CHAPTER 3

STORM DRAINAGE SYSTEM

22 February 2000

Chapter Three - Storm Drainage System

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3.1 Overview

3.1.1 Introduction

Every urban area has two separate and distinct drainage systems, whether or not they are actually planned for and designed. One is the *minor* system and the other is 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 of the minor system. The minor drainage system is typically thought of as storm drains and related appurtenances, such as inlets, curbs and gutters. The minor system is normally designed for floods with return frequencies of 5-years to 10-years, depending upon the kind of land use. The minor system has also been termed the “convenience” 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 using “City of Lincoln Standard Specifications for Municipal Construction.”

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	Depression section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
T_s	Spread above depressed section	ft
V	Velocity of flow	ft/s
W	Width of depression for curb opening inlets	ft
Z	T/d, reciprocal of the cross slope	-

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.

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 parking lot, alley or area drain. Grated inlets are not allowed in standard roadway sections.

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 Spread

For multi-laned curb and gutter or guttered roadways with no parking, it is not practical to avoid travel lane flooding when grades are flat. Allowable maximum encroachment is provided in the following table.

Table 3-2 Allowable Maximum Encroachment for Minor Storms

Street Classification	Maximum Encroachment
Local	No curb overtopping.
Collector	No curb overtopping.
Arterial	No curb overtopping. Flow spread must leave at least one lane free of water in each direction.
Freeway	Refer to Nebraska Department of Roads design criteria.

When these encroachments are met, the storm drain system shall commence.

For the major storm runoff, the following street inundation is allowable:

Table 3-3 Allowable Maximum Encroachment for Major Storms

Street Classification	Maximum Encroachment
Local and Collector	The depth of water over the gutter flowline shall not exceed the right-of-way width.
Arterial	The depth of water at the street crown shall not exceed 6 inches.
Freeway	Refer to Nebraska Department of Roads design criteria.

Table 3-4 provides recommendation for allowable cross street flow.

Table 3-4 Allowable Cross Street Flow

Street Classification	Minor Storm Design Runoff	Major Storm Design Runoff
Local	Flow equivalent to 5" depth in upstream curb and gutter section	The depth of water over the gutter flowline shall not exceed the right-of-way width.
Collector	Where cross pans allowed, depth of flow shall not exceed 6 inches.	The depth of water over the gutter flowline shall not exceed the right-of-way width.
Arterial	None	6 inches or less over crown.
Freeway	Refer to Nebraska Department of Roads design criteria.	Refer to Nebraska Department of Roads design criteria.

3.2.4 Longitudinal Slope

A minimum longitudinal gradient is important for a curbed pavement, since it is susceptible to stormwater spread. Flat gradients on uncurbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge.

Curb and gutter grades that are equal to pavement slopes shall not exceed 8 percent or fall below 0.5 percent without approval from the Director of Public Works and Utilities.

3.2.5 Cross Slope

Roadway cross slopes are determined by the City of Lincoln 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 City Standard Drawings 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 and routed away from the highway pavement.

Large median areas and inside shoulders should be sloped to a center swale, preventing drainage from the median area from running across the pavement. This is particularly important for high-speed facilities, and for facilities with more than two lanes of traffic in each direction.

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.

3.2.9 Median/Barriers

Weep holes are often used to prevent ponding of water against barriers (especially on superelevated 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

Gutter flow capacities for City of Lincoln standard street cross-sections are provided in Figure 3-1 for 2.5% pavement cross-slope and in Figure 3-2 for 3.0% pavement cross-slope. For non-standard applications, refer to Sections 3.3.2 through 3.3.7.

3.3.2 Formula

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

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

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. Manning's n values for various pavement surfaces are presented in Table 3-2.

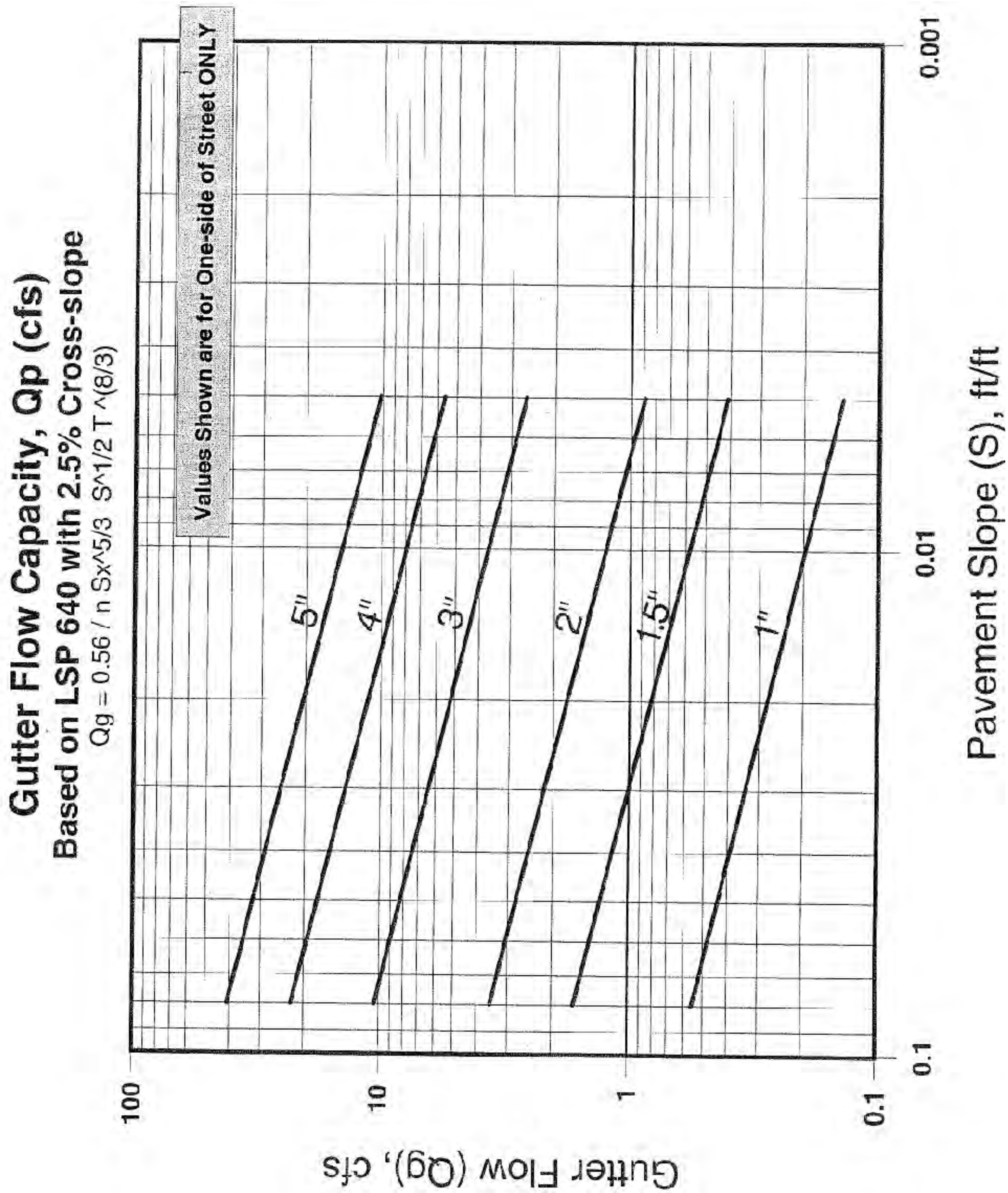


Figure 3-1 Gutter Flow Capacity for 2.5% Pavement Cross-Slope

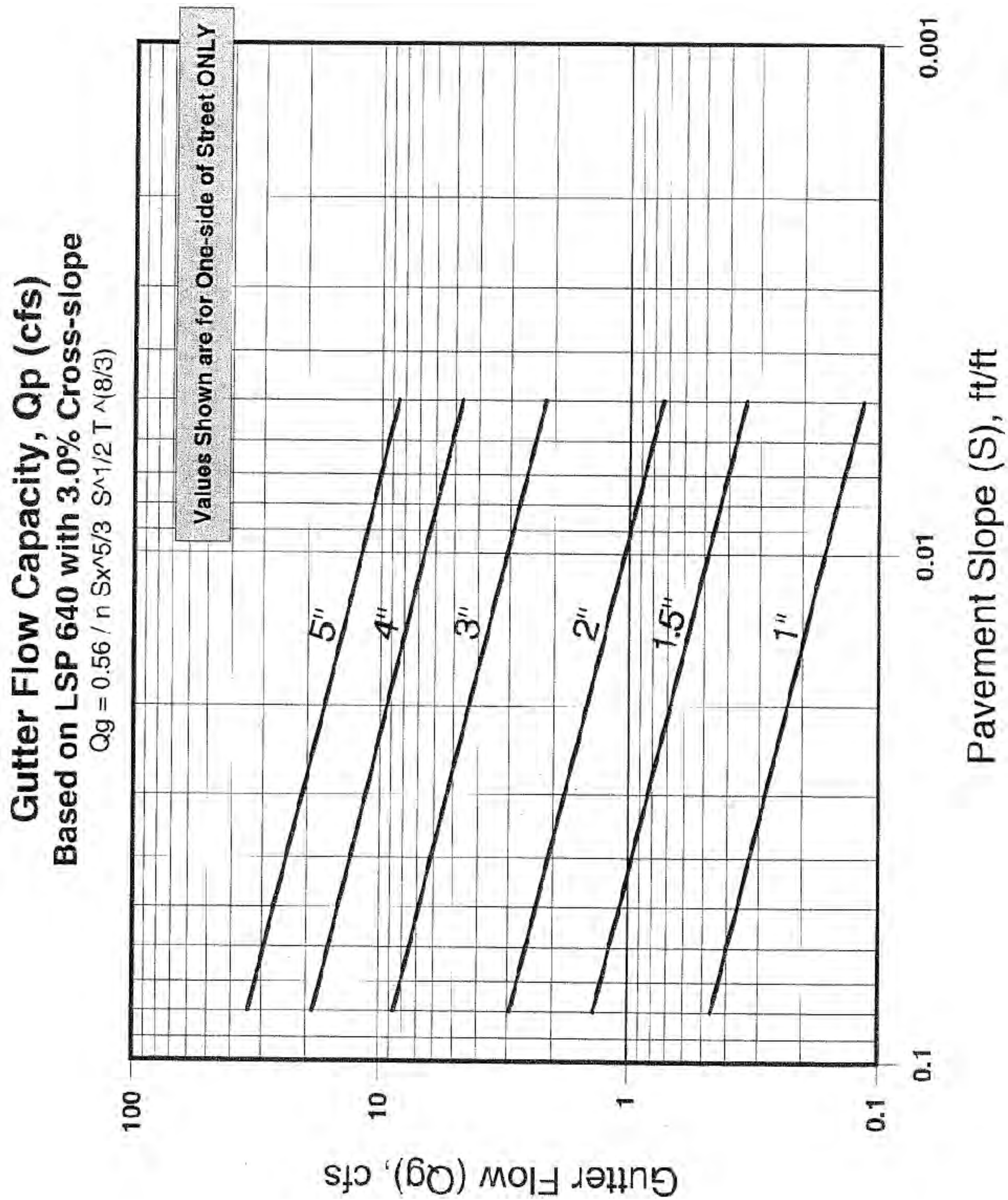


Figure 3-2 Gutter Flow Capacity for 3.0% Pavement Cross-Slope

3.3.4 Manning's n Table

Table 3-2 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.

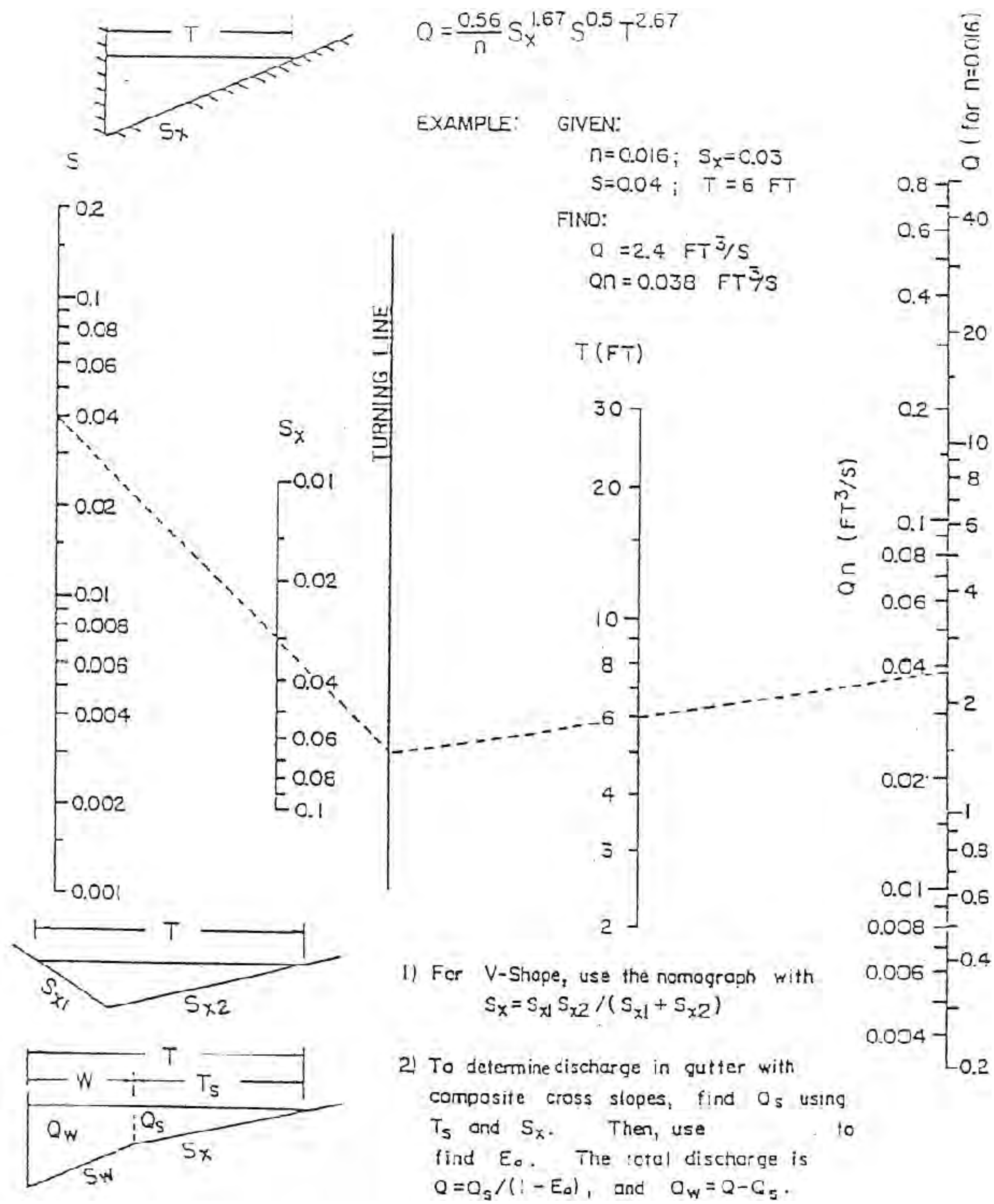


Figure 3-3 Flow In Triangular Gutter Sections

Source: AASHTO Model Drainage Manual, 1991

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 Qn (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 (Qn) 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:

$$\mathbf{Q_w = Q - Q_s} \qquad \textbf{(3.2)}$$

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.

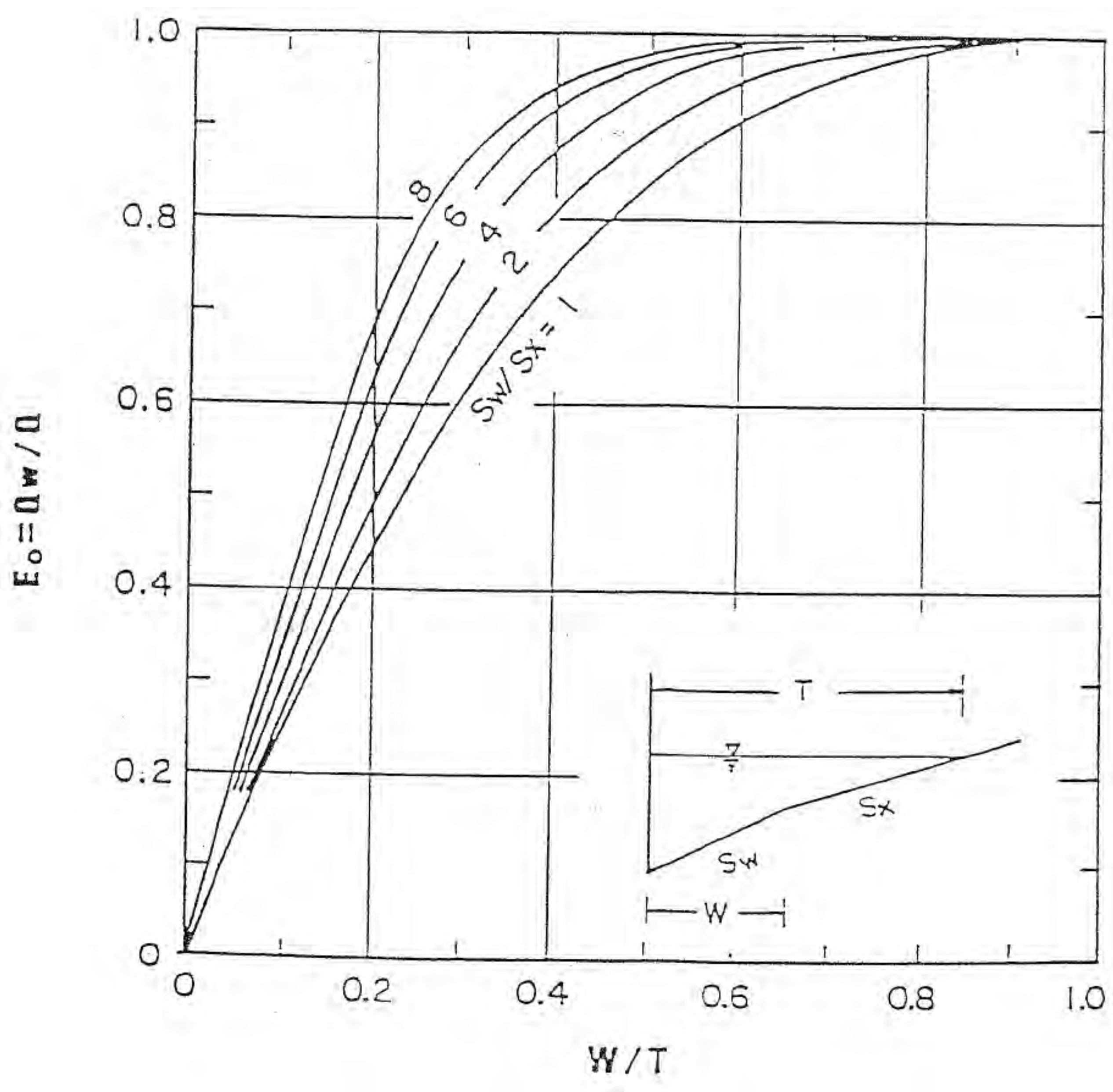


Figure 3-4 Ratio Of Frontal Flow To Total Gutter Flow

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 3-2 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:

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

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, Qn = 0.03
Q = Qn/n = 0.03/0.015 = 2.0 cfs

(2) T_s = 8 - 2 = 6 ft
$(Qn)_2$ = 0.014 (Figure 3-1) (flow in 6 ft width outside of width W)

Q = 0.014/0.015 = 0.9 cfs

Q_w = 2.0 - 0.9 = 1.1 cfs

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

Example 2

Given: $T = 6$ ft $\quad S_w = 0.0833$ ft/ft $\quad n = 0.014$
$T_s = 6 - 1.5 = 4.5$ ft $\quad S = 0.04$ ft/ft
$S_x = 0.03$ ft/ft $\quad W = 1.5$ 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 Storm Water 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-12 (USDOT, FHWA, 1984). 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 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 should not be greater than the width of spread encountered on continuous grades. 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.

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

1. Grate Inlets - These inlets include grate 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.

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

3.4.2 Criteria

The following criteria shall be used for inlet design:

Land Use	Average Return Frequency (years)
Residential Areas	5
Commercial, Industrial, and Arterial Roads	10

Inlets

- 72-inch straight and canted inlets shall be used in the public street system
- Grate inlets may be used for parking lot drains, area drains, etc.
- Flow in the gutter should not exceed five (5) inches.
- Inlets should be placed at the low points in the street grade.

Design charts for standard City of Lincoln inlets are provided in the Chapter 3 of the Manual. The location of the first inlet shall be determined by a trial and error process based upon a point where the maximum depth of flow in the gutter is five inches. Subsequent inlets downstream from the initial inlets shall be located at or before points where the depth of flow in gutter is five inches. Usually inlets shall 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. It may be necessary at some locations to use more than one inlet to pick up the contributing flow. Canted inlets Inlets shall not be placed along intersection radii, unless approved by the Director of Public Works and Utilities.

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 5 inches 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 approach shall be reduced at a gradual rate from 3% 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 exceed 8 percent or fall below 0.5 percent without approval from the Director of Public Works and Utilities.

The detailed procedures and necessary charts to design inlets are described in Chapter 3 of the Manual. Curb and gutter installation shall be in accordance with the current Lincoln Standard Plans and Specifications.

3.4.3 Manholes

Manholes shall be installed at the upper end of all storm drain lines and at all changes in grade, size, or alignment. The recommended maximum spacing is 600 feet for storm drain lines, 36 inches and less in diameter. Greater spacings than this will require approval by the Director of Public Works and Utilities. The crowns of all storm drain pipes entering and leaving a junction shall be at the same elevation. 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. If the diameter of the lateral does exceed one-half the diameter of the pipe being tapped, a storm drain manhole or inlet will be required. The crown of the lateral pipe shall match the crown of the main storm drain pipe. Storm drain manhole shall be constructed in accordance with the most current City Standard Drawings and Specifications.

3.4.4 Grate Inlets

A design chart for the three standard City of Lincoln grate inlets plus curb inlet in a sump condition is provided in Figure 3-5.

The capacity of an 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 gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small. 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 Federal Highway Administration and inlet grate capacity charts prepared by grate manufacturers.

3.4.5 Curb Inlets

Capacities for the standard City of Lincoln 72-inch straight and canted curb inlets are provided in Figures 3-6 and 3-7. Capacity of 48-inch straight inlets used for median drainage is provided in Figure 3-8. In extraordinary conditions, non-standard designs may be needed. Refer to “Drainage of Highway Pavements” Hydraulic Engineering Circular No. 12. A sample Inlet Design Computation Form is provided in Figure 3-9.

Formulas for inlet capacities

Inlet capacity in a sump is controlled by two flow conditions, weir flow and orifice flow. Figure 3-5 was developed using the following two equations:

Sump conditions

$$\mathbf{Q_w = 3.0\ Pd^{1.5}} \qquad \textbf{(3.4a)}$$

and

$$\mathbf{Q_o = 0.67\ A\ (2gd)^{0.5}} \qquad \textbf{(3.4b)}$$

Where, Q_w = weir capacity (cfs)
P = perimeter (ft)
d = depth (ft)

and, Q_o = orifice capacity (cfs)
A = clear opening area (sf)
g = acceleration of gravity - 32.2 ft/s^2
d = depth (ft)

Curb inlet interception capacity is the flow intercepted by an inlet under a given set of conditions. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and to a lesser extent, pavement roughness.

Curb inlets on grade formula

$$\mathbf{Q_i = Q_g\ (1\text{-}\ (1\text{-}E_L/kQ_g)^{0.42}\ s^{0.3}\ (1/\ n\ S_e)^{1.8}} \qquad \textbf{(3.5)}$$

Where, Q_i = inlet capacity (cfs)
Q_g= flow in the gutter (cfs)
E_L = effective length of inlet (ft)
k = 0.6 for US customary units
s = pavement slope (ft/ft)
n = Manning’s roughness coefficient
S_e = composite slope of cross-section (ft/ft)

3.4.6 Flared End Sections

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

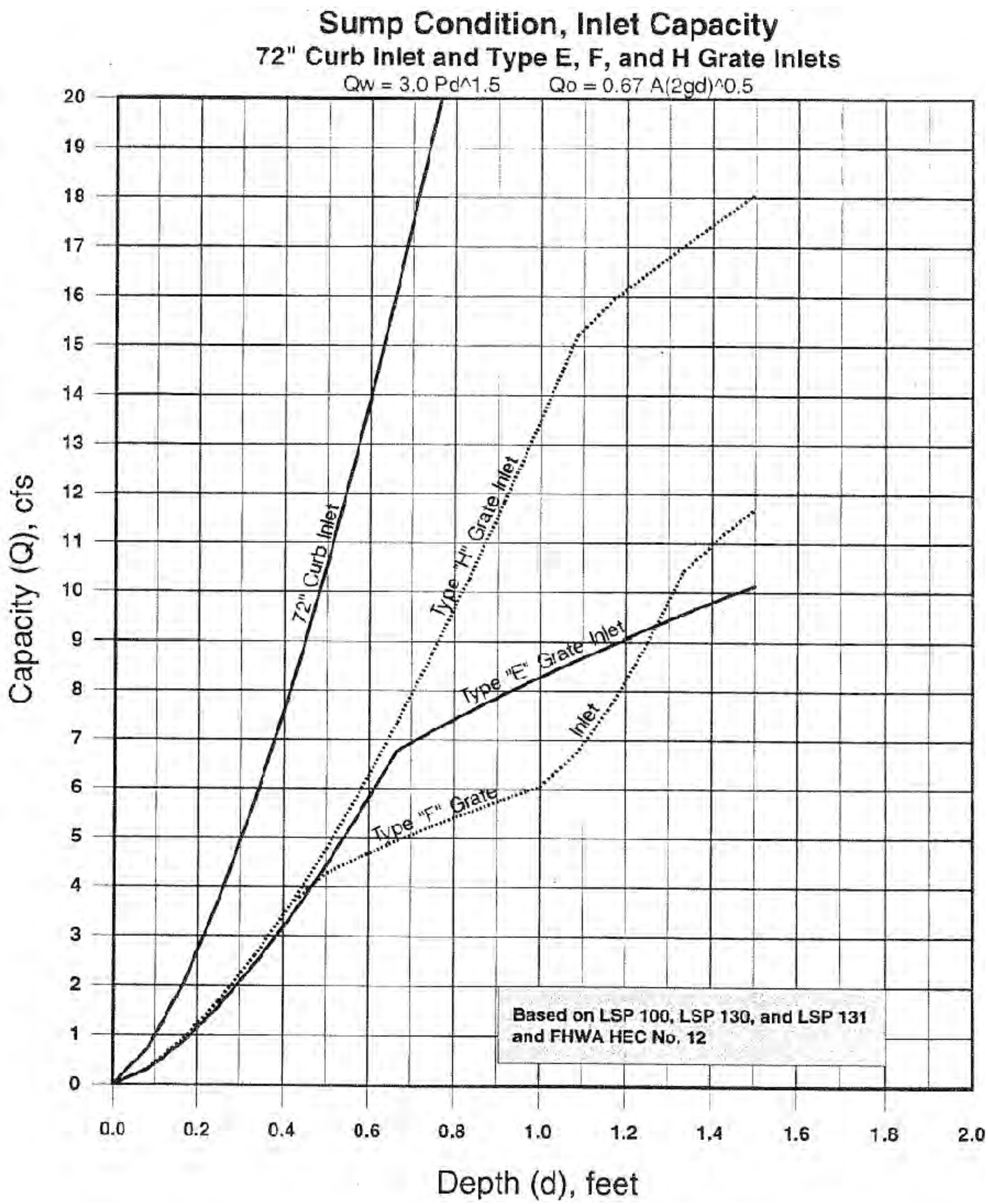


Figure 3-5 Capacity for City of Lincoln Standard Grate Inlets

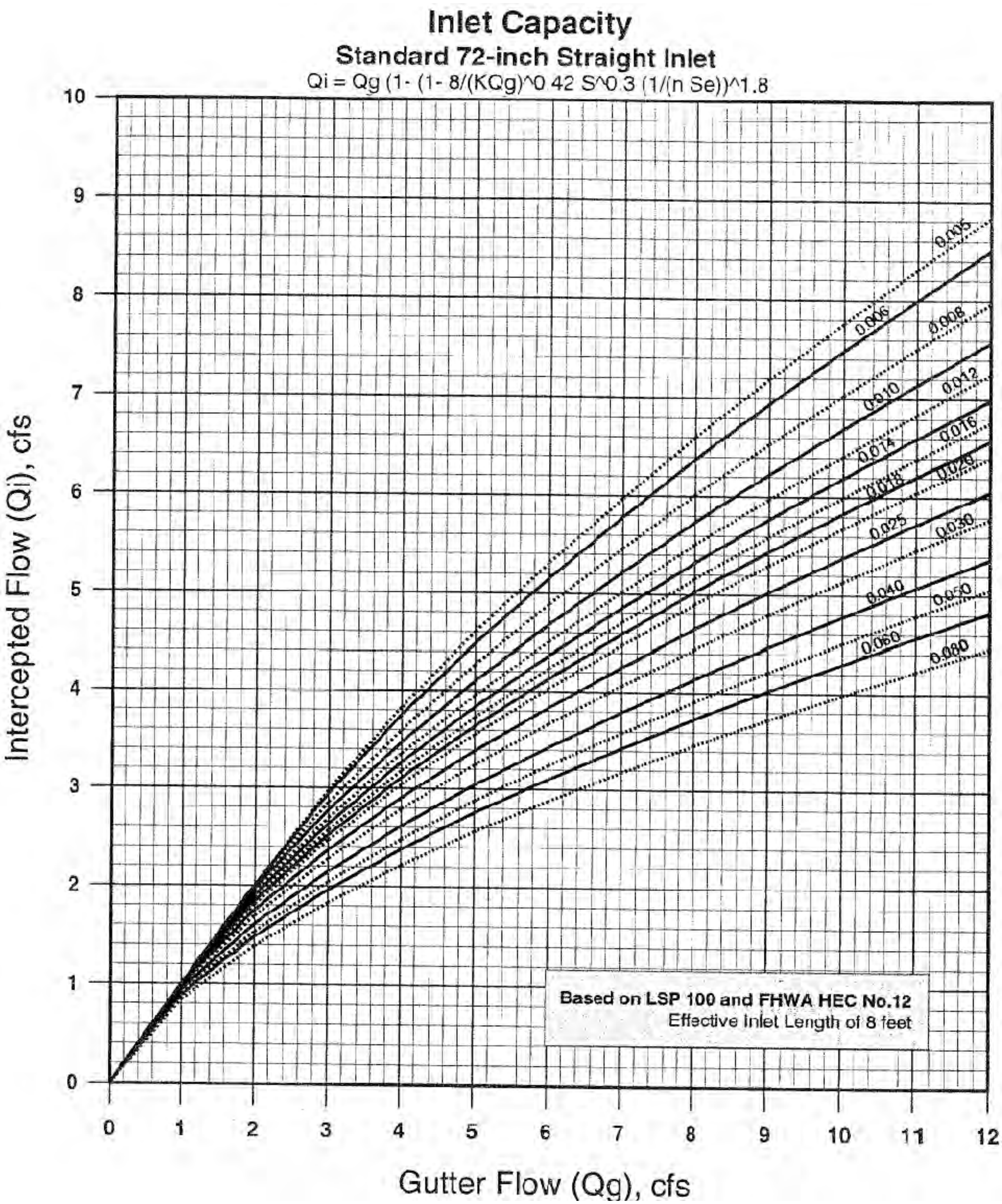


Figure 3-6 Inlet Capacity for City of Lincoln Standard 72-Inch Standard Straight Curb Inlet

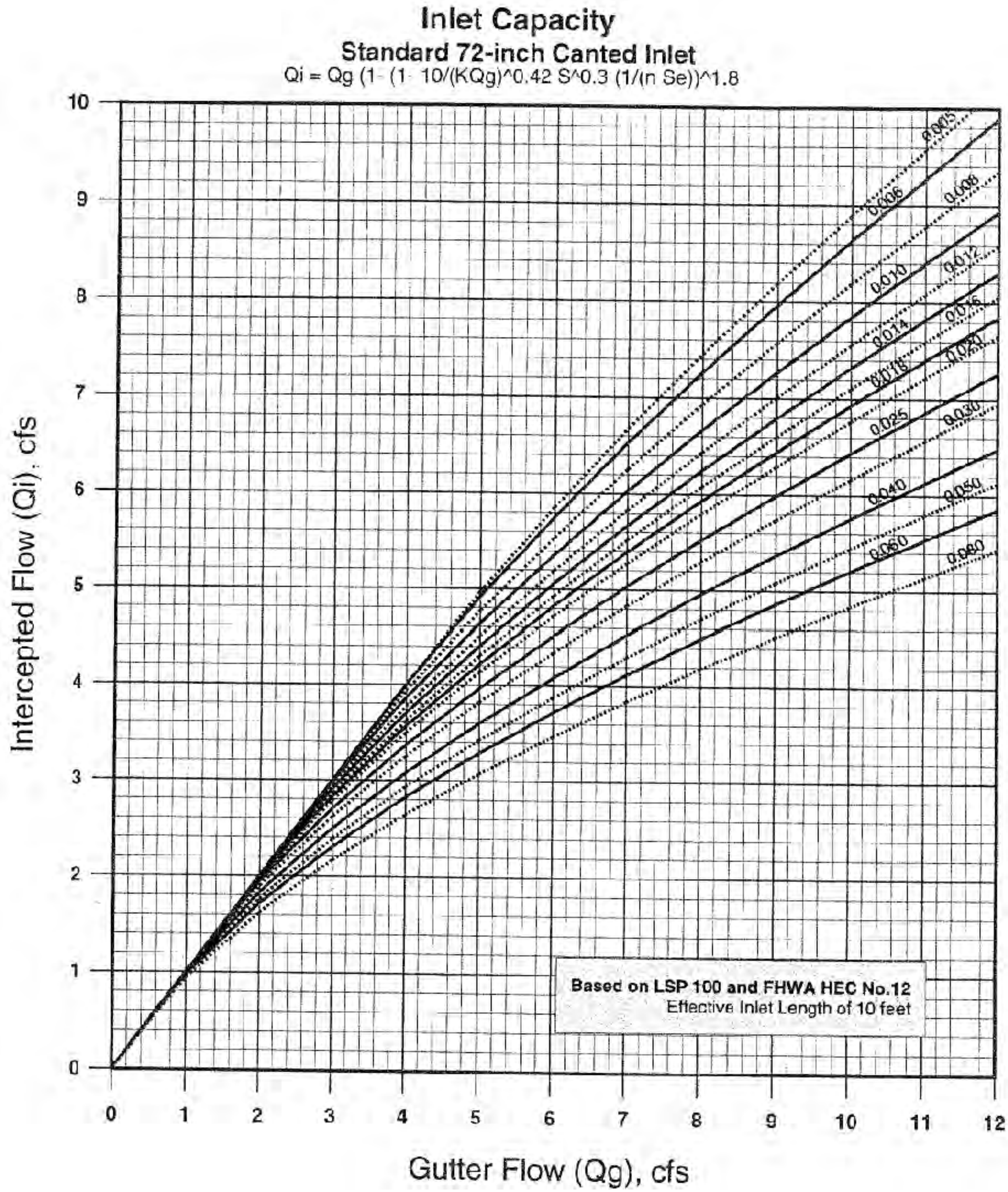


Figure 3-7 Inlet Capacity for City of Lincoln Standard 72-Inch Canted Curb Inlet

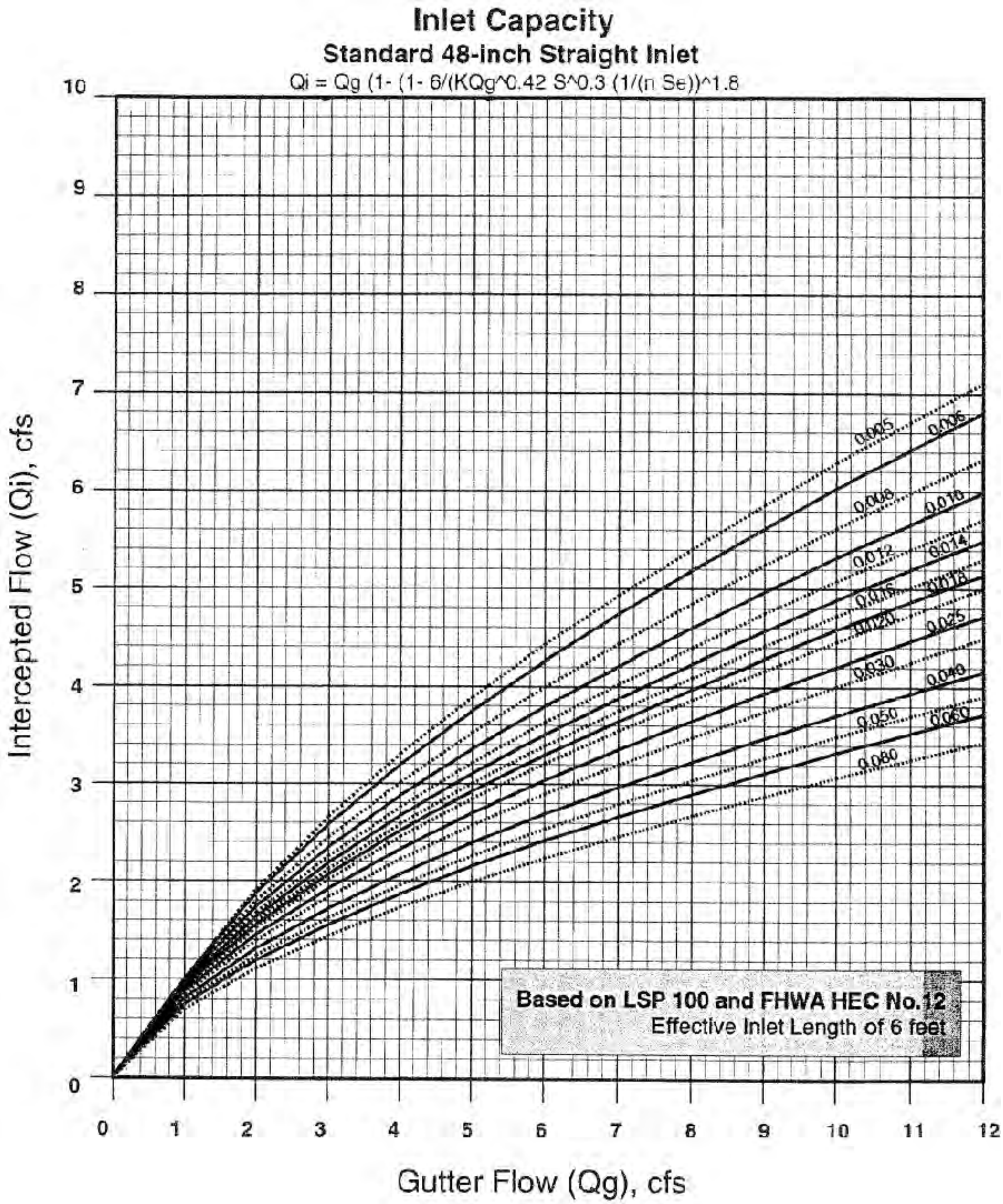


Figure 3-8 Inlet Capacity for City of Lincoln Standard 48-Inch Straight Curb Inlet

Inlet Design

Q = CIA Minor Storm Average Return Frequency ______

Location	Area, A	Runoff Coeficient, C	Time of Concen-tration	Intensity, I	Runoff, Q	By-Pass, Qb	Crossover, Qc	Total, Q	Paving Slope, S	Depth of Curb Flow	Inlet Type	Intercepted, Qi	Bypassed, Qb
	acres	constant	minutes	inch/hour	cfs	cfs	cfs	cfs	ft/ft	inches		c's	cfs

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. The Manning Formula is recommended for capacity calculations.

3.5.2 Design Criteria

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

1. The maximum hydraulic gradient shall not produce a velocity that exceeds 20 feet per second.
2. The minimum desirable physical slope shall be 0.5 percent or the slope which will produce a velocity of 3.0 feet per second when the storm drain is flowing full, whichever is greater.

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 shall be followed when determining the elevation along the hydraulic grade line (HGL):

- The hydraulic grade line shall be 0.75 feet below the intake lip of any affected inlet, any manhole cover, or any entering nonpressurized system.
- The energy grade line shall not rise above the intake lip of any affected inlet, any manhole cover or any entering nonpressurized system.

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

<u>Location and Alignment</u>

In new subdivisions the center of the street is reserved for storm drain system. When construction of a storm drain system is necessary in the older parts of the town, the location is determine by the City. No structures may be placed over a public storm drain system.

<u>Depth of Cover</u>

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

<u>Material and Joints</u>

Only reinforced concrete storm drain pipe shall be used within the City limits, unless approved by the Director of Public Works and Utilities. Construction of pipe and joint shall conform to the City of Lincoln Standard Specification.

<u>Bar Grates on End Sections</u>

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 Director of Public Works and Utilities.

3.5.3 Design Procedures

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

3. The design of the storm drain system is then accomplished by determining drainage areas, computing runoff by rational method, and computing the hydraulic capacity by Manning's equation.
4. The storm drain design computation sheet (Figure 3-12) shall be used to summarize the preliminary system design computations.
5. The hydraulic grade line computation from Figure 3-14 shall be used to determine the hydraulic gradient. 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 3-13. 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:

$$\mathbf{V = [1.486\ R^{2/3}S^{1/2}]/n} \qquad \textbf{(3.6)}$$

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:

$$\mathbf{Q = [1.486\ AR^{2/3}S^{1/2}]n} \qquad \textbf{(3.7)}$$

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

For pipes flowing full, the above equations become:

$$\mathbf{V = [0.590\ D^{2/3}S^{1/2}]/n} \qquad \textbf{(3.8)}$$
$$\mathbf{Q = [0.463\ D^{8/3}S^{1/2}]/n} \qquad \textbf{(3.9)}$$

Where: D = diameter of pipe (ft)

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

$$\mathbf{H_f = [2.87\ n^2V^2L]/[S^{4/3}]} \qquad \textbf{(3.10)}$$

$$H_f = [29\ n^2LV^2]/[(R^{4/3})(2g)] \qquad \textbf{(3.11)}$$

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 Street Right-of-way and Overland Swale

Street right-of-ways convey the portion runoff in excess of pipe capacity, whether planned or not. Street right-of-way capacity is determined using Manning's equation for open channel flow conditions.

$$Q = \frac{1.486}{n} AR^{2/3}S^{1/2} \qquad \textbf{(3.6)}$$

The City of Lincoln uses standard street and right-of-way cross-sections for municipal streets, the formula can be simplified to:

$Q = K\,S^{1/2}$, where conveyance constant, $K = \frac{1.486}{n} AR^{2/3}$

Area, wetted perimeter, and roughness coefficient are constant, the only variable being the street slope.

(i.e., For residential

A = 21.662 square feet,
R = 0.360 feet, and
$n_{(wtd)}$ = 0.026

For overland swales

A = 22.5 square feet,
R = 0.149 feet, and
n = 0.032)

The following table gives the conveyance constants for residential, commercial and major two-lane streets and a 30-foot wide swale with 10:1 side slopes.

Table 3-6 Conveyance Constants for Standard Street Right-of-Ways and 30' Swale

Residential	620
Business with parking	970
Business without parking	790
Major two-lane	1100
30-foot Swale	780

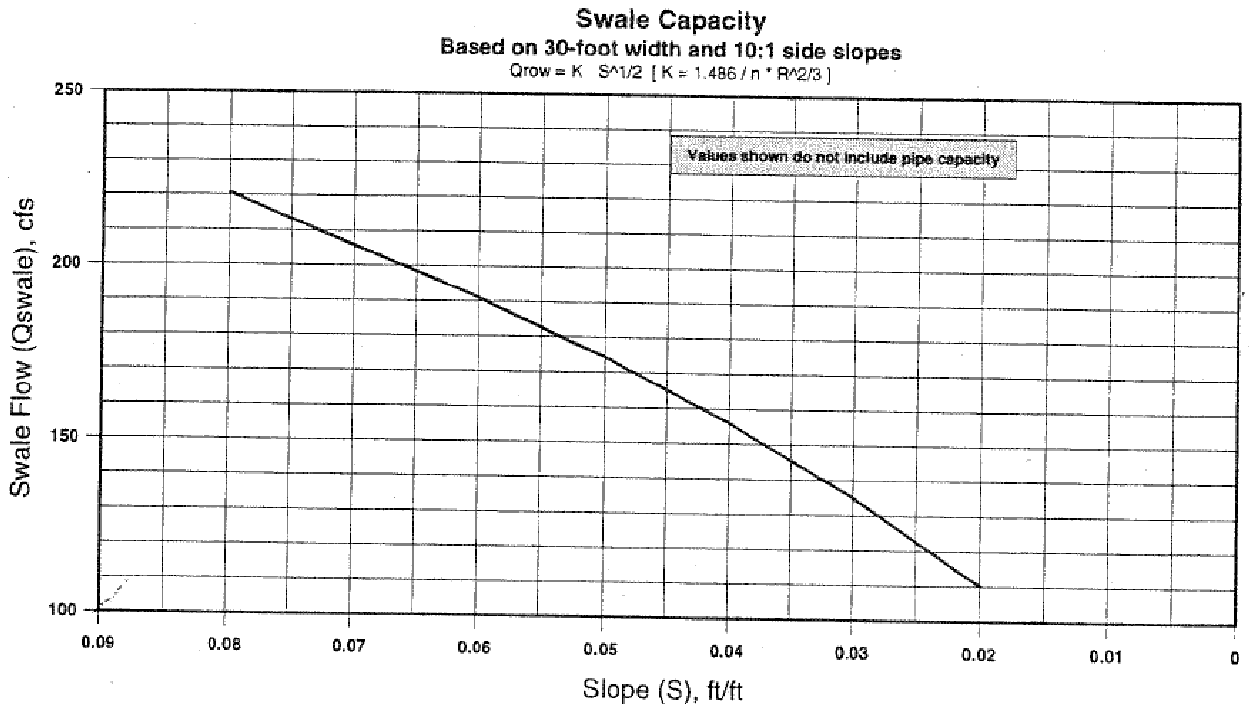


Figure 3-10 Swale Capacity Chart

TABLE III: Circular Concrete and Corrugated Metal Pipe

Pipe Dia. (Inch)	A Area (Square Feet)	R Hydraulic Radius (Feet)	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

Table 3-7 Values of 1.486/n x A x $R^{2/3}$ for Circular Concrete and Corrugated Metal Pipe

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

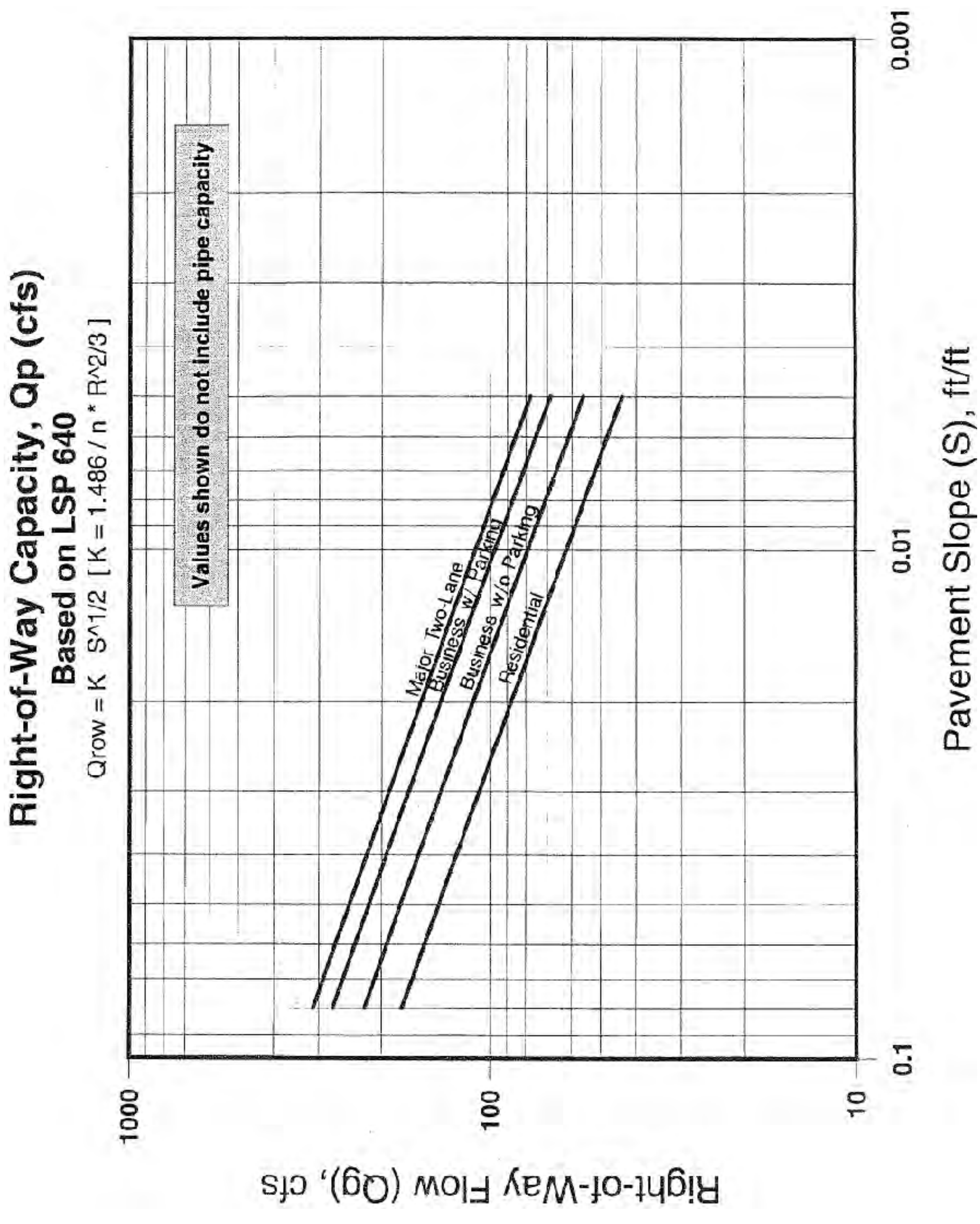


Figure 3-11 Right-of-Way Capacity Chart

Storm Drainage System

Column (1) - Contributing area at the point-of-study
Column (2) - Coefficient of runoff for Rational Method, see Table 2-3 and Table 2-4
Column (3) - Product of area and coefficient of runoff, C x A or Col. (3) = Col. (1) x Col. (2)
Column (4) - Summation of Col. (3) for all contributing drainage basins to the point-of-study
Column (5) - Time of concentration to the point of study for the drainage basin or the accumulated travel time of the aggregate drainage basins, whichever is greater, T_c
Column (6) - Minor storm rainfall intensity, from Figure 2-3
or $I = 42.456\ F^{0.1943} / (T_c+14.0)^{0.7912}$; F = Average Return Frequency
Column (7) - Peak rate of flow for minor storm runoff at the point-of-study, $Q_r = CIA$
or Col. (7) = Col. (4) x Col. (6)
Column (8) - Preliminary pipe slope
Column (9) - Pipe length segment from center to center of structures
Column (10) - Preliminary pipe size required to convey minor storm runoff. Indicate diameter or span x rise
Column (11) - Capacity of pipe for full flow conditions

$$Q = \frac{1.486}{0.013} A R^{2/3} S^{1/2}$$ or Figure 3-13

Column (12) - Velocity in the pipe for full-flow conditions, V = Q/A or Figure 3-13
Column (13) - Time of travel in pipe segment, $T_p = \frac{L}{60V}$ or Col. (13) = Col. (9) / Col. (12) / 60

Column (14) - 100-year storm rainfall intensity, from Figure 2-3
or $I_{100} = 103.882 / (T_c + 14)^{0.7912}$
Column (15) - Peak rate of flow for 100-year storm runoff at the point-of-study, $Q_{100} = CI_{100}A$
or Col. (15) = Col. (4) x Col. (14)
Column (16) - Slope of overland flow route for 100-year storm runoff
Column (17) - Street and right-of-way width
Column (18) - Street capacity for flow to the limits of right-of-way for LSP-640

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

$K = \frac{1.486}{n} A R^{2/3}$, is constant for full depth flow conditions.

K for each standard street and ROW width is provided below (e.g., K 26/60 for a 26' street with a 60-foot ROW)

Residential	$K_{(26/60)} = 620$	Business with parking	$K_{(38/72)} = 970$
Major two lane	$K_{(32/80)} = 1100$	Business without parking	$K_{(33/66)} = 790$
30-foot Swale	$K_{swale} = 780$		

or See Figure 3-10 or Figure 3-11

Column (19) - Combined capacity of the street and minor drainage systems must be equal to or greater than the peak rate of flow for the 100-year storm.
Column (20) - Swale width, where flow from major storms is not contained in the street system an overland flow route must be provided.
Column (21) - Combined capacity of the swale and minor systems must be equal to or greater than the peak rate of flow for the major storm.
Column (22) - Clarifying comments

Preliminary Pipe Sizing Calculations

Minor Storm System Conveyance Analysis

Minor Storm Average Return Frequency, years ________

Major Storm System Conveyance Analysis

Major Storm Average Return Frequency, 100 years

Location	Area, A acres	Coefficient, C	A * C	Sum A * C	Time of Concentration, Tc minutes	Intensity, I in/hour	Runoff, Qr cfs	Pipe Slope, Sp ft/ft	Pipe Length, L feet	Pipe Diameter, D inches	Pipe Capacity, Qp cfs	Pipe Velocity, Vp fps	Time in Section, Tp minutes	Intensity, I 100 in/hour	Flow, Q 100 cfs	Overflow Route Slope, Sov ft/ft	Street Width, Wst feet	Street Capacity, Qst cfs	Swale Width, W feet	Swale Capacity, Qsw cfs	Overflow plus Pipe Capacity, Qov cfs	Comments
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

(3) = (1) x (2) (6) $I_F = 42.456\, F^{0.1943} / (T + 14)^{0.7912}$ (7) $Q_F = C \times I_F \times A$ (11) $Q_P = (1.486 / n) \times A \times R^{2/3} \times S^{1/2}$ (12) $V_P = Q_P / A_P = (11) / A_F$

(13) $T_P = L / 60 / V_P$ (14) $Q_{100} = C \times I_{100} \times A$ (18) $Q_{ST} = K_{ST} \times S_{OV}$ (20) $Q_{SW} = K_{SW} \times S_{OV}$ (21) $Q_{OV} = (11) + (18) + (20)$

Figure 3-12 Storm Drain Computation Form

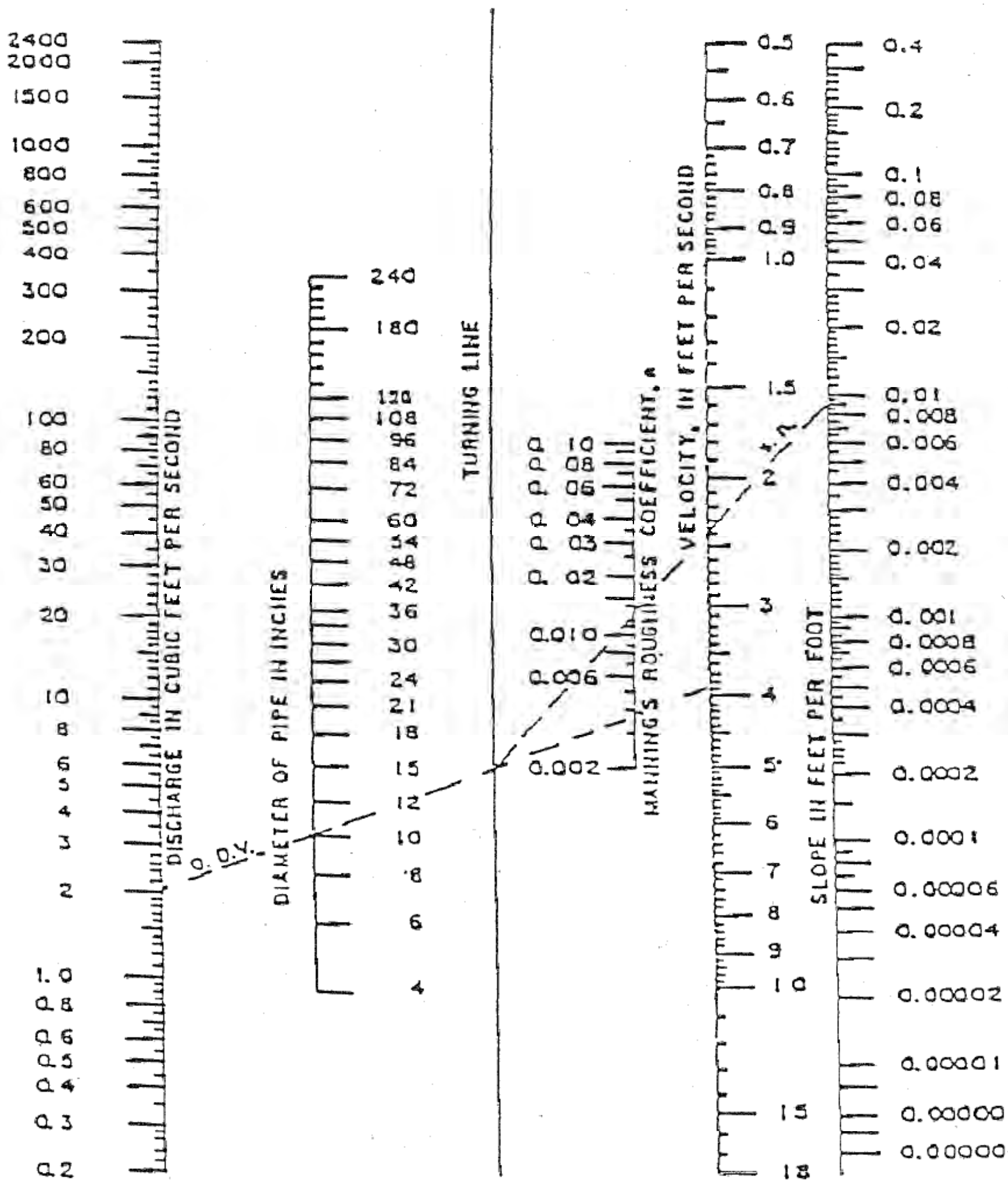


Figure 3-13 Nomograph For Solution of Manning's Formula In Storm Drains

3.5.5 Hydraulic Gradient

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. Computing the hydraulic gradient will determine the elevation to which water will rise in inlets and manholes. The following sections provide the necessary procedures and equations to determine the hydraulic gradient.

3.5.5.1 Friction Losses

Energy losses from pipe friction may be determined by rewriting the Manning equation.

$$\mathbf{S_f = [Qn/1.486\ A(R^{2/3})]^2} \qquad \textbf{(3.12)}$$

Then the head losses due to friction may be determined by the formula:

$$\mathbf{H_f = S_f L} \qquad \textbf{(3.13)}$$

Where: H_f = friction head loss (ft)
S_f = friction slope (ft/ft)
L = length of outflow pipe (ft)

3.5.5.2 Velocity Head Losses

From the time storm water first enters the sewer system at the inlet until it discharges at the outlet, it will encounter a variety of hydraulic structures such as inlets, manholes, junctions, bends, contractions, enlargements and transitions, which will cause velocity head losses. Velocity losses may be expressed in a general form derived from the Bernoulli and Darcy-Weisback equations.

$$\mathbf{H = KV^2/2g} \qquad \textbf{(3.14)}$$

Where: H = velocity head loss (ft)
K = loss coefficient for the particular structure
V = velocity of flow (ft/s)
g = acceleration due to gravity (32.2 ft/s)

3.5.5.3 Entrance Losses

Following are the equations used for entrance losses.

$$\mathbf{H_{tm} = V^2/2g} \qquad \textbf{(3.15)}$$
$$\mathbf{H_e = KV^2/2g} \qquad \textbf{(3.16)}$$

Where: H_{tm} = terminal (beginning of run) loss (ft)
H_e = entrance loss for outlet structure (ft)
K = 0.5 (assuming square-edge)
(Other terms defined above.)

3.5.5.4 Junction Losses

Incoming Opposing Flows

The head loss at a junction, H_{j1} for two almost equal and opposing flows meeting head on with the outlet direction perpendicular to both incoming directions, head loss is considered as the total velocity head of outgoing flow.

$$\mathbf{H_{j1} = (V^2)/2g} \qquad \mathbf{(3.17)}$$

Where: H_{j1} = junction losses (ft)
(Other terms are defined above.)

Changes in Direction of Flow

When main storm drain pipes or lateral lines meet in a junction, velocity is reduced within the chamber and specific head increases to develop the velocity needed in the outlet pipe. The sharper the bend (approaching 90º) the more severe this energy loss becomes. When the outlet conduit is sized, determine the velocity and compute head loss in the chamber by the formula:

$$\mathbf{H_b = K_b(V^2)/2g} \qquad \mathbf{(3.18)}$$

Where: H_b = bend head loss (ft)
K_b = junction loss coefficient

The following Table 3-8 lists the values of K_b for various changes in flow direction and junction angles.

Table 3-8 Values Of K_b For Change In Direction Of Flow In Lateral

K	Degree of Turn (In Junction)
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90 and greater

K values for other degree of turns can be obtained by interpolating between values.

Table 3-9 lists the values for the junction loss coefficient for various conditions at pipe junctions.

Table 3-9 Values Of K At Junctions

For no bends at junctions -	K = 0.20
For bends at junctions of 25 degrees -	K = 0.30
For bends at junctions of 45 degrees -	K = 0.40
For bends at junctions of 90 degrees -	K = 0.60
For junctions of three pipes -	K = 0.80
For junctions of four or more pipes -	K = 1.00

Several Entering Flows

The computation of losses in a junction with several entering flows utilizes the principle of conservation of energy. For a junction with several entering flows, the energy content of the inflows is equal to the energy content of outflows plus additional energy required by the collision and turbulence of flows passing through the junction. The total junction losses can be determined from equation 3-17. See also Figure 3-14.

$$\mathbf{H_{j2} = [(Q_4V_4^2)-(Q_1V_1^2)-(Q_2V_2^2)+(KQ_1V_1^2)]/(2gQ_4)] \qquad (3.19)}$$

Where: H_{j2} = junction losses (ft)
Q = discharges (cfs)
V = horizontal velocities (ft/s) (V_3 is assumed to be zero)
g = acceleration due to gravity (32.2 ft/s^2)
K = bend loss factor

Where subscript nomenclature is as follows:

Q_1 = 90º lateral (cfs)
Q_2 = straight through inflow (cfs)
Q_3 = vertical dropped-in flow from an inlet (cfs)
Q_4 = main outfall = total computed discharge (cfs)
V_1,V_2,V_3,V_4 are the horizontal velocities of foregoing flows, respectively, in feet per second
V_3 assumed to be = 0

Also Assume:

- $H_b = K(V_1^2)/2g$ for change in direction.
- No velocity head of an incoming line is greater than the velocity head of the outgoing line.
- Water surface of inflow and outflow pipes in junction to be level.

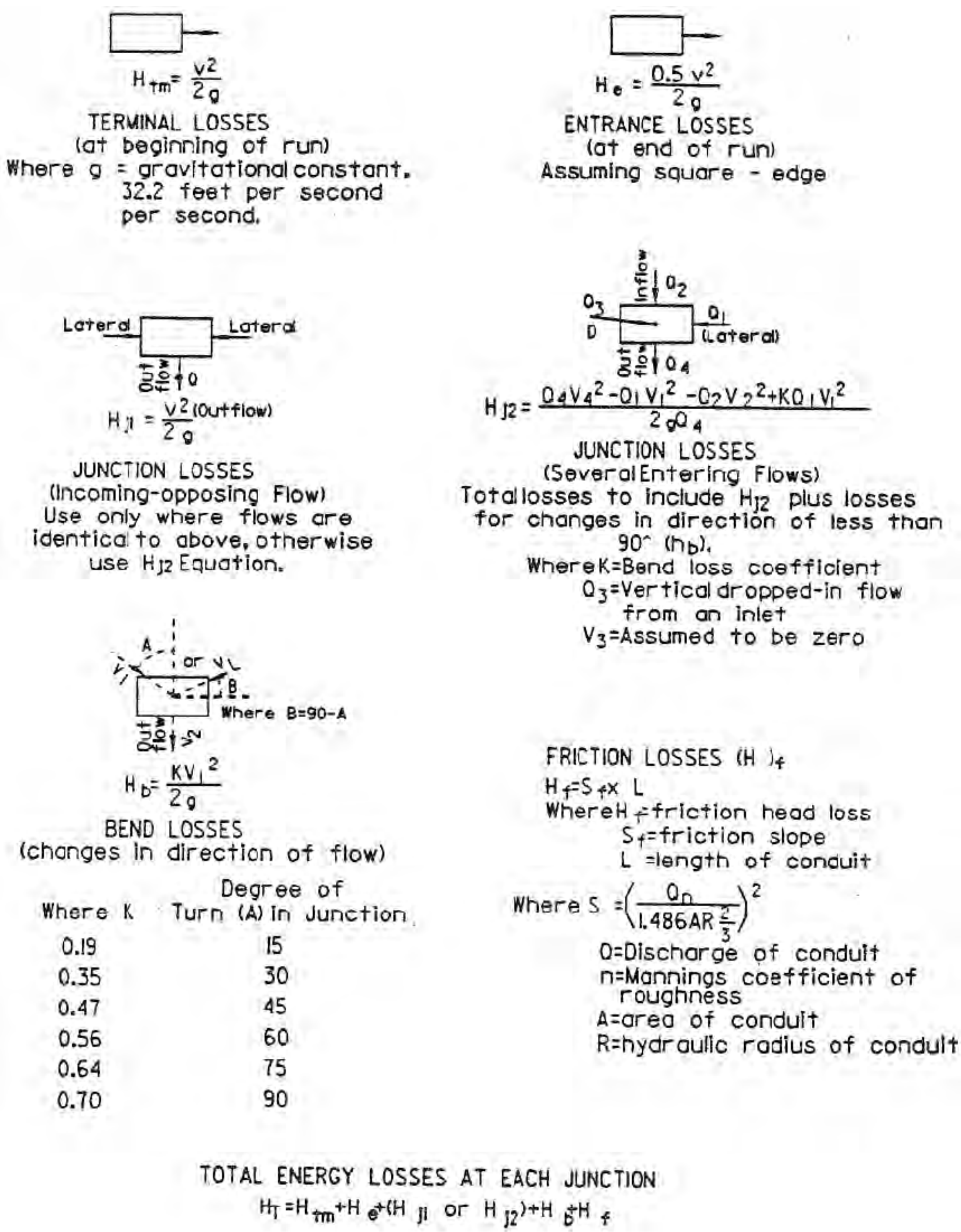
When losses are computed for any junction condition for the same or a lesser number of inflows, the above equation will be used with zero quantities for those conditions not present. If more directions or quantities are at the junction, additional terms will be inserted with consideration given to the relative magnitudes of flow and the coefficient of velocity head for directions other than straight through.

3.5.5.5 Summary

The final step in designing a storm drain system is to check the hydraulic grade line (HGL) as described in the next section of this chapter. Computing the HGL will determine the elevation, under design conditions, to which water will rise in various inlets, manholes, junctions, and etc. The following design criteria shall be followed when determining the elevation at the HGL:

- The hydraulic grade line shall be 0.75 feet below the intake lip of any affected inlet, any manhole cover, or any entering nonpressurized system.
- The energy grade line shall not rise above the intake lip of any affected inlet, any manhole cover or any entering nonpressurized system.

A summary of energy losses which shall be considered is presented in Figure 3-14.



Where K	Degree of Turn (A) in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

Figure 3-14 Summary Of Energy Losses

Source: AASHTO Model Drainage Manual, 1991

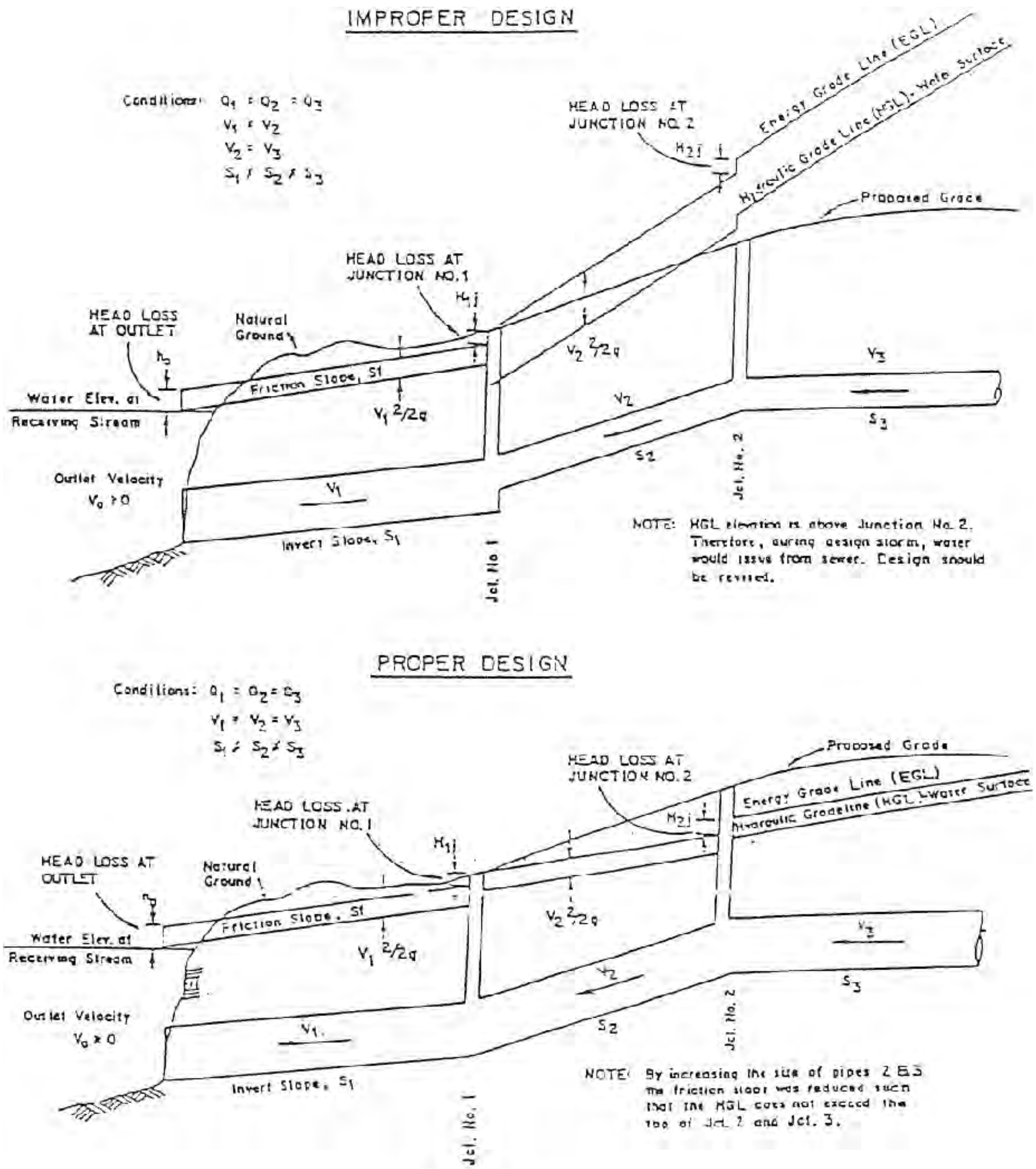


Figure 3-15 Energy And Hydraulic Grade Lines For Storm drain Under Constant Discharge

Source: AASHTO Model Drainage Manual, 1991

3.5.6 Hydraulic Grade Line Design Procedure

The hydraulic grade line is calculated beginning at the system outlet proceeding upstream. Conditions expected at the outlet for the minor design storm shall be used for the starting water surface elevation.

Column (1) - Design flow to be conveyed by pipe segment.
Column (2) - Length of pipe segment.
Column (3) - Pipe Size; Indicate pipe diameter or span x rise.
Column (4) - Constant K, from American Concrete Pipe Association Design Data:

$$K_p = \frac{1{,}486}{n} AR^{2/3} \text{ ; or from Table 3-7}$$

Column (5) - Flowline Outlet Elevation of pipe segment.
Column (6) - Flowline Inlet Elevation of pipe segment.
Column (7) - Barrel Area is the full cross sectional area of the pipe.
Column (8) - Barrel Velocity is the full velocity in the pipe as determined by:

$V = Q/A$ or Col. (8) = Col. (1) / Col. (7)

Column (9) - Barrel Velocity Head = $V^2/2g$ or Col. $(8)^2 / 2g$
Where, $g = 32.2$ ft/sec^2 (acceleration due to gravity)

Column (10) - Tailwater (TW) Elevation; this is the water surface elevation at the outlet of the pipe segment. If the pipe's outlet is not submerged by the TW and the TW depth is less than $(D+d_c)/2$, set the TW elevation equal to $(D+d_c)/2$. This will keep the analysis simple yet still obtain reasonable results (D = pipe barrel height and d_c = critical depth, both in ft. See Appendix 4-B for determination of d_c).

Column (11) - Friction Loss = S_1 x L or S_1 x Col. (2)
Where, S_1 is the friction slope or head loss per lineal foot of pipe as determined by Manning's Equation expressed in the form:

$S_1 = S_f = (Q / K_p)^2$; K from Table 3-7

Column (12) - Hydraulic Grade Line (HGL) Elevation just inside the entrance of the pipe barrel; this is determined by adding the friction loss to the TW elevation:

Col. (12) = Col. (11) + Col. (10)

If this elevation falls below the pipe's inlet crown, it no longer represents the true HGL when computed in this manner. The true HGL will fall somewhere between the pipe's crown and either normal flow depth or critical flow depth, whichever is greater. To keep the analysis simple and still obtain reasonable results (i.e., erring on the conservative side), set the HGL elevation equal to the crown elevation.

Column (13) - Entrance Head Loss = K_e x $V^2 / 2g$ or K_e x Col. (9) Where, K_e = Entrance Loss Coefficient (0.5 assuming square-edge) This is the head lost due to flow contractions at the pipe entrance.

Column (14) - Exit Head Loss = 1.0 x $V^2 / 2g$ or 1.0 x Col. (9)
This is the velocity head lost or transferred downstream.

Column (15) - Outlet Control Elevation = Col. (12) + Col. (13) + Col. (14)
This is the maximum headwater elevation assuming the pipe's barrel and inlet/outlet characteristics are controlling capacity. It does not include structure losses or approach velocity considerations.

Column (16) - Inlet Control Elevation (See Figure 4-2 for computation of inlet control on culverts). This is the maximum headwater elevation assuming the pipe's inlet is controlling capacity. It does not include structure losses or approach velocity considerations.

Column (17) - Approach Velocity Head; this is the head (energy) being supplied by the discharge form an upstream pipe or channel section, which serves to reduce the headwater elevation. If the discharge is from a pipe, the approach velocity head is equal to the barrel velocity head computed for the upstream pipe. If the upstream pipe outlet is significantly higher in elevation (as in a drop manhole) or lower in elevation such that its discharge energy would be dissipated, an approach velocity head of zero should be assumed.

Column (18) - Bend Head Loss = K_b x V^2 / 2g or k_b x Col. (17)
Where, K_b = Bend Loss Coefficient (from Table 3-7). This is the loss of head/energy required to change direction of flow in an access structure.

Column (19) - Junction Head Loss; this is the loss in head (energy) that results from the turbulence created when two or more streams are merged into one within the access structure. Table 3-8 can be used to determine junction loss coefficients for use in the following equations given in Figure 3-14.

Column (20) - Headwater (HW) Elevation; this is determined by combining the energy heads in Columns 17, 18, and 19 with the highest control elevation in either Column 15 or 16, as follows:

Col. (20) = Col. (15 or 16) - Col. (17) + Col. (18) + Col. (19)

Column (21) - Top of curb elevation at an inlet or rim elevation at a storm sewer manhole.

Column (22) - Inlet capacity is reduced if the hydraulic gradeline elevation interferes with the napping effect during weir or orifice flow conditions.

Hydraulic Grade Line Calculation Sheet

Minor Storm Ave. Return Frequency ____

Pipe Segment from	to	Q (cfs)	Length (feet)	Pipe Size (in)	Ko	Outlet Elevation (feet)	Inlet Elevation (feet)	Barrel Area (sqft)	Barrel Velocity (fps)	Barrel Velocity Head (feet)	Tail Water Elevation (feet)	Friction Loss (feet)	Entrance HGL Elevation (feet)	Entrance Head Loss (feet)	Exit Head Loss (feet)	Outlet Control Elevation (feet)	Inlet Control Elevation (feet)	Appr. Vel. Head (feet)	Bend Head Loss (feet)	Junc. Head Loss (feet)	HW Elevation (feet)	Top Curb Elevation (feet)	Elevation Difference (ft)	Comment
from	to	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)

(8) = Q/A = (1)/(7) (9) = $h_v = V^2/2g = (8)/64.4$ (11) = $h_f = SL$; $S = Q/K$; K from Table 3-7 (12) 10 + 11

(13) = $K_e(V^2/2g) = K_e(9)$ (14) = $1.0(V^2/2g) = 1.0(9)$ (15) 12 + 13 + 14 (16) Figure 4-2 (17) = (9)

(18) = $K_b(V^2/2g) = K_b(17)$ (19) = $K_j(V^2/2g) = K_j(17)$ (20) = (15 or 16) − (17) + (18) + (19) (22) Reduces Inlet Capacity if < 1.5 feet

Figure 3-16 Hydraulic Grade Line Computation Form

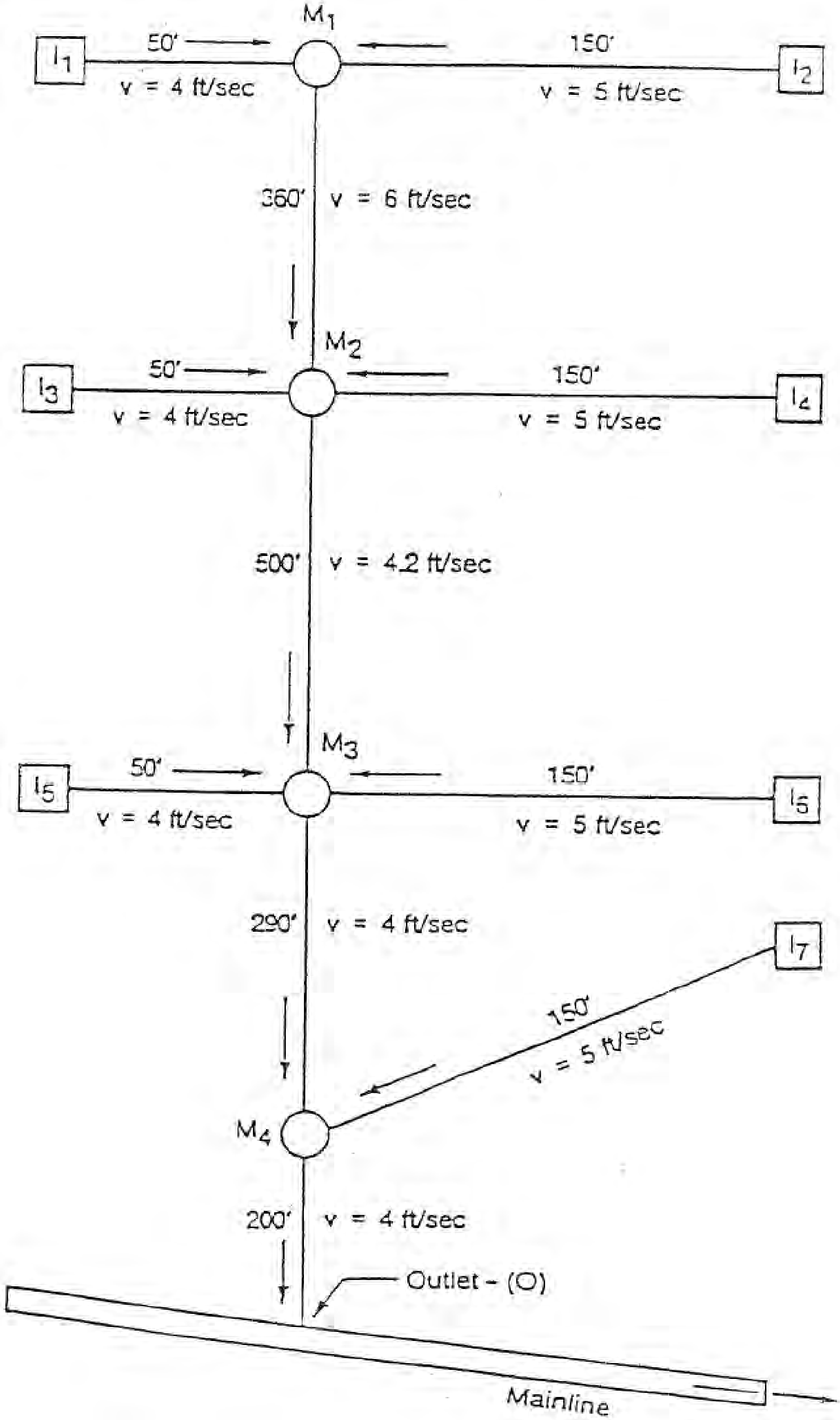


Figure 3-17 Hypothetical Storm Drain System Layout

3.6 Computer Programs

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.

References

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

American Concrete Pipe Association, March 1968, Design Data.