A). Place the EGL_o in Column 9A.

(iii) The HGL_o (at inside face of conduit outfall) will be the EGL_o (Column 9A) minus the velocity head (Column 8A). Place in Column 10A.

Case E: If the TW elevation at the conduit outfall is greater than the conduit normal depth elevation (i.e., Column 6A plus BOC), but less than TOC:

(i) Set D/S conduit face depth equal to TW elevation minus BOC.

(ii) Use the hydraulic elements graph in Figure 3-19 with the ratio of part face depth to diameter to compute the partial flow area of the D/S conduit face. Use flow (Column 3A) and continuity to determine D/S conduit face velocity. Compute the D/S conduit face velocity head ($V^2/2g$) (outlet velocity head).

(iii) The exit loss (H_o) is calculated (Equation 3.22) based on the velocity head for the conduit flow condition (Column 8A). Place the H_o value in Column 2B.

(iv) The EGL_o will be this TW elevation plus the outlet velocity head plus exit loss (Column 2B). Place the EGL_o in Column 9A.

(v) The HGL_o (at inside face of conduit outfall) will be the EGL_o (Column 9A) minus the velocity head (Column 8A). Place in Column 10A.

Case F: If the access hole energy gradeline (EGL_a) is greater than the conduit normal depth elevation (i.e., Column 6A plus BOC), but less than TOC:

(i) Set D/S conduit face depth equal to EGL_a minus BOC.

(ii) Use the hydraulic elements graph in Figure 3-19 with the ratio of part face depth to diameter to compute the partial flow area of the D/S conduit face. Use flow (Column 3A) and continuity to determine D/S conduit face velocity. Compute the D/S conduit face velocity head ($V^2/2g$) (outlet velocity head).

(iii) The exit loss (H_o) will equal to 0.4 times the outlet velocity head. Place the H_o value in Column 2B.

(iv) The EGL_o will be the EGL_a plus the exit loss (Column 2B). Place the EGL_o in Column 9A.

(v) The HGL_o (at inside face of conduit outfall) will be the EGL_o (Column 9A) minus the velocity head (Column 8A). Place in Column 10A.

Step 8 Compare the flow depth in Column 6A with the critical depth in Column 7A to determine the flow regime state in the conduit.

Case A: If the flow depth in Column 6A is greater than the critical depth in Column 7A, the flow is subcritical, continue with Step 11.

Case B: If the flow depth in Column 6A is less than or equal to the critical depth in Column 7A, the flow is supercritical, continue with Step 9.

Note: In either case, remember that the EGL must be higher upstream for flow to occur. If after checking for supercritical flow in the upstream section of pipe, assure that the EGL is higher in the pipe than in the structure.

Step 9 Pipe losses in a supercritical pipe section are not carried upstream. Therefore, enter a zero (0) in Column 8B for this structure.

Step 10 Assume that the hydraulic gradeline at the upstream end of the conduit (HGL_i) is equal to the normal depth (Column 6A) plus the upstream conduit invert elevation. Place in Column 14A. The EGL_i would then be HGL_i plus the conduit velocity head (Column 8A). After adding the EGL_i value to Column 13A, skip to Step 15.

Step 11 Compute the friction slope (S_f) for the pipe using Equation 7-3: Sf

$$S_f = H_f / L = [Qn / (K_Q D^{2.67})]^2 \quad \textbf{(7.30)}$$

Enter this value in Column 11A of the current line. Equation 7-3 assumes full flow in the outlet pipe. If full flow does not exist, set the friction slope equal to the pipe slope.

Step 12 Compute the friction loss (H_f) by multiplying the length (L) in Column 4A by the friction slope (S_f) in Column 11A and enter this value in Column 3B.

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_i) will be not apply since the City of Omaha requires access holes in these locations. Place the value of 0 in Columns 4B, 5B, 6B, and 7B, respectively. Add the values in Columns 3B, 4B, 5B, 6B, and 7B and place the total in Column 8B and 12A.

Step 13 Compute the energy grade line value at the U/S end of the conduit (EGL_i) as the EGL_o from D/S end of the conduit (Column 9A) plus the total pipe losses (Column 12A). Enter the EGL_i in Column 13A.

Compute the hydraulic grade line value at the U/S end of the conduit (HGL_i) as the EGL_i minus the velocity head to get HGL_i. Enter the HGL_i in Column 14A.

Step 14 Verify flow conditions at inlet end of conduit.

Case A: If HGL_i is greater than or equal to the TOC of the inlet end of the conduit, the conduit is full flow. Put “full” in Column 6A (normal depth) and not applicable “n/a” in Column 7A (critical depth), and continue with Step 15.

Case B: If HGL_i is less than TOC but greater than normal depth and greater than critical depth (see procedures of Steps 5 and 6) conduit is not in full flow, but tailwater or D/S access hole conditions still control. Put the normal depth value in Column 6A (normal depth) and critical depth value in Column 7A (critical depth), and continue with Step 15.

Case C: If HGL_i is less than TOC but greater than critical depth and less than or equal to normal depth (see procedures of Steps 5 and 6), this indicates subcritical partial flow conditions. Put the normal depth value in Column 6A (normal depth) and critical depth value in Column 7A (critical depth). Revise EGL_i (Column 12A) by adding normal depth plus the velocity head to the conduit invert elevation. Go to Step 15.

Case D: If HGL_i is less than critical depth (see procedures of Steps 5 and 6) conduit is in supercritical partial flow conditions. Put the normal depth value in Column 6A (normal depth) and critical depth value in Column 7A (critical depth) and proceed to Step 9.

Step 15 Estimate outflow pipe energy head (E_i) by subtracting the pipe invert elevation (from the Preliminary Storm Drain Computation Form) from the EGL_i (Column 13A). Enter this value in Column 9B. Determine sum of pressure head and potential head ($y + P/\gamma$) by subtracting velocity head from E_i (Column 9B). Place this term in Column 10B. Compute the discharge intensity (Equation 3.29) and place in Column 11B.

Step 16 Determine initial access hole energy level (E_{ai}) as the maximum energy level of (a) outlet control - partial or full flow, (b) inlet control - unsubmerged, and (c) inlet control - submerged:

(1) Outlet control - partial or full flow (E_{aio}): If the outflow conduit is in supercritical flow,

then E_{aio} is equal to zero (see the two conditions associated with Equation 3.27). If not, E_{aio} equals E_i (Column 8B) plus Equation 3.28 (i.e., the conduit velocity head [Column 8A] times 0.2).

(2) Inlet control - submerged (E_{ais}): Place the discharge intensity (Column 11B) into Equation 3.30 to yield E_{ais}.

(3) Inlet control - unsubmerged: (E_{aiu}): Place the discharge intensity (Column 11B) into Equation 3.31 to yield E_{aiu}.

Place the maximum of E_{aio} , E_{ais} , and E_{aiu} into Column 12B.

However, if this value is less than Column 9B, the head loss through the access hole will be zero (Column 16B), and E_a is equal to E_i (Column 9B). Enter E_a in Column 17B and Column 15A and skip to Step 22.

Step 17 Obtain energy loss coefficient for benching (CB) from Table 3-9 and place into Column 13B. Linear interpolation between the two columns of values will most likely be necessary. If benching type is unknown, assume flat benching (making CB equal to -0.05).

Step 18 For non-plunging inflow pipes (Column 9B greater than inflow pipe invert), use Equation 3.34 and 3.35 to obtain energy loss coefficient for angled inflows (C_θ) and enter this value into Column 14B.

Step 19 Using Equations 3.37 and 3.38 to compute energy loss coefficient for plunging flow (C_P) and enter this value in Column 15B.

Note: the value for depth in Equation 3.37 is conservatively estimated by using Eai (Column 12B).

Step 20 Subtract Column 9B (E_i) from Column 12B (E_{ai}). If less than zero, place E_i (Column 9B) into Column 17B and Column 15A and skip to Step 22. Otherwise, multiply the remainder by the sum of values of Columns 13B, 14B, and 15B. If greater than zero, place this result in Column 16B.

Step 21 Add Column 12B to Column 16B. If greater than E_i (Column 9B) place value in Column 17B and 15A (this is E_a). If less than or equal to E_i, place the value of E_i in Column 17B and 15A.

Step 22 Add Column 15A to outflow pipe invert elevation to compute EGL_a. Place EGL_a in Column 16A. (Make notation "access hole energy" somewhere on that row). Assume the hydraulic grade line (HGL_a) at the access hole structure is equal to the EGL_a (Column 16A).

Step 23 Determine the top of conduit (TOC) value for the inflow pipe (using information from the Preliminary Storm Drain Computation Form) and enter this value in Column 17A.

Step 24 Enter the ground surface, top of grate elevation or other high water limits at the structure in Column 18A. If the EGL_a value in Column 16A exceeds the limiting elevation, design modifications will be required.

Step 25 Continue to compute the EGL through the system by repeating Step 3 through Step 24 for each successive upstream access hole or inlet structure. For each inflow conduit, the hydraulic and other elevation information for the access hole would be repeatedly transferred to appropriate Columns in Table A and B.

Figure 3-19 Hydraulic Elements

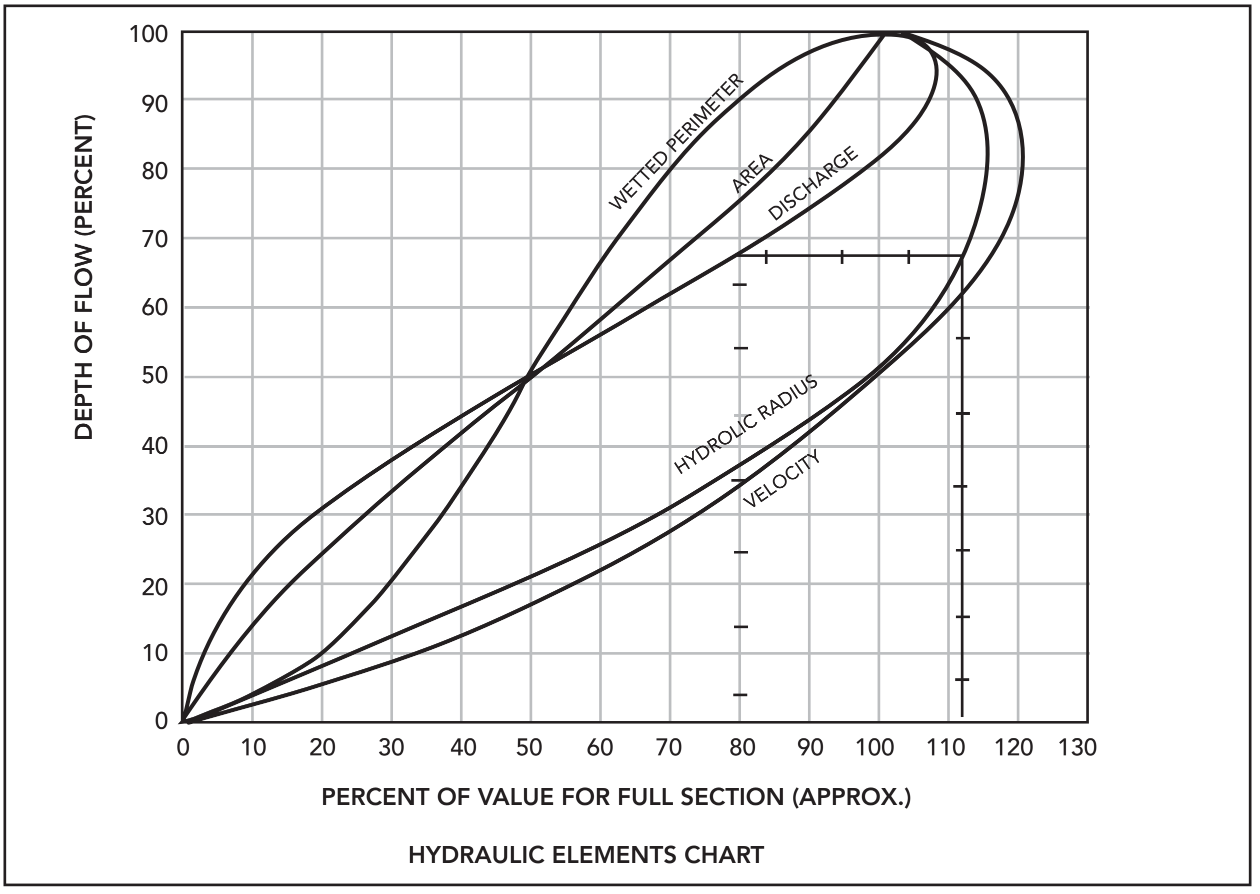
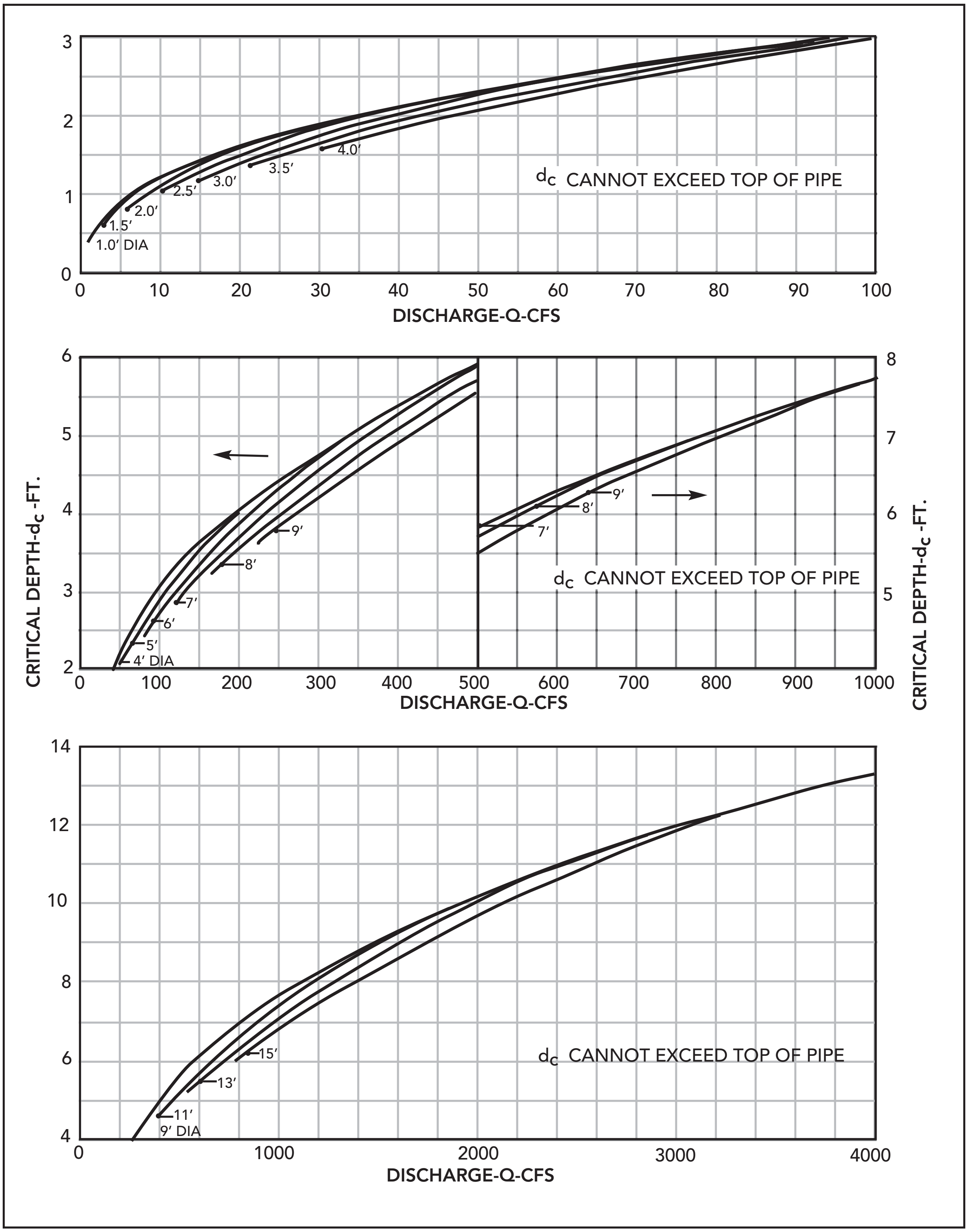


Figure 3-20 Critical Depth in Circular Pipe - English Units



3.6 Computer Programs

3.6.1 Computer Design

There are numerous proprietary and non-proprietary computer models that may be used to design components of the minor storm drainage system. The reader is referred to the user manual for any particular program to determine its suitability for solving storm drainage problems. Programs proposed for use on a specific project by a designer must receive written approval from the Public Works Department prior to application in the project design.

3.7 Outlet into Channels

3.7.1 Pipe Outlets

In addition to requirements of applicable Chapters of this manual, all storm drainage system pipe outlets into channels shall be designed and constructed in accordance with the requirements of Papio-Missouri River NRD regarding pipe installations and trail pavement replacement. The following design criteria are applicable when designing storm drainage pipe outlets into channels:

- All pipe outlets shall have an invert elevation within 2 ft. of the normal (average annual) water surface elevation in the channel, and be located near the normal stream edge (e.g. pipe should not project out into the stream).
- All pipe outlets need to be properly supported and protected. Pipe supports (e.g. timber piles or similar) shall be placed at the pipe outlets, if the pipe is 24 in. in diameter or greater.
- Pipe outlets shall be angled downstream 10 to 15 degrees off perpendicular with respect to the channel flowline to help prevent trash and debris from collecting on the pipe and protect the opposing streambank.
- Pipe sections shall be connected via secure, water tight bands (4 sets of rods and lugs are required for corrugated metal pipes).
- Adequate energy dissipation and stream bank protection as outlined in Chapters 5 and 7 shall be provided in order to meet maximum channel velocities from Chapter 5.
- Backfill material around the pipe shall be placed in 6 in. lifts and compacted to 95% of the maximum density of the Standard Proctor Test (ASTM D698). Trench side slopes shall be 3:1 (horizontal/vertical) or less.
- An easement or permit needs to be obtained for any work or activity that is planned within public right-of-way along a channel.
- Floodgates will be required on any pipe outlet when the pipe is through a public flood control levee. The same level of flood protection must be maintained at all times during pipe installation and construction. Backfill material must meet or exceed the original design specifications of the levee and will need to be overbuilt at cut sections to provide for expected settlement.
- Any concrete trail sections disturbed by construction must be rebuilt to original grade and dimensions.
- Any area disturbed within public right-of-way will need to be reseeded or restored to its original ground cover.

Appendix 3-A

References

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References

American Concrete Pipe Association, March 1968, Design Data.

City of Lincoln, Nebraska Drainage Criteria Manual. February 2000, Revision – May 2004.

AASHTO 2005 Model Drainage Manual

U. S. Department of Transportation, Federal Highway Administration, 2009. Hydraulic Engineering Circular No. 22 - Urban Drainage Design Manual