

# **STORM DRAINAGE DESIGN MANUAL**

**City of Farmers Branch, Texas**

**SEPTEMBER 2019**



**FARMERS  
BRANCH**

City of Farmers Branch, Texas  
Public Works Department

# STORM DRAINAGE DESIGN MANUAL

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## **1.0 INTRODUCTION**

### **1.1 GENERAL**

The purpose of this “Storm Drainage Design Manual” is to provide guidelines for designing drainage facilities in the City of Farmers Branch. This manual is for use by the Public Works Department, other City departments, consulting engineers employed by the City, and engineers for private development in the City. Unusual circumstances or special designs requiring variance from standards within this manual may be approved by the City Engineering Division. All requests for variance must be submitted in writing to the City Engineering Division.

### **1.2 SCOPE**

The information included in this manual has been developed through a comprehensive review of basic design technology as published in various sources listed in the Bibliography, and as developed through the experience of individual Engineers who have contributed to the content.

The manual concerns itself with storm drainage conditions which are generally relative to the City of Farmers Branch and the immediate geographical area. Accepted engineering principles are applied to these situations in detailed documented procedures. The documentation of the procedures is not intended to limit initiative, but rather, is included as a standardized procedure to aid in design and provide a record source for the City.

### **1.3 ORGANIZATION OF MANUAL**

This manual is divided into six basic sections:

Section 1.0 is the INTRODUCTION, which is a general discussion of the intended use of the material, and an explanation of its organization.

Section 2.0, DRAINAGE DESIGN THEORY, explains the basic technical theory employed by the design procedures prescribed in this manual.

Section 3.0, CRITERIA AND DESIGN PROCEDURES, lists recommended design criteria, and outlines the design procedures followed by the City of Farmers Branch.

Section 4.0, CONSTRUCTION PLAN PREPARATION, describes construction plans for drainage facilities in the City of Farmers Branch.

Section 5.0, APPENDIX, contains definitions of terms and symbols, abbreviations, and the Bibliography.

Section 6.0, TABLES, contains the tables which are used in the design of drainage facilities.

Section 7.0, FIGURES, contains the basic graphs, nomographs and charts for use in design of drainage facilities.

Section 8.0, FORMS, contains forms with detailed instructions for their use.

## 2.0 DRAINAGE DESIGN THEORY

### 2.1 GENERAL

This section covers the technical theory utilized in the design procedures outlined in the manual. It is intended as an application of basic hydraulic and hydrologic theory to specific storm drainage situation.

### 2.2 HYDROLOGY ITEMS

#### 2.2.1 Drainage Area Determination and System Designation

The size and shape of each drainage area and sub-area must be determined for each storm drainage facility. This determination should be based on recent aerial imagery and topographic maps, utilizing at maximum two-foot (2') contour intervals. When two-foot (2') contour intervals are insufficient, or physical conditions may have changed from those shown on existing maps, it may be necessary to supplement the maps with field topographic surveys. The actual conditions should always be verified by a reconnaissance survey.

In preparing the drainage area maps, careful attention must be given to the gutter configurations at intersections. The direction of flow in the gutters should be shown on the maps and on the construction plans. The performance of these surveys is the responsibility of the Engineer designing the drainage facility.

#### 2.2.2 Rainfall

**Table 1**, which shows anticipated rainfall rates for storm durations from 5 minutes to 24 hours, has been prepared utilizing the National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume 11 Precipitation-Frequency Atlas of the United States, Texas (2018). The City has selected a single coordinate to define standard rainfall intensities for the entire City.

#### 2.2.3 Storm Frequency

The storm return periods in **Table 1** are referred to as "Design Storm Frequency". The term "100-year storm" means that a storm of that severity has a one in one hundred chance of occurring in any given calendar year. It does not mean that a storm of that severity can be expected once in any 100-year period.

Each storm drainage system shall be designed to convey the runoff which results from the 100-year design storm, as shown in Section 3.0, CRITERIA AND DESIGN PROCEDURES.

#### 2.2.4 Design Discharge

Prior to hydraulic design of drainage facilities, the amount of runoff from the particular drainage area must be determined. The Rational, the Unit Hydrograph, and the HEC-HMS Computer Program are the accepted methods for computing volumes of stormwater runoff. Data from an appropriate flood study shall be used in lieu of Rational Method, Unit Hydrograph, or HEC-HMS for determination of drainage and floodway easement elevations and design discharge flows, if such data is available. However, all discharge values shall be based on full development of the drainage basin as outlined on the current Zoning Maps and Comprehensive Plan available from the City of Farmers Branch.

#### 2.2.5 Methods for Determining Design Discharge

##### **Rational Method**

The use of the Rational Method, introduced in 1889, is based on the following assumptions:

- A. The peak rate of runoff at any point is a direct function of the average rainfall intensity during the time of concentration to that point;
- B. The frequency of the peak discharge is the same as the frequency of the average rainfall intensity; and
- C. The time of concentration is the time required for the runoff to become established and flow from the most remote part of the drainage area to the design point.

The Rational Method is based on the direct relationship between rainfall and runoff expressed in the following equation:

$$Q = C I A, \quad \text{where}$$

...“Q” is the storm flow at a given point in cubic feet per second (cfs);

... “C” is a coefficient of runoff representing the ratio of runoff to rainfall;

...“I” is the average intensity of rainfall in inches per hour, for a period equal to the time of flow from the farthest point of the drainage area to the point of design, and is obtained from **Table 1**;

...“A” is the area in acres that is tributary to the point of design.

The determination of the factors, runoff coefficient and time of concentration shown in this manual have been developed through past experience in the City’s system, and by review of values recommended by others. Maximum area for Rational Method shall be as outlined in Section 3.4.

### Runoff Coefficient

The runoff coefficient “C” in the Rational Method equation is dependent on the character of the soil, and the degree and type of development in the drainage area. The nature and condition of the soil determine its ability to absorb precipitation. The absorption ability generally decreases as the duration of the rainfall increases until saturation occurs. Infiltration rates in the Farmers Branch area generally are low due to the cohesive soils.

Normally, as the drainage area develops, the amount of runoff increases in proportion to the amount of impervious areas. Examples of impervious areas are streets, parking areas and buildings. **Table 2** lists the accepted runoff coefficients for different land uses.

### Time of Concentration

The time of concentration is defined as the longest time, without interruption of flow by detention devices, that will be required for water to flow from the upper limit of a drainage area to the point of concentration. This time is a combination of the inlet time, which is the time for water to flow over the surface of the ground from the upper limit of the drainage area to the first storm sewer inlet, and the flow time in the conduit or channel to the point of concentration. The flow time in the conduit or channel is computed by dividing the length of the conduit by the average velocity in the conduit.

Although the basic principles of the Rational Method are applicable to all sizes of drainage areas, natural retention of flow and other interruptions cause an attenuation of the runoff hydrograph, resulting in over-estimation of flow rates for larger areas. For this reason, in development of runoff rates in drainage areas over 100 acres, use of the Unit Hydrograph Method is recommended, but HEC-HMS is acceptable, as outlined in Section 3.4.

### Unit Hydrograph Method

The Unit Hydrograph Method to be used in calculation of runoff shall be in accordance with Snyder’s synthetic relationships.

The computation of runoff quantities utilizing the Unit Hydrograph Method is based on the following equations:

$$T_p = C_t (L L_{ca})^{0.3}$$
$$q_p = \frac{C_p^{640}}{t_p}$$

$$Q_p = q_p A$$

$$S_D = I \times 2$$

$$R_T = S_D - L_{is}$$

$$Q_u = R_T Q_p$$

...“ $t_p$ ” is the lag time, in hours, from the midpoint of the unit rainfall duration to the peak of the unit hydrograph;

...“ $C_t$ ” and “ $C_{p640}$ ” are coefficients related to drainage basin characteristics. Recommended values for these coefficients are found in **Table 3**;

...“ $L$ ” is the measured stream distance in miles from the point of design to the upper limit of the drainage area;

...“ $L_{ca}$ ” is the measured stream distance, in miles, from the point of design to the centroid of the drainage area.

...“ $q_p$ ” is the peak rate of discharge of the unit hydrograph for unit rainfall duration in cubic feet per second per square mile;

...“ $Q_p$ ” is the peak rate of discharge of the unit hydrograph in cubic feet per second;

...“ $A$ ” is the area in square miles that is tributary to the point of design;

...“ $I$ ” is the rainfall intensity at two hours, in inches per hour, for the appropriate design storm frequency;

...“ $S_D$ ” is the design storm rainfall in inches for a two-hour period;

...“ $L_{is}$ ” is the initial and subsequent losses which have a recommended constant value of 1.11 inches;

...“ $R_T$ ” is the total runoff in inches;

...“ $Q_u$ ” is the design storm runoff in cubic feet per second

### Unit Hydrograph Coefficients

In August 1952, the U.S. Army Corps of Engineers published a report which contains observed unit hydrographs from records on several storms which occurred during the period from May 1948 through May 1950, on the Turtle Creek drainage basin. Data developed in that report, which is entitled “Definite Project Report on Dallas Floodway, Volume I – General, Hydrologic and Economic Data”, together with additional measurements made since that time, was used to establish the coefficients for the Farmers Branch area.

In Section 3.0 of the manual, certain values for factors involved in a unit hydrograph analysis are recommended. These values are not to be considered inflexible but are intended as guidelines when more specific data is not available. Detailed review of the development of all these factors is not warranted, but several factors should be discussed where the documentation for the selected values may not be apparent.

The recommended rainfall intensity to be used is selected based on a duration of two hours. The two hours are representative of the time elapsed from the beginning of the rainfall to the peak rate of runoff. Where more definite relationships are known to exist on any particular stream, this time should be adjusted accordingly. When using a duration of two hours, multiply the rainfall rate (intensity) by two hours, subtract the losses, and the total runoff is obtained.

There are two losses to be considered when determining the total runoff. These are termed the “initial” and “subsequent” losses, and are shown in Section 3.0, Criteria and Design Procedures, as having a constant value of 1.11 inches. This is arrived at by assigning a value of 0.75 inches as the total initial loss occurring during the first one-half hour of rainfall, and a loss of 0.24-inch per hour for the remaining one and one-half hour rainfall period, calculated as follows:

Initial Loss	0.75 inch
Subsequent Loss (1.5 hrs x 0.24 inch/hr)	0.36 inch
Total Losses	1.11 inches

As in the case of other recommended specific values, where more definite information is available, it should be used.

## 2.3 HYDRAULIC THEORY FOR DRAINAGE RELATED STRUCTURES

### 2.3.1 Paved Areas

#### **Flow in Gutters – Definition of Terms**

In the design of storm drainage facilities, the geometrics of specific types of streets are an integral part of drainage design. Throughout this manual, reference is made to certain types and widths of streets with specific crown characteristics. The following terms are defined for reference purposes:



*Major Thoroughfare:* A street that moves traffic from one section of the city to another section.

*Collector Street:* A street that has the dual purpose of traffic movement plus providing access to abutting properties.

*Residential Street:* A street whose primary function is to provide local access to abutting properties.

*Width of Street:* The horizontal distance between the faces of the curbs.

*Straight Crown:* A constant slope from one gutter flow line across a street to the other gutter flow line. Most generally found on divided thoroughfares.

*Parabolic Crown:* A pavement surface shaped in a parabola from one gutter flow line to the other. Most generally found on undivided secondary thoroughfares, collector streets, and residential streets.

*Vertical Displacement between Gutter Flow Lines:* Due to topography, it may be necessary at times for the curbs on a street to be placed at different elevations. This will be done only in exceptional cases, and only with the prior approval of the City Engineering Division.

### **Straight Crown Streets**

Stormwater flow in a street having a straight crown slope may be expressed as follows:

$$Q = 0.56 \frac{Z}{n} S^{1/2} y^{8/3} \quad (\text{Equation 1})$$

...“Q” is quantity of gutter flow in cubic feet per second;

...“Z” is the reciprocal of the crown slope;

...“n” is the coefficient of roughness as used in Manning’s Equation; a value of 0.0175 was used;

...“S” is the longitudinal slope of the street gutter in feet per foot;

...“y” is the depth of flow in the gutter at the curb in feet

This formula is an expression of Manning’s Equation, as referenced in Highway Research Board Proceedings, 1946, Page 150, Equation 14.

Based on this equation, **Figure 2** was prepared, and inlet design calculations, as explained elsewhere in this manual, were made.

### **Parabolic Crown Streets**

**Figure 3** shows the capacity of gutters in streets with parabolic crowns. The following formulas can be used for determining the gutter capacity, or refer to the figure for solution.

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \quad (\text{Equation 2})$$

$$R = \frac{A}{P} \quad (\text{Equation 3})$$

$$A = \frac{w_o c_o}{2} - \frac{8c_o}{w_o^2} \int_0^{w_o/2} x^2 dx \quad (\text{Equation 4})$$

- ...“Q” is quantity of gutter flow in cubic feet per second;
- ...“n” is the coefficient of roughness; a value of 0.0175 was used;
- ...“A” is the cross section flow area in square feet;
- ...“R” is the hydraulic radius in feet;
- ...“S” is the longitudinal slope of the street gutter in feet per foot
- ...“P” is the wetted perimeter in feet;
- ...“w<sub>o</sub>” is the width of the street in feet;
- ...“c<sub>o</sub>” is the crown height of the street in feet.

As discussed in Section 3.0, Criteria and Design Procedures, it may, at times, be necessary for one curb to be at a different elevation than the opposite curb due to the topography. Where parabolic crowns are involved, the gutter capacities will vary radically as one curb becomes higher or lower. The maximum vertical displacement values shown in **Figure 3** were developed based on a minimum depth of flow of approximately two inches, in the high gutter. Vertical displacement is rarely allowed and is discussed in Section 2.3.1.

### **Alley Capacity**

Alley capacity shall be calculated based on solution of Manning’s Equation:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \quad (\text{Equation 2})$$

- ...“Q” is the alley capacity, flowing full, in cubic feet per second;
- ...“n” is the coefficient of roughness; a value of 0.0175 was used;
- ...“A” is the cross section flow area in square feet;
- ...“R” is the hydraulic radius in feet;
- ...“S” is the longitudinal slope in feet per foot.

### 2.3.2 Inlet

#### **Inlet Capacity Curves**

The primary objective in developing the curves shown in **Figure 5** through **Figure 19**, was to provide the Engineer with a direct method for sizing inlets which would yield answers within acceptable accuracy limits.

#### **Recessed and Standard Curb Opening Inlets on Grade**

The basic curb opening inlet capacity curves, **Figure 5** through **Figure 9**, “Recessed and Standard Curb Opening Inlets on Grade”, were based on solution of the following equation:

$$L = \frac{Q (H_1 - H_2)}{(H_1^{5/2} - H_2^{5/2})(0.70)} \quad (\text{Equation 5})$$

...“L” is the length of inlet, in feet, required to intercept the gutter flow;

...“Q” is the gutter flow in cubic feet per second;

...“H<sub>1</sub>” is the depth of flow, in feet, in the gutter approaching the inlet plus the inlet depression, in feet;

...“H<sub>2</sub>” is the inlet depression, in feet.

This is an empirical equation from Hydraulic Manual, Texas Highway Department, dated September 1970. The data from solution of this equation was used to plot the curves shown on **Figure 5** through **Figure 9**.

#### **Recessed and Standard Curb Opening Inlets at Low Point**

**Figure 10**, “Recessed and Standard Curb Opening Inlets at Low Point”, was plotted from the solution of the following equation:

$$Q = 3.087 L h^{3/2} \quad (\text{Equation 6})$$

“Q” is the gutter flow in cubic feet per second;

...“L” is the length of inlet, in feet, required to intercept the gutter flow;

...“h” is the depth of flow, in feet, at the inlet opening. This is the sum of the depth of the flow in the gutter,  $y_o$ , plus the depth of the inlet depression;

This equation expresses the capacity of a rectangular weir and is referenced in “The Design of Stormwater Inlets,” John Hopkins University, dated June 1956.

The calculated inlet capacities were reduced by ten (10) percent for preparation of **Figure 10**, due to the tendency of inlets at low points to clog from the collection of debris at their entrance.

#### Combination Inlet on Grade

**Figure 11** through **Figure 13**, “Combination Inlet on Grade”, were prepared based on the length of grate in feet,  $L_o$ , required to capture the portion of the gutter flow which crosses the upstream side of the grade, and on the length of grate in feet,  $L'$ , required to capture the outer portion of gutter flow. The figures were prepared with the solution of Equation 1 and the following equations:

$$L_o = 4 v_o \left[ \frac{y_o}{g} \right]^{1/2} \quad (\text{Equation 7})$$

$$L' = 1.2 v_o \tan \theta_o \left[ \frac{y_o - \frac{w}{\tan \theta_o}}{g} \right]^{1/2} \quad (\text{Equation 8})$$

$$q_2 = \frac{L' - L}{4} g^{1/2} \left[ y_o - \frac{w}{\tan \theta_o} \right]^{3/2} \quad (\text{Equation 9})$$

$$q_3 = Q_o \left[ 1 - \frac{L^2}{L_o^2} \right]^2 \quad (\text{Equation 10})$$

$$Q = Q_o (q_2 + q_3) \quad (\text{Equation 11})$$

...“ $L_o$ ” is the length of grate required to capture 100% of all flow over grate in feet;

...“ $v_o$ ” is the gutter velocity in feet per second;

...“ $y_o$ ” is the depth of gutter flow in feet

...“ $g$ ” is the gravitational acceleration (32.2 feet per second per second);

...“ $L$ ” is the length of grate required to capture the outer portion of the gutter flow in feet;

...“ $\theta_o$ ” is the crown slope of pavement

...“ $w$ ” is the width of grate in feet;

...“ $q_2$ ” is the carry-over flow in cfs outside of the grate;

...“ $L$ ” is the length of grate in feet;

...“ $q_3$ ” is the carry-over flow in cfs over the grate;

...“ $Q_o$ ” is the gutter flow in cfs

...“ $Q$ ” is the capacity of grate inlet in cfs

These equations are from “The Design of Stormwater Inlets,” John Hopkins University, dated June 1956.

#### Combination Inlet at Low Point

**Figure 17**, “Combination Inlet at a Low Point”, was prepared based on the inlet, a capacity equal to 90% of the quantity derived from solution of Equation 6, and 70% of quantity derived from solution of the following Equation 12.

(Grates are based on 1.72 square feet of opening per grate (Bass & Hayes #814 Grate).

$$Q = 3.087 L h^{3/2} \quad (\text{Equation 6})$$

$$Q = 0.6 A \sqrt{(2gh)} \quad (\text{Equation 12})$$

...“Q” is the gutter flow in cubic feet per second;

...“A” is the net cross sectional area, in square feet, of the grate opening;

...“g” is the gravitational acceleration 32.2 feet per second per second);

...“h” is the head, in feet, on the grate.

Combination inlets shall be used in locations with limited space clearance such as in Alley ROW, or with the approval of the City Engineering Division.

#### Grate Inlet on Grade

**Figure 13** through **Figure 16**, “Grate Inlet on Grade”, were prepared based on the solution of Equations 1, 7, 8, 9, 10 and 11, as described in the previous Sections, and with the assumption that the inlet was located in a curbed gutter. Grate Inlet on Grade shall only be used with the approval of the City Engineer for thoroughfare construction. Private systems may construct grate inlets as outlined in the manual.

#### Grate Inlet at Low Point

**Figure 18**, “Grate Inlet at Low Point”, was prepared on the assumption that the inlet has a capacity of 50 percent of the quantity derived from solution of Equation 12, as shown above. While this particular inlet capacity may appear to be considerably less than would be expected, it has been calculated based on observed clogging effects, primarily due to paper. The velocity of the gutter flow across the same inlet on grade tends to clear the grate openings. Grate Inlet at Low Point shall only be used with the approval of the City Engineering Division for thoroughfare construction. Private systems may construct grate inlets as outlined in this manual provided a clogged inlet will not cause flow to leave the property and overload the public inlets to the public drainage system.

### Drop Inlet at Low Point

**Figure 19**, “Drop Inlet at Low Point”, was prepared based on solution of Equation 6, as previously referenced, using a ten percent reduction in capacity due to clogging.

### 2.3.3 Hydraulic Design of Closed Conduits

All closed conduits shall be hydraulically designed through the application of Manning’s Equation, (non-critical flows) expressed as follows:

$$Q = A V$$

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

$$R = \frac{A}{P}$$

...“Q” is the flow in cubic feet per second;

...“A” is the cross sectional area of the conduit in square feet;

...“V” is the velocity of flow in the conduit in feet per second;

...“n” is the roughness coefficient of the conduit;

...“R” is the hydraulic radius which is the area of flow divided by the wetted perimeter. ( $R = A/P$ );

...“ $S_f$ ” is the friction slope of the conduit in feet per foot;

...“P” is the wetted perimeter.

Box culvert pipe will be designed as if flowing full. Design flow depth of less than full to get a lesser wetted perimeter is not acceptable. Four (4) wall wetted perimeter is required in the calculations unless the City Engineering Division approves a variance from these criteria.

### Velocity in Closed Conduits

Storm sewers should operate within certain velocity limits to prevent excessive deposition of solids due to low velocities, and to prevent invert erosion and undesirable and hazardous outlet conditions due to excessively high velocity. A minimum velocity of 2.5 feet per second and a maximum velocity of 12 feet per second shall be observed. In extreme conditions where the maximum velocity must be exceeded, prior approval must be obtained from the City Engineering Division.

### **Roughness Coefficients for Closed Conduits**

Roughness coefficients are directly related to construction procedures. When alignment is poor and joints have not been properly assembled, extreme head losses will occur. Coefficients used in this manner are related to construction procedures and assume that the pipe will be manufactured with a consistently smooth surface. Normally, .013 will be used as a minimum for new reinforced concrete pipe.

### **Minor Head Losses in Closed Conduits**

Head losses at structures shall be determined for manholes, junction boxes, wye branches, bends, curves and changes in pipe sizes in the design of closed conduits. Minimum head loss used at any structure shall be 0.10 foot. Properly designed curves may have zero losses.

- A. Head losses and gains for wyes and pipe size changes will be calculated by the following formulas:

Where  $V_1 < V_2$ :

$$\frac{V_2^2}{2g} - \frac{V_1^2}{2g} = HL$$

Where  $V_1 > V_2$ :

$$\frac{V_2^2}{4g} - \frac{V_1^2}{4g} = HL$$

and  $V_1$  is upstream velocity and  $V_2$  is downstream velocity.

- B. Head losses and gains for manholes, bends, curves and junction boxes will be calculated as shown in **Table 7**.

1. The basic equation for most cases, where there is both upstream and downstream velocity, takes the form as set forth below with the various conditions of the coefficient " $K_j$ " shown in **Table 7**.

$$h_j = \frac{V_2^2}{2g} - K_j \frac{V_1^2}{2g}$$

... " $h_j$ " is the junction or structure head loss in feet;

... " $V_1$ " is the velocity in upstream pipe in fps;

... " $V_2$ " is the velocity in downstream pipe in fps;

... " $K_j$ " is the junction or structure coefficient of loss.

2. In the case where the inlet is at the very beginning of a line, or the line is laid with bends or obstructions, the equation is revised as follows, without any approach velocity.

$$h_j = K_j \frac{V_2^2}{2g}$$

### 2.3.4 Hydraulic Design of Open Channels

Channel design involves the determination of a channel cross-section required to convey a given design flow. The method outlined in Section 3.0 of this manual may be used for analysis of an existing channel, or for the design of a proposed channel. Minimum slope is 0.3% for all channels.

#### Analysis of Existing Channels

The analysis of the carrying capacity of an existing channel is an application of Bernoulli's Energy Equation, which is as follows:

$$Z_1 + d_1 + h_{v1} = Z_2 + d_2 + h_{v2} + h_f + \text{other losses, where}$$

...“ $Z_1$ ” and “ $Z_2$ ” are the streambed elevations with respect to a given datum at upstream and downstream sections, respectively;

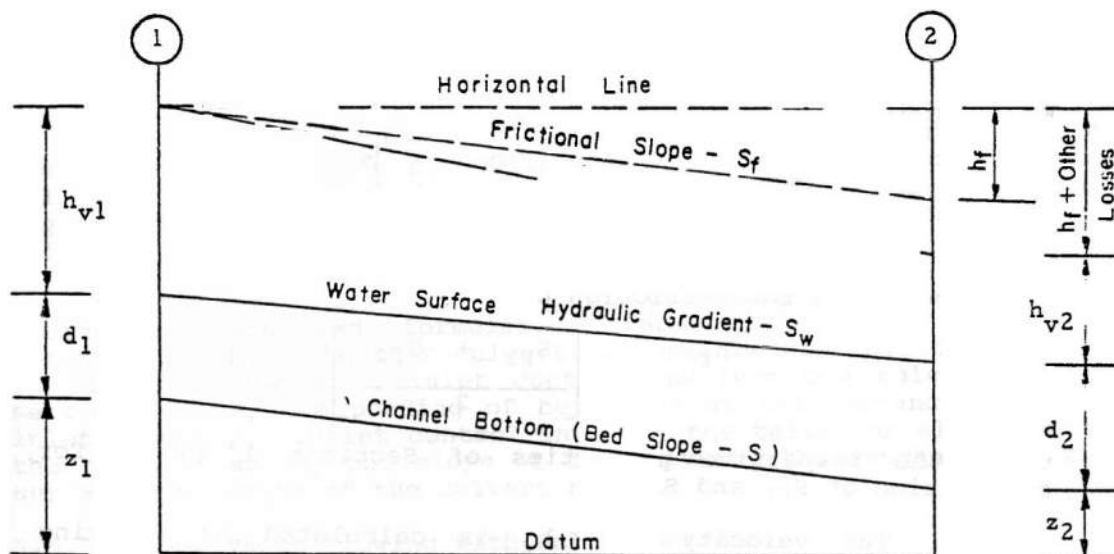
...“ $d_1$ ” and “ $d_2$ ” are depth of flows at the upstream and downstream sections, respectively;

...“ $h_{v1}$ ” and “ $h_{v2}$ ” are velocity heads of the upstream and downstream sections, respectively;

...“ $h_f$ ” is the friction head loss.

Other losses, such as eddy losses, are estimated as 10 percent (10%) of the friction head loss, where the quantity  $h_{v2}$  minus  $h_{v1}$  is positive and 50 percent thereof when it is negative. Bend losses are disregarded as an unnecessary refinement.

Bernoulli's Energy Equation is illustrated in graphic form as shown below:





The basic equations involved are:

$$Q = A V$$

$$h_v = \frac{v^2}{2g}$$

And Manning's Equation:

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

Which is defined previously in this chapter.

The friction head can be determined by using Manning's Equation in terms of the friction slope  $S_f$ , where:

$$S_f = \left[ \frac{Qn}{1.486 AR^{2/3}} \right]^2$$

Thus giving the total friction head:

$$H_f = L \left[ \frac{S_{f1} + S_{f2}}{2} \right]$$

Using the respective properties of Sections 1 and 2 for the calculation of  $S_{f1}$  and  $S_{f2}$ .

The velocity head  $h_v$  is calculated by weighing the partial discharges for each subdivision of the cross-section, i.e.,

$$h_v = \frac{V_s^2}{2g} \times \frac{Q_s}{Q}$$

Where " $V_s$ " is the velocity in subsection of the cross section;

" $A_s$ " is the area of the subsection of the cross section;

..." $Q_s$ " is the discharge in the subsection of the cross section;

And is

$$\frac{Q_s}{A_s}$$

When severe constrictions occur, the Momentum Equation may be required in the determination of losses.

### **Design of Improved Channels**

The hydraulic characteristics of improved channels are to be determined through the application of Manning's Equation as previously defined. In lieu of Manning's Equation, a HEC-RAS computer analysis can be utilized. The City, at its option, can require the use of HEC-RAS computer analysis in lieu of Manning's Equation. All channels shall have a minimum slope of 0.3%.

#### 2.3.5 Culverts

##### **Concrete Box and Pipe Culverts**

The design theory outlined herein is a modification of the method used in the hydraulic design of concrete box and pipe culverts, as discussed in the Department of Commerce, Hydraulic Engineering Circular No. 5, entitled "Hydraulic Charts for the Selection of Highway Culverts", dated December 1965.

The hydraulic capacity of culverts is computed using various factors and formulas. Laboratory tests and field observations indicate that culvert flow may be controlled either at the inlet or outlet. Inlet control involves the culvert cross-sectional area, the ponding of headwater at the entrance, and the inlet geometry. Outlet control involves the tailwater elevation in the outlet channel, the slope of the culvert, the roughness of the surface and length of the culvert barrel.

##### **Culverts Flowing with Inlet Control**

Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of the headwater and entrance geometry, including the barrel shape and cross-sectional area, and the type of inlet edge. Culverts flowing with inlet control can flow as shown on Form F, "Hydraulic Design of Culverts", Case I (inlet not submerged), or Case II (inlet submerged).

Nomographs for determining culvert capacity for inlet control are shown on **Figure 23** and **Figure 24**. These nomographs were developed by the Division of Hydraulic Research, Bureau of Public Roads, from analysis of laboratory research reported in the National Bureau of Standards Report No. 4444, entitled "Hydraulic Characteristics of Commonly Used Pipe Entrances", by John L. French, and "Hydraulics of Conventional Highway Culverts", by H. G. Bossy. Experimental data for box culverts with headwalls and wingwalls were obtained from an unpublished report of the U.S. Geological Survey.

##### **Culverts Flowing with Outlet Control**

Culverts flowing with outlet control can flow full as shown on Form F, Case III (outlet submerged), or part full for part of the barrel, as shown on Form F, Case IV (outlet not submerged).

The culvert is designed so that the depth of headwater, which is the vertical distance from the upstream culvert flow line to the elevation of the ponded water surface, does not encroach on the allowable freeboard during the design storm.

Headwater depth, HW, can be expressed by a common equation for all outlet control conditions:

$$HW = H + h_o - L (S_o)$$

... "HW" is the headwater depth in feet;

... "H" is the head or energy required to pass a given discharge through a culvert;

... "h<sub>o</sub>" is the vertical distance from the downstream culvert flow line to the elevation from which H is measured, in feet;

... "L" is the length of culvert in feet;

... "S<sub>o</sub>" is the culvert barrel slope in feet per foot.

The head, H, is made up of three parts, including the velocity head, exit loss (H<sub>v</sub>) and entrance loss (H<sub>e</sub>), and a friction loss (H<sub>f</sub>). This energy is obtained from the ponding of water at the entrance and is expressed as:

$$H = H_v + H_e + H_f$$

... "H" is head or energy in feet of water;

... "H<sub>v</sub>" is  $V^2/2g$  where V is average velocity in culvert or  $Q/A$ ;

... "H<sub>e</sub>" is  $K_e V^2/2g$  where K<sub>e</sub> is entrance loss coefficient;

... "H<sub>f</sub>" is the energy required to overcome the friction of culvert barrel and expressed as:

$$H_f = \left[ \frac{29.2n^2L}{R^{1.33}} \right] \left[ \frac{V^2}{2g} \right]$$

Where

... "n" is the coefficient of roughness (See **Table 6**);

... "L" is the length of culvert barrel in feet;

... "V" is the average velocity in the culvert in feet per second;

... "g" is the gravitational acceleration (32.2 feet per second per second);

... "R" is the hydraulic radius in feet

Substituting into the previous equation:

$$H = \left[ \frac{V^2}{2g} \right] + K_e \left[ \frac{V^2}{2g} \right] + \left[ \frac{29.2n^2L}{R^{1.33}} \right] \left[ \frac{V^2}{2g} \right]$$

And simplifying:

$$H = \left[ 1 + K_e + \frac{29.2n^2L}{R^{1.33}} \right] \left[ \frac{V^2}{2g} \right]$$

For full flow.

This equation for H may be solved using **Figure 25** and **Figure 26**.

For various conditions of outlet control flow,  $h_o$  is calculated differently. When the elevation of the water surface in the outlet channel is equal to or above the elevation of the top of the culvert opening at the outlet,  $h_o$  is equal to the tailwater depth or:

$$h_o = TW$$

If the tailwater elevation is below the top of the culvert opening at the outlet,  $h_o$  is the greater of two values: (1) Tailwater, TW, as defined above, or (2)  $(d_c + D)/2$ , where  $d_c$  = critical depth. The critical depth,  $d_c$ , for box culverts may be obtained from **Figure 27** or may be calculated from the formula:

$$d_c = 0.315 \left[ \frac{Q}{B} \right]^{2/3}$$

“ $d_c$ ” is the critical depth for box culvert in feet;

“Q” is the discharge in cubic feet per second;

“B” is the bottom width of box culvert in feet.

The critical depth for circular pipes may be obtained from **Figure 28** or may be calculated by trial and error. Charts developed by the Federal Highway Administration may be used for determining the critical depth. Utilize values of D, A and  $d_c$ , which will satisfy the equation:

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

...“ $d_c$ ” is the critical depth for pipe in feet;

...“Q” is the discharge in cubic feet per second;

...“D” is the diameter of pipe in feet;

...“g” is the gravitational acceleration (32.2 feet per second per second);

...“A” is the cross-sectional area of the trial critical depth of flow.

The equation is also applicable for trapezoidal or irregular channels, in which instances “D” becomes the channel top width in feet.

### 2.3.6 Bridges

Once a design discharge and depth of flow have been established, the size of the bridge opening may be determined. Specific effects of columns and piers may usually be neglected in the hydraulic calculations for determination of bridge openings. This is based on the assumption that all bents will be placed parallel to the direction of flow. Only in extenuating circumstances would it be desirable for bents to be placed at an oblique angle to the flow.

The basic hydraulic calculations involved in the hydraulic design involve solution of the following:

$$V = Q/A$$

“V” is the average velocity through the bridge in feet per second;

“Q” is the flow in cubic feet per second;

“A” is the actual flow area through the bridge in square feet.

$$h_f = K_b \frac{V^2}{2g}$$

...“h<sub>f</sub>” is the head loss through the bridge in feet;

...“K<sub>b</sub>” is a head loss coefficient (Normally .2 to.5);

...“V” is the average velocity through the bridge in feet per second;

...“g” is the gravitational acceleration (32.2 feet per second per second).

As can be seen from the above, the loss of head through the bridge is a function of the velocity head. The section of a head loss coefficient as recommended in Section 3.0, Criteria and Design Procedures, will determine the exact hydraulic conditions.

## 3.0 CRITERIA AND DESIGN PROCEDURES

### 3.1 GENERAL

This section contains storm drainage design criteria and demonstrates the design procedures to be employed on drainage projects in the City of Farmers Branch.

Applicable forms which can be used for the design of various storm drainage facilities are contained in Section 8 of this manual and the appropriate forms shall be part of the drainage submittal to the City and as part of the plan set.

### 3.2 HYDROLOGIC ITEM

#### 3.2.1 Rainfall

In determining the estimated runoff from a drainage area, it is necessary to predict the amount of rain which can be expected. **Table 1** "Rainfall Intensity Values", has been prepared as a reference for anticipated rainfall intensity for storm duration from 5 minutes to 24 hours for selected return frequencies and shall be used for determining rainfall rates as required.

### 3.3 DESIGN CAPACITIES/CONSIDERATIONS

Each storm drainage facility, including street capacities, shall be designed to convey the runoff which results from the 100-year design storm.

Drainage design requirements for open and closed systems shall provide protection for property during a 100-year design frequency storm. The design flow will result from assuming fully developed conditions as projected by the City's current zoning maps, and this projected flow shall be carried in the streets and closed drainage systems in accordance with the guidelines.

#### 3.3.1 Streets

##### Residential Streets

Based on a transverse slope of ¼-inch per foot, the 100-year design frequency flows shall not exceed a depth of the lowest top of curb. The entire street capacity may be used to convey drainage in the 100-year event. A maximum point flow from outside the right-of-way, such as a parking lot, side street, right-

of-way, etc., shall be based on the allowance designed into the drainage system when the street was constructed.

### **Collector Streets**

Based on a transverse slope of  $\frac{1}{4}$ -inch per foot, the 100-year design frequency flows shall not exceed a depth of the lowest top of curb. One moving traffic lane (12' minimum) is to remain open during the 100-year event. Maximum flow from outside the right-of-way, such as a parking lot, side street, right-of-way, etc., shall be based on the allowance designed into the drainage system when the street was constructed.

### **Major Thoroughfares**

Based on a transverse slope of  $\frac{1}{4}$ - inch per foot on the pavement, the 100-year design frequency flows shall not exceed the elevation of the lowest top of curb. One traffic lane (12' minimum, each) in each direction is to remain open during the 100-year event. A maximum point flow from outside the right-of-way, such as a parking lot, side street, right-of-way, etc., shall be based on the allowance designed into the drainage system when the street was constructed. Flow through major thoroughfare intersections shall not be permitted.

### **Alleys**

The 100-year design frequency flows shall not exceed the capacity of the alley pavement.

## **3.3.2 Areas**

### **Residential**

The drainage system shall be in accordance with the Subdivision Ordinance. In areas where the flow is small, paved flumes may be used in lieu of closed systems upon approval of the City Engineering Division. Natural or excavated channels may be utilized in accordance with Figure 24. A lot grading plan will be provided so that the surface flow pattern from lot to lot can be established.

### **Non-Residential**

The drainage system shall be as specified in the Subdivision Ordinance and in accordance with Section 3.3.

### 3.3.3 Open Channel

#### **Easement**

Drainage and floodway easements shall be provided for all open channels. Easements shall encompass all areas lower than a ground elevation defined as being the highest of the following:

- A. One (1) foot above the calculated water surface elevation based on a design storm whose frequency is 100 years. All watersheds are to be treated as fully developed.
- B. The top of the high bank, if higher than (a), above.

An additional easement of 15 feet on each side of a channel is required by the City for maintenance and access purposes. The slope within the easement shall be no steeper than 6:1.

#### **Excavated Channels**

Excavated channels shall have concrete pilot channels, if deemed necessary by the City Engineer, for access or erosion control as outlined below. All excavated channels shall have a design water surface as outlined in Section 3.4.2, and be in accordance with Figure 24, Type II. Concrete lined channels shall be not less than Type III, shown in Figure 24. At locations where earth channel improvements are required to carry a flood discharge through an undeveloped area, the channel grade can be “daylighted” and no freeboard required until the area is developed. Minimum slope of 0.3% for all channels.

#### **Storm Sewer Easement**

The storm sewer easement shall be the outside diameter of the storm sewer pipe plus 10 feet. The minimum is 15 feet.

### 3.3.4 Floodway and Drainage Related Minimum Elevations

#### **Minimum Lot and Floor Elevations**

Minimum lot and floor elevations shall be established as follows:

- A. For lots abutting a natural or excavated channel:
  - 1. Lots shall have a minimum elevation for the buildable area (including parking areas) of the lot at one (1) foot above the 100-year water surface elevation, or as directed by the City Engineering Division.
  - 2. Any inhabitable structure shall have a finished floor elevation at least two (2) feet above the 100-year water surface elevation.



- B. Where lots are positioned on a downhill side of a steep lead-in road to a “T” intersection, or a sharp turn in a steep alley, the downhill portion of the lot will be at least the same level as the top of curb or edge of alley right-of-way grade.

For lots adjacent to or in the influence of a sag area and a positive overflow, the lot elevation will be at least one (1) foot above the sag area top of curb, or one (1) foot above the possible maximum pool elevation when the positive overflow is functioning, whichever elevation is higher. (See Positive Overflow (Section 3.3.5 below).

Where lots do not abut a natural or excavated channel, minimum floor elevations shall be a minimum of two (2) feet above the street curb, edge of alley, or rear property line, whichever is lower, unless otherwise approved by the City Engineer.

### **Minimum Street or Alley Elevations**

Streets or alleys adjacent to an open channel shall have the edge of the pavement designed with an elevation not lower than the drainage and floodway easement elevation, as defined in Section 3.3.3 above, or as directed by the City Engineering Division.

#### **3.3.5 Positive Overflow**

The approved drainage system shall provide for positive overflow at all low points. The term “positive overflow” means that when the inlets do not function properly, or when the design capacity of the conduit is exceeded, the excess flow can be conveyed overland along a paved course. Normally, this would mean along a street or alley, but can require the dedication of special drainage easements on private property. Reasonable judgment should be used to limit the easements on private property to a minimum. In specific cases where the chances of substantial flood damages could occur, the City of Farmers Branch may require special investigations and designs. The overflow elevation shall not be higher than 0.5 feet above the top of the curb at the low point. Artificial sags created by “seesaw” of street or alley grades will not be permitted.

#### **3.3.6 Inlet Design**

Inlet spacing shall be in accordance with the design criteria contained in Section 3.5.1. Maximum length of inlet at any one curb location shall be 20 feet on each side of the street. Inlets will be placed only in straight sections of curb, and with curb returns at least 10 feet from the inlet box. Prior approval from the City Engineering Division is required for any deviations.

### 3.3.7 Culverts and Bridges

All drainage structures shall be designed to carry the fully developed 100-year design frequency flow. Bridges and culverts shall be designed for a water surface elevation as outlined in Section 3.4.2.

## 3.4 DETERMINATION OF DESIGN DISCHARGE

The Rational Method for computing stormwater runoff is to be used for hydraulic design of facilities serving a drainage area of less than 100 acres, unless otherwise directed by the City Engineering Division. For drainage areas larger than 100 acres, the ultimate drainage runoff shall be calculated by the Unit Hydrograph Method as described in Section 2.2.5, or an approved alternate method.

In lieu of the Unit Hydrograph Method, a HEC-HMS computer analysis can be utilized. The City, at its option, can require the use of HEC-HMS computer analysis in lieu of the Unit Hydrograph Method. Other recognized sources of data are the approved ultimate discharge studies.

### 3.4.1 Runoff Coefficients and Time of Concentration

Runoff coefficients, as shown in **Table 2**, shall be used, based on total development under the existing Farmers Branch land zoning map and regulations. Where land uses other than those listed in **Table 2** are planned, a coefficient shall be developed utilizing values comparable to those shown.

Times of concentration shall be computed based on the normal minimum inlet times shown in **Table 2**. Where conditions obviously warrant a deviation from the normal minimum inlet times as shown, Figure 2 may be used.

### 3.4.2 Criteria for Channels, Bridges and Culverts

Discharge flows and water surface elevations shall be based on the design storm frequency of 100 years, calculated by the City's design criteria. Where a unit hydrograph is used to determine the design flows, coefficients for " $C_t$ " and  $C_{p640}$ " should be as shown in **Table 3**.

### 3.4.3 Procedure for Determination of Design Discharge

A standard form, "Stormwater Runoff Calculations", Form A, is included in Section 8 to record the data used in various drainage area calculations. This form may be used in a modified form for calculations of runoff for design of open channels, culverts and bridges. Explanation for use of this form is included in Section 8.

## 3.5 PAVED AREAS

### 3.5.1 Flow in Gutters and Inlet Design

Unless there are specific agreements to the contrary prior to beginning design of the particular storm drainage project, the City of Farmers Branch requires a storm drain conduit to begin, and consequently, the first inlet to be located, at the point where the street gutter flows full. Location of the first inlet may be adjusted, with prior approval of the City Engineering Division.

### 3.5.2 Capacity of Straight Crown Streets

Figure 3, “Capacity of Triangular Gutters”, applies to all width streets having a straight cross slope varying from 1/8-inch per foot to ¼-inch per foot, which are the minimum and maximum allowable slopes. Cross slopes other than ¼-inch per foot shall not be used without prior approval from the City Engineering Division.

### 3.5.3 Capacity of Parabolic Crown Streets

Figures 4 and 5, “Capacity of Parabolic Gutters”, apply to streets with parabolic crowns.

### 3.5.4 Street Intersection Drainage

The use of surface drainage to convey stormwater across a street intersection is subject to the following criteria:

- A. An arterial or collector street shall not be crossed with surface drainage over and above that specified in Section 3.3.1, unless approved by the City Engineering Division. Intersections of arterial or collector streets shall not be crossed with surface drainage unless approved by the Engineering Division.
- B. At any intersection, only one street shall be crossed with surface drainage, and this shall be the lower classified street.

### 3.5.5 Alley Capacities

Alley capacity may be calculated through use of Manning’s Equation, as outlined in Section 2.3.1. Alley capacities are calculated to allow only the alley paving to carry the flow.

### 3.5.6 Inlet Design

**Figure 4**, “Storm Drain Inlets”, is a tabulation for various types and sizes of inlets and their prescribed uses.

The information in **Figure 4** and the general requirements of beginning the storm drain conduit where the street gutter or inlet capacity is reached, will furnish the information necessary to establish inlet locations.

**Figure 5** through **Figure 19** shall be used to determine capacity of specific inlets under various conditions.

In using the graphs for selection of inlet sizes, care must be taken where the gutter flow exceeds the capacity of the largest available inlet size. This is illustrated with the following example:

Known: Major Street  
Pavement Width = 22 Feet  
Gutter Slope = 1.00%  
Pavement Cross Slope = ¼-inch/1 Foot  
Gutter Flow = 11 cfs

Find: Length of Inlet Required ( $L_i$ )

Solution: Refer to **Figure 5**  
Enter Graph at 11 cfs  
Intersect Slope = 1.00%  
Read  $L_i$  = 16.9 Feet  
Enter Graph at  $L_i$  = 18 Feet  
Intersect Slope = 1.00%  
Read Q = 12.0 cfs

Therefore, the one inlet has a total capacity of 12.0 cfs, which is more than the gutter flow of 11 cfs.

The first upstream inlet shall be sized to intercept flows before they exceed the street capacity.

### 3.5.7 Procedure for Sizing and Locating Inlets

In order that the design procedure for determining inlet locations and sizes may be facilitated, a standard form, "Inlet Design Calculations", Form B, has been included in Section 8, along with instructions for completing the form. Inlet spacing shall be determined in accordance with Section 3.3.1 and Section 3.5.1 of this manual. The maximum length of inlet in any one curb location is 20 feet.

The fully developed runoff which is not designed to flow into the street (offsite) will be collected in storm sewer laterals. Undeveloped offsite flows that do not overload the inlets or curb capacity may be allowed to flow into the street until development is accomplished.

Inlet sizing in non-residential areas along major streets will include drainage areas that extend 100 feet past the ROW line for collection of surface drainage from non-residential zoning. Downhill extension from

the ROW line vary depending on the steepness of slope away from the ROW. In residential areas, the extension should be a minimum of 50 feet to allow side/backyards along the road to flow into the street drainage system as appropriate for the topography.

In handling undeveloped flows, the design for collection of stormwater should consider the undeveloped flow: 1) going into the street curb inlets, or 2) collected in drop ("Y") inlets on a lateral stubout. Drainage interceptor swales or berms should be used, as required, to direct runoff to the drop inlets.

### 3.6 CLOSED CONDUIT

#### 3.6.1 Hydraulic Gradient of Conduits

$$S_f = \left[ \frac{Q_n}{1.486 AR^{2/3}} \right]^2$$

A storm drainage conduit must have sufficient capacity to discharge a design storm with a minimum of interruption and inconvenience to the public using streets and thoroughfares. The size of the conduit is determined by accumulating runoff from contributing inlets and calculating the slope of a hydraulic gradient from Manning's Equation:

The hydraulic gradient for the selected conduit size shall be designed to carry the design flow at an elevation not less than 1.5 feet below the curb profile. As the conduit size is selected, and the tentative hydraulic gradient is plotted between each inlet pickup point, a head loss due to a change in velocity and pipe size must be incorporated in the gradient profile. (See **Table 7** for Junction or Structure Coefficients of Loss).

Also at each point where an inlet lateral enters the main conduit the gradient must be sufficiently low to allow the hydraulic gradient in the inlet to be below the gutter grade.

At the discharge end of the conduit (generally a creek or stream), the hydraulic gradient of the creek for the design storm must coincide with the gradient of the storm drainage conduit. An adjustment is usually required in the tentative conduit gradient and, necessarily, the initial pipe selection could also change. The hydraulic gradient of the creek or stream for the design storm can be calculated using design flows obtained by methods approved in Section 3.4 (Determination of Design Discharge).

Concrete pipe conduit shall be used to carry the stormwater. A flow chart, (**Figure 20**), based on Manning's Equation, may be used to determine the various hydraulic elements including the conduit size, the hydraulic gradient, Manning full flow capacity at the conduit slope, velocity, and  $V^2/2g$ .

In addition to concrete pipe, other frequently used conduit types include cast-in-place concrete box conduit and precast concrete box conduit. If flow charts are not available, the hydraulic gradient, conduit size and velocity of each of these conduits can be determined from the basic equation for flow in closed conduits, Manning's Equation:

$$\left[ \frac{Qn}{1.486 S^{1/2}} \right] = AR^{2/3}$$

The roughness coefficients for each of these conduit types are shown in **Table 6** of this manual.

With the hydraulic gradient established, considerable latitude is available for establishment of the conduit flow line. The inside top of the conduit must be at or below the hydraulic gradient thus allowing the conduit to be lowered where necessary. The hydraulic gradient for the storm sewer line and associated laterals should be plotted directly on the construction plan profile worksheet and adjusted as necessary. The  $Q_{100}$ ,  $C_m$  (Manning's capacity using the pipe slope as " $S_f$ "),  $S_f$ ,  $V$ ,  $S_{HG}$  (hydraulic grade line slope),  $V^2/2g$  shall be provided for each segment of the pipe profile.

There will be hydraulic conditions which cause the conduits to flow partially full. Where this occurs, the hydraulic gradient should be shown at the inside crown (soffit) of the conduit. This procedure provides a means for conservatively selecting a conduit size which will carry the design flood discharge.

### 3.6.2 Velocity in Closed Conduits

**Table 4** is a tabulation of minimum pipe grades which will produce a velocity of not less than 2.5 fps when flowing full. Grades less than those shown will not be allowed. Only those pipe sizes shown in **Table 4** should be considered for use in designing concrete pipe storm sewer systems.

**Table 5** shows the maximum allowable velocities in closed conduits.

### 3.6.3 Roughness Coefficients for Conduits

Recommended values for the roughness coefficient " $n$ " are tabulated in **Table 6**. Where engineering judgment indicates values other than those shown should be used, special note of this variance should be taken, and the appropriate adjustments made in the calculations.

#### 3.6.4 Minor Head Losses

The values of  $K_f$  to be used are tabulated for various conditions in **Table 7**. In designing storm sewer systems, the head losses which occur at points of turbulence shall be computed and reflected in the profile of the hydraulic gradient.

#### 3.6.5 Procedure for Hydraulic Design of Closed Conduits

“Storm Sewer Calculations”, Form C, has been included in Section 8, together with explanation for its use, to facilitate the hydraulic design of a storm sewer.

### 3.7 OPEN CHANNELS

Open channels are to be used to convey stormwaters where closed conduits are not justified. Consideration must be given to such factors as relative location to streets, schools, parks and other areas subject to frequent pedestrian use as well as basic economics.

Type II Channel (**Figure 21**) is an improved section recommended for use where larger storm flows are to be conveyed. The concrete flume in the channel bottom, including slope protection, is to be used as a maintenance aid. The indicated width of the flumes is a minimum width and, as the width of the channel increases, the required width of the flume may be increased.

Type III Channel (**Figure 21**) is a concrete lined section to be used for large flows in higher valued property areas and where exposure to pedestrian traffic is limited.

Where a recommended side slope and a maximum side slope are shown on a channel section, the Engineer shall use the recommended slope unless prior approval has been obtained from the City Engineering Division, or soil conditions require a flatter slope. Channel flow line gradient shall not be less than 0.3%.

The most efficient hydraulic cross-section of an open channel is the one which, with a given slope, area and roughness coefficient, will have the maximum capacity. This cross-section is the one having the smallest wetted perimeter. There are usually practical obstacles to using cross-sections of the greatest hydraulic efficiency, but the dimensions of such sections should be considered and adhered to as closely as conditions will allow.

Landscaping is intended to protect the channel right-of-way from erosion and present an aesthetically pleasing view. In erosion prone and disturbed ground areas, the Engineer shall provide for good grass coverage. Full coverage of grass must be established prior to acceptance by the City.

Design water surface shall be as shown on **Figure 21** and as outlined in Section 3.4.2. Floodway or drainage easements shall be provided as shown in Section 3.3.3.

Special care must be taken at entrances to closed conduits, such as culverts, to provide for the headwater requirements. These calculations and the required explanations are included on Form F.

As required by City Ordinances, erosion and sedimentation control measures shall be shown on the plans. The following items shall be considered for use: dikes, dams, berms, sediment basins, fiber mats, jute netting, temporary seeding, straw mulch, asphalt mulch, rubble liners, plastic liners, baled-hay retards, slope drains, and other devices as specified by the City Engineering Division. Construction and installation of all these items shall conform to the most recent version of the North Central Texas Council of Governments Standard Specifications for Public Works.

On all channels, the  $Q_{100}$  flood water surface elevations will normally be coincident with the culvert hydraulic gradient at the outfall, and will be shown on the construction plans. One exception to the water surface coinciding with the hydraulic gradient would be in supercritical flow, which generally is not encountered in this geographical area. Designs utilizing supercritical flow should be discussed with the City of Farmers Branch in the preliminary stages of design. A Froude Number between 0.8 and 1.2 is to be avoided in any flat bottom channel due to unstable flow conditions.

$$\text{Froude Number} = V / (g(A_s/b_s))^{1/2}$$

“ $A_s$ ” is the area of cross section;

“ $b_s$ ” is the width of stream at the surface;

“ $g$ ” is 32.2 ft/second<sup>2</sup>

“ $V$ ” is velocity in ft/second.

Hydraulic calculations for Type I Channels (**Figure 21**) shall be made as outlined on Form D, “Water Surface Profile Calculations”. This procedure is applicable to a stream with an irregular channel, and utilizes Bernoulli’s Energy Equation to establish the water surface elevations at succeeding points along the channel.



Hydraulic calculations for Types II and III Channels shall be made as outlined on Form E, "Open Channel Calculations".

In general, existing channels should be left in their natural condition if reasonable safety factors are present.

A hydraulic/hydrologic analysis may be required by the City Engineering Division for any drainage channel/watershed. The analysis is to be based on a fully developed watershed and adhere to the criteria set forth in Section 3.9.2.

Supercritical flow is only allowed at drop structures and other energy dissipaters. Channel armoring for erosion control shall be provided on curves where deemed necessary by the City Engineering Division.

The following hydraulic data should be submitted to the Engineering Division, preferably using the HEC-RAS program or the method in the appendix to compute the channel's water surface elevation. The data should be submitted electronically and in a bound report.

1. Duplicate of the effective City of Farmers Branch fully developed backwater model.
2. Modified existing condition hydraulic model – this model should include pre-development cross-sections through the project site obtained from field surveys or updated topographic information.
3. Proposed condition reflecting the development's impact on the floodplain area.
4. Water surface elevation and velocity summary tables tabulating the results of the above analysis.
5. Topographic map at a suitable scale with cross-sections, existing and proposed 100-year fully developed floodplain delineated, and the area being developed shown.
6. Analysis of the existing and proposed valley storage conditions of the area.
7. Documentation from the Corps of Engineers determining if a 404 Permit is required for the project.

### 3.7.1 Types of Channels

**Figure 21** illustrates the classifications and geometrics of various channel types which are to be used wherever possible.

Type I Channel is to be used whenever the development of land will allow. It is intended to be left as nearly as possible in its natural state, with improvements primarily limited to those which will allow the safe conveyance of stormwaters, minimize public health hazards and make the floodplain usable for recreation purposes. In some instances, it may be desirable to remove undergrowth.

### 3.7.2 Quantity of Flow

In the design of open channels, it is usually necessary that quantities of flow be estimated for several points along the channel. These are locations where recognized discharge points enter the channel and the flows are calculated as previously outlined under “Determination of Design Discharge.”

### 3.7.3 Channel Alignment and Grade

While it is recognized that channel alignments must be controlled primarily by existing topography and right-of-way, changes in alignment should be as gradual as possible. Whenever practicable, changes in alignment should be made in sections with flatter grades.

Normally, the grade of channels will be established by existing conditions, such as an existing channel at one end and a storm sewer at the other end. There are times, however, when the grade is subject to modification, especially between controlled points.

Whenever possible, the grades should be sufficient to prevent sedimentation and should not be overly steep to cause excessive erosion. Sediment control and collection points may be required by the Engineering Division.

For any given discharge and cross-section of channel, there is always a slope just sufficient to maintain flow at critical depth. This is termed critical slope, and a relatively large change in depth corresponds to relatively small changes in energy. Because of this instability, slopes at or near critical values should be avoided. (Froude Number = 1.0)

Maximum allowable velocities are shown in **Table 8**. When normal available grade would cause velocities in excess of maximums, plans shall include details for any special structures required to retard this flow. Velocity dissipation shall be provided at all outfalls where velocities exceed eight (8) feet per second or exceed the maximum allowable velocity for a soil type (**Table 9**).

### 3.7.4 Roughness Coefficients for Open Channels

Roughness coefficients to be used in solving Manning’s Equation are shown in **Table 8**, together with maximum allowable velocities.

### 3.7.5 Procedure for Calculation of Water Surface Profile for Unimproved Channels

Form D, included in Section 8, together with the explanation for its use, shall be used for calculating a profile of the water surface along an unimproved channel. The HEC-RAS computer program is an alternate method to the use of Form D and may be required by the City.

### 3.7.6 Procedure for Hydraulic Design of Open Channels

Form E, included in Section 8, together with the explanation for its use, shall be used in the design for open channels. The HEC-RAS computer program is an alternate method to the use of Form E and may be required by the City.

## 3.8 CULVERTS AND BRIDGES

### 3.8.1 Hydraulic Design of Culverts

The function of a culvert or bridge is to pass stormwater from the upstream side of a roadway to the downstream side without submerging the roadway or causing excessive backwater which floods upstream property.

The Engineer shall keep head losses and velocities within reasonable limits while selecting the most economical structure. In general, this means selecting a structure which creates a headwater condition and has a maximum flow velocity safely below the allowed maximums.

The vertical distance between the upstream design water surface and the roadway elevation should be maintained to provide a safety factor to protect against unusual clogging of the culvert, and to provide a margin for future modifications in surrounding physical conditions. In general, a minimum of one foot of freeboard shall be used when the structure is designed to pass a design storm frequency of 100 years calculated by the City's criteria. Unusual surrounding physical conditions may be cause for an increase in this requirement.

Culverts should always be aligned to follow the natural stream channel. Survey information of the stream channel should be provided for 200 feet upstream and downstream from the proposed culverts so that the channel alignment is evident.

A culvert which could become part of a storm drain pipe system will be sized to handle the worst-case flow as a culvert or storm drain in a fully developed drainage area.

Form F, included in Section 8, along with the explanation for its use, shall be used for the hydraulic design of culverts.

### 3.8.2 Culvert Hydraulics

In the hydraulic design of culverts an investigation shall be made of four different operating conditions, all as shown on Form F. It is not necessary that the Engineer know prior to the actual calculations which condition of operation (Case I, II, III, or IV) exists. The calculations will make this known.

1. Case I operation is a condition where the capacity of the culvert is controlled at the inlet with the upstream water level at or below the top of the culvert, and the downstream water level below the top of the culvert.
2. Case II operation is also a condition where the capacity of the culvert is controlled at the inlet with the upstream water level above the top of the culvert, with the downstream water level below the top of the culvert.
3. Case III operation is a condition where the capacity of the culvert is controlled at the outlet, with the upstream and downstream water levels above the top of the culvert.
4. Case IV operation is a condition where the capacity of the culvert is controlled at the outlet with the upstream water level above the top of the culvert, and the downstream water level equal to one or two levels to be calculated.

### 3.8.3 Quantity of Flow-Culverts

The quantity of flow which the structure must convey shall be calculated in accordance with the "Procedure for Determination of Design Discharge", utilizing Form A. Alternate methods to the use of Form A are named in Section 3.4, and may be required by the City.

### 3.8.4 Headwalls and Entrance Conditions

Headwalls are used to retain the fill material and reduce erosion of embankment slopes; to improve hydraulic efficiency; to provide structural stability to the culvert ends and serve as a counterweight to offset buoyant or uplift forces. The headwalls, with or without wingwalls and aprons, shall be constructed in accordance with the Texas Department of Transportation standard drawings as required by the physical conditions of the particular installation.

In general, straight headwalls (Type A) should be used where the approach velocities in the channel are below 6 feet per second, where headwater pools are formed and where no downstream channel protection is required. Headwalls with wingwalls and aprons (Type B) should be used where the approach velocities are from 6 to 12 feet per second and downstream channel protection is desirable.

Special headwalls and wingwalls shall be constructed where approach velocities are in excess of 12 feet per second, and where the flow must be directed in order to enter the culvert more effectively. This requirement varies according to the axis of the approach velocity with respect to the culvert entrance.

A table of culvert entrance data is shown on Form F and **Table 10**. The values of the entrance coefficient,  $K_e$ , are a combination of the effects of entrance and approach conditions. It is recognized that all possible conditions may not be tabulated, but an interpolation of values should be possible from the information shown. Where the term “round” entrance edge is used, it means a 6-inch radius on the exposed edge of the entrance.

### 3.8.5 Culvert Discharge Velocities

Velocities in culverts should be limited to no more than 15 feet per second, but downstream conditions very likely will impose more stringent controls. Consideration must be given to the effect of high velocities and turbulence on the channel, adjoining property and embankment. **Table 9** is a tabulation of maximum allowable velocities based on downstream channel conditions. Discharge velocities that are too high, per Section 3.7.3, must be reduced to allowable velocities using appropriate energy dissipation structures or techniques.

### 3.8.6 Hydraulic Design of Bridges

Wherever possible, the proposed bridge should be designed to span a channel section equal to the approaching channel section. If a reduction in channel section is desired, this should be accomplished upstream of the bridge, and appropriate adjustments made in the hydraulic gradient.

Wherever possible, bridges should be constructed to cross channels at a 90 degree angle, which normally will result in the most economical construction. Wherever the bridge structure is skewed, the bents should be constructed parallel to the flow of water. Values of  $K_b$ , head loss coefficient, shall be determined by an appraisal of the particular hydraulic conditions associated with the specific project. With a minimum of constriction and change in velocity, a clear span bridge would have a minimum coefficient. This would increase for a multispan bridge, skewed or with piers not placed parallel to the

flow. The Texas Department of Transportation “Hydraulic Design Manual” should be used for determining the K coefficient.

A minimum distance of 2 feet between the 100-year water surface elevation as calculated using the City’s criteria, and the lowest point of the bridge stringers, shall be maintained.

### 3.8.7 Quantity of Flow – Bridges

The quantity of flow which the structure must convey shall be calculated in accordance with the criteria set forth in Section 3.4.2.

### 3.8.8 Procedure for Hydraulic Design of Bridges

Form G, “Bridge Design Calculations”, included in Section 8, together with the explanation for its use, shall be used for the hydraulic design of bridges.

In more complex bridge design (such as long multiple spans and relief structures crossing an irregular channel section), the procedures outlined in the Texas Department of Transportation “Hydraulic Design Manual” should be used.

The Engineer should investigate several different bridge configurations on each project to determine the most economical that can be constructed within the velocity limitations and other criteria included in this manual.

## 3.9 FLOODPLAIN RECLAMATION

### 3.9.1 Definitions

*One Hundred (100) Year Water Surface Elevation (100- Yr WSEL)* – That water surface elevation established by hydrologic/hydraulic analysis of a stream, river, creek, or tributary, using the 100-year fully developed watershed, based upon the 100-year rainfall event.

*Floodplain* – Area of land lying below the 100-year water surface elevation.

*Equal Conveyance Principle* – An area of the cross-section of a stream, in its existing condition, carrying a percentage of the stream flow, will continue to carry the same percentage of the stream flow after filling of the floodplain occurs, without any rise in the 100-year floodplain elevation.

### 3.9.2 Procedures for Floodplain Alterations

Fill and alteration of floodplains which is not unreasonable damaging to the environment and which complies with the City Floodplain Ordinance and other applicable City flood protection policies and standards is permitted where it will not create other flood problems. The following are the engineering criteria for such requests:

#### **FEMA Submittal**

Developers of sites which impact designated Federal Emergency Management Agency (FEMA) floodplains in the City of Farmers Branch will be required to prepare the appropriate documentation for submittal to FEMA for conditional approval of the proposed project. The developer must submit the FEMA package to the City Floodplain Administrator for review and approval in advance of submitting to FEMA. The submittal package may include but is not limited to:

1. A written description of the scope of the proposed project and the methodology used to analyze the project's effects.
2. Hydraulic backwater models of the 10-, 50-, 100-, and 500- year floods for the following:
  - a. Duplicate of the effective Flood Insurance Study (FIS) model.
  - b. Existing conditions (effective FIS model including cross-sections through the project site – all cross-sections should reflect conditions prior to construction of the project).
  - c. Proposed conditions (existing conditions model reflecting the proposed project).
3. Floodway hydraulic backwater models of the following:
  - a. Duplicate effective
  - b. Existing conditions
  - c. Proposed conditions
4. A copy of the Flood Insurance Rate map with the project area indicated.
5. Topographic mapping of the entire area covered by the proposed conditions model, indicating the locations of all cross-sections used in the hydraulic model and delineating the proposed 100-year floodplain boundary.
6. Topographic mapping of the entire area covered by the proposed conditions model, indicating the locations of all cross-sections used in the hydraulic model and delineating:
  - a. The proposed 100- and 500- year floodplain boundaries;

b. The proposed floodway boundary

7. Certification that the project meets the requirements of the 44 CFR 60.3 (d)(2).

In order to recoup the costs associated with the review of Conditional Letters of Map Revision, FEMA has established fees which will be submitted with the above data. The developer is responsible for full payment of all fees associated with review of the project. Please refer to the [FEMA website](#) for the most recent fee schedule.

Upon completion of the proposed project, “as-built” plans, certified by a registered professional engineer, should be submitted.

As-built conditions are required in lieu of proposed condition backwater models for projects constructed without conditional approval. FEMA requires that individual legal notices be sent to all affected property owners when developments (cut or fill) occurs in the regulatory floodway that would cause any rise in the 100-year FIS water surface elevation. Public notice in the official community newspaper is required for proposed modifications to the regulatory floodway.

In all of the above hydraulic models, the following rules will apply:

1. The hydraulic parameters, such as bridge loss coefficients, “n” values, etc., used in the effective FIS models will only be changed where obvious errors or changes have taken place and must be documented.
2. The computed water surface elevation profiles have to converge with the existing profiles upstream and downstream of the project.
3. The information should be shown on a map of suitable scale and topographic definition to provide reasonable accuracy.
4. All items should be labeled for easy cross-referencing to the hydraulic model and summary data.

FEMA may have questions regarding the project. The engineer must address all of FEMA’s comments. It is not anticipated, but if revisions to the development are required by FEMA, the developer will be responsible to do so.

### 3.9.3 Floodplain Reclamation – Engineering Criteria

#### **Water Surface Elevation**

Alterations of the floodplain shall result in no increase in the 100-year fully developed watershed water surface elevation on other properties. No alteration of the floodplain will be permitted which could result in any degree of increased flooding to other properties, either adjacent, upstream, or downstream.



### **Stream Velocity**

Alterations of the floodplain shall not create an erosive water velocity on- or off-site. The mean velocity of stream flow at the site after fill shall be no greater than the mean velocity of the stream flow under existing conditions. No alteration to the floodplain will be permitted which would increase velocities of flood waters to the extent that significant erosion of floodplain soils will occur either on the subject property or on other properties up or downstream. Staff's determination of what constitutes an "erosive" velocity will be based on analysis of the surface material and permissible velocities for specific cross-sections affected by the proposed alteration, using standard engineering tables as a general guide (see **Table 8**).

### **Valley Storage**

Encroachment in the flood fringe area reduces the storage capacity of creeks and drainageways. This causes increased discharges downstream of the encroachment and hence increases the water surface elevation onto downstream property owners. Encroachments and/or channelization is strongly discouraged along Farmers Branch, Cooks, and Rawhide Creeks. The City of Farmers Branch has adopted the policy of restricting the valley storage loss to zero percent (0%) reduction, unless otherwise approved by the City Engineering Division.

### **Conveyance**

Alterations of the floodplain shall be permitted only to the extent permitted by equal conveyance on both sides of the natural channel. Staff's calculation of the impact of the proposed alteration will be based on the "equal conveyance" principle in order to insure equitable treatment for all property owners. Under equal conveyance, if the City allows a change in the flood carrying capacity (capacity to carry a particular volume of water per unit of time) on one side of the creek due to a proposed alteration of the floodplain, it must also allow an equal change to the owner of the other side. The combined change in flood carrying capacity, due to the proposed alteration, plus corresponding alteration to the other side of the creek, may not cause either an increase in flood elevation or an erosive velocity or violate the other criteria.

Conveyance is mathematically expressed as

$$KD = (1.486/n) A R^{2/3}$$

Where:

“n” is the Manning’s friction factor;

“A” is the cross sectional area;

“R” is the hydraulic radius.

### **Toe of Fill Alignment**

The toe of any fill slope shall parallel the natural channel to prevent an unbalancing of stream flow in the altered floodplain. If the alignment of the proposed fill slope departs from the contours of the natural floodplain, the flow characteristics of the flood waters may be altered. The erosion and deposition experienced in the altered floodplain may be damaging. If the fill slope follows the natural channel, it will also tend to minimize the visual impact of the alteration.

### **Side Slope**

To ensure maximum accessibility to the floodplain for maintenance and other purposes, and to lessen the probability of slope erosion during periods of high water, maximum slopes of filled area shall usually not exceed 3 feet horizontal to 1 foot vertical. Grass cover is required for all cut and fill slopes 3:1 or flatter. Concrete riprap or an approved equal erosion protection measure is required on slopes steeper than 3:1. Vertical walls, terracing and other slope treatments will be considered only as (a) part of a landscaping plan submission, and (b) if no unbalancing of stream flow results.

### **Vegetation/Landscaping**

Engineering plan submission shall include plans for: (a) erosion control of cut and fill slopes, (b) restoration of excavated areas, and (c) tree protection where possible in and below fill area. Landscaping should incorporate natural materials (earth, stone, and wood) on cut or fill slopes wherever possible. Applicant shall show in the plan the general nature and extent of existing vegetation on the tract, the location of trees 6-inch and larger in diameter, the areas which will be preserved, altered, or removed as a result of the proposed alterations. Locations and construction details should be provided, showing how trees will be preserved in areas which will be altered by filling or paving within the drip line of those trees. Applicant should also submit plans showing location, type, and size of new plant materials and other landscape features planned for altered floodplain areas.

### **Erosion Control**

Erosion control plans should indicate how the developer intends to minimize soil erosion and sedimentation from his site during and after the fill operation. Plans should include a timing schedule showing anticipated starting and completion dates for each step of the proposed operation. Soil areas exposed by grading, and length of time of exposure should be minimized. Existing vegetation should be retained and protected wherever feasible. Disturbed areas should have vegetation re-established as quickly as possible. Erosion control structures (e.g., drop structures, sediment ponds, etc.) should be utilized where necessary for effective erosion control, but should also be designed to blend in with the natural appearance of the floodplain.

## 4.0 CONSTRUCTION PLANS PREPARATION

### 4.1 GENERAL

This section covers the preparation of drainage construction plans for the City of Farmers Branch.

### 4.2 DESIGN PHASE

Plans shall be submitted in accordance with the City of Farmers Branch Checklist for Storm Drainage Plans. The first engineering plan set submission shall be complete, and in sufficient detail to allow review by the City of Farmers Branch. All topographic surveys should be furnished to allow establishment of alignment, grades and right-of-way requirements. These may be accomplished by on-the-ground field surveys, aerial imagery, or the use of the two- (2) foot contour topography. All field surveys shall utilize monuments and benchmarks listed on the [Survey Benchmarks page](#) on the City of Farmers Branch website.

The hydraulic design of the proposed facilities shall be accomplished based on the procedures and criteria outlined in Section 3.0 – Criteria and Design Procedures, of this manual. Calculations shall be made on the appropriate forms and submitted as part of the plan set. These plans shall show the alignment, drainage areas, size of facilities, and grades.

Storm drainage plans shall include a drainage area map, plan-profile sheets and channel cross-sections, if applicable. The proposed improvements shall be produced on 24" x 36" sheets. The sheet scale shall be sufficient to allow for accurate understanding of the design and contributing drainage areas as well as general project location within the City. A horizontal scale of 1" = 100' is usually sufficient and is generally preferred by the City.

#### 4.2.1 Drainage Area Map

The following items/information shall be included:

1. Acres, coefficient, and intensity for each drainage sub-area;
2. Inlets, their size and location, the flow bypass for each, the direction of flow as indicated by flow arrows, the station for the centerline of the line;
3. A chart including data shown shall be submitted with the first review, and included on the map with the final review;
4. Existing and proposed storm sewers;

5. Sub-areas for alleys, streets, and off-site areas;
6. Points of concentration;
7. Runoff to all inlets, dead-end streets, and alleys or to adjacent additions and/or lots;
8. A table for runoff computations;
9. Flow arrows to indicate all crests, sags and street and alley intersections;
10. North arrow;
11. Any off-site drainage shall be included;
12. Street names shall be indicated;
13. 100-year floodplain shall be indicated on the drainage area map.

When calculating runoff, the drainage area map shall show the boundary of the drainage area contributing runoff into the proposed system. This boundary should be determined from a map having a maximum contour interval of 2 feet. The area shall be further divided into sub-areas to determine flow concentration points or inlet locations. The centerline of all streets (except Residential or Local Streets) will normally be a boundary of a drainage area, to ensure that inlets are sized and positioned to fill the need without depending on stormwater crossing over the street crown for proper drainage.

In residential areas, the centerline of the street will only be used as a drainage area boundary if the flow in either gutter has not exceeded the street crown elevation (**Figure 3**).

Direction of flow within streets, alleys, natural and man-made drainage ways, and at all system intersections, shall be clearly shown on the drainage area map and/or paving plans. Existing and proposed drainage inlets, storm sewer pipe systems and drainage channels shall also be clearly shown and identified on the drainage area map. Storm sewers shall show and mark station tic-marks at 100-foot intervals. Plan-profile storm sewer or drainage improvement sheet limits and match lines shall be shown with pipes and channels identified.

The drainage area map should show enough topography to easily determine its location within the City.

## **4.3 PLAN-PROFILE SHEETS**

### **4.3.1 Inlets**

Inlets shall be given the same number designation as the area or sub-area contributing runoff to the inlet. The inlet number designation shall be shown opposite the inlet. Inlets shall be located at or immediately downstream of drainage concentration points. At intersections, where possible, the end of the inlet shall be ten feet from the curb return P.T., and the inlet location shall also provide minimum interference with the use of adjacent property. Inlets in residential areas should be located in streets and alleys so that driveway access is not prohibited to the lots. Inlets located directly above storm sewer lines, as well as laterals passing through an inlet, shall be avoided. Drainage from abutting properties shall not be impaired and shall be designed into the storm drainage system.

Data opposite each inlet shall include paving or storm sewer stationing at centerline of inlet, size and type of inlet, number or designation, top of curb elevation and flow line of inlet as shown on the construction plans.

### **4.3.2 Laterals**

Inlet laterals leading to storm sewers, where possible, shall enter the inlet and the storm drain main at a 60 degree (60°) angle from the street side. Laterals shall be four feet from top of curb to flow line of inlet, unless utilities or storm sewer depth requires otherwise. Laterals shall not enter the corners or bottoms of inlets. Lateral profiles shall be drawn showing appropriate information including the hydraulic gradient and utility crossings. Short lateral (30 feet or less) crossings utility lines will be profiled.

### **4.3.3 Storm Sewer**

In the plan view, the storm sewer designation, size of pipe, and length of each size pipe shall be shown adjacent to the storm sewer. The sewer plan shall be stationed at one hundred- (100) foot intervals, and each sheet shall begin and end with even or fifty- (50) foot stationing. All storm sewer components shall be stationed.

The profile portion of the storm sewer plan-profile sheet shall show the existing and proposed ground profile along the centerline of the proposed sewer, the hydraulic gradient of the sewer, the proposed storm sewer, and utilities which intersect the alignment of the proposed storm sewer. Also shown shall be the diameter of the proposed pipe in inches, and the physical grade in percent. Hydraulic data for each

length of storm sewer between interception points shall be shown on the profile. This data shall consist of pipe diameter in inches, the 100-year design storm discharge in cubic feet per second, slope of hydraulic gradient in percent, Manning capacity of the pipe flowing full in cubic feet per second, velocity in feet per second, and  $V^2/2g$ . Also, the head loss at each interception point shall be shown.

Elevations of the flow line of the proposed storm sewer shall be shown at one hundred-(100) foot intervals on the profile. Stationing and flow line elevations shall also be shown at all pipe grade changes, pipe size changes, lateral connections, manholes and wye connections. All soffits shall be connected.

#### 4.3.4 Creek Cross-Sections

### 4.4 MISCELLANEOUS

All plan sheets shall be generated using computer aided design and drafting software and plotted on 24" x 36" material, to a standard engineering scale, and shall be clearly legible when sheets are reduced to half scale. After each review, all review comments shall be addressed, additional data incorporated, and drafting of plans completed. Each plan-profile sheet shall have a benchmark shown, and the City of Farmers Branch Survey Benchmarks used.

### 4.5 CHECKLIST FOR STORM DRAINAGE PLANS

#### 4.5.1 Drainage Area Map

1. Normally, use 1" = 100' scale. Show match lines between any two (2) or more maps.
2. Show existing and proposed storm drains and inlets with designations.
3. Indicate sub-areas for alley, street, and off-site areas.
4. Indicate contours on map for on- and off-site, using two (2) foot contour maps.
5. Use design criteria as shown in design manual.
6. Indicate zoning on drainage area.
7. Show points of concentration and their designations.
8. Indicate runoff at all inlets, dead-end streets and alleys, or to and from adjacent additions or acreage.
9. Provide runoff calculations for all areas showing acreage, runoff coefficient, and inlet time. (Q = CIA Table or Form A)

10. For cumulative runoff, show calculations.
11. Indicate all crests, sags, and street and alley intersections with flow arrows.
12. Identify direction of north to top page or to the left.
13. Show limits of 100-year fully developed floodplain on drainage area map.

#### 4.5.2 Storm Sewers

1. Diversion of flow from one natural drainage area to another will not be allowed.
2. Show plan and profile of all storm sewers.
3. Specify Class III pipe unless otherwise noted.
4. Use heavier than Class III pipes where crossing railroads, areas of deep fill and areas subjected to heavy loads.
5. Specify concrete strength for all structures. The minimum allowable is 4200 psi.
6. Provide inlets where street capacity is exceeded. Provide inlets where alley runoff exceeds intersecting street capacity.
7. Do not allow stormwater flow from streets into alleys.
8. Do not use high velocities in storm sewer design. A maximum discharge velocity of eight (8) fps at the outfall is required. Velocity dissipation may be necessary to reduce erosion.
9. Flumes may not be allowed unless specifically designated, and will not be allowed on major thoroughfares.
10. Provide headwalls and aprons for all storm sewer outfalls. Provide riprap around headwalls where slopes exceed 3:1.
11. Discharge flow lines of storm sewers to be two (2) feet above the flow line of creeks and channels, unless channel lining is present. Energy dissipation shall be provided when specified by the Engineering Division.
12. Where fill is proposed for trench cut in creeks or outfall ditches, compaction shall be 95% of the maximum density as determined by ASTM D 698.
13. Investigation shall be made by the engineer to validate the adequacy of the storm sewer outfall to a major stream.
14. Outfall area must have adequate capacity to carry the discharge. Provide erosion control facilities with hydraulic data.



15. Any off-site drainage work or discharge to downstream property will require an easement. Easement shall be sized such that the developed flows can be conveyed within the easement. Submit field notes for off-site easement that may be required (Private development only).

#### 4.5.3 Plan and Profile

1. Indicate property lines and lot lines along storm sewers and show easements with dimensions.
2. If necessary, provide separate plan and profile of storm sewers. The storm drain pipes should also be shown on paving plans with a dashed line, and on sanitary sewer profiles showing the full pipe section.
3. Tie storm sewer system stationing with paving stations.
4. Show pipe size in plan and profile.
5. Show hydraulics on each segment of pipe profile to include:  $Q_{100}$ ,  $C$  = Manning full flow capacity;  $S$ ,  $V$ ,  $V^2/2g$ .
6. Show curve data for all storm sewer system.
7. Show all existing utilities in plan and profile. On storm sewer profiles, as a minimum, the sanitary sewer profile will be shown.
8. Indicate existing and proposed ground line and improvements on all street, alley, and storm sewer profiles.
9. Show future streets and grades where applicable.
10. Where connections are made to existing storm sewer, show computations of existing system when available. HGL will be calculated from the outfall to the connection point including the designed flows of the added on system.
11. Indicate flow line elevations of storm sewers on profile, show pipe slope (percent grade). Match top inside of pipe where adjacent to other size pipe.
12. Intersect laterals at sixty 60° degrees with trunk line.
13. Show details of all junction boxes, headwalls, storm sewers, flumes, and manholes, when more than one pipe intersects the drainage facility or any other item not a standard detail.
14. Pipe direction changes will be curves using radius pipe unless approved by the City Engineering Division.
15. Bends in pipe may be used in unusual circumstances with approval of the City Engineering Division. No bend at one location may exceed thirty (30°) degrees.
16. Do not use 90-degree (90°) turns on storm sewers or outfalls. Provide good alignment with junction structures or manholes (for small systems).

17. Profile outfall with typical flat bottom section.
18. Show all hydraulics, velocity head changes, gradients, and computations.
19. Show water surface at outfall of storm drain.
20. On all dead-end streets and alleys, show grade out to "daylight" for drainage on the profiles and provide erosion control. Show typical section and slope of "daylight" drainage.
21. At sags in pavement, provide a positive overflow (paved sidewalk in a swale) to act as a safety path for failure of the storm drain system. Minimum finished floor elevations will be shown on the plat to protect building against flooding should the positive overflow be used.
22. Where quantities of runoff are shown on plans or profiles, indicate storm frequency design.
23. Provide sections for road, railroad and other ditches with profiles and hydraulic computations. Show design water surface on profile.

#### 4.5.4 Laterals

1. Show laterals on trunk profile with stations.
2. Provide lateral profiles for laterals exceeding thirty (30) feet in length. Profile short laterals that pass over a sanitary sewer or other profiled utility.
3. Where laterals tie into trunk lines, place at sixty-degree (60°) angles with centerlines. Connect them so that the longitudinal centers intersect.
4. Calculate hydraulic grade line for laterals and inlets to insure collection of stormwater. Check  $1.5 V^2/2g$ , using trunk line velocity on laterals less than 80-feet long. Finalize the H.G.L. at the gutter or inlet lip by adding the  $1.5 V^2/2g$  to the hydraulic gradient of the trunk line at the lateral connection. For all inlets, provide H.G.L. and hydraulic data on profile for all profiled laterals. Laterals longer than eighty (80) feet require special analysis.
5. All inlets shall have a minimum eighteen-inch (18") laterals.

#### 4.5.5 Inlets and Intakes

1. Provide inlets where street capacity is exceeded. Provide inlets where runoff from alley causes the capacity of the intersection street to be exceeded.
2. Indicate runoff concentrating at all inlets and direction of flow. Show runoff for all stub outs, pipes and intakes.
3. On plan view, indicate size of inlet, lateral size, flow line, top-of-curb elevations, paving station, and inlet designation number.

4. Use standard curb inlets in streets. Use recessed inlets in divided streets. Use combination inlets in alleys when on a straight run. Do not use grate or combination inlet unless other solution is not available (special situations).
5. Use type "Y" or special "Y" inlets in ditches or swales. No "Glory Holes" allowed as intake for a storm sewer or at a culvert. A three (3) foot concrete apron shall be constructed around "Y" inlets.

#### 4.5.6 Paving

1. Provide six (6) inch curb on alleys parallel to creek or channel on creek side of alley.
2. For a proposed driveway turnout, curb return P.T. must be 10 feet upstream from any existing or proposed inlet, or 5 feet downstream of a standard inlet. (See SD 9)
3. Check the need for curbing at all alley turns and "T" intersections. Flatten grades ahead of turns and intersections.
4. Where inlets are placed in an alley, provide curbing for 10 feet on each side of combination inlet.

#### 4.5.7 Detention

When required by the City Engineering Division:

1. Provide drainage area map and show all computations for runoff affecting the detention basin.
2. Provide a plot plan with existing and proposed contours for the detention basin and plan for structural measures.
3. Where earth embankment is proposed for impoundment, furnish a typical embankment section and specifications for fill include profile for the structural outflow structure and geotechnical report.
4. Provide structural details and calculations for any item not a standard detail.
5. Provide detention basin volume calculations and elevation versus storage curve.
6. Provide hydraulic calculations for outflow structure and elevation versus discharge curve.
7. Provide routings or modified rational determination of storage requirements, demonstrating that critical duration is used (permitted for areas of 100 acres or less).
8. Fencing may be required around detention area.

#### 4.5.8 Bridges

1. Clear the lowest member of the bridge by 2 feet above the design water surface, unless otherwise directed by the City Engineering Division.
2. Show geotechnical soil boring information on plans.

3. Show bridge sections upstream and downstream.
4. Provide hydraulic calculations on all sections.
5. Provide structural details and calculations with dead load deflection diagram.
6. Provide vertical and horizontal alignment.
7. Show soil erosion protection measures and concrete riprap.

## 5.0 APPENDIX

### 5.1 DEFINITION OF TERMS

<u>Acceptable Outlet:</u>	That point where stormwater runoff can be released into a watercourse or drainageway of adequate capacity without causing scour or erosion.
<u>Angle of Flare:</u>	Angle between direction of wingwall and center line of culvert or storm drain outlet.
<u>Backwater Curve:</u>	The surface curve of a stream of water when backed up by a dam or other obstruction.
<u>Berm:</u>	A shelf that breaks the continuity of a slope.
<u>CFS:</u>	Quantity of flow in cubic feet per second.
<u>Channel Stabilization:</u>	Erosion prevention and stabilization of velocity distribution in a channel using jetties, drops, revetments, structural linings, vegetation and other measures.
<u>Conduit:</u>	Any closed device for conveying flowing water.
<u>Control:</u>	The hydraulic characteristic which determines the stage-discharge relationship in a conduit.
<u>Critical Flow:</u>	The state of flow for a given discharge at which the specific energy is a minimum with respect to the bottom of the conduit.
<u>Crushed Stone:</u>	Aggregate consisting of angular particles produced by mechanically crushing rock.
<u>Dike (Engineering):</u>	An embankment to confine or control water; for example, one built along the banks of a river to prevent overflow of lowlands; a levee.
<u>Disturbed Area:</u>	An area in which the natural vegetation soils cover has been removed or altered, which is therefore susceptible to erosion.
<u>Diversion:</u>	A channel with a supporting ridge on the lower side constructed across the slope to divert water from areas where it is in excess, to sites where it can be used or disposed of safely. Diversions differ from terraces in that they are individually designed.
<u>Entrance Head:</u>	The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head.
<u>Entrance Loss:</u>	Head lost in eddies or friction at the inlet to a conduit, headwall or structure.

<u>Erosion:</u>	<p>(a) The wearing away of land surface by running water, wind, ice, or other geological agents, including such processes as gravitational creep.</p> <p>(b) Detachment and movement of soil or rock fragments by water, wind, ice or gravity.</p>
<u>Accelerated Erosion:</u>	Erosion much more rapid than normal, natural or geologic erosion, primarily as a result of the influence of the activities of man, or in some cases, of other animals or natural catastrophes that expose base surfaces; for example, fires.
<u>Gully Erosion:</u>	The erosion process whereby water accumulates in narrow channels, and over short periods removes the soil from this narrow area to considerable depths, ranging from 1 to 2 feet to as much as 75 to 100 feet (see Gully).
<u>Rill Erosion:</u>	An erosion process in which numerous small channels, only several inches deep, are formed (see Rill).
<u>Sheet Erosion:</u>	The removal of a fairly uniform layer of soil from the land surface by runoff water.
<u>Splash Erosion:</u>	The splattering of small soil particles caused by the impact of raindrops on wet soils. The loosened and spattered particles may or may not be subsequently removed by surface runoff.
<u>Flume:</u>	Any open conduit on a prepared grade trestle or bridge.
<u>Freeboard:</u>	The distance between the normal operating level and the top of the side of an open channel left to allow for wave action, floating debris, or any other condition or emergency without overflowing structure.
<u>Grade:</u>	<p>(a) The slope of a road, channel or natural ground.</p> <p>(b) The finished surface of a canal bed, roadbed, top of embankment, or bottom of excavation; any surface prepared for the support of construction, like paving or laying a conduit.</p> <p>(c) To finish the surface of a canal bed, roadbed, top of embankment, or bottom of excavation.</p>
<u>Headwater:</u>	Depth of water in the stream channel measured from the invert of culvert.
<u>HEC-HMS:</u>	Hydrologic Engineering Center – Hydrologic Modeling System; Computer program that simulates the precipitation-runoff processes of watersheds, producing flood hydrographs. This program is available from the U.S. Army Corps of Engineers.
<u>HEC-RAS:</u>	Hydrologic Engineering Center – River Analysis System; Computer program that models the hydraulics of water flow through natural rivers

or other channels, producing water surface profiles. This program is available from the U.S. Army Corps of Engineers.

<u>Highwater Elevation:</u>	The water surface elevation during the peak of the design storm.
<u>Hydraulic Gradient:</u>	A line representing the pressure head available at any given point within the system.
<u>Invert:</u>	The flowline of pipe or box (inside bottom).
<u>Manning's Equation:</u>	The uniform flow equation used to relate velocity, hydraulic radius and energy gradient slope.
<u>Open Channel:</u>	A channel in which water flows with a free surface.
<u>Outfall:</u>	The point where water flows from a conduit, stream or drain.
<u>Outlet:</u>	The point at which water discharges from sources such as streams, rivers, lakes, tidal basins, pipes, channels, or drainage areas.
<u>Permanent Seeding:</u>	Results in establishing perennial vegetation which may remain on the area for many years.
<u>Permissible Velocity:</u>	The highest average velocity at which water may be carried safely in a channel or other conduit. The highest velocity that can exist through a substantial length of a conduit and not cause scour of the channel. Syn.: safe, non-eroding or allowable velocity.
<u>Rational Formula:</u>	The means of relating runoff with the area being drained and the intensity of the storm rainfall. ( $Q_R = CIA$ )
<u>Rill:</u>	A small channel cut by concentrated runoff, but through which water commonly flows during and immediately after rains. A rill is usually only a few inches deep (but no more than a foot), and hence, no obstacle to tillage operations.
<u>Riprap:</u>	Broken rock, cobbles, or boulders, placed on earth surfaces, such as the face of a dam or the bank of a stream, for protection against the action of water (waves); also applies to brush or pole mattresses, or brush and stone, or similar materials used for soil erosion control.
<u>Sediment:</u>	Solid material, both mineral and organic, that is in suspension, is being transported, or has been moved from its site of origin by air, water, gravity, or ice, and has come to rest on the earth's surface either above or below sea level.
<u>Sedimentation:</u>	Deposition of detached soil particles.

<u>Silt:</u>	(a) (Agronomy): A soil separate consisting of particles between 0.05 and 0.002 millimeter in equivalent diameter.  (b) A soil textural class.  (c) (Engineering): According to the Unified Soil Classification System, a fine-grained soil (more than 50 percent passing the NO. 200 sieve) that has a low plasticity index in relation to the liquid limit.
<u>Soffit:</u>	The inside top of pipe or box. Also called Crown.
<u>Stabilization:</u>	Providing adequate measures, vegetative and/or structural, that will prevent erosion from occurring.
<u>Stabilized Grade:</u>	The slope of a channel at which neither erosion nor deposition occurs.
<u>Steady Flow:</u>	Constant discharge.
<u>Surcharge:</u>	Height of water surface above the crown of a closed conduit at the upstream end.
<u>Tailwater:</u>	Total depth of flow in the downstream channel measured from the invert at the culvert outlet.
<u>Time of Concentration:</u>	The estimated time in minutes required for runoff to flow from the most remote section of the drainage area to the point at which the flow is to be determined ( $t_c$ ).
<u>Total Head Line (Energy Line):</u>	A line representing the energy in flowing water. It is plotted a distance above the profiles of the flow line of the conduit equal to the normal depth plus the normal velocity head plus the pressure head for conduits flowing under pressure.
<u>Uniform Channel:</u>	A channel with a constant cross-section and roughness coefficient.
<u>Uniform Flow:</u>	A condition of flow in which the discharge, or quantity of water flowing per unit of time, and the velocity are constant. Flows will be at normal depth and can be computed by the Manning Equation.
<u>Watershed:</u>	The area drained by a stream or drainage system.



## 5.2 ABBREVIATION OF TERMS AND SYMBOLS

"A"	Drainage area in acres of tributary watershed. Cross-sectional area of gutter flow in square feet. Cross-sectional area of flow through conduit in square feet.
"A <sub>s</sub> "	Sub-section area in square feet as used on unimproved channel calculations.
"b"	Bottom width of channel in feet.
"b <sub>s</sub> "	Width of spread at water surface (Froude number equation).
	Runoff coefficient, for use in Rational Formula, representing the estimated ration of runoff to rainfall which is dependent on the slope of the watershed, the land use and the character of soil.
"C <sub>m</sub> "	Pipe capacity under Manning full flow conditions, using the storm sewer pipe slope for sf in the Manning equation.
"C <sub>o</sub> "	Street crown height in feet.
"C <sub>t</sub> "	A coefficient related to drainage basin characteristics and used in Unit Hydrograph calculations.
"C <sub>p640</sub> "	A coefficient related to drainage basin characteristics and used in Unit Hydrograph calculations.
"cfs"	Cubic feet per second.
"d"	Depth of flow, in feet.
"d <sub>n</sub> "	Normal depth of flow in conduit, in feet.
"d <sub>c</sub> "	Critical depth of flow in conduit, in feet.
"FL"	Flow line.
"FR"	Froude Number = $V/[g(A_s/b_s)]^{1/2}$
"fps"	Feet per second.
"g"	Gravitational acceleration (32.2 feet per second per second).
"H"	Depth of flow, in feet, required to pass a given discharge.
"h"	Depth of flow, in feet.
"HW"	Headwater elevation or depth above invert at storm drain entrance in feet.
"h <sub>o</sub> "	Vertical distance from downstream culvert flow line to the elevation from which H is measured, in feet.

" $h_f$ "	Head loss due to friction in a length of conduit, in feet.
" $h_j$ "	Head loss at junction structures, inlets, manholes, etc., due to turbulence in feet.
" $h_v$ "	Velocity head loss in feet.
" $I$ "	Intensity, in inches per hour, for rainfall over an entire watershed or a sub-basin.
" $K_b$ "	Head loss coefficient at bridges.
" $K_e$ "	Coefficient of entrance loss.
" $K_j$ "	Coefficient for head loss at junctions, inlets and manholes.
" $L$ "	Length of channel, in miles, measured along flow line.
" $L_{ca}$ "	Length of stream, in miles, from design point to center of gravity of drainage area and used in Unit Hydrograph calculations.
" $L_i$ "	Length of curb opening inlet in feet.
" $L_{is}$ "	Initial and subsequent rainfall losses in inches and used in Unit Hydrograph calculations.
" $n$ "	Coefficient of roughness for use in Manning's Equation.
" $P$ "	Length, in feet, of contact between flowing water and the conduit measured on a cross-section. (Wetted Perimeter)
" $Q$ "	Stormwater flow in cfs
" $Q_R$ "	Peak flow, in cfs, as determined by Rational Method.
" $Q_u$ "	Peak flow, in cfs, as determined by Unit Hydrograph Method.
" $q_p$ "	Peak rate of discharge of the Unit Hydrograph for unit rainfall duration in cfs per square mile.
" $Q_p$ "	Peak rate of discharge of the Unit Hydrograph in cfs
" $R$ "	Hydraulic Radius = $A_s/P$
" $R_T$ "	Total runoff in inches as used in Unit Hydrograph calculations.
" $S$ "	Slope of street, gutter or hydraulic gradient in feet per foot or percent.
" $S_c$ "	That particular slope in feet, per foot, of a given uniform conduit operating as an open channel, at which normal depth and velocity equal critical depth and velocity for a given discharge.
" $S_D$ "	Design storm runoff, in inches, for a two-hour period.

" $S_f$ "	Friction slope in feet per foot in a conduit. This represents the rate of loss in the conduit due to friction.
" $t_c$ "	Time of Concentration, in minutes.
" $t_p$ "	Lag time, in hours, from the midpoint of the unit rainfall duration to the peak of the Unit Hydrograph.
"TW"	Tailwater elevation of depth above invert at culvert outlet.
"V"	Velocity of flow in feet per second.
" $\bar{v}$ "	Mean velocity of flow at upstream end of inlet opening in feet per second.
" $v_c$ "	Critical velocity of flow in a conduit in feet per second.
" $V^2/2g$ "	Velocity head. A measure, in feet, of the kinetic energy in flowing water.
" $V_1$ "	Upstream Velocity
" $V_2$ "	Downstream Velocity
"W"	Street width from face of curb, in feet.
"WP"	Wetted perimeter, in feet.
"Y"	Conveyance factor calculated for unimproved channels.
" $\theta$ "	Reciprocal of crown slope, $1/\theta_o$ .
" $\theta_o$ "	Crown slope of pavement, in feet per foot.

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**Table 1: Rainfall Intensity Values**

Duration		Return Period (Years)				
		5	10	25	50	100
		Rainfall Intensity (inches per hour)				
Hours	Minutes					
0.083	5	7.04	8.22	9.74	10.90	12.00
0.167	10	5.64	6.58	7.81	8.71	9.60
0.250	15	4.67	5.44	6.46	7.23	7.99
0.500	30	3.25	3.78	4.48	4.99	5.50
1	60	2.13	2.49	2.95	3.28	3.62
2	120	1.33	1.56	1.88	2.11	2.35
3	180	0.99	1.17	1.42	1.61	1.81
6	360	0.59	0.71	0.87	0.99	1.12
12	720	0.35	0.42	0.51	0.58	0.66
24	1440	0.20	0.24	0.30	0.34	0.39

Source: NOAA Atlas-14 Point Precipitation Frequency Estimates – Precipitation Frequency Data Server  
[https://hdsc.nws.noaa.gov/hdsc/pfds/pfds\\_map\\_cont.html](https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html)

**Table 2: Coefficients of Runoff and Normal Minimum Inlet Times**

Description of Area	Runoff Coefficient C	Minimum Inlet Time in Minutes
Areas Zoned Industrial, Commercial, Local Retail, Office or Similar Use	0.90	10
Areas Zoned for Multi-Family Dwelling Apartments >12 unit/acre	0.90	10
Areas Zoned for Patio Homes, Duplexes, Single Family Attached, and Townhouses	0.70	15
Areas Zoned for Single Family Residential, High Density,	0.70	15
Medium Density	0.60	15
Low Density	0.50	15
Schools	0.70	15
Churches	0.80	10
Parks, Cemeteries, Pasture	0.40	15
Major Thoroughfare R.O.W. (when it is a drainage area)	0.90	10



**Table 3: Coefficients “Ct” and Cp640”**

Drainage Area Characteristics	Approximate Value of “Ct”	Value of “Cp640”
Sparsely Sewered Area		
Flat Basin Slope (less than 0.50%)	0.65	350
Moderate Basin Slope (0.50% to 0.80%)	0.60	370
Steep Basin Slope (greater than 0.80%)	0.55	390
Moderately Sewered Area		
Flat Basin Slope (less than 0.50%)	0.55	400
Moderate Basin Slope (0.50% to 0.80%)	0.50	420
Steep Basin Slope (greater than 0.80%)	0.45	440
Highly Sewered Area		
Flat Basin Slope (less than 0.50%)	0.45	450
Moderate Basin Slope (0.50% to 0.80%)	0.40	470
Steep Basin Slope (greater than 0.80%)	0.35	490

**Table 4: Minimum Slopes for Concrete Pipes**  
**(n = .013)**

Pipe Diameter (Inches)	Slope Feet/100 Feet	Pipe Diameter (Inches)	Slope Feet/100 Feet
18	.180	51	.045
21	.150	54	.041
24	.120	60	.036
27	.110	66	.032
30	.090	72	.028
33	.080	78	.025
36	.070	84	.023
39	.062	90	.021
42	.056	96	.019
45	.052	102	.018
48	.048	108	.016

**NOTE:** Minimum pipe diameter to be used in construction of storm sewers shall be eighteen (18) inches.

**Table 5: Velocities in Closed Conduits**

Type of Conduit	Minimum Velocity	Maximum Velocity
Culverts	2.5 fps	15 fps
Inlet Laterals	2.5 fps	No Limit
Storm Sewers	2.5 fps	12 fps

Storm sewers shall discharge into open channels at a maximum velocity of 8 feet per second, unless erosion protection is provided.

**Table 6: Roughness Coefficients for Closed Conduits**

<b>Material of New Construction</b>	<b>Recommended Roughness Coefficient “n”</b>
New Monolithic Concrete Conduit	.015
Concrete Pipe Storm Sewer – New Construction	.013
<b>Materials of Existing Systems</b>	<b>Recommended Roughness Coefficient “n”</b>
Concrete Pipe Storm Sewer (Old System)	
Good Alignment, Smooth Joints	.013
Fair Alignment, Ordinary Joints	.015
Poor Alignment, Poor Joints	.017
Concrete Pipe Culverts	.012
Monolithic Concrete Culverts	.012
*Corrugated Metal Pipe	.024
*Corrugated Metal Pipe (Smooth Lined)	.013
*Corrugated Metal Pipe Arch	.024

Note: “n” values for Concrete Box Storm Sewers are same as Concrete Pipe Storm Sewers.

\*Information Only: Reinforced concrete pipe is the accepted material for construction of storm drains. The use of other materials for the construction of storm drains shall have prior approval from the City Engineering Division.

**Table 7: Junction or Structure Coefficient of Loss**

Case No.	Reference Figures	Description of Condition	Coefficient K <sub>j</sub>	Equation h <sub>j</sub> =
I	Table 7, Sheet 2	Inlet on Main Line	.50	$\frac{V_2^2}{2g} - \frac{K_j V^2}{2g}$
II	Table 7, Sheet 2	Inlet on Main Line with Branch Lateral	.25	
III	Table 7, Sheet 2	Manhole on Main Line with 90° 60° 45° 22 ½°	.25 .35 .50 .75	
IV	Table 7, Sheet 2	Wye Connection or Cut In 60° 45° 22 ½°	.60 .75 .95	
V	Table 7, Sheet 3	Inlet or Manhole at Beginning of Line	1.25	$\frac{K_j V^2}{2g}$
VI	Table 7, Sheet 3	Conduit Curves for 90°** Curve Radius 2 to 8D** 8 to 20D > 20D	.40 .25 .00	
VII	Table 7, Sheet 3	Bend Where Radius is Equal to Diameter 90° Bend 60° Bend 45° Bend 22 ½° Bend	.50 .43 .35 .20	

The values of the coefficient “K<sub>j</sub>” for determining the loss of head due to obstruction in pipes are shown in Table 7-A and the coefficients are used in the following equation to calculate the head loss at the obstruction:

$$h_j = \frac{K_j V^2}{2g}$$

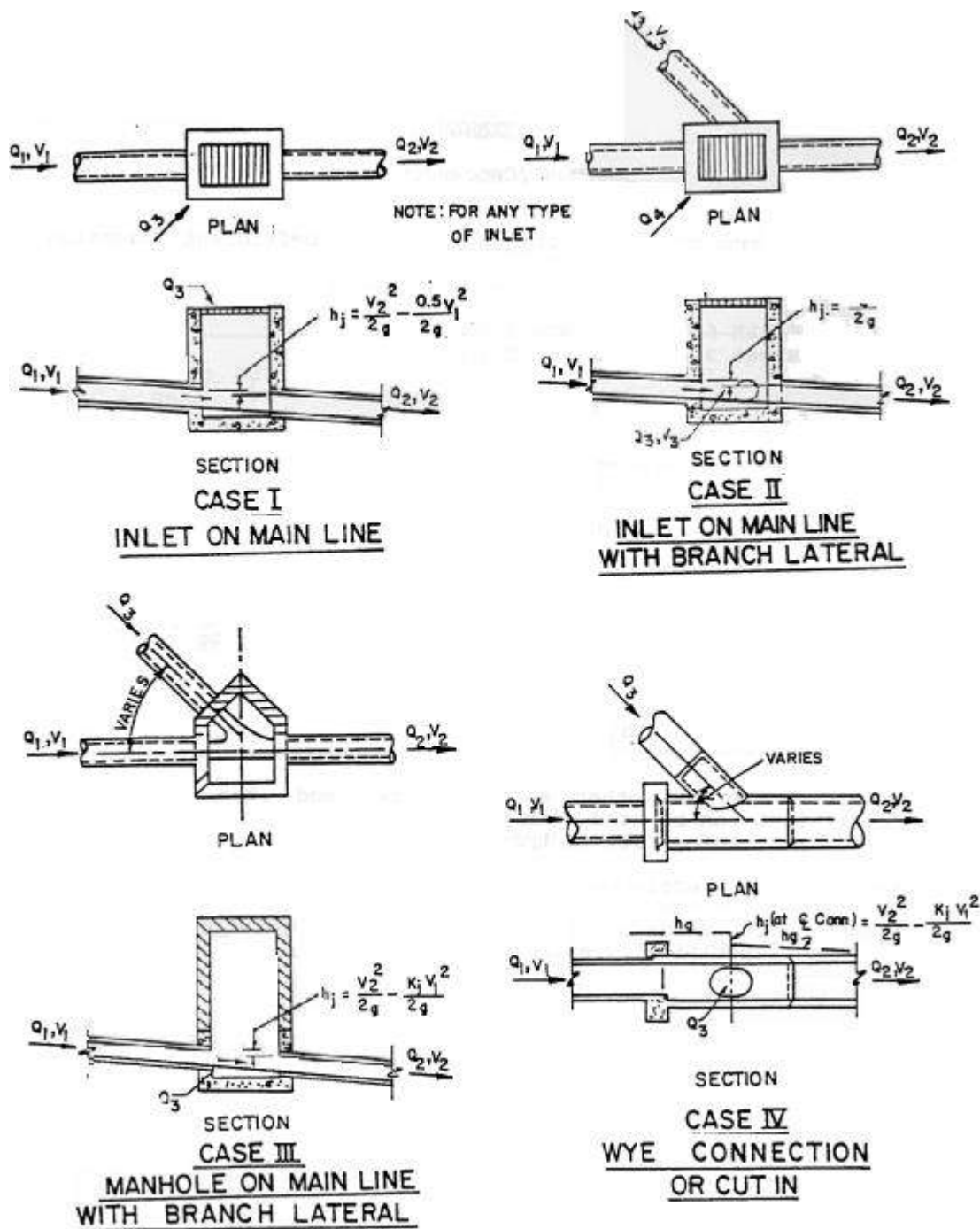
\*Where deflection other than 90° are used, the 90° deflection coefficient can be used with the following percentage factors: 60° Bend – 85%; 45° - 70%; 22 ½° Bend – 40%

Inside Diameter of Pipe

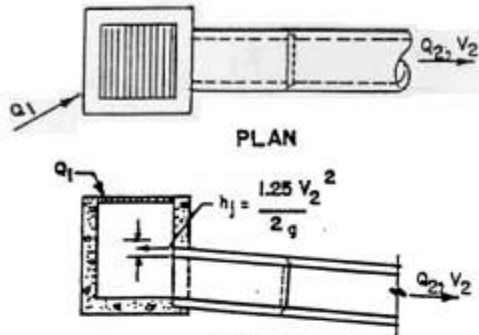
Note: 90° Bends are not to be used in Storm Sewer System unless specifically approved by City Engineering Division.

**Table 7-A: Head Loss Coefficients Due to Obstructions**

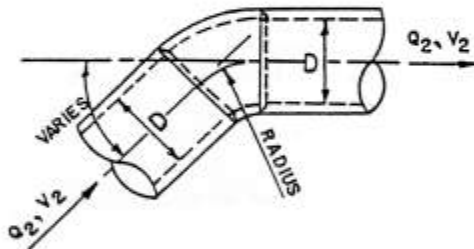
Fitting	Coefficient $K_j$
Globe valve, wide open	10
Angle valve, wide open	5
Close-return bend	2.2
T, through side outlet	1.8
Short-radius elbow	0.9
Medium-radius elbow	0.75
Long-radius elbow	0.60
45° elbow	0.42
Gate valve, wide open	0.19
half open	2.06



Minor Head Losses Due to Turbulence at Structures  
Table 7



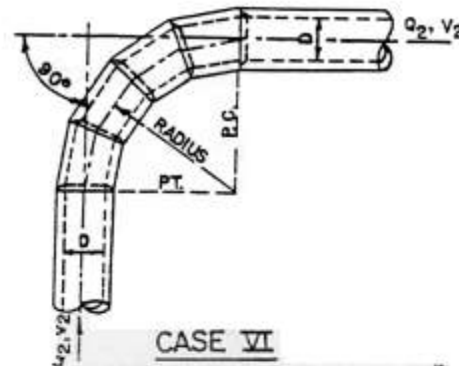
**CASE V**  
**INLET OR MANHOLE AT**  
**BEGINNING OF LINE**



**CASE VII**  
**BENDS WHERE RADIUS IS**  
**EQUAL TO DIAMETER OF PIPE**

NOTE : HEAD LOSS APPLIED AT BEGINNING OF BEND. BENDS TO BE USED ONLY WITH THE PERMISSION OF THE DRAINAGE DESIGN ENGINEER.

$$\begin{aligned} 90^\circ \text{ BEND } h_f &= 0.50 \frac{V_2^2}{2g} & 60^\circ \text{ BEND } h_f &= 0.43 \frac{V_2^2}{2g} \\ 45^\circ \text{ BEND } h_f &= 0.35 \frac{V_2^2}{2g} & 22.5^\circ \text{ BEND } h_f &= 0.20 \frac{V_2^2}{2g} \end{aligned}$$



**CASE VI**  
**CONDUIT ON 90° CURVES**

NOTE : HEAD LOSS APPLIED AT P.C. FOR LENGTH OF CURVE

$$\text{RADIUS} = (2-8) \times \text{DIA. OF PIPE } h_f = 0.40 \frac{V_2^2}{2g}$$

$$\text{RADIUS} = (8-20) \times \text{DIA. OF PIPE } h_f = 0.25 \frac{V_2^2}{2g}$$

$$\text{RADIUS} = \text{GREATER THAN } 20 \times \text{DIA. OF PIPE } h_f = 0$$

\* WHEN CURVES OTHER THAN 90° ARE USED, APPLY THE FOLLOWING FACTORS TO 90° CURVES

- 60° CURVE 85 %
- 45° CURVE 70 %
- 22 1/2° CURVE 40 %

**Minor Head Losses Due to Turbulence at Structures**  
**Table 7**



**Table 8: Roughness Coefficients for Open Channels**

Channel Description	Roughness Coefficient			Maximum Velocity ft/sec
	Minimum	Normal	Maximum	
Minor Natural Streams – Type I Channel				
Moderately Well Defined Channel				
Grass and Weeds, Little Brush	0.025	0.030	0.033	8
Dense Weeds, Little Brush	0.030	0.035	0.040	8
Weeds, Light Brush on Banks	0.030	0.035	0.040	8
Weeds, Heavy Brush on Banks	0.035	0.050	0.060	8
Weeds, Dense Willows on Banks	0.040	0.060	0.080	8
Irregular Channel with Pools and Meanders				
Grass and Weeds, Little Brush	0.030	0.036	0.042	8
Dense Weeds, Little Brush	0.036	0.042	0.048	8
Weeds, Light Brush on Banks	0.036	0.042	0.048	8
Weeds, Heavy Brush on Banks	0.042	0.060	0.072	8
Weeds, Dense Willows on Banks	0.048	0.072	0.090	8
Floodplain, Pasture				
Short Grass, No Brush	0.025	0.030	0.035	8
Tall Grass, No Brush	0.030	0.035	0.050	8
Floodplain, Cultivated				
No Crops	0.025	0.030	0.035	8
Mature Crops	0.030	0.040	0.050	8
Floodplain, Uncleared				
Heavy Weeds, Light Brush	0.035	0.050	0.070	8
Medium to Dense Brush	0.070	0.100	0.160	8
Trees with Flood Stage below Branches	0.080	0.100	0.120	8
Major Natural Streams – Type I Channel				
The roughness coefficient is less than that for minor streams or similar description because banks offer less effective resistance.				
Moderately Well Defined Channel	0.025	-	0.060	8
Irregular Channel	0.035	-	0.100	8
Unlined Vegetated Channels – Type II Channel				
Mowed Grass, Clay Soil	0.025	0.030	0.035	8
Mowed Grass, Sandy Soil	0.025	0.030	0.035	6
Unlined Non-Vegetated Channels – Type II Channel				
Clean Gravel Section	0.022	0.025	0.030	8
Shale	0.025	0.030	0.035	10
Smooth Rock	0.025	0.030	0.035	15
Lined Channels – Type III				
Smooth Finished Concrete	0.013	0.015	0.020	15
Riprap (Rubble)	0.030	0.040	0.050	12

**Table 9: Culvert Discharge Velocities**

Culvert Discharges On	Maximum Allowable Velocity (fps)
Earth (Sandy)	6
Earth (Clay)	8
Sodded Earth	8
Concrete	15
Shale (Rock)	10
UngROUTED Rip-rap	10

**Table 10: Culvert Entrance Losses**

Culvert Entrance Losses Where:

$$h_j = K_e \frac{v^2}{2g}$$

...“ $h_e$ ” is the entrance head loss (ft).

...“ $K_e$ ” is the entrance loss coefficient as shown in the table below.

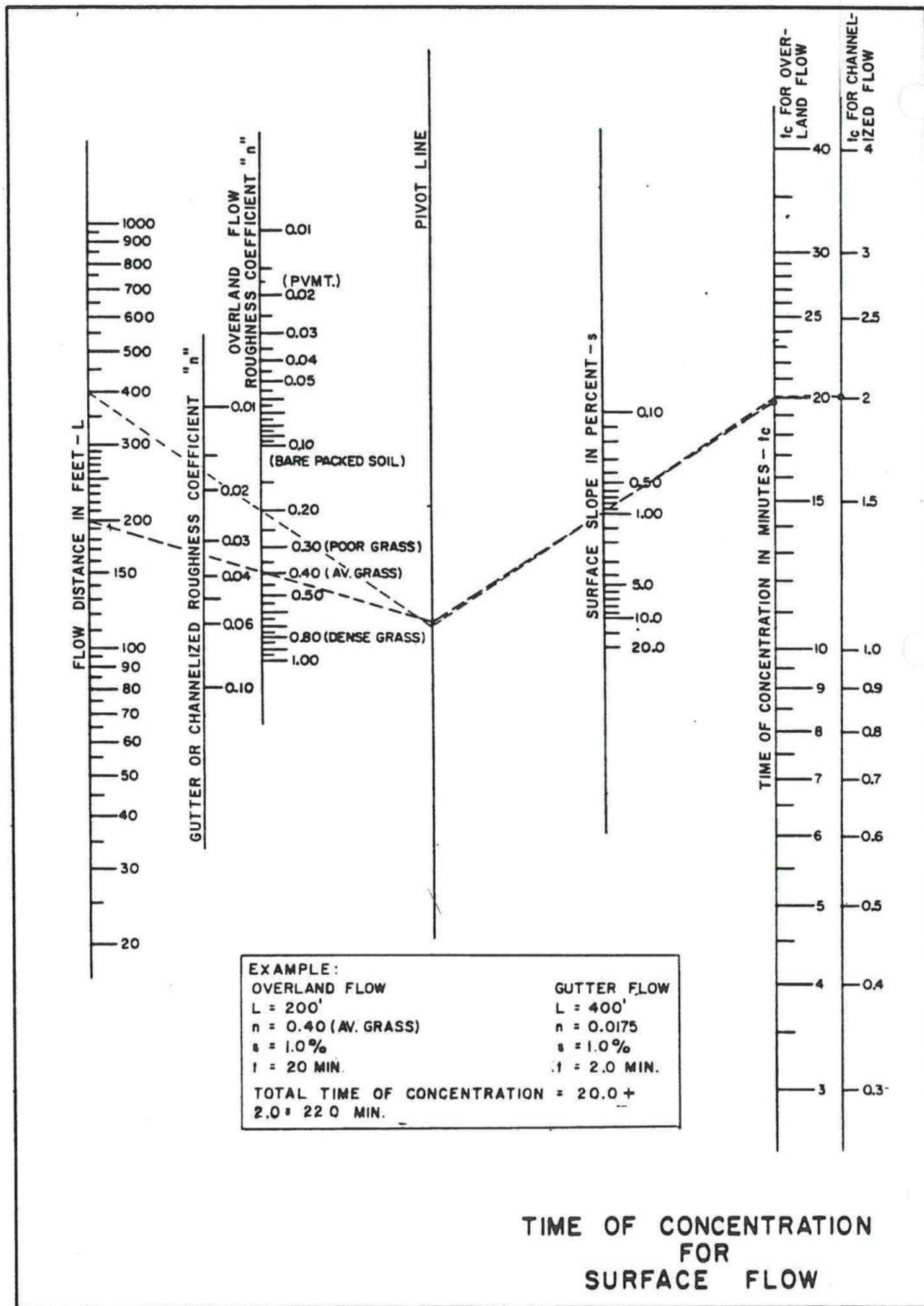
...“ $V$ ” is the velocity of flow in culvert (ft/s).

The following table gives  $K_e$  values for different entrance conditions:

Type of Structure	$K_e$
<b>Pipe, Concrete</b>	
Projecting from fill, socket and (groove end)	0.2
Projecting from fill, square cut end	0.5
<b>Headwall or headwall and wingwalls</b>	
Socket end of pipe (groove end)	0.2
Square edge	0.5
Rounded (radius = 0.0933D)	0.2
Mitered to conform to fill slope	0.7
Beveled edges, 33.7° or 45°	0.2
Side or sloped tapered inlet	0.2
<b>Pipe, or Pipe-Arch, Corrugated Metal</b>	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls, square edge	0.5
Mitered to conform to fill slope, paved/unpaved slope	0.7
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope tapered inlet	0.2
<b>Box, Reinforced Concrete</b>	
<b>Headwall parallel to embankment (no wingwalls)</b>	
Squared on three sides	0.5
Rounded on three sides to radius 1/12 barrel dimension or bevelled on three sides	0.2
<b>Wingwalls at 30° to 75° to barrel</b>	
Square edged at crown	0.4
Crown edge rounded to radius of 2/12 barrel dimension, or beveled top edge	0.2
<b>Wingwall at 10° to 25° to barrel</b>	
Square edged at crown	0.5
<b>Wingwalls parallel (extension of sides)</b>	
Square edged at crown	0.7
Side or slope tapered inlet	0.2

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## EXAMPLE

### Known:

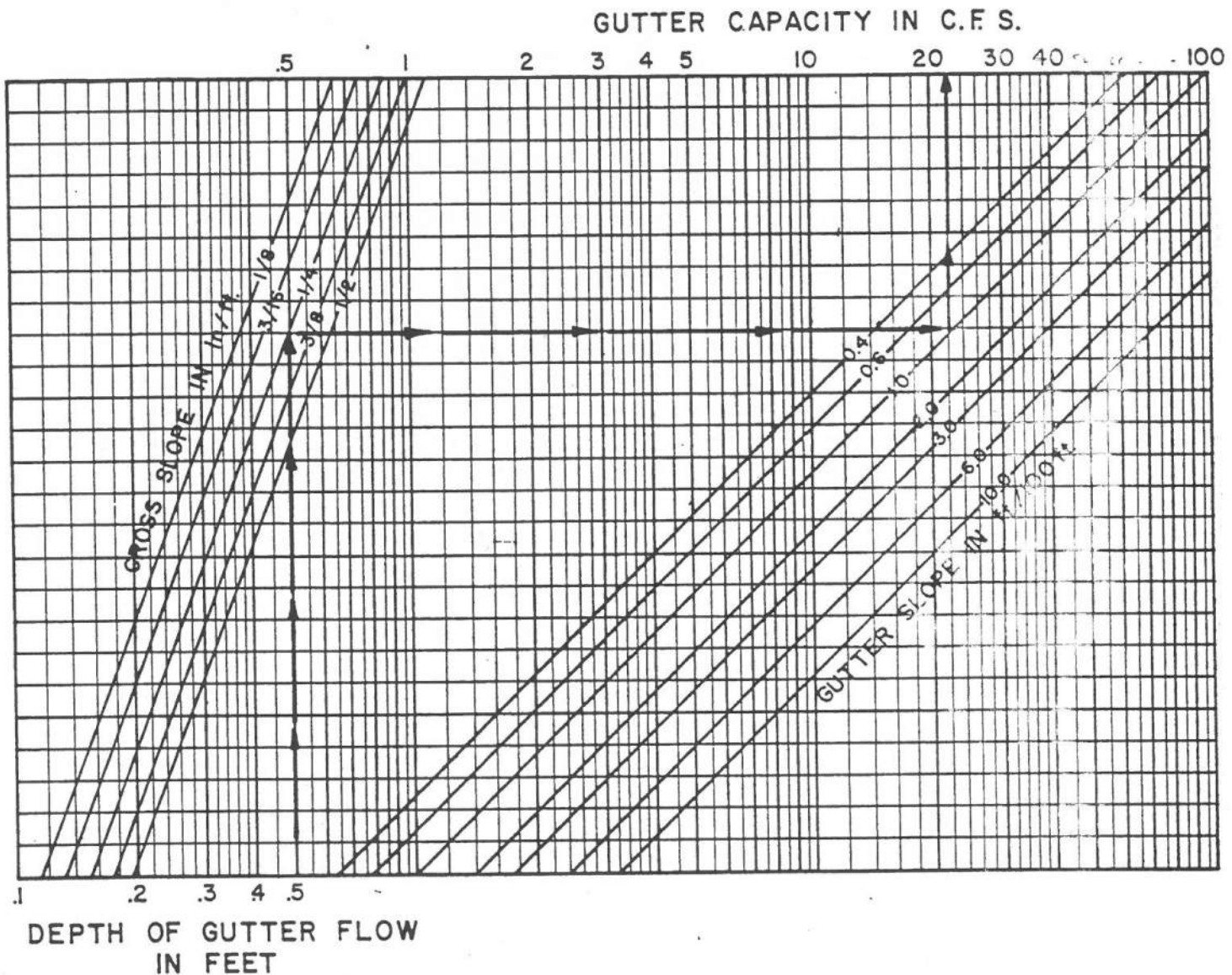
Major Thoroughfare, Type C  
 Pavement Width = 33'  
 Gutter Slope = 1.0%  
 Pavement Cross Slope = 1/4"/1'  
 Depth of Gutter Flow = .5'

### Find:

Gutter Capacity

### Solution:

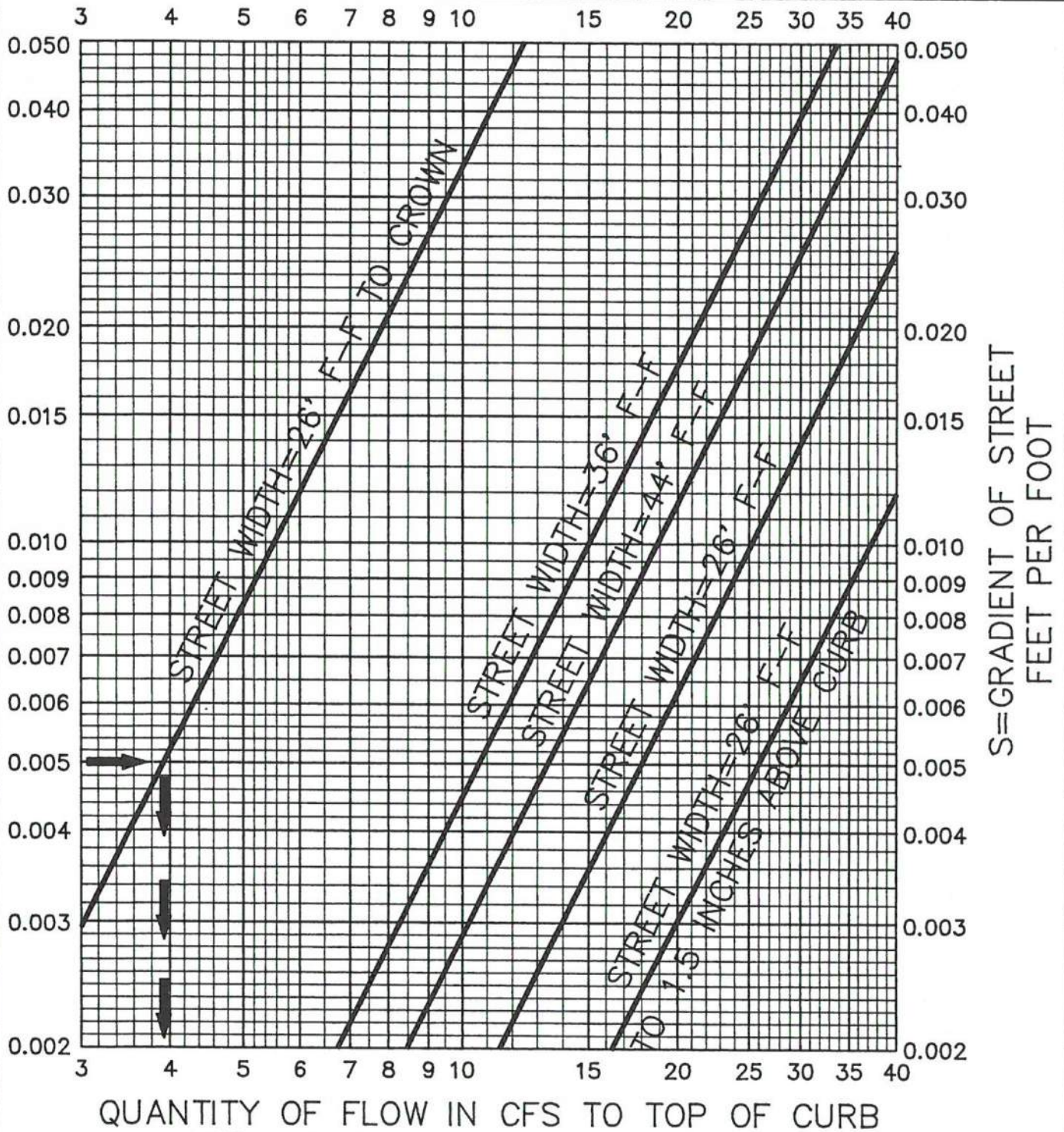
Enter Graph at .5'  
 Intersect Cross Slope = 1/4"/1'  
 Intersect Gutter Slope = 1.0%  
 Read Gutter Capacity = 22 c.f.s.



**CAPACITY OF  
TRIANGULAR GUTTERS**

(Roughness Coefficient  $n = .0175$ )





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

WHERE  $Q_{ST}$ =Flow capacity to top of streets crown  
 $n=0.0175$   
 $A$ =Area carrying flow  
 $R = \frac{A}{P}$  where  $P$ =wetted perimeter  
 $S$ =Gradient of street ft./ft.

$Q_{26'ST} = 55.71 S^{1/2}$  TO CROWN

$Q_{36'ST} = 151.32 S^{1/2}$

$Q_{44'ST} = 185.48 S^{1/2}$

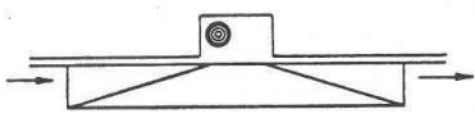
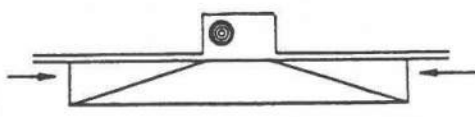
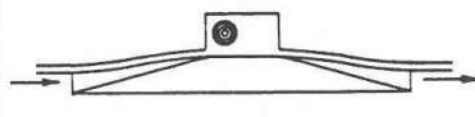
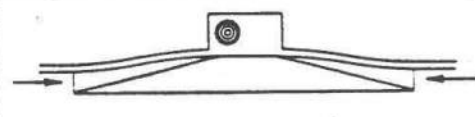
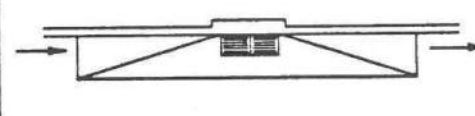
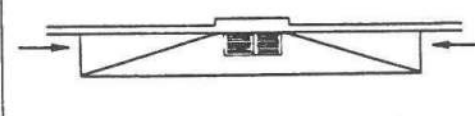

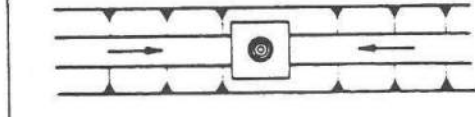
$Q_{26'ST} = 254.43 S^{1/2}$

$Q_{26'ST \text{ to } 1 1/2' \text{ Above T.C.}} = 363.30 S^{1/2}$   
 $(n=0.02)$

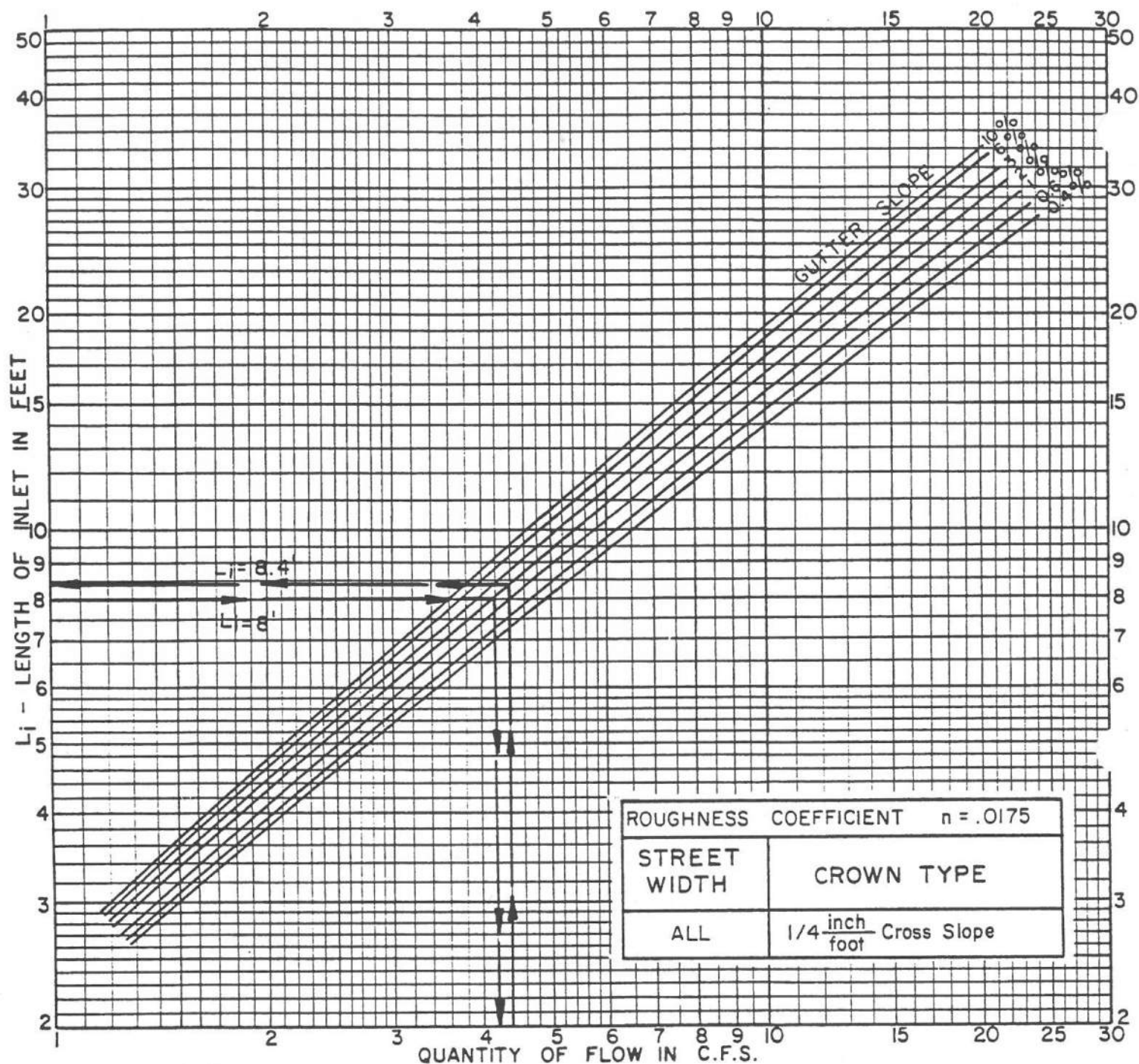
EXAMPLE  
 KNOWN:

STREET WIDTH=26' F-F  
 $S=0.005$   
 $Q_{ST}=3.94$  CFS

PARABOLIC CROWN  
 STREET FLOW CAPACITY

STORM DRAIN INLETS				
INLET TYPE	INLET DESCRIPTION	AVAIL. INLET SIZES	WHERE USED	DESIGN CURVES
I	 <p>STANDARD CURB OPENING INLET ON GRADE</p>	4' 6' 8' 10' 12' 14'	26' LOCAL STREET, TYPE H 36' COLLECTOR STREET, TYPE F ALLEY	FIGURES 8 THROUGH 12
IA	 <p>STANDARD CURB OPENING INLET AT LOW POINT</p>	4' 6' 8' 10' 12' 14'	26' LOCAL STREET, TYPE H 36' COLLECTOR STREET, TYPE F ALLEY	FIGURE 13
II	 <p>RECESSED CURB OPENING INLET ON GRADE</p>	4' 6' 8' 10' 12' 14'	44' COLLECTOR STREET, TYPE F 48' SECONDARY STREET, TYPE E 2-24' MAJOR STREET, TYPE D 2-33' MAJOR STREET, TYPE C 2-36' MAJOR STREET, TYPE B 2-36' MAJOR STREET, TYPE A	FIGURES 8 THROUGH 12
IIA	 <p>RECESSED CURB OPENING INLET AT LOW POINT</p>	4' 6' 8' 10' 12' 14'	44' COLLECTOR STREET, TYPE F 48' SECONDARY STREET, TYPE E 2-24' MAJOR STREET, TYPE D 2-33' MAJOR STREET, TYPE C 2-36' MAJOR STREET, TYPE B 2-36' MAJOR STREET, TYPE A	FIGURE 13
III	 <p>COMBINATION INLET ON GRADE</p>	4' 6' 8'	COMBINATION INLETS TO BE USED WHERE SPACE BEHIND CURB PROHIBITS OTHER INLET TYPES	FIGURES 14 THROUGH 16
IIIA	 <p>COMBINATION INLET AT LOW POINT</p>	4' 6' 8'	COMBINATION INLETS TO BE USED WHERE SPACE BEHIND CURB PROHIBITS OTHER INLET TYPES	FIGURE 20
IV	 <p>GRATE INLETS</p>	2 GRATE 3 GRATE 4 GRATE 6 GRATE	GRATE INLETS TO BE USED WHERE SPACE RESTRICTIONS PROHIBIT OTHER INLET TYPES OR AT LO- CATIONS WITH NO CURB.	FIGURES 16, 17, 18, 19 & 21
V	 <p>DROP INLET</p>	2 x 2' 3 x 3' 4 x 4'	OPEN CHANNELS	FIGURE 22





### EXAMPLE

#### Known:

Pavement Width = 24'  
 Gutter Slope = 2.0 %  
 Pavement Cross Slope =  $1/4'' / 1'$   
 Gutter Flow = 4.4 cfs

#### Find:

Length of Inlet Required ( $L_i$ )

#### Solution:

Enter Graph at 4.4 cfs  
 Intersect Slope = 2.0 %  
 Read  $L_i = 8.4'$

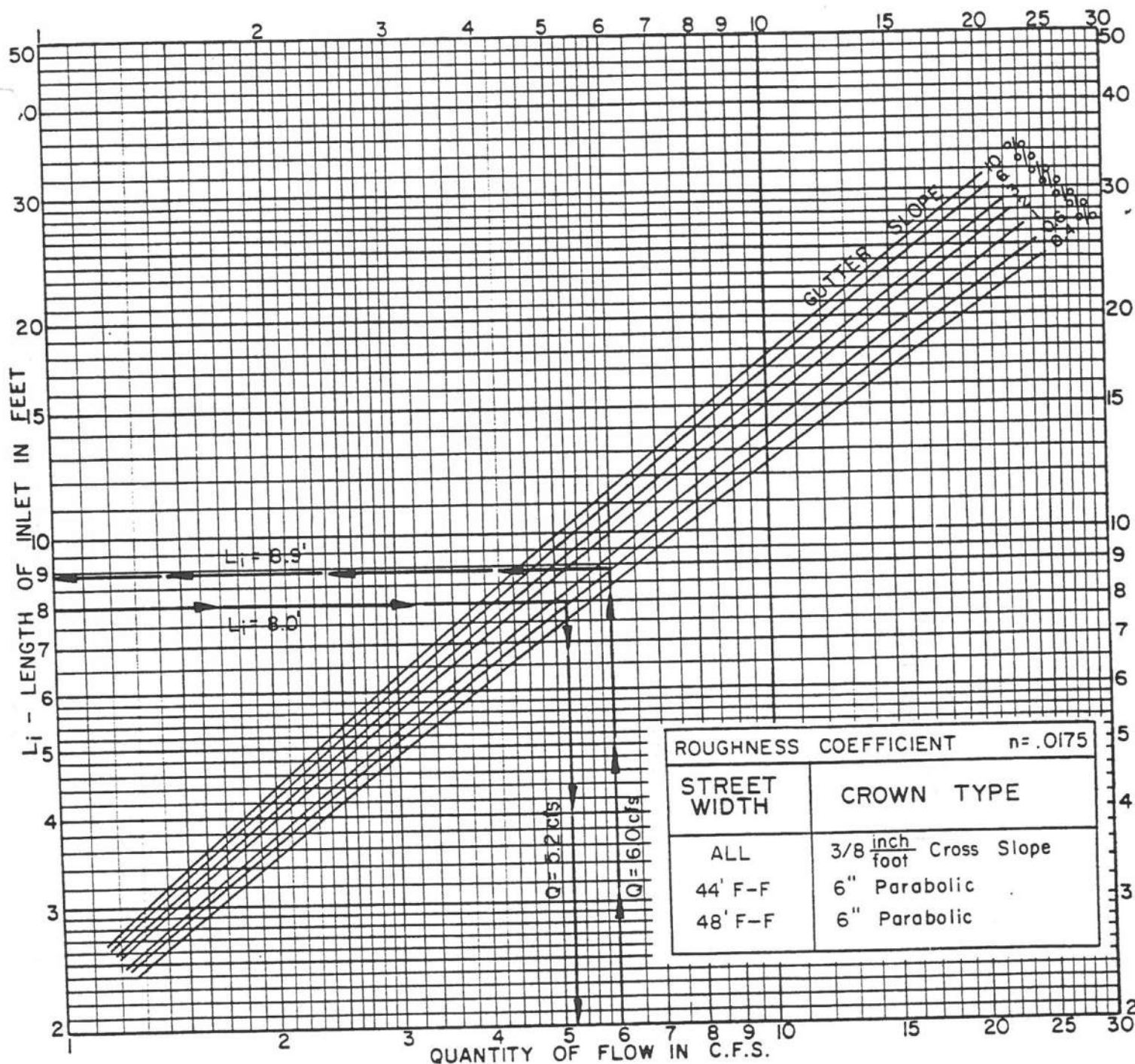
#### Decision:

1. Use 10' Inlet  
 No Flow Remains in Gutter  
 2. Use 8' Inlet  
 Intercept Only Part of Flow

#### Use 8' Inlet

Enter Graph at  $L_i = 8'$   
 Intersect Slope = 2.0 %  
 Read  $Q = 4.2$  cfs  
 Remaining Gutter Flow =  
 $4.4 \text{ cfs} - 4.2 \text{ cfs} = 0.2 \text{ cfs}$

RECESSED AND STANDARD  
 CURB OPENING INLET  
 CAPACITY CURVES  
 ON GRADE



### EXAMPLE

Known:

Pavement Width = 44'  
 Gutter Slope = 0.6 %  
 6" Parabolic Crown  
 Gutter Flow = 6.0 cfs

Find:

Length of Inlet Required ( $L_i$ )

Solution:

Enter Graph at 6.0 cfs  
 Intersect Slope = 0.6 %  
 Read  $L_i = 8.9'$

Decision:

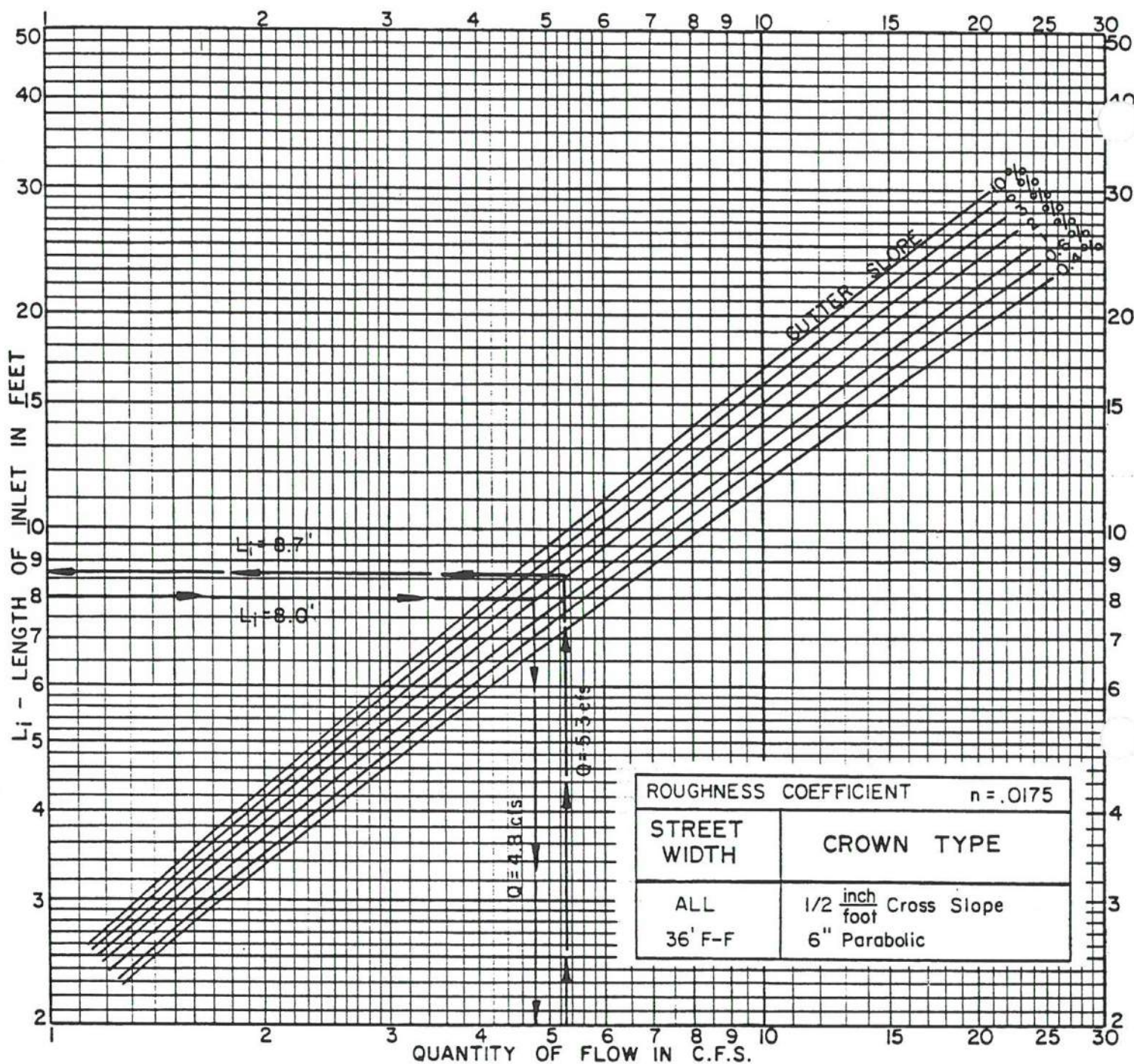
1. Use 10' Inlet  
 No Flow Remains in Gutter
2. Use 8' Inlet  
 Intercept Only Part of Flow

Use 8' Inlet

Enter Graph at  $L_i = 8'$   
 Intersect Slope = 0.6 %  
 Read  $Q = 5.2$  cfs  
 Remaining Gutter Flow =  
 $6.0 \text{ cfs} - 5.2 \text{ cfs} = 0.8 \text{ cfs}$

RECESSED AND STANDARD  
 CURB OPENING INLET  
 CAPACITY CURVES  
 ON GRADE





### EXAMPLE

#### Known:

Pavement Width = 36'  
 Gutter Slope = 2%  
 6" Parabolic Crown  
 Gutter Flow = 5.3 cfs

#### Find:

Length of Inlet Required ( $L_i$ )

#### Solution:

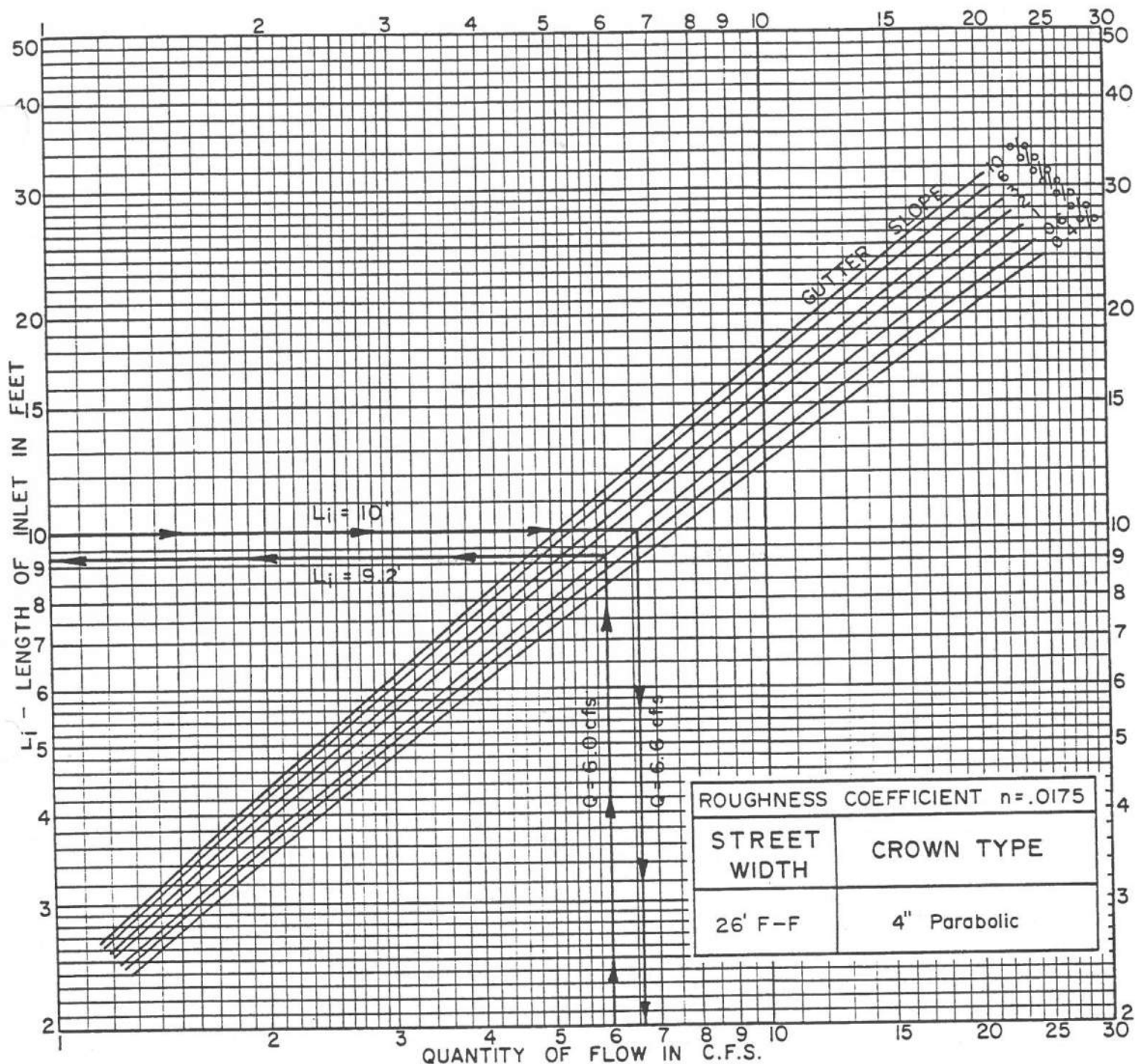
Enter Graph at 5.3 cfs  
 Intersect Slope = 2%  
 Read  $L_i = 8.7'$

#### Decision:

1. Use 10' Inlet  
 No Flow Remains in Gutter  
 2. Use 8' Inlet  
 Intercept Only Part of Flow  
 Use 8' Inlet

Enter Graph at  $L_i = 8'$   
 Intersect Slope = 2%  
 Read  $Q = 4.8$  cfs  
 Remaining Gutter Flow =  
 $5.3 \text{ cfs} - 4.8 \text{ cfs} = 0.5 \text{ cfs}$

RECESSED AND STANDARD  
 CURB OPENING INLET  
 CAPACITY CURVES  
 ON GRADE



### EXAMPLE

#### Known:

Pavement Width = 26'  
 Gutter Slope = 1%  
 4" Parabolic Crown  
 Gutter Flow = 6.0 cfs

#### Find:

Length of Inlet Required ( $L_i$ )

#### Solution:

Enter Graph at 6.0 cfs  
 Intersect Slope = 1%  
 Read  $L_i = 9.2'$

#### Decision:

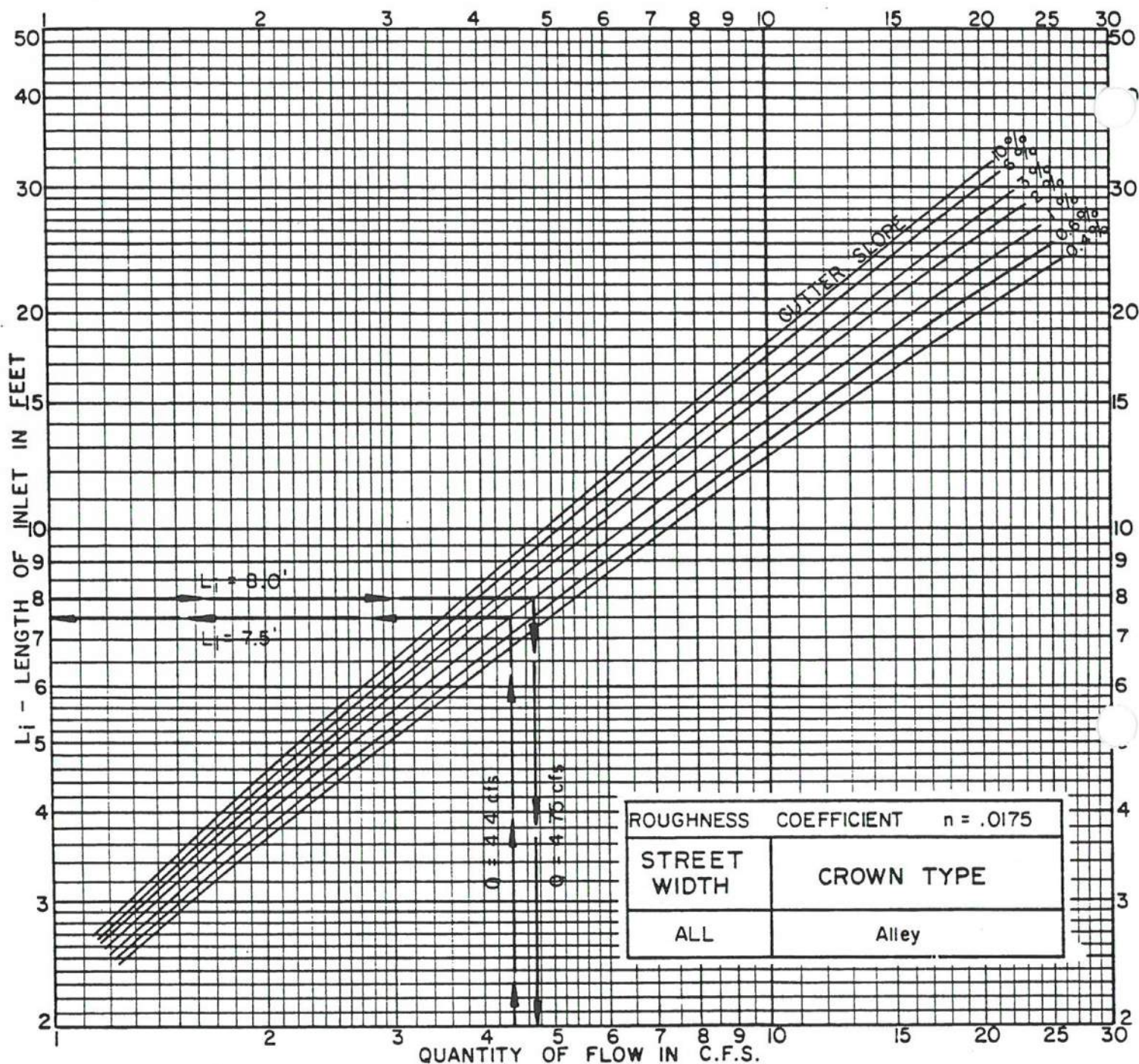
1. Use 10' Inlet  
 No Flow Remains in Gutter
2. Use 8' Inlet  
 Intercept Only Part of Flow

#### Use 10' Inlet

Enter Graph at  $L_i = 10'$   
 Intersect Slope = 1%  
 Read  $Q = 6.6$  cfs  
 No Flow Remains in Gutter

RECESSED AND STANDARD  
 CURB OPENING INLET  
 CAPACITY CURVES  
 ON GRADE





### EXAMPLE

Known:

Pavement Width = 16'  
 Gutter Slope = 1%  
 Pavement Cross Slope = 1/4"/1'  
 Gutter Flow = 4.4 cfs

Find:

Length of Inlet Required ( $L_i$ )

Solution:

Enter Graph at 4.4 cfs  
 Intersect Slope = 1%  
 Read  $L_i = 7.5'$

Decision:

1. Use 8' Inlet  
 No Flow Remains in Gutter  
 2. Use 6' Inlet  
 Intercept Only Part of Flow

Use 8' Inlet

Enter Graph at  $L_i = 8'$   
 Intersect Slope = 1%  
 Read  $Q = 4.75$  cfs  
 No Flow Remains in Gutter

RECESSED AND STANDARD  
 CURB OPENING INLET  
 CAPACITY CURVES  
 ON GRADE

# EXAMPLE

## Known:

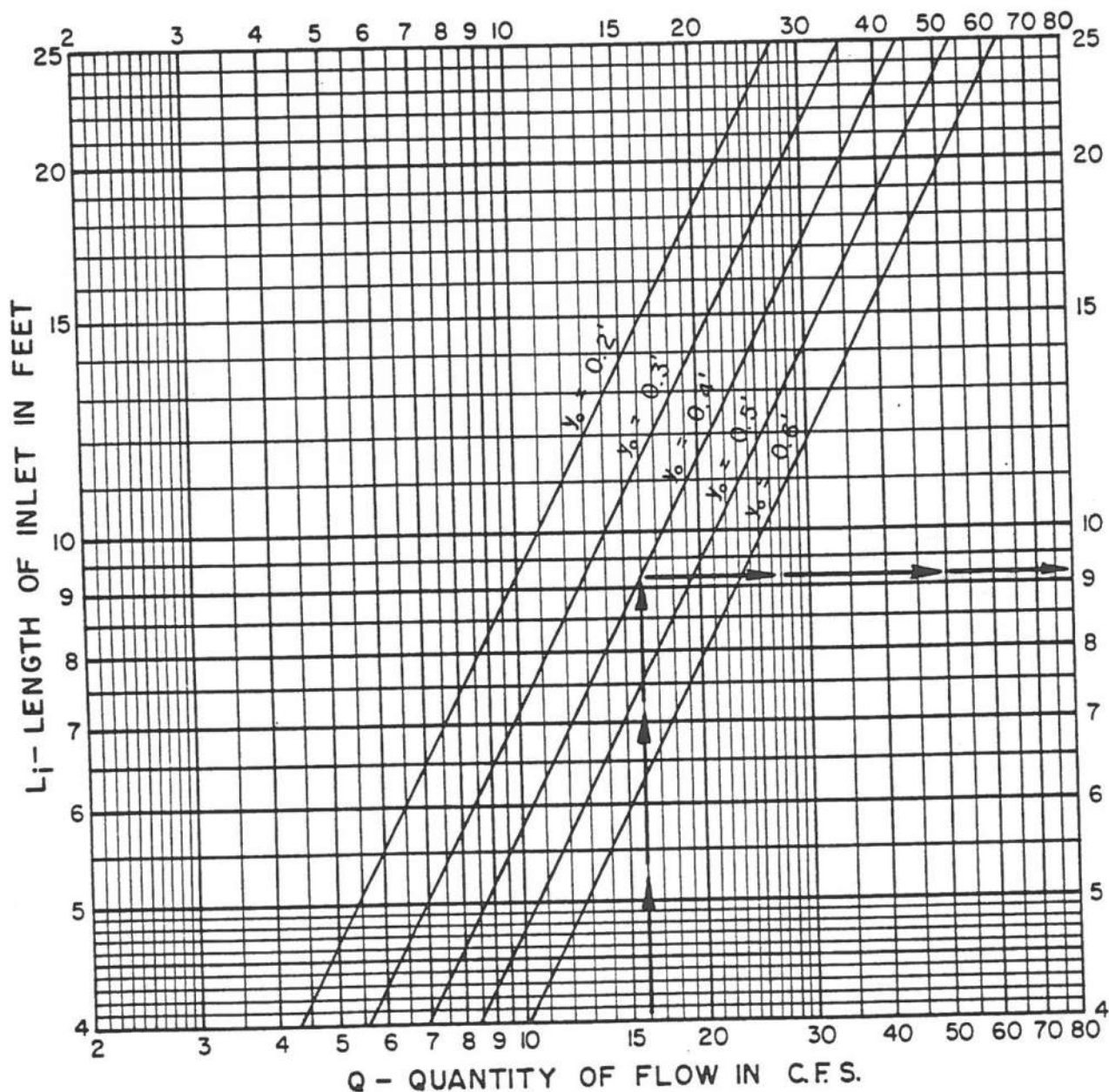
Quantity of Flow = 16.0 c.f.s.  
Maximum Depth of Flow Desired  
in Gutter At Low Point ( $y_o$ ) = 0.4'

## Find:

Length of Inlet Required ( $L_i$ )

## Solution:

Enter Graph at 16.0 c.f.s.  
Intersect  $y_o = 0.4'$   
Read  $L_i = 9.2'$   
Use 10' Inlet



ROUGHNESS COEFFICIENT $n = .0175$	
STREET WIDTH	CROWN TYPE
ALL	Straight and Parabolic

RECESSED AND STANDARD  
CURB OPENING INLET  
CAPACITY CURVES  
AT LOW POINT



## EXAMPLE

### Known:

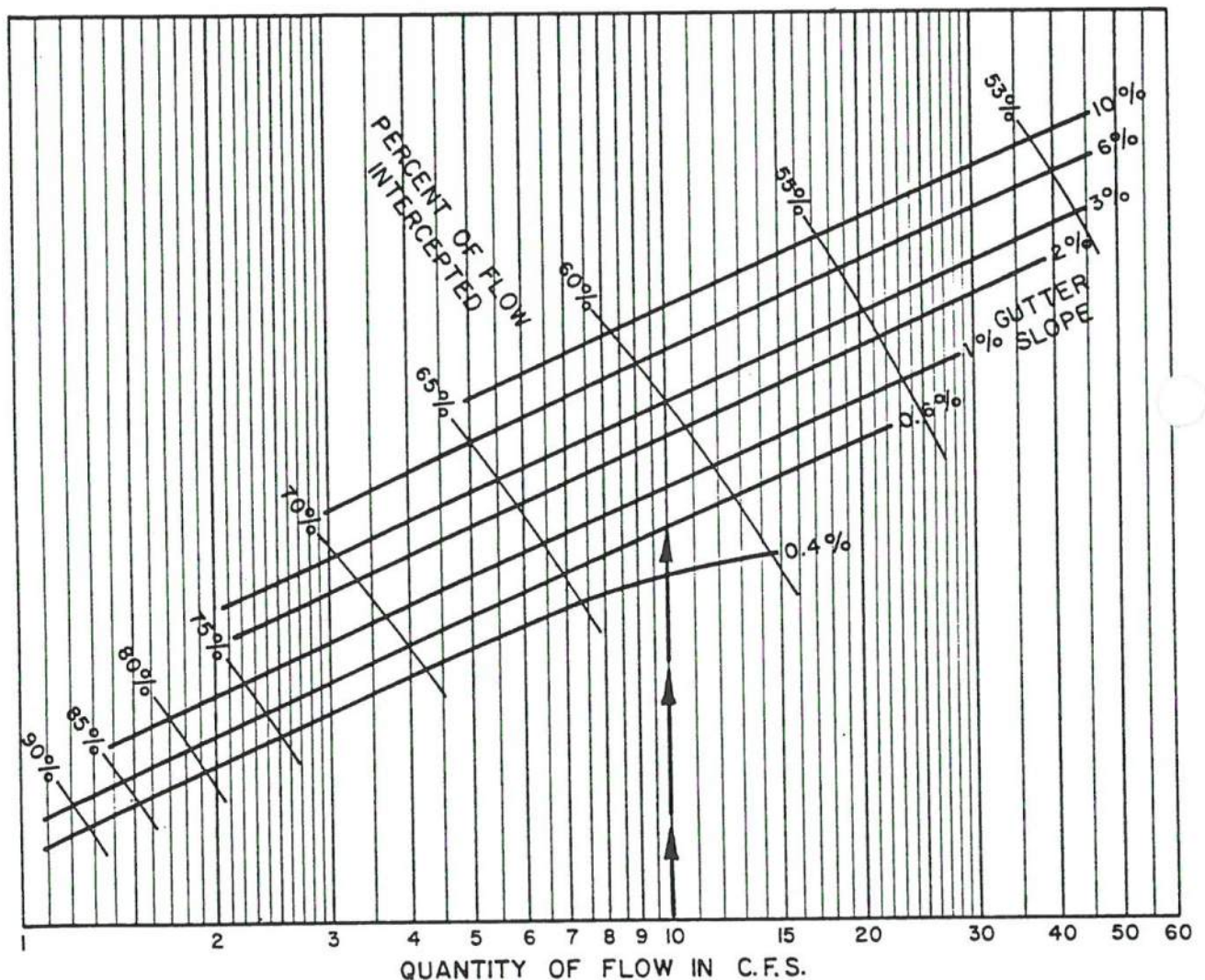
Quantity of Flow = 10.0 c.f.s.  
Gutter Slope = 0.6 %

### Find:

Capacity of Two Grate Combination  
Inlet

### Solution:

Enter Graph at 10.0 c.f.s.  
Intersect Slope = 0.6 %  
Read Percent of Flow  
Intercepted = 62 %  
62 % of 10.0 c.f.s. = 6.2 c.f.s.  
as Capacity of Two Grate  
Combination Inlet  
Remaining Gutter Flow =  
 $10.0 \text{ c.f.s.} - 6.2 \text{ c.f.s.} = 3.8 \text{ c.f.s.}$



TWO GRATE COMBINATION INLET  
CAPACITY CURVES  
ON GRADE

## EXAMPLE

### Known:

Quantity of Flow = 6.0 c.f.s.

Gutter Slope = 1.0 %

### Find:

Capacity of Four Grate Combination Inlet

### Solution:

Enter Graph at 6.0 c.f.s.

Intersect Slope = 1.0 %

Read Percent of Flow

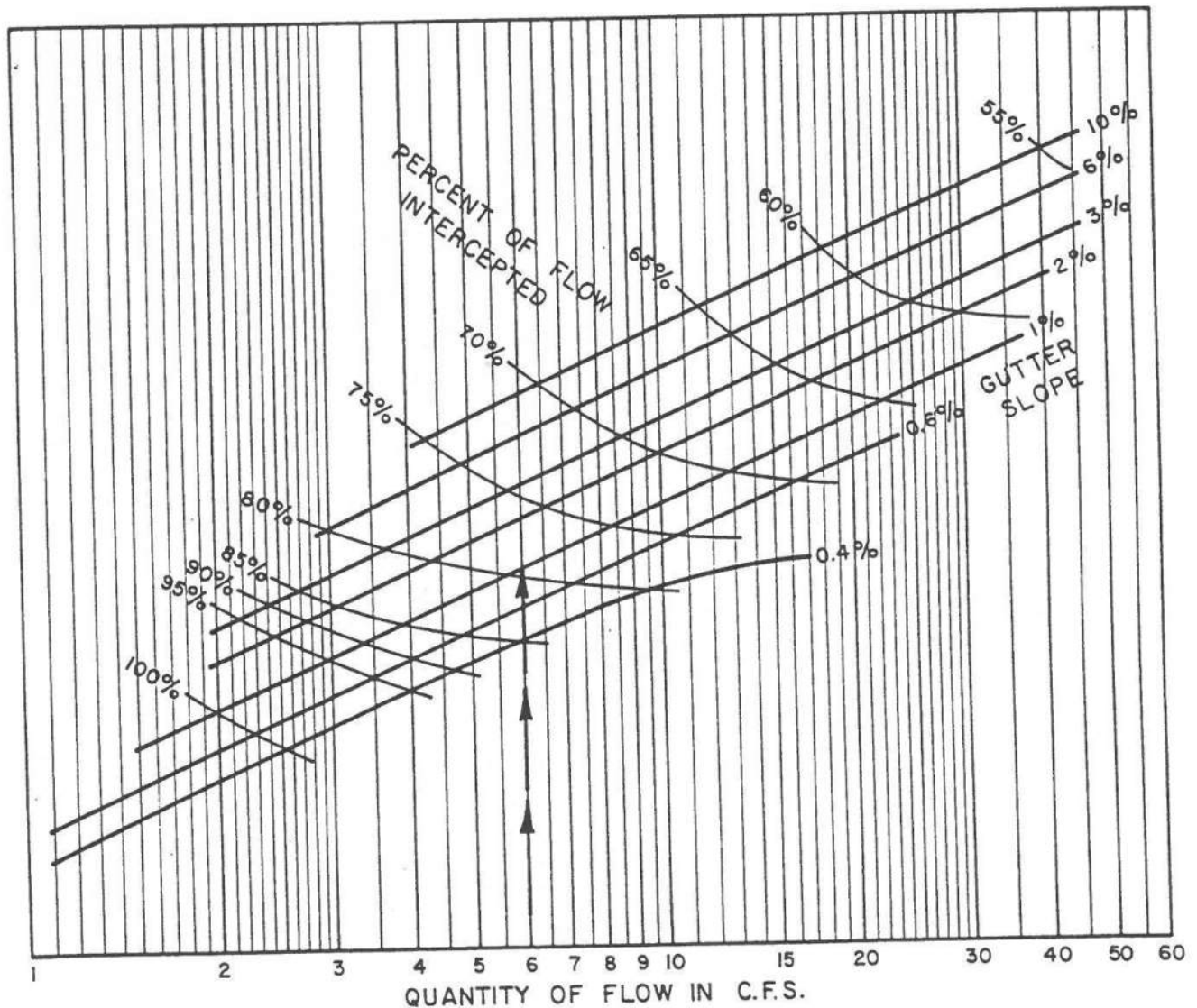
Intercepted = 79 %

79 % of 6.0 c.f.s. = 4.7 c.f.s.

as Capacity of Four Grate Combination Inlet

Remaining Gutter Flow =

6.0 c.f.s. - 4.7 c.f.s. = 1.3 c.f.s.



FOUR GRATE COMBINATION INLET  
CAPACITY CURVES  
ON GRADE



## EXAMPLE

### Known:

Quantity of Flow = 8.0 c.f.s.

Gutter Slope = 0.4%

### Find:

Capacity of Three Grate Inlet

### Solution:

Enter Graph at 8.0 c.f.s.

Intersect Slope = 0.4%

Read Percent of Flow

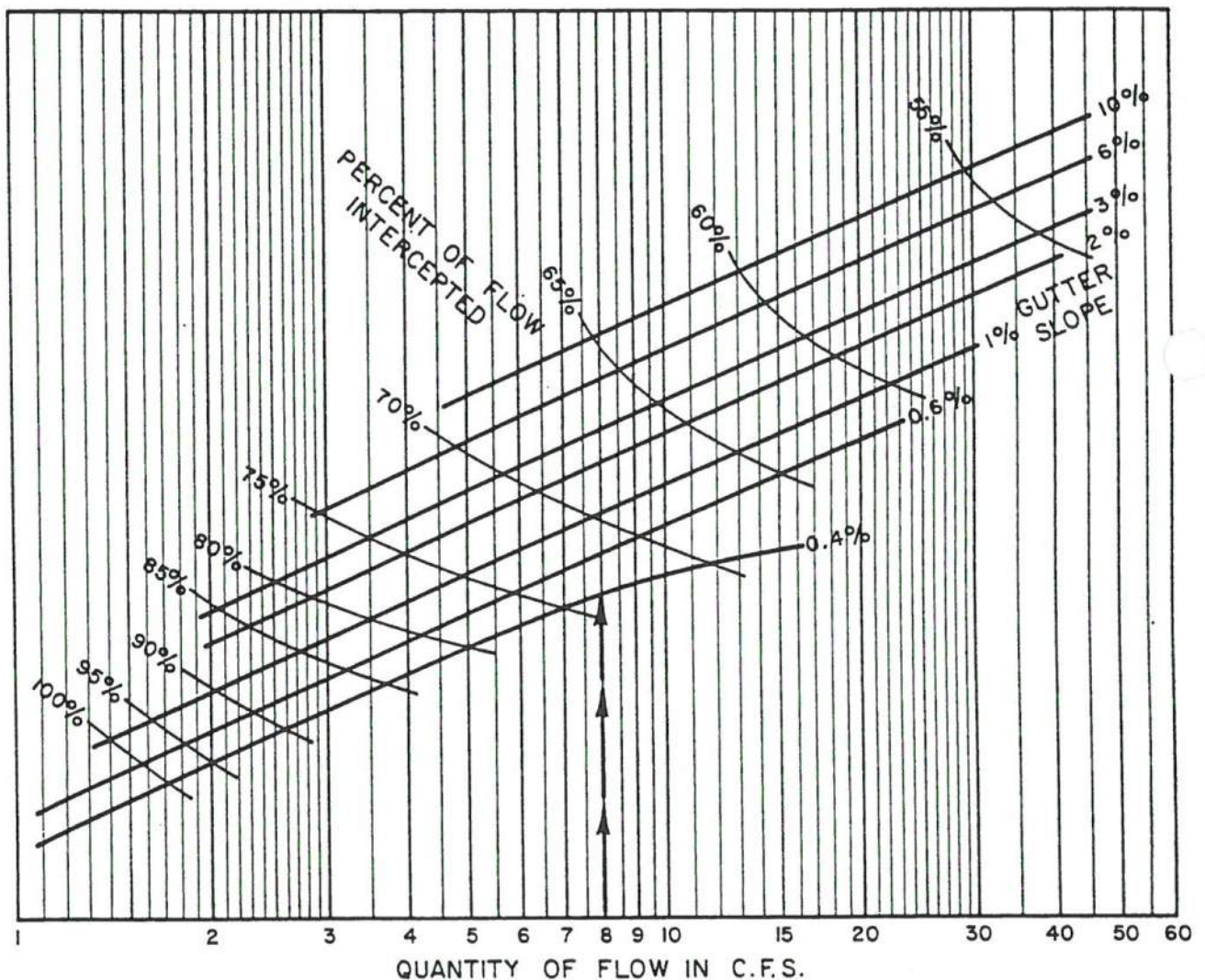
Intercepted = 74 %

74 % of 8.0 c.f.s. = 5.9 c.f.s.

as Capacity of Three Grate Inlet

Remaining Gutter Flow =

8.0 c.f.s. - 5.9 c.f.s. = 2.1 c.f.s.



THREE GRATE INLET AND  
THREE GRATE COMBINATION INLE  
CAPACITY CURVES  
ON GRADE

## EXAMPLE

### Known:

Quantity of Flow = 6.0 c.f.s.

Gutter Slope = 1.0%

### Find:

Capacity of Two Grate Inlet

### Solution:

Enter Graph at 6.0 c.f.s.

Intersect Slope = 1.0%

Read Percent of Flow

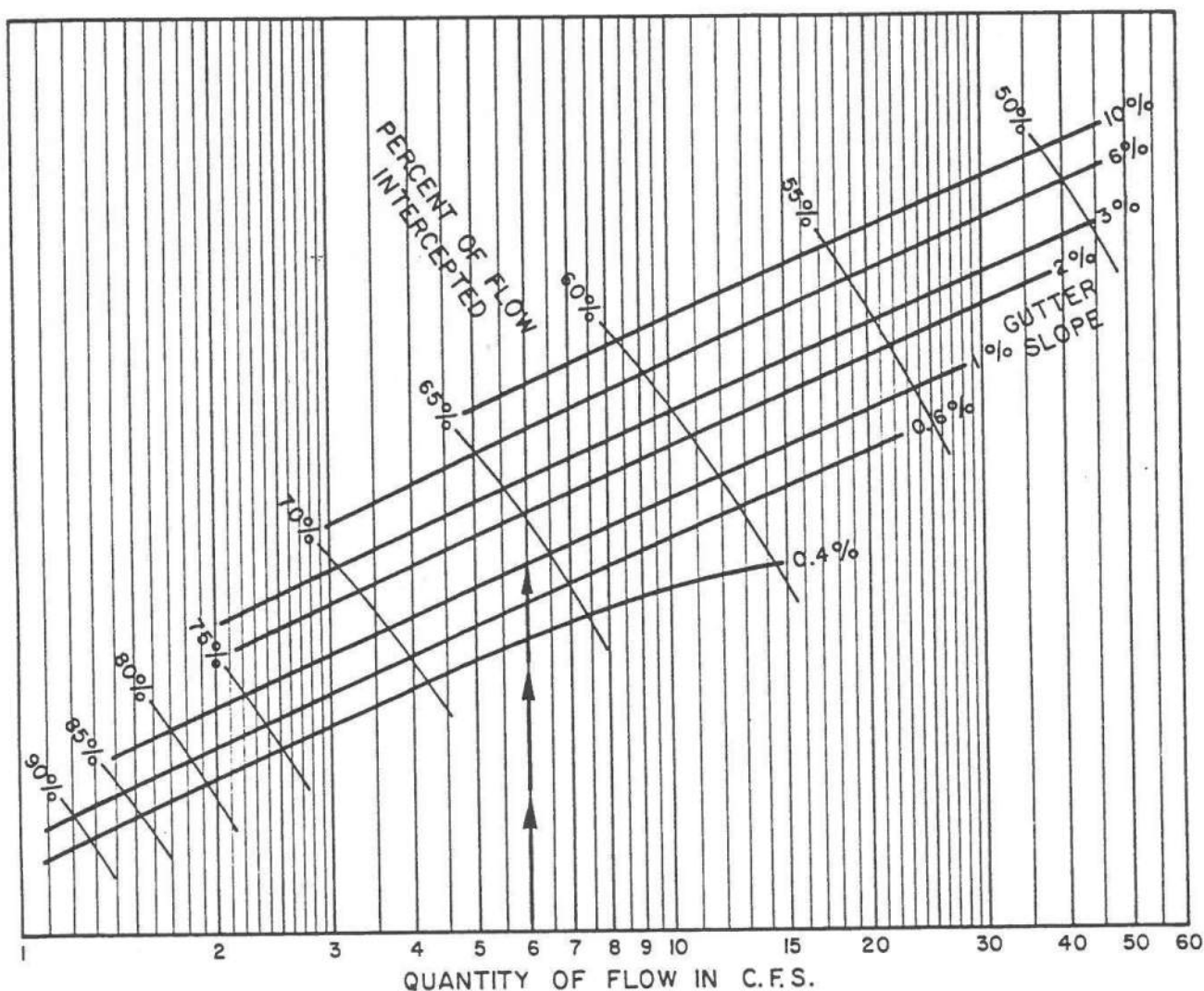
Intercepted = 66%

66% of 6.0 c.f.s. = 4.0 c.f.s.

as Capacity of Two Grate Inlet

Remaining Gutter Flow =

6.0 c.f.s. - 4.0 c.f.s. = 2.0 c.f.s.



TWO GRATE INLET  
CAPACITY CURVES  
ON GRADE

## EXAMPLE

### Known:

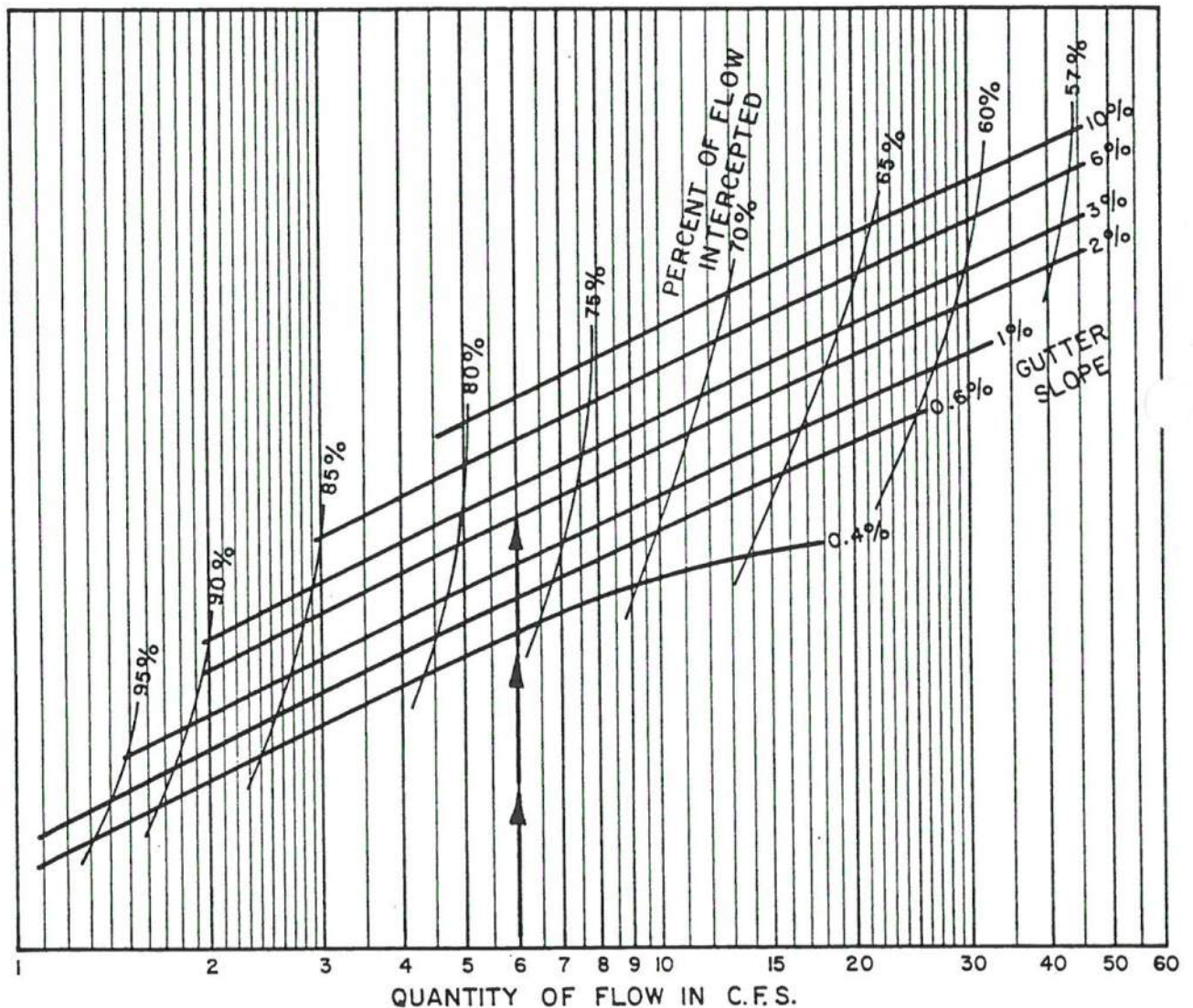
Quantity of Flow = 6.0 c.f.s.  
Gutter Slope = 1.0%

### Find:

Capacity of Four Grate Inlet

### Solution:

Enter Graph at 6.0 c.f.s.  
Intersect Slope = 1.0%  
Read Percent of Flow  
Intercepted = 77 %  
77% of 6.0 c.f.s. = 4.6 c.f.s.  
as Capacity of Four Grate Inlet  
Remaining Gutter Flow =  
6.0 c.f.s. - 4.6 c.f.s. = 1.4 c.f.s.



FOUR GRATE INLET  
CAPACITY CURVES  
ON GRADE



## EXAMPLE

### Known:

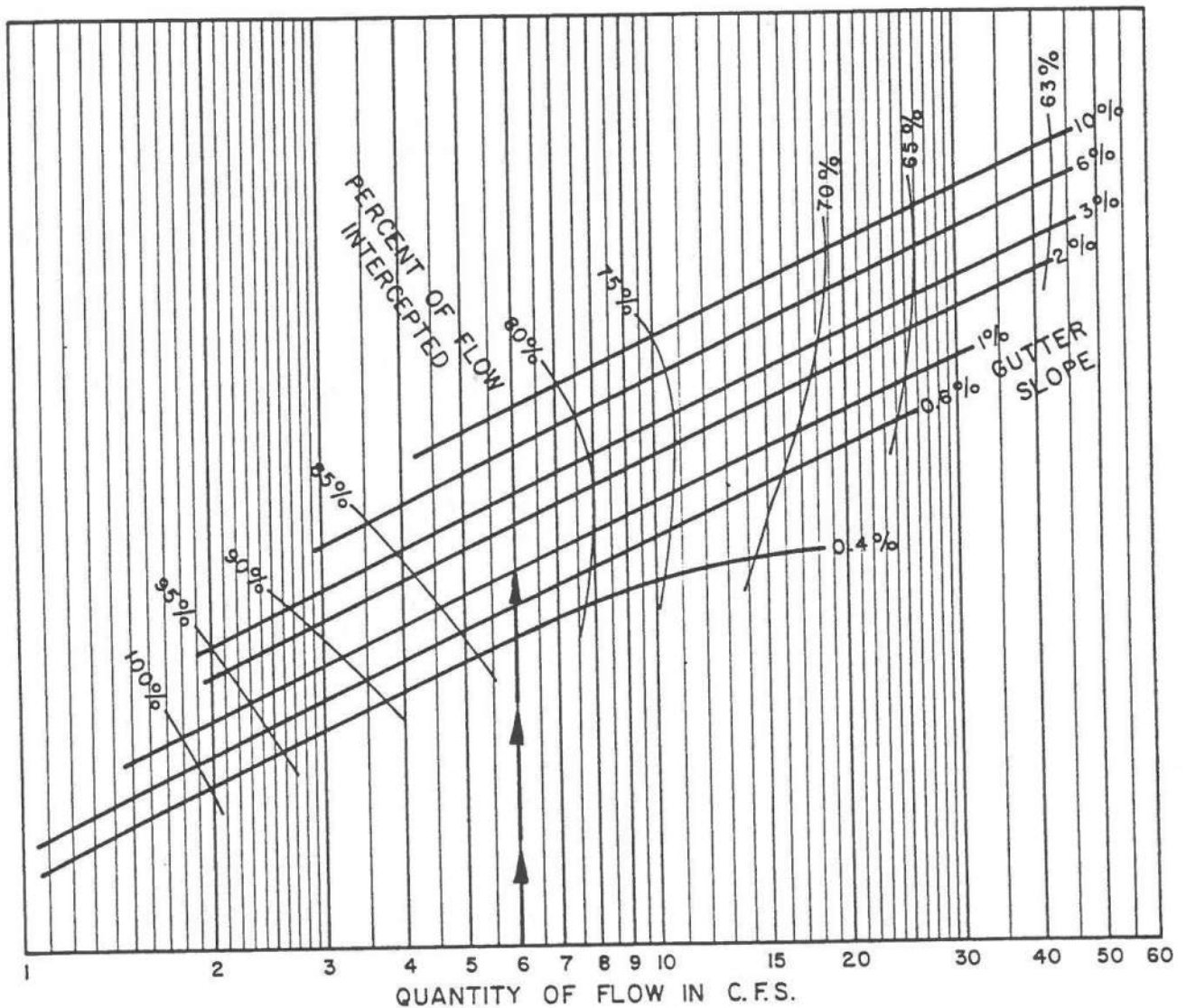
Quantity of Flow = 6.0 c.f.s.  
Gutter Slope = 1.0%

### Find:

Capacity of Six Grate Inlet

### Solution:

Enter Graph at 6.0 c.f.s.  
Intersect Slope = 1.0%  
Read Percent of Flow  
Intercepted = 82 %  
82 % of 6.0 c.f.s. = 4.9 c.f.s.  
as Capacity of Six Grate Inlet  
Remaining Gutter Flow =  
6.0 c.f.s. - 4.9 c.f.s. = 1.1 c.f.s.



SIX GRATE INLET  
CAPACITY CURVES  
ON GRADE

# EXAMPLE

## Known:

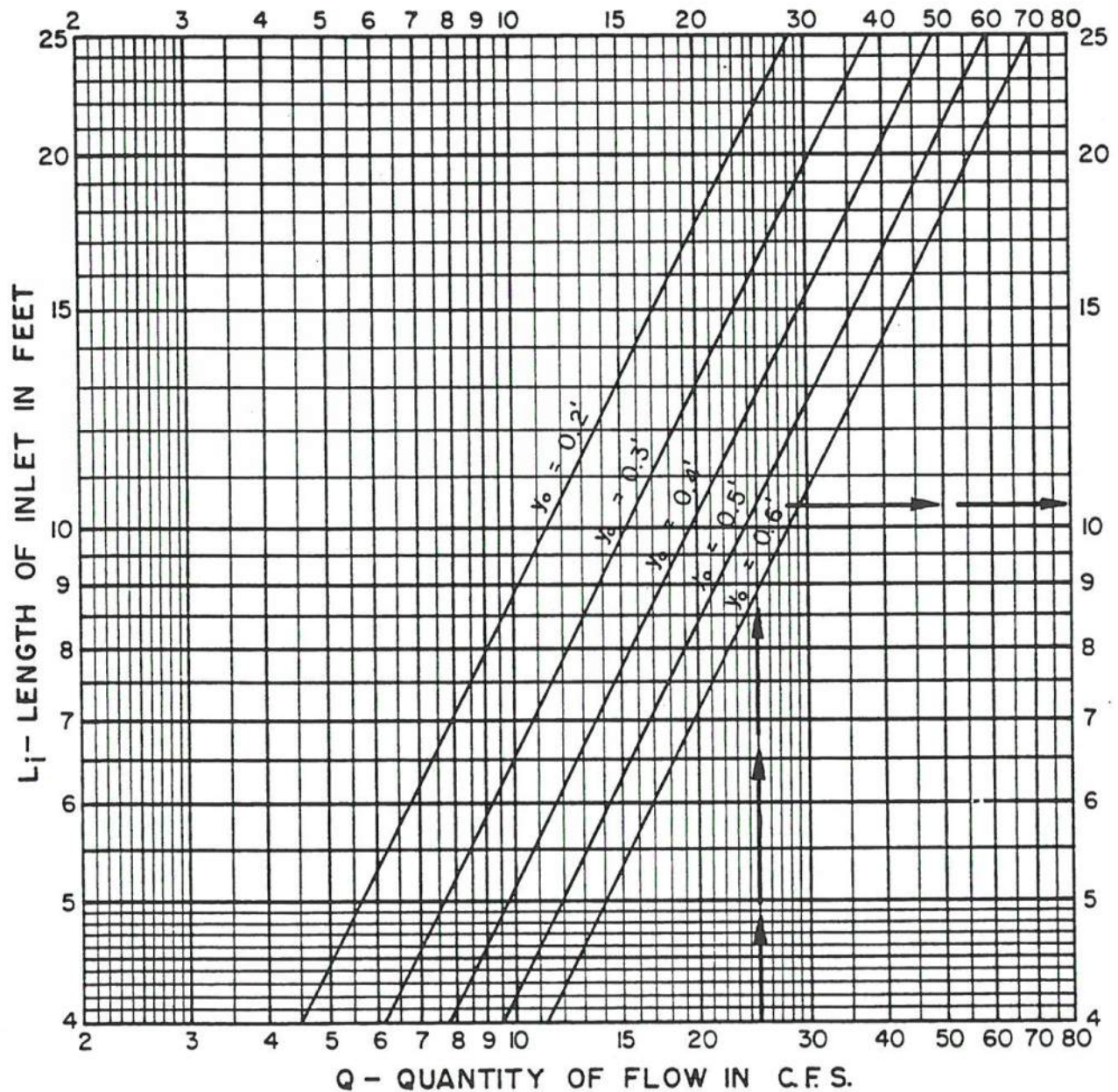
Quantity of Flow = 25.0 c.f.s.  
Maximum Depth of Flow Desired  
At Low Point ( $y_o$ ) = 0.5'

## Find:

Length of Inlet Required ( $L_i$ )

## Solution:

Enter Graph at 25.0 c.f.s.  
Intersect  $y_o = 0.5'$   
Read  $L_i = 10.4'$   
Use 12' Inlet



ROUGHNESS COEFFICIENT $n = .0175$	
STREET WIDTH	CROWN TYPE
ALL	Straight and Parabolic

COMBINATION INLET  
CAPACITY CURVES  
AT LOW POINT

### EXAMPLE

#### Known:

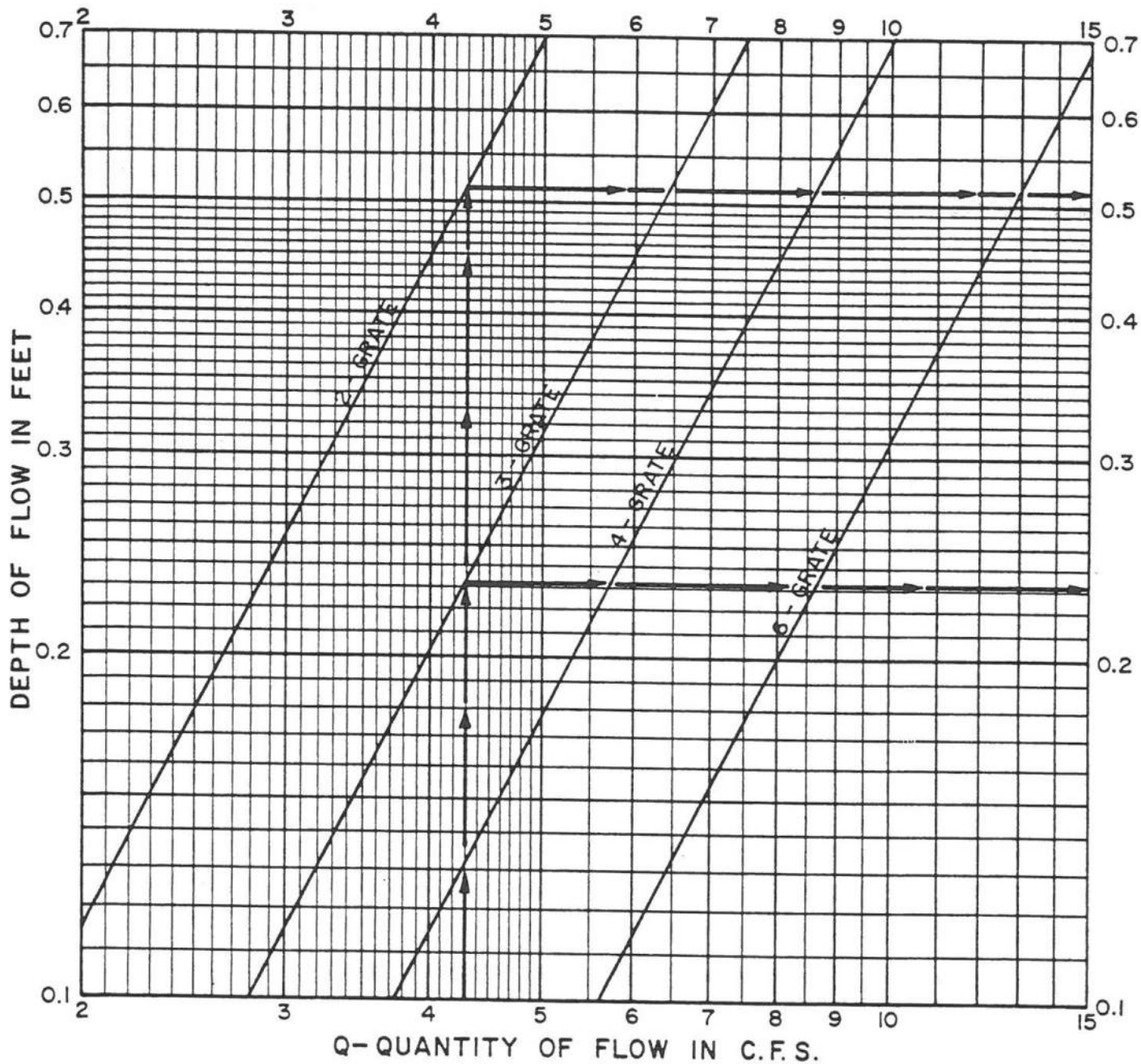
Quantity of Flow = 4.3 c.f.s.  
Maximum Depth of Flow Desired  
at Low Point = 0.3'

#### Find:

Inlet Required

#### Solution:

Enter Graph at 4.3 c.f.s.  
Intersect 3 - Grate at 0.23'  
Intersect 2 - Grate at 0.51'  
Use 3 - Grate



GRATE INLET  
CAPACITY CURVES  
AT LOW POINT



# EXAMPLE

## Known:

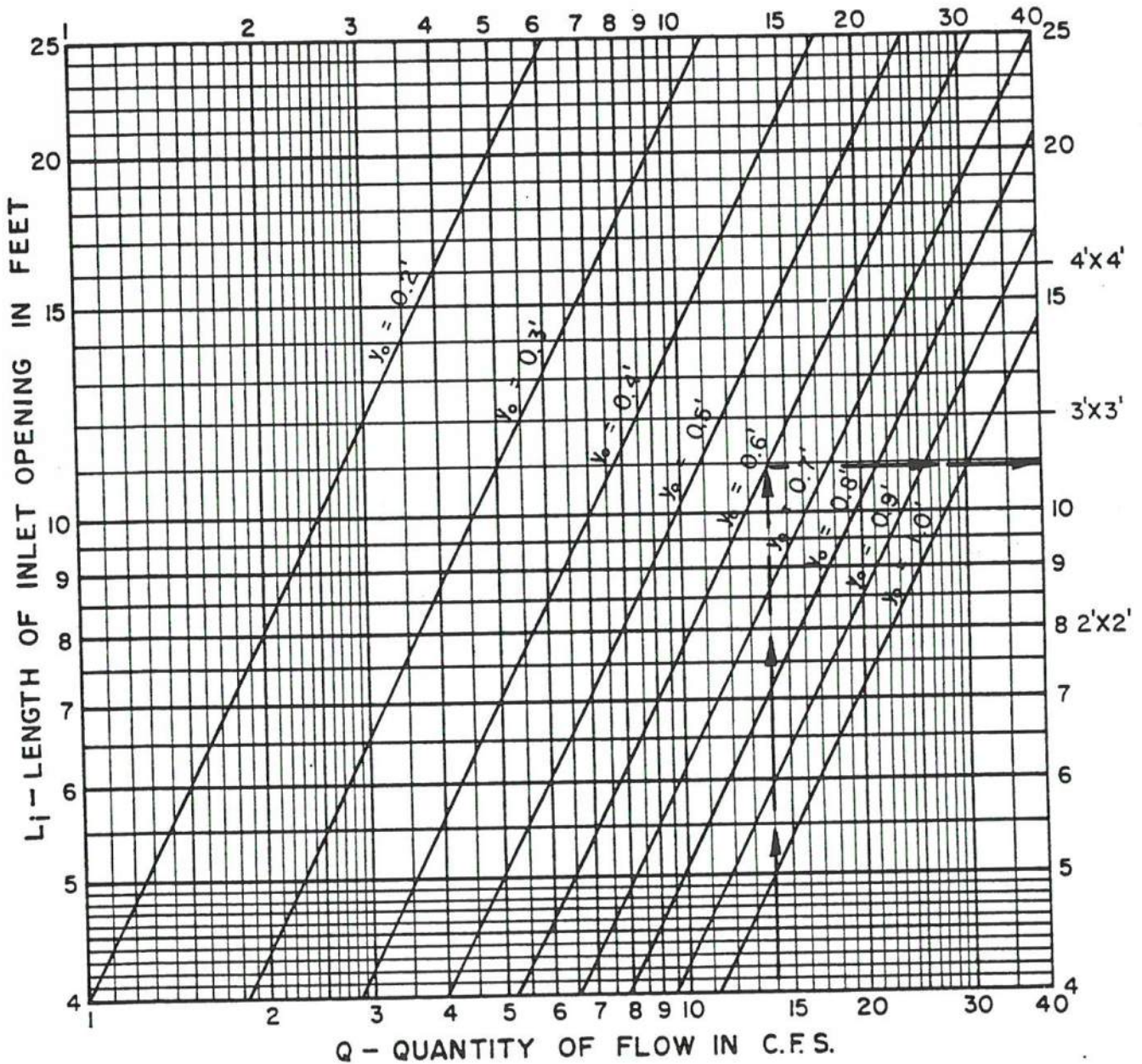
Quantity of Flow = 14.0 c.f.s.  
Maximum Depth of Flow Desired  
( $y_o$ ) = 0.6'

## Find:

Length of Inlet Opening Required ( $L_i$ )

## Solution:

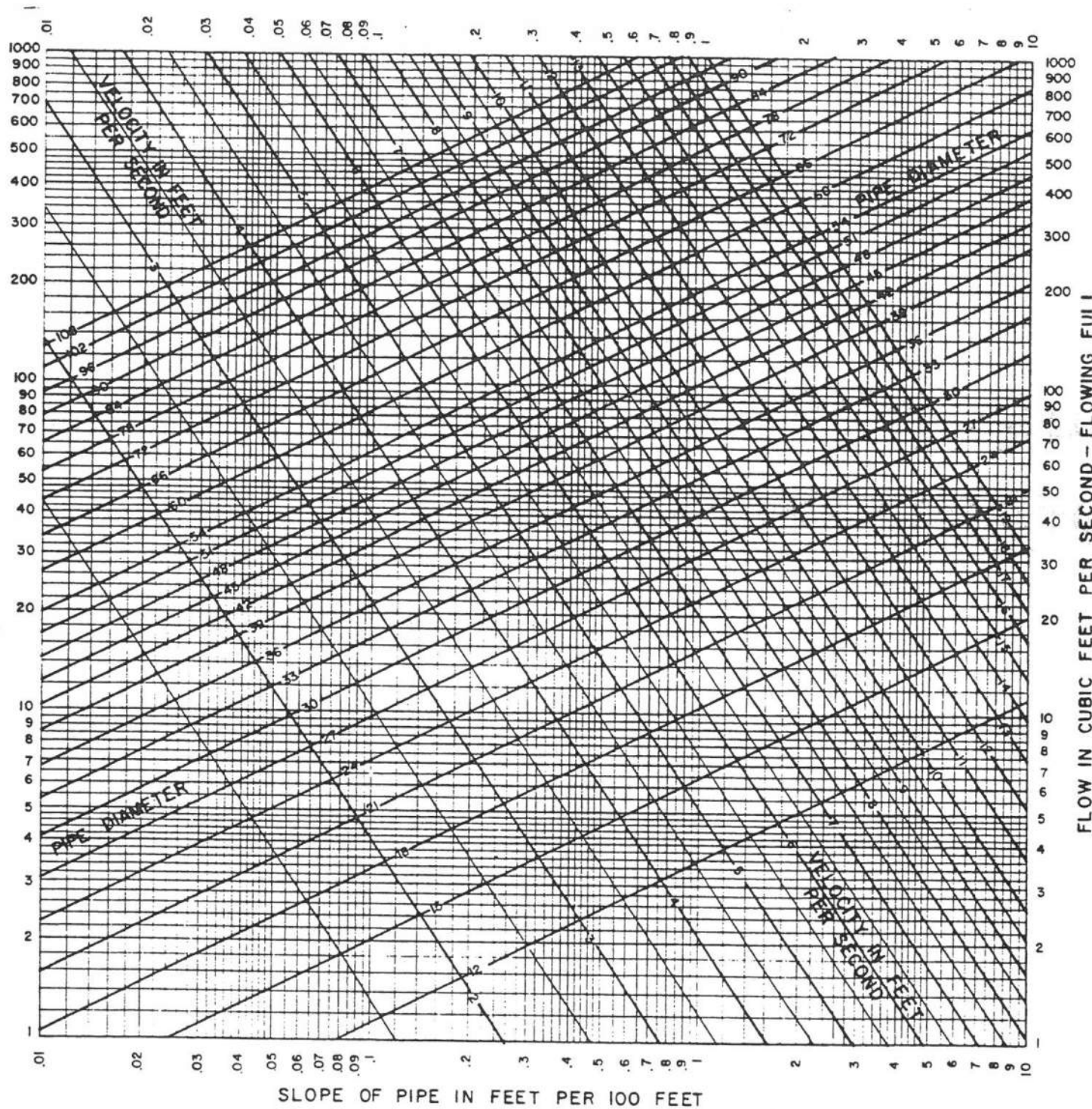
Enter Graph at 14.0 c.f.s.  
Intersect  $y_o = 0.6'$   
Read  $L_i = 10.9'$   
Use 12' of Inlet ; 3'x3'



Standard Drop Inlet Sizes:

2'x2' ;  $L_i = 8'$   
3'x3' ;  $L_i = 12'$   
4'x4' ;  $L_i = 16'$

DROP INLET  
CAPACITY CURVES  
AT LOW POINT



CAPACITY OF CIRCULAR  
PIPES FLOWING FULL

A GRAPHICAL SOLUTION  
OF  
MANNING'S EQUATION

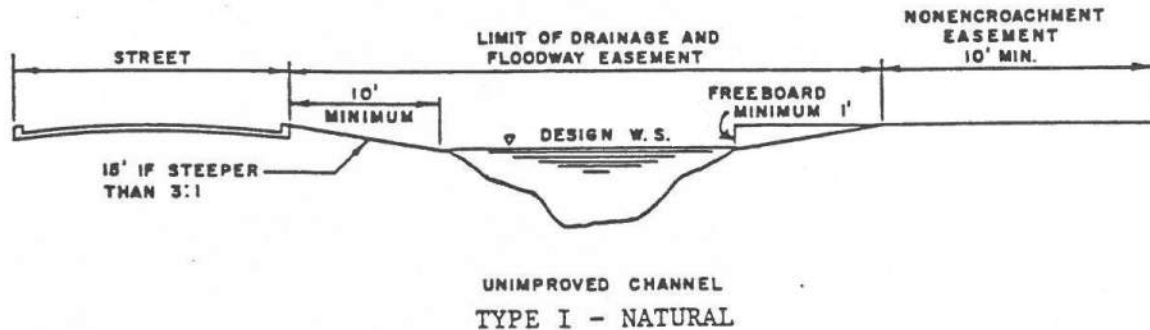
$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

$$n = 0.013$$



CREEKS MAY REMAIN IN OPEN NATURAL CONDITION IF:

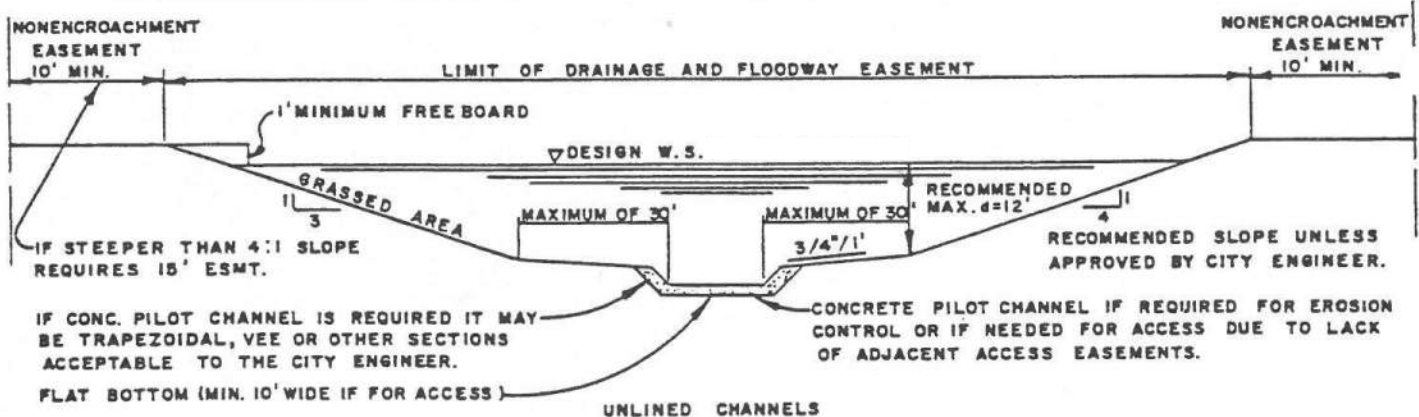
- (1) THEY COMPLY WITH THE SUBDIVISION ORDINANCE;
- (2) TREE COVERAGE IS ADEQUATE TO BE ACCEPTABLE TO THE CITY OF FARMERS BRANCH;
- (3) UNSANITARY OR UNACCEPTABLE DRAINAGE CONDITIONS DO NOT EXIST IN THE CREEK;
- (4) APPROVED BY THE CITY ENGINEER.



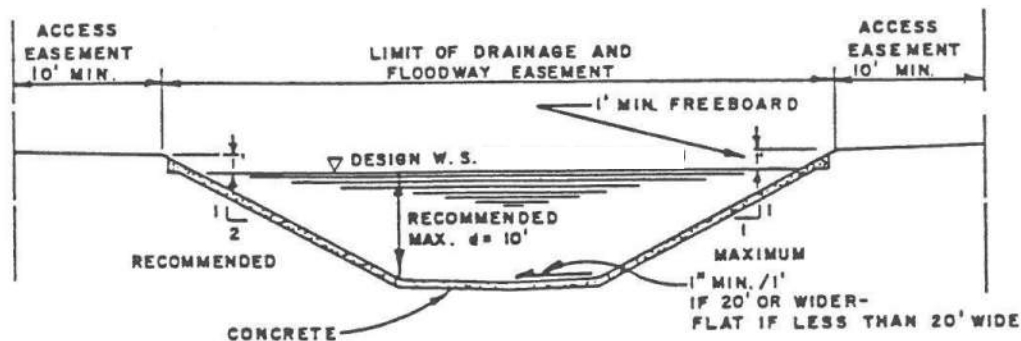
NOTE: TYPE I OR II - IF STEEPER THAN 3:1 SLOPE ABOVE DESIGN W.S., THE NON-ENCROACHMENT ESMT. SHALL BE 15 FEET WIDE TO PROVIDE A STABLE ACCESS ESMT., IF ACCESS HAS NOT OTHERWISE BEEN PROVIDED.

NOTE: A PARALLEL STREET IS RECOMMENDED ON AT LEAST ONE SIDE OF TYPE I CHANNELS IF THE DRAINAGE AND FLOODWAY IS DEDICATED TO PUBLIC USE.

NOTE: NO ENCROACHMENTS SHALL BE PERMITTED IN ACCESS EASEMENTS.



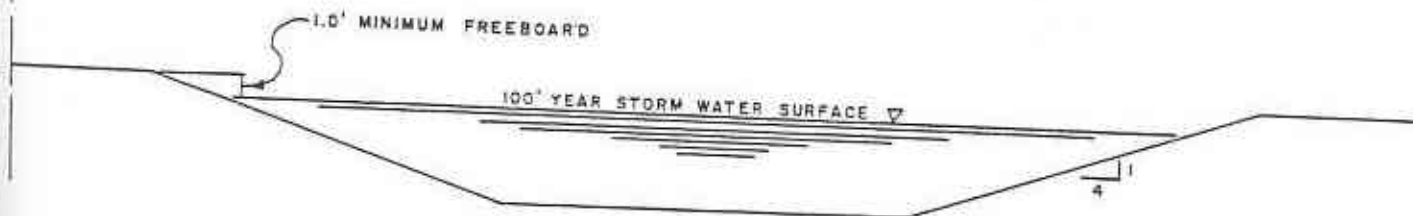
TYPE II - UNLINED WITH MAINTENANCE SECTION  
TYPICAL



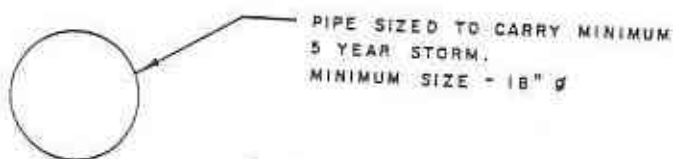
TYPE III - LINED

NOTE: WHEN CHANNEL IS DESIGNED USING PEAK DISCHARGE FLOWS FROM THE FLOOD INSURANCE STUDY, FREEBOARD MAY BE DELETED.

FIGURE 21



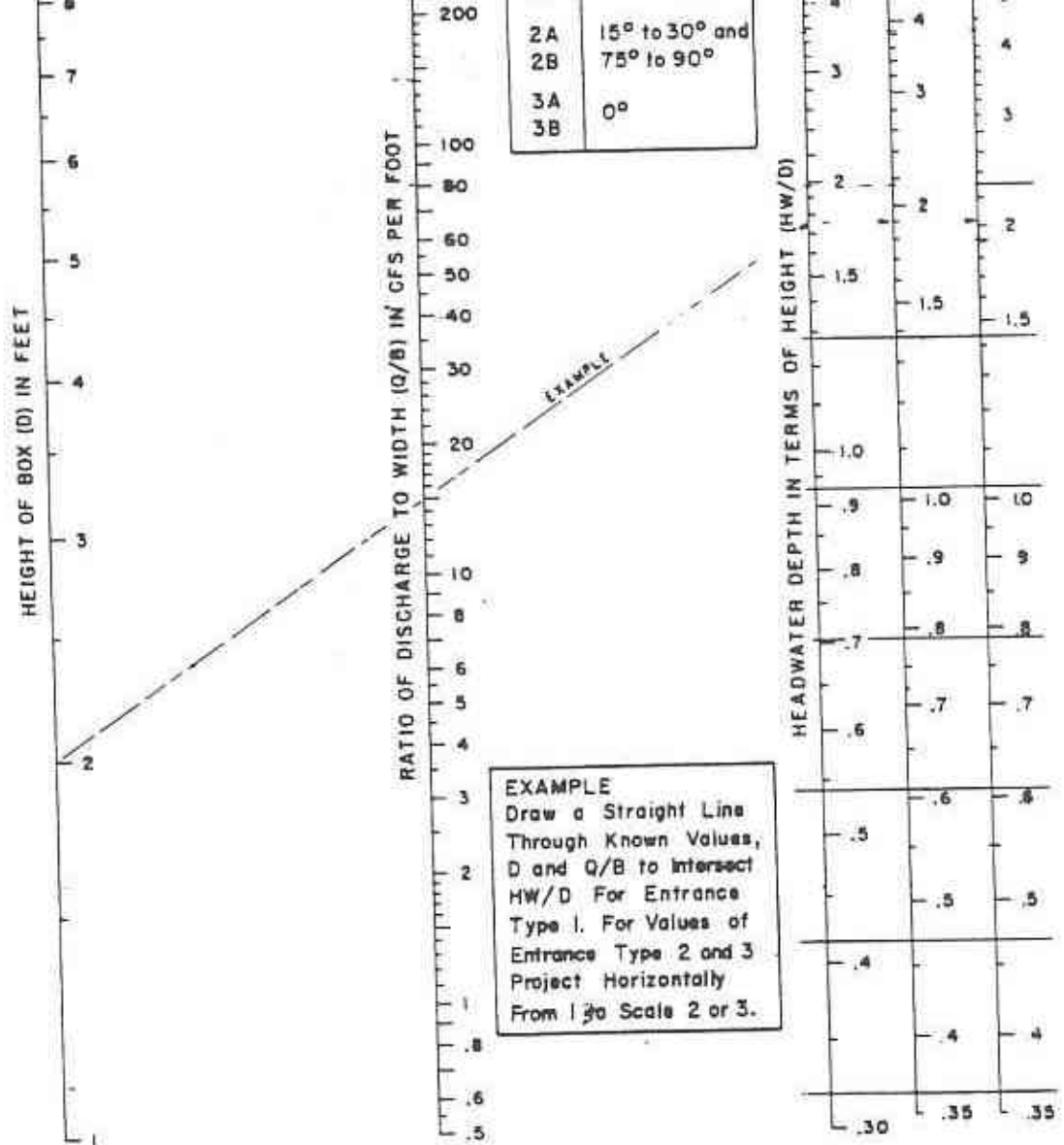
NOTE: Bank slopes and non-encroachment easement requirements same as for Type II.



NOTE: There are conditions due to the excessive capacity of the open ditch section where a pilot pipe carrying less than a five-year storm may be used if approved by the City Engineer.

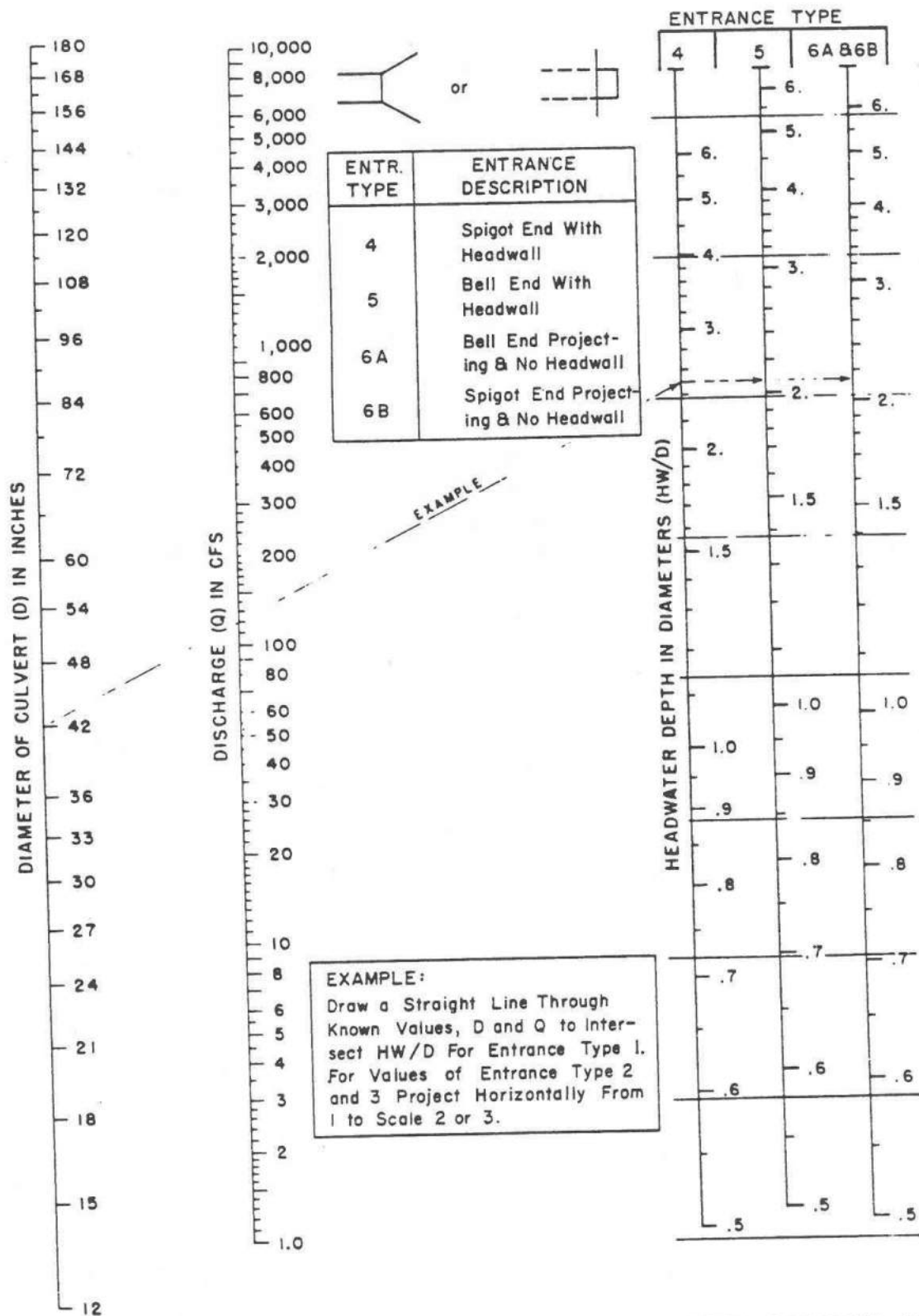
FIGURE 22

ALTERNATE OPEN  
CHANNEL TYPE II

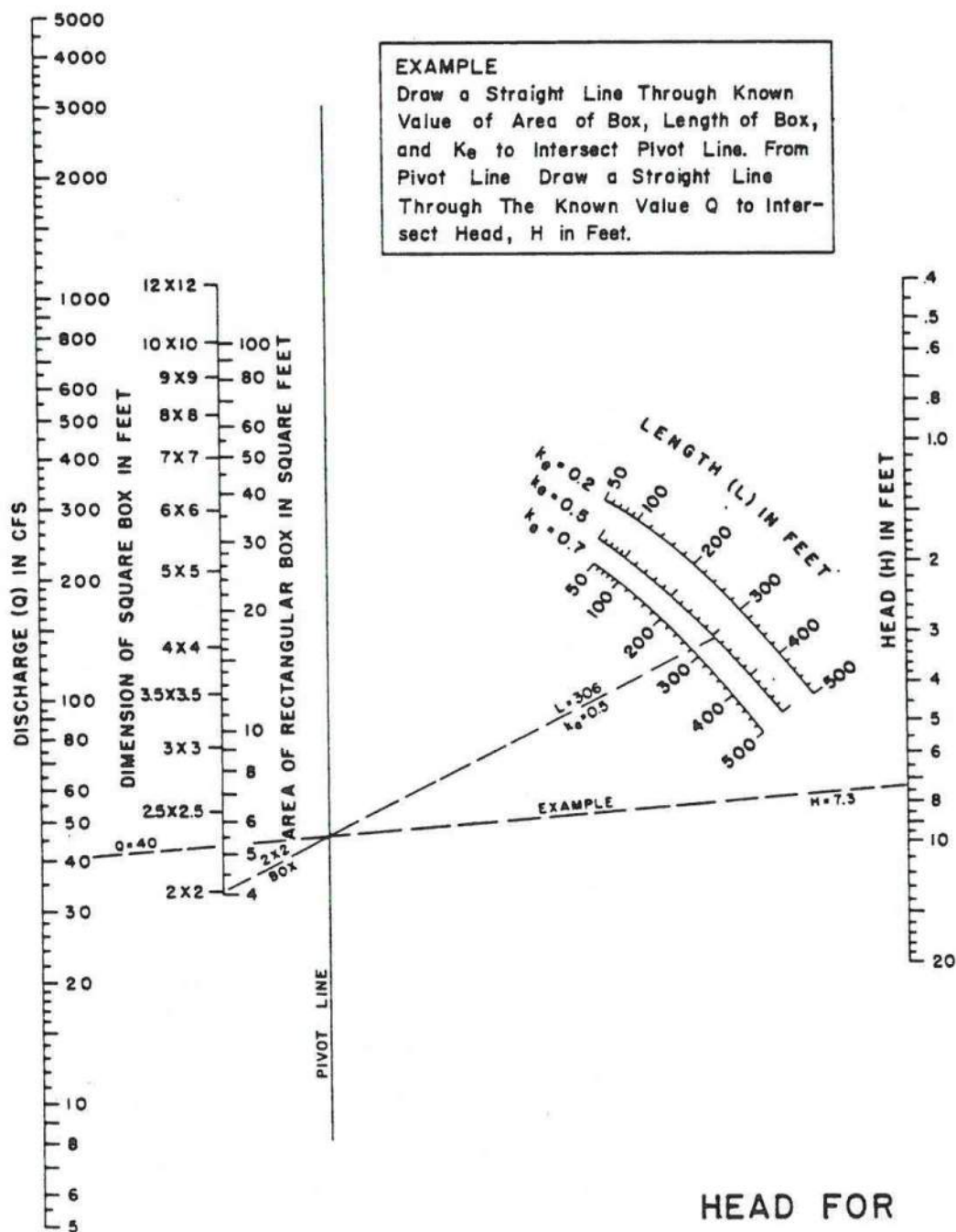


BUREAU OF PUBLIC ROADS JAN 1963

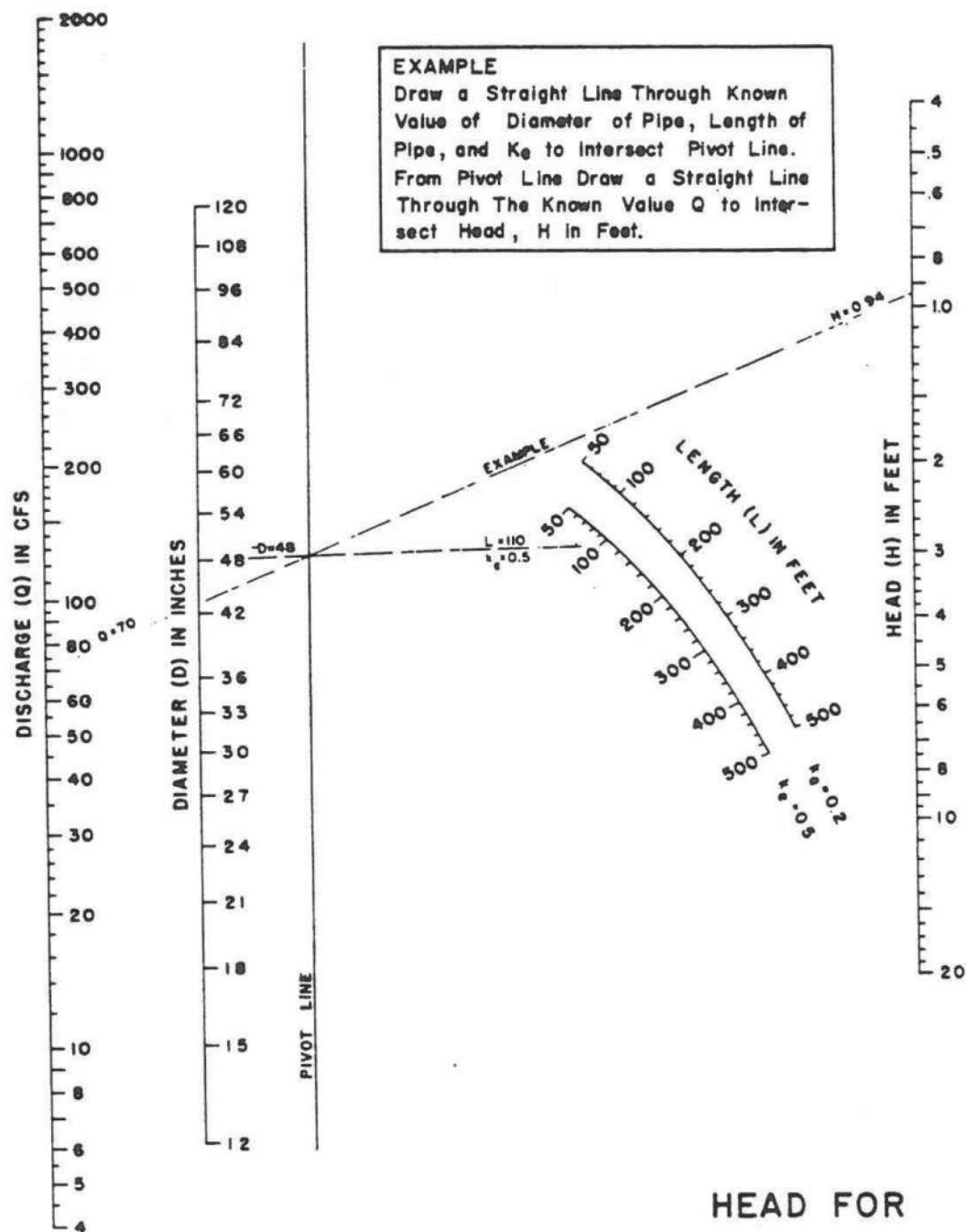
HEADWATER DEPTH  
FOR CONCRETE BOX  
CULVERT WITH  
INLET CONTROL



HEADWATER DEPTH FOR  
CONCRETE PIPE CULVERTS  
WITH INLET CONTROL



HEAD FOR  
 CONCRETE BOX CULVERTS  
 FLOWING FULL  
 $n = 0.012$



HEAD FOR  
 CONCRETE PIPE CULVERTS  
 FLOWING FULL  
 $n = 0.012$

BUREAU OF PUBLIC ROADS JAN. 1963



### EXAMPLE

#### Known:

Discharge = 200 c.f.s.

Width of Conduit = 5'

$Q/B = 40$

#### Find:

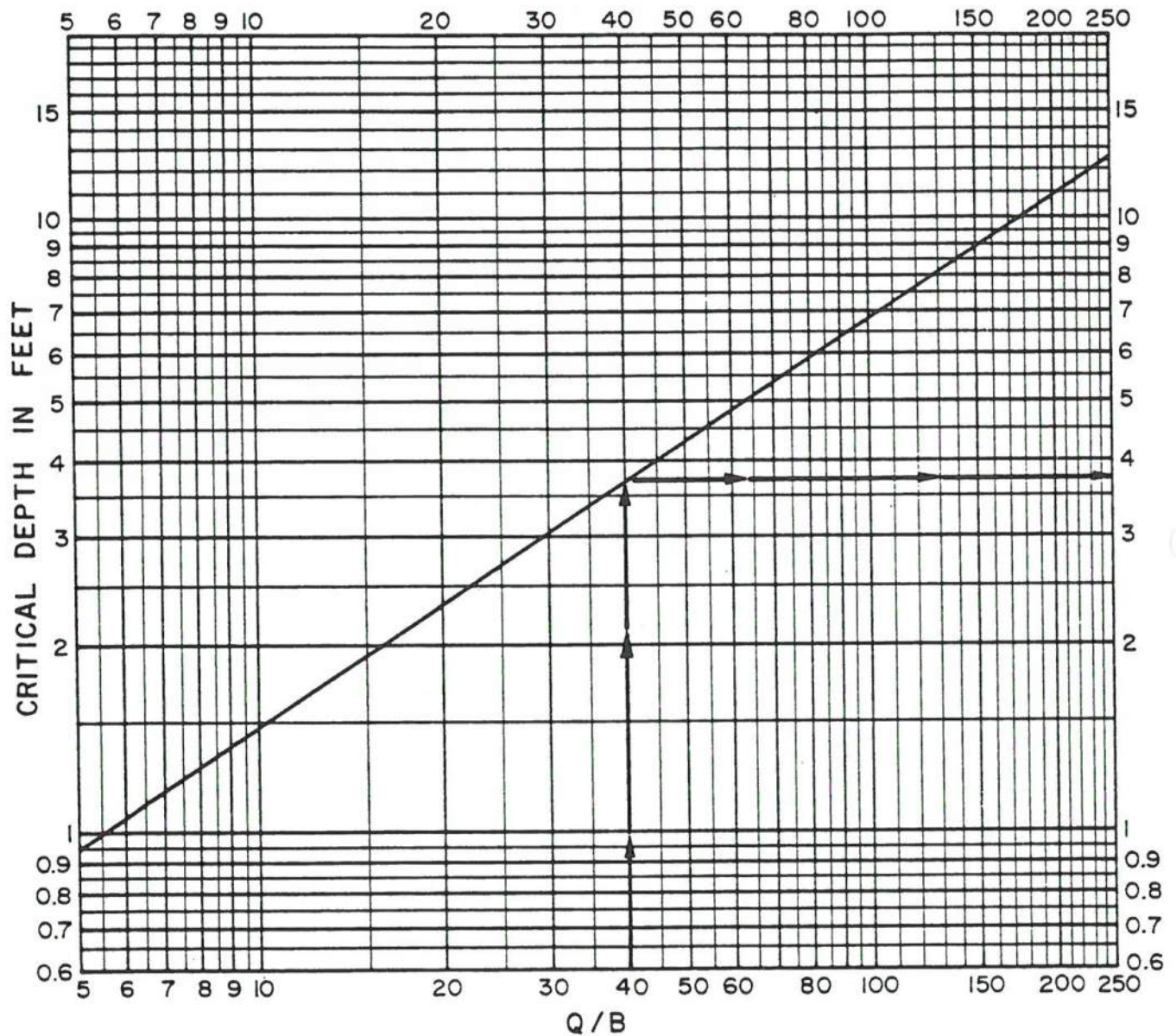
Critical Depth

#### Solution:

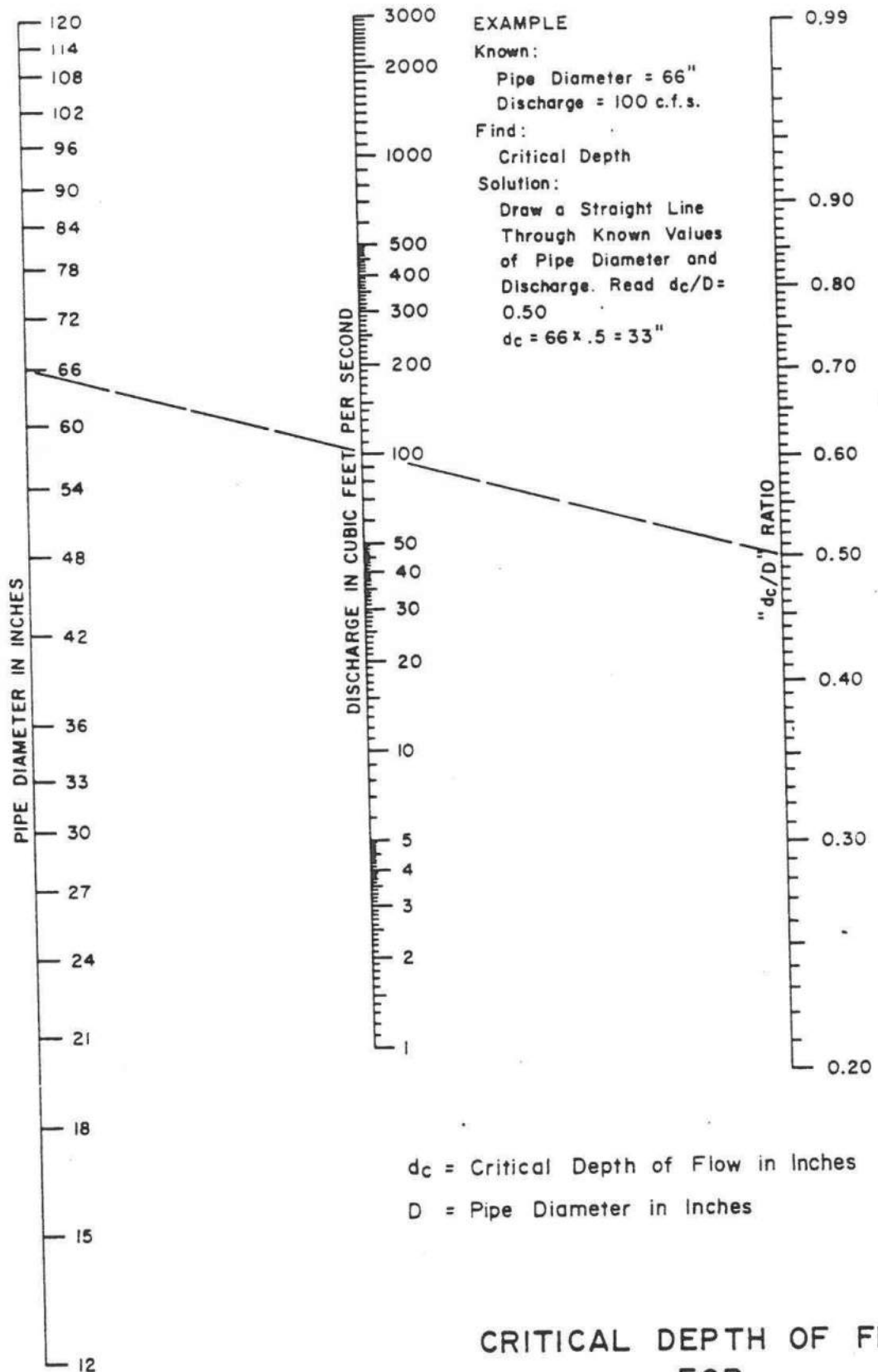
Enter Graph at  $Q/B = 40$

Intersect Critical Depth

at 3.7



CRITICAL DEPTH  
OF FLOW FOR  
RECTANGULAR CONDUITS



# CRITICAL DEPTH OF FLOW FOR CIRCULAR CONDUITS



## 8.0 LIST OF FORMS

Form A: Stormwater Runoff Calculations .....	8-2
Form B: Inlet Design Calculations .....	8-3
Form C: Storm Sewer Calculations .....	8-4
Form D: Water Surface Calculations .....	8-5
Form E: Open Channel Calculations .....	8-6
Form F: Hydraulic Design of Culverts .....	8-7
Form G: Bridge Design Calculations .....	8-8

**Note:**      **A copy of each applicable form must be submitted to the Engineering Division showing design calculation with all storm drainage construction plans.**

## CITY OF FARMERS BRANCH, TEXAS

DATE \_\_\_\_\_

DESIGN	STORM	FREQUENCY-
--------	-------	------------

DATE \_\_\_\_\_

[illegible]

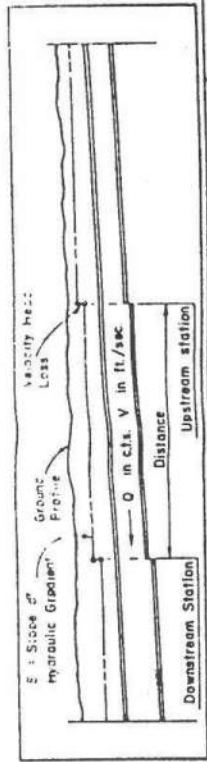
**FORM A**

FORM A

**CITY OF FARMERS BRANCH, TEXAS**

BY \_\_\_\_\_  
DATE \_\_\_\_\_

[illegible]



CITY OF FARMERS BRANCH, TEXAS

BY \_\_\_\_\_

DATE \_\_\_\_\_

STORM SEWER LINE \_\_\_\_\_  
INITIAL INLET TIME \_\_\_\_\_ MINUTES

STORM SEWER LINE

INITIAL	INITIAL TIME	MINUTES
1	10:00	10
2	10:05	15
3	10:10	20
4	10:15	25
5	10:20	30
6	10:25	35
7	10:30	40
8	10:35	45
9	10:40	50
10	10:45	55
11	10:50	60
12	10:55	65
13	11:00	70
14	11:05	75
15	11:10	80
16	11:15	85
17	11:20	90
18	11:25	95
19	11:30	100
20	11:35	105
21	11:40	110
22	11:45	115
23	11:50	120
24	11:55	125
25	12:00	130
26	12:05	135
27	12:10	140
28	12:15	145
29	12:20	150
30	12:25	155
31	12:30	160
32	12:35	165
33	12:40	170
34	12:45	175
35	12:50	180
36	12:55	185
37	13:00	190
38	13:05	195
39	13:10	200
40	13:15	205
41	13:20	210
42	13:25	215
43	13:30	220
44	13:35	225
45	13:40	230
46	13:45	235
47	13:50	240
48	13:55	245
49	14:00	250
50	14:05	255
51	14:10	260
52	14:15	265
53	14:20	270
54	14:25	275
55	14:30	280
56	14:35	285
57	14:40	290
58	14:45	295
59	14:50	300
60	14:55	305
61	15:00	310
62	15:05	315
63	15:10	320
64	15:15	325
65	15:20	330
66	15:25	335
67	15:30	340
68	15:35	345
69	15:40	350
70	15:45	355
71	15:50	360
72	15:55	365
73	16:00	370
74	16:05	375
75	16:10	380
76	16:15	385
77	16:20	390
78	16:25	395
79	16:30	400
80	16:35	405
81	16:40	410
82	16:45	415
83	16:50	420
84	16:55	425
85	17:00	430
86	17:05	435
87	17:10	440
88	17:15	445
89	17:20	450
90	17:25	455
91	17:30	460
92	17:35	465
93	17:40	470
94	17:45	475
95	17:50	480
96	17:55	485
97	18:00	490
98	18:05	495
99	18:10	500
100	18:15	505
101	18:20	510
102	18:25	515
103	18:30	520
104	18:35	525
105	18:40	530
106	18:45	535
107	18:50	540
108	18:55	545
109	19:00	550
110	19:05	555
111	19:10	560
112	19:15	565
113	19:20	570
114	19:25	575
115	19:30	580
116	19:35	585

[illegible]

FORM C



## CITY OF FARMERS BRANCH, TEXAS

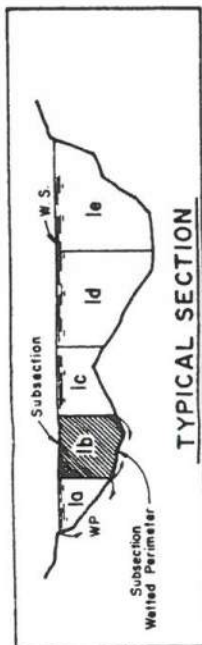
# WATER SURFACE PROFILE CALCULATIONS FOR UNIMPROVED CHANNELS

OPEN CHANNEL

TOTAL FLOW,  $Q_T =$  \_\_\_\_\_ c.f.s.

BY

DATE \_\_\_\_\_

[illegible]





BY \_\_\_\_\_

DATE \_\_\_\_\_