STORM DRAINAGE DESIGN MANUAL

City of Richardson, Texas

CITY OF RICHARDSON, TEXAS

ENGINEERING DEPARTMENT

DESIGN MANUAL
FOR
STORM DRAINAGE FACILITIES

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RICHARDSON DRAINAGE MANUAL

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I - INTRODUCTION

1.01 GENERAL

Storm water runoff is that portion of the precipitation which flows over the ground surface during and for a period after a storm. The objective of designing storm sewer systems is to conduct runoff in a functional and efficient way from places it is not wanted to the nearest acceptable discharge point. This transfer of runoff is done in sufficient time and methods to avoid damage and unacceptable amounts of inconvenience to the general public.

This manual provides guidelines for design of storm drainage facilities in the City of Richardson. The procedures outlined herein shall be followed for all drainage design and review of plans submitted to the City.

1.02 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 Richardson 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 as a record source for the City.

1.03 ORGANIZATION OF MANUAL

This manual is divided into six basic sections. The first section is the INTRODUCTION, which is a general discussion of the intended use of the material and an explanation of its organization.

* SECTION II, DRAINAGE DESIGN THEORY, explanation of the basic technical theory employed by the design procedures prescribed in this manual.
* SECTION III, CRITERIA AND DESIGN PROCEDURES, lists recommended design criteria and outlines the design procedures followed by the City of Richardson.
* SECTION IV, CONSTRUCTION PLAN PREPARATION, describes construction plans for drainage facilities in the City of Richardson.
* SECTION V, APPENDIX, contains a definition of terms, definition of symbols and abbreviations and the Bibliography.
* SECTION VI, TABLES, contains all the tables which are used in the design of drainage facilities.
* SECTION VII, FIGURES, contains all of the basic graphs, nomographs and charts for use in design of drainage facilities.
* SECTION VIII, FORMS, contains forms with detailed instructions for their use.

SECTION II

II - DRAINAGE DESIGN THEORY

2.01 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 situations.

2.02 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 size and shape should be determined from topographic maps at scale of 1 inch = 200 feet or larger. Topographic maps at 1 inch = 200 feet, are available from the City.

Where the contour interval is 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.

2.03 RAINFALL

FIGURE 1, which shows anticipated rainfall rates for storm durations from 5 minutes to 6 hours, has been prepared utilizing the information contained in the U. S. Department of Commerce, Weather Bureau, HYDRO-35 (National Technical Information Service Publication No. PB272-112, dated June, 1977). Interpolation of rainfall rates versus durations from the isopluvial maps contained in HYDRO-35 were used to prepare FIGURE 1 for durations less than 60 minutes. For durations beyond 60 minutes the information shown in FIGURE 1 was derived from Weather Bureau Technical Paper No. 40, dated May, 1961.

2.04 DESIGN STORM FREQUENCY

The individual curves shown on FIGURE 1 labeled "5 Yr.," "10 Yr.," "25 Yr.," "50 Yr.," and "100 Yr." 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 III, CRITERIA AND DESIGN PROCEDURES.

2.05 DETERMINATION OF DESIGN DISCHARGE

Prior to hydraulic design of drainage facilities, the amount of runoff from the particular drainage area must be determined. The Rational Method, the Unit Hydrograph, and the HEC-I Computer Program are the accepted procedures for computing volumes of storm water runoff. Data from the Flood Insurance Study shall be used in lieu of Rational Method, Unit Hydrograph or HEC-I 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 available from the City.

2.06 RATIONAL METHOD

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

(1) 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.

(2) The frequency of the peak discharge is the same as the frequency of the average rainfall intensity.

(3) 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:

$Q = C\ I\ A$, 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 FIGURE 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.

2.07 RUNOFF COEFFICIENT

The runoff coefficient "C" in the Rational Formula 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 Richardson area generally are low due to the cohesive soils.

As a drainage area develops, the amount of runoff increases generally in proportion to the amount of impervious areas such as streets, parking areas and buildings. Table 1 lists the accepted runoff coefficients for different land uses.

2.08 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 overestimation of rates of flow for larger areas. For this reason, in development of runoff rates in drainage areas over 500 Acres, use of the Unit Hydrograph Method or HEC-I is required as outlined in Section 3.04.

2.09 UNIT HYDROGRAPH METHOD

A Flood Insurance Study has been prepared for the Federal Emergency Management Agency by the U. S. Army Corps of Engineers for streams in the City of Richardson. Flows contained in this report must be used for design of drainage facilities along these streams. Where information is not available on a stream, a synthetic unit hydrograph must be used as outlined in Section 3.04 or HEC-I Computer Program Run.

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:

$$t_p = C_t \ (L\ L_{ca})^{0.3}$$

$$q_p = \frac{C_p 640}{t_p}$$

$$Q_p = q_p\ A$$

$$S_D = I \times 2$$

$$R_T = S_D - L_{1s}$$

$$Q_u = R_T\ Q_p$$

... "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 "$C_p 640$" are coefficients related to drainage basin characteristics. Recommended values for these coefficients are found in TABLE 2 (p. VI-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. This value may obtained in the following manner:

Trace the outline of the drainage basin on a piece of cardboard and trim to shape. Suspend the cardboard before a plumb bob by means of a pin near the edge of the cardboard and draw a vertical line. In a similar manner, draw a second line at approximately a 90° angle to the first line. The intersection of the two lines is the centroid of gravity of the 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.

2.10 UNIT HYDROGRAPH COEFFICIENTS

The U. S. Army Corps of Engineers published, in August 1952, 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 Richardson area.

In Section III 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 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 arriving at the total runoff. These are termed the "initial" and "subsequent" losses and are shown in Section III, 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.

2.11 FLOW IN GUTTERS AND INLET DESIGN

In the design of storm drainage facilities, the geometrics of specific types of streets are an integral part of drainage design. Throughout this manual references are made to certain types and widths of streets with specific crown characteristics. These roadway sections are defined in the City's Thoroughfare Plan. The following terms are defined for reference purposes:

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.

PARABOLIC CROWN: A pavement surface shaped in a parabola from one gutter flow line to the other.

VERTICAL DISPLACEMENT BETWEEN GUTTER FLOW LINES: Due to topography, it will be necessary at times that the curbs on a street be placed at different elevations.

2.12 STRAIGHT CROWN STREETS

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

$$Q = 0.56 * \frac{Z}{n} * S^{1/2} * Y^{8/3}$$ (Equation 1), where ...

... "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 3 was prepared and inlet design calculations, as explained elsewhere, were made.

2.13 PARABOLIC CROWN STREETS

FIGURES 4 and 5 show the capacity of gutters in streets with parabolic crowns. The following formulas can be used for determining the gutter capacity or refer to the figures for solution.

$$Q = \frac{1.486}{n} * AR^{2/3} * S^{1/2} \qquad \text{(Equation 2)}$$

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

$$A = \left(\frac{W_o\ C_o}{2}\right) - \left(\frac{8C_o}{W_o^2}\right)\int_o^{\frac{W_o}{2}} X^2dx \qquad \text{(Equation 4)}$$

where ...

... "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_o" is the width of the street in feet.

... "C_o" is the crown height of the street in feet.

As discussed in Section III, 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 FIGURES 4 and 5 were developed based on a minimum depth of flow in the high gutter of approximately two inches.

2.14 ALLEY CAPACITY

FIGURE 6, CAPACITY OF ALLEY SECTIONS, was prepared based on solution of Manning's Equation:

$Q = \frac{1.486}{n} * AR^{2/3} * S^{1/2}$ (Equation 2), where ...

... "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.

2.15 INLET CAPACITY CURVES

The primary objective in developing the curves shown in FIGURES 8 through 22 was to provide the Engineer with a direct method for sizing inlets which would yield answers within acceptable accuracy limits.

2.16 RECESSED AND STANDARD CURB OPENING INLETS ON GRADE

The basic curb opening inlet capacity curves, FIGURES 8 through 12, Recessed and Standard Curb Opening Inlets on Grade, were based upon solution of the following equation:

$$L = \frac{Q(H_1 - H_2)}{(H_1^{5/2} - H_2^{5/2})\ (.70)} \quad \text{(Equation 6), where ...}$$

... "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_1" is the depth of flow, in feet, in the gutter approaching the inlet plus the inlet depression, in feet.

... "H_2" 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 were used to plot the curves shown on FIGURES 8 through 12.

2.17 RECESSED AND STANDARD CURB OPENING INLETS AT LOW POINT

FIGURE 13, Recessed and Standard Curb Opening Inlets at Low Point, was plotted from the solution of the following equation:

$Q = 3.087\ L\ h^{3/2}$ (Equation 7), where ...

... "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 Storm Water Inlets," dated June 1956, The John Hopkins University.

The calculated inlet lengths were doubled for preparation of FIGURE 13 due to the tendency of inlets at low points to clog from the collection of debris at their entrance.

2.18 COMBINATION INLET ON GRADE

FIGURES 14 through 16, 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 grate 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:

$$L_o = 4 v_o \left[\frac{Y_o}{g}\right]^{1/2} \qquad \text{(Equation 8)}$$

$$L' = 1.2 v_o \tan \theta o \left[\frac{y_o - \frac{W}{\tan \theta_o}}{g}\right]^{1/2} \qquad \text{(Equation 9)}$$

$$q_2 = \frac{L' - L}{4} (g)^{1/2} \left[y_o - \frac{W}{\tan \theta_o}\right]^{3/2} \qquad \text{(Equation 10)}$$

$$q_3 = Q_o \left[1 - \frac{L^2}{Lo^2}\right]^2 \qquad \text{(Equation 11)}$$

$$Q = Q_o - \left[q_2 + q_3\right] \qquad \text{(Equation 12)}$$

where ...

... L_o = Length of grate required to capture 100% of all flow over grate in feet.

... v_o = Gutter velocity in feet per second.

... y_o = Depth of gutter flow in feet.

... g = Gravitational acceleration (32.2 feet per second per second).

... L' = Length of grate required to capture the outer portion of the gutter flow in feet.

... θ_o = Crown slope of pavement.

... w = Width of grate in feet.

... q_2 = Carry-over flow in c.f.s. outside of the grate.

... L = Length of grate in feet.

... q_3 = Carry-over flow in c.f.s. over the grate.

... Q_o = Gutter flow in c.f.s.

... Q = Capacity of grate inlet in c.f.s.

These equations are from "The Design of Storm Water Inlets," The John Hopkins University, June 1956.

2.19 COMBINATION INLET AT LOW POINT

FIGURE 20, Combination Inlet at a Low Point, was prepared based on 4-foot inlets having one grate and inlets 6-foot and greater having two grates. Grates are based on 1.72 square feet of opening (Bass & Hayes #814 Grate). Figure 20 curves reflect sizing an inlet for twice its capacity due to the tendency of low point inlets to clog from collection of debris at their entrance.

$$Q = 3.087\ Lh^{3/2} \qquad \text{(Equation 7)}$$

$$Q = 0.6A\sqrt{2gh} \qquad \text{(Equation 13)}$$

where ...

... "Q" is the gutter flow in cubic feet per second.

... "A" is the net cross section area, in square feet, of the grate opening.

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

Combination inlets shall only be used with written approval from the City Engineer.

2.20 GRATE INLET ON GRADE

FIGURES 16 through 19, Grate Inlet on Grade, were prepared based on the solution of Equations 1, 8, 9, 10, 11, and 12 as described in Paragraph 2.18, 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 can construct grate inlets as outlined in this manual.

2.21 GRATE INLET AT LOW POINT

FIGURE 21, Grate Inlet at Low Point, was prepared on the inlet having a capacity of 50 percent of the quantity derived from solution of Equation 13 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 Engineer for thoroughfare construction. Private systems can construct grate inlets as outlined in this manual.

2.22 DROP INLET AT LOW POINT

FIGURE 22, Drop Inlet at Low Point, was prepared based on solution of Equation 7 as previously referenced, using a fifty percent reduction in capacity due to clogging. Drop inlet shall be used in thoroughfare construction only with the permission of the City Engineer.

2.23 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:

$$Q = A\,V$$

$$Q = \frac{1.486}{n} * AR^{2/3} * S_f^{1/2}$$

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

where ...

... "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, (TABLE 5).

... "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.

2.24 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.

2.25 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.

2.26 MINOR HEAD LOSSES IN CLOSED CONDUITS

The basic equation for calculation of minor head losses at wye branches (lateral connections to main storm sewer line) is as follows:

WHERE $V_1 < V_2$

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

WHERE $V_1 > V_2$

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

"h_j" is head loss in feet

"V_2" is the downstream velocity in feet per second

"V_1" is the upstream velocity in feet per second

"g" is gravitational acceleration (32.2 feet per second per second.

The basic equation for calculation of minor head losses at manholes and bends in closed conduits is as follows:

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

... "h_j" is head loss in feet.

... "K_j" is coefficient of loss (TABLE 6)

... "V" is velocity in feet per second in conduit immediately downstream of point of loss.

... "g" is gravitation acceleration (32.2 feet per second per second).

2.27 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 of this manual may be used for analysis of an existing channel or for the design of a proposed channel.

2.28 ANALYSIS OF EXISTING CHANNELS

The analysis of the carrying capacity of an existing channel is an application of Bernoulli's energy equation which is written:

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

where ...

... "Z_1" and "Z_2" is the streambed elevation with respect to a given datum at upstream and downstream sections, respectively.

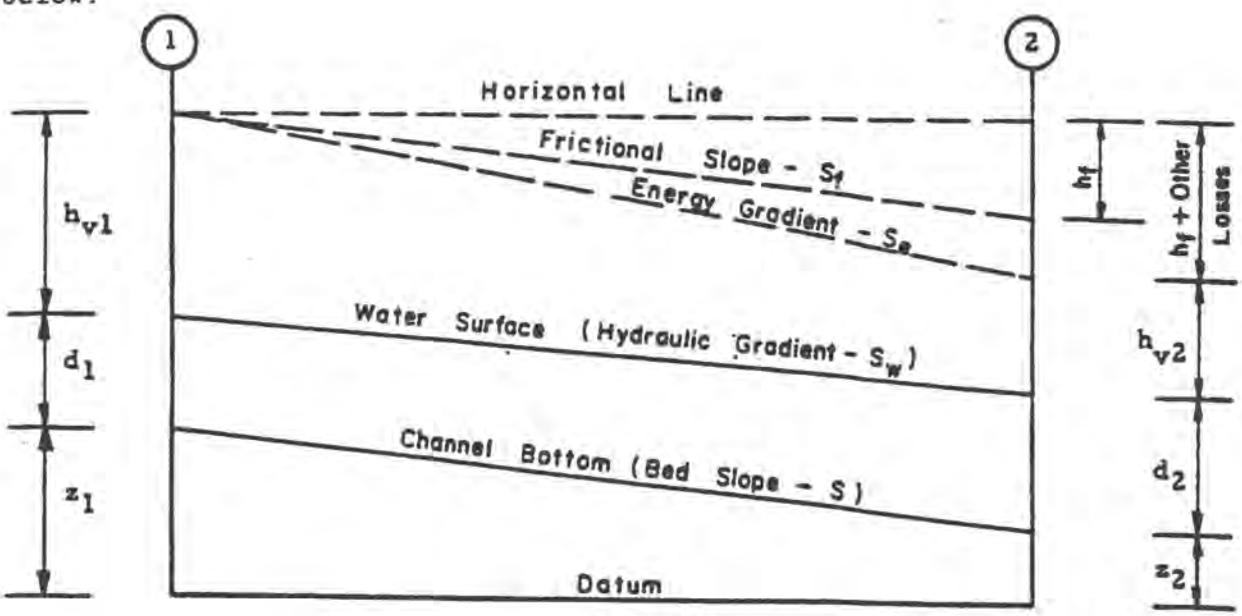
... "d_1" and "d_2" is depth of flow at upstream and downstream sections, respectively.

... "h_{v1}" and "h_{v2}" is velocity head of upstream and downstream sections, respectively.

... "h_f" is friction head loss.

Other losses such as eddy losses are estimated as 10 percent 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:

$$Q = \frac{1.486}{n} * AR^{2/3} * S^{1/2}$$

which is defined elsewhere in this chapter.

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

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

thus giving the total friction head

$$h_f = L\ \frac{S_{f1} + S_{f2}}{2}$$

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

The velocity head h is found by weighing the partial discharges v for each subdivision of the cross section, i.e.,

$$h_v = \frac{V_s^2 * Q_s}{2g * Q}$$, where ...

... "V_s" is velocity in subsection of the cross section.

... "A_s" is area of the subsection of the cross section.

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

... "V_s" is $\frac{Q_s}{A_s}$

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

2.29 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-2 (Water Surface Profile) computer analysis can be utilized. The City, at its option, can require the use of HEC-2 Computer Analysis in lieu of Manning's Equation. The HEC-2 Computer Program is available from the U. S. Army Corps of Engineers, Hydrologic Engineering Center, 609 Second Street, Davis, California 95616, 916/440-2105.

2.30 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 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 formulae. Laboratory tests and field observations indicate 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.

2.31 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", Case I (inlet not submerged) or as shown on FORM "F", Case II (inlet submerged).

Nomographs for determining culvert capacity for inlet control are shown on FIGURES 25 and 26. These nomographs were developed by the Division of Hydraulic Research, Bureau of Public Roads from analysis of laboratory research reported in 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.

2.32 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:

$HW = H + h_o - L\,(S_o)$, where ...

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

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

... "h_o" is the vertical distance from the downstream culvert flow line to the elevation from which H is measured, in feet.

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

... "S_o" 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_v, an entrance loss, H_e, and a friction loss, H_f. This energy is obtained from ponding of water at the entrance and is expressed as:

$H = H_v + H_e + H_f$, where ...

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

... "H_v" is $\frac{V^2}{2g}$ where V is average velocity in culvert or $\frac{Q}{A}$.

... "H_e" is $K_e \frac{V^2}{2g}$ where K_e is entrance loss coefficient.

... "H_f" is energy required to overcome the friction of the culvert barrel and expressed as:

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

... "n" is the coefficient of roughness (See TABLE 5).

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

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

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

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

Substituting into previous equation:

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

and simplifying:

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

This equation for H may be solved using FIGURES 27 and 28.

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 29 or may be calculated from the formula:

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

... "d_c" is critical depth for box culvert in feet.

... "Q" is discharge in cubic feet per second.

... "B" is bottom width of box culvert in feet.

The critical depth for circular pipes may be obtained from FIGURE 30 or may be calculated by trial and error. Charts developed by the Bureau of Public Roads may be used for determining the critical depth. Try values of D, A and d_c which will satisfy the equation:

$$\frac{Q^2}{g} = \frac{A^3}{D}, \text{ where } \ldots$$

... "d_c" is critical depth for pipe in feet.

... "Q" is discharge in cubic feet per second.

... "D" is diameter of pipe in feet.

... "g" is 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 applicable also for trapezoidal or irregular channels, in which instances "D" becomes the channel top width in feet.

2.33 BRIDGES

Once a design discharge and the 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:

$$V = \frac{Q}{A}$$

using the respective properties of Sections 1 and 2 for the calculations of S_{f1} and S_{f2}.

The velocity head h is found by weighing the partial discharges v for each subdivision of the cross section, i.e.,

$$h_v = \frac{V_s^2 * Q_s}{2g * Q}, \text{ where ...}$$

... "V_s" is velocity in subsection of the cross section.

... "A_s" is area of the subsection of the cross section.

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

... "V_s" is $\frac{Q_s}{A_s}$

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

2.29 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 an HEC-2 (Water Surface Profile) computer analysis can be utilitized. The City, at its option, can require the use of HEC-2 Computer Analysis in lieu of Manning's Equation. The HEC-2

Computer Program is available from the U. S. Army Corps of Engineers, Hydrologic Engineering Center, 609 Second Street, Davis, California 95616, 916/440-2105.

2.30 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 Department of Commerce Hydraulic Engineering Circular No. 5 entitled "Hydraulic Charts for the Selection of Highway Culverts," dated December, 1965.

SECTION III

III - CRITERIA AND DESIGN PROCEDURES

3.01 GENERAL

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

Applicable forms which can be used for the design of various storm drainage facilities are contained in Section VIII of this manual and the appropriate forms shall be part of the drainage submittal if required by the City Engineer.

3.02 RAINFALL

In determining the estimated runoff from a drainage area, it is necessary to predict the amount of rain which can be expected. FIGURE 1, RAINFALL INTENSITY AND DURATION, has been prepared to graphically illustrate anticipated rainfall intensity for storm duration from five minutes to six hours for selected return frequencies and shall be used for determining rainfall rates as required.

3.03 DESIGN STORM FREQUENCY

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 for all reaches of a drainage area utilizing ultimate land uses as outlined in the City's current Zoning Maps, with this projected flow carried in the streets and closed drainage systems in accordance with the following:

(a) RESIDENTIAL STREETS: The 100-year Design Storm Frequency flows shall be fully contained within the street right-of-way. A maximum point flow (such as a parking lot, side street, etc.) shall be 10 cfs.

Bypass from upstream inlets shall not exceed 5 cfs through residential street intersections.

(b) COLLECTOR STREETS: The 100-year Design Storm Frequency flows shall be fully contained within the street and allow one lane of traffic to be open for vehicular travel. A maximum point flow (such as a parking lot, side street, etc.) shall be 10 cfs. Bypass from upstream inlets shall not exceed 5 cfs through collector street intersections.

(c) MAJOR THOROUGHFARES: The 100-year Design Storm Frequency shall be fully contained within the street and allow one lane of traffic in each direction to be open for vehicular travel. A maximum point flow (such as a parking lot, side street, etc.) shall be 10 cfs. Bypass from upstream inlets shall not exceed 1 cfs through major thoroughfare intersections.

(d) ALLEYS: The 100-year Design Frequency flows shall not exceed the capacity of the alley right-of-way.

(e) EXCAVATED CHANNELS: Excavated channels shall have concrete pilot channels if deemed necessary by the City Engineering Department, for access or erosion control as outlined below. All excavated channels shall have a design water surface as outlined in 3.06 and be in accordance with FIGURE 24, Type II. Concrete lined channels shall be not less than Type III shown in FIGURE 24.

(f) MINIMUM LOT AND FLOOR ELEVATIONS: Minimum lot and floor elevations shall be established as follows:

(1) Lots abutting a natural or excavated channel shall have a minimum elevation for the finished floor of the lot at two (2) feet above the 100-year design flow or as directed by the City Engineer.

(2) Where lots do not abut a natural or excavated channel, minimum floor elevations shall be a minimum of one (1) foot above the street curb or edge of alley, whichever is higher, unless otherwise approved by the City Engineering Department. A lot grading plan may be required by the City Engineer.

(3) Where lots are located adjacent to a floodplain and a street, the street shall be constructed with a top-of-curb elevation a minimum two (2) feet above the BFE and the finished floor of the lot one (1) above the top-of-curb. The lot shall be graded so that there is positive drainage away from the slab.

(g) EASEMENTS: Drainage or floodway easements shall be provided for all open channels. Floodway easements shall be reserved for FEMA Regulated Floodways only. Easements shall encompass all areas beneath a Ground Elevation Defined as being the highest of the following:

(1) Two (2) feet above a design storm whose frequency is 100 years.
(2) The top of the high bank.

Additional easement may be required by the City for maintenance purposes.

(h) 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 Richardson may require special investigations and designs.

(i) INLET DESIGN: Inlet spacing shall be in accordance with the design criteria contained in this manual, as required in Section 3.08, maximum length of inlets at one location shall not exceed 2-10 foot inlets each side of street without prior approval from the City Engineering Department.

(j) CULVERTS AND BRIDGES: All drainage structures shall be designed to carry the 100-year Design Frequency flow. Bridges and culverts shall be designed so that there is a minimum of two (2) feet of freeboard between the 100-year water surface and the low steel of a bridge or soffit of a box culvert.

(k) MINIMUM STREET OR ALLEY ELEVATIONS: Streets or alleys adjacent to an open channel shall be designed with an elevation not lower than two (2) feet above the base flood elevation or as directed by the City Engineer.

3.04 DETERMINATION OF DESIGN DISCHARGE

The Rational Method for computing storm water runoff is to be used for hydraulic design of facilities serving a drainage area of less than 500 acres. For drainage areas larger than 500 acres the runoff shall be calculated by the Unit Hydrograph Method. The City of Richardson uses Snyder's Modified Unit Hydrograph using the U. S. Army's Corps of Engineers Modified Parameters for Urban Areas.

In lieu of the Unit Hydrograph Method a HEC-1 (Flood Hydrograph) Computer Analysis can be utilized. The City at its option can require the use of HEC-1 Computer Analysis in lieu of the unit Hydrograph Method. The HEC-1 Computer Program is available from the U.S. Army Corps of Engineers, Hydrologic Engineering Center, 609 Second Street, Davis, California 95616, 916/440-2105.

3.05 RUNOFF COEFFICIENTS AND TIME OF CONCENTRATION

Runoff coefficients, as shown in TABLE 1, shall be used, based on total development utilizing the existing City Zoning Map.

Times of concentration shall be computed based on the minimum inlet times shown in TABLE 1.

3.06 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 "Ct" and "Cp640" should be as shown in Table 2.

3.07 PROCEDURE FOR DETERMINATION OF DESIGN DISCHARGE

A standard form, STORM WATER RUNOFF CALCULATIONS, FORM A, is included in the Section VIII to record the data used in various drainage area calculations. This form will be used in calculation of runoff for design of open channels, culverts and bridges. Explanation for use of this form is included in the Section VIII.

3.08 FLOW IN GUTTERS AND INLET DESIGN

Unless there are specific agreements with the City of Richardson to the contrary, prior to beginning design of the particular storm drainage project, the City of Richardson requires a storm drain conduit to begin, and consequently the first inlet to be located, at the point where the street gutter flows full or accumulates 10 cfs of storm water based on the 100-year design storm.

3.09 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 1/2-inch per foot which are the minimum and maximum allowable slopes. Cross slopes other than 1/4-inch per foot shall not be used without prior approval from the City Engineering Department.

3.10 CAPACITY OF PARABOLIC CROWN STREETS

FIGURES 4 and 5, CAPACITY OF PARABOLIC GUTTERS, apply to streets with parabolic crowns.

3.11 STREET INTERSECTION DRAINAGE

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

(1) An arterial or collector street shall not be crossed with surface drainage over and above what is outlined in this manual, unless approved by the City Engineering Department.

(2) At any intersection, only one street shall be crossed with surface drainage and this shall be the lower classified street.

3.12 ALLEY CAPACITIES

FIGURE 6 is a nomograph to allow determination for the storm drain capacity of various standard alley sections. In residential areas where the standard 10-foot wide alley section capacity is exceeded, a wider alley may be used to provide storm drain capacity.

As can be seen on FIGURE 6, the 20-foot wide alley section has the largest storm drain capacity. Curbs shall not be added to alleys to increase the capacity unless approved by the City Engineering Department. Where a particular width alley is required, such as a 12-foot width, a wider alley, such as a 16-foot width, may be required for greater capacity. Increases in right-of-way widths will be necessary.

3.13 INLET DESIGN

FIGURE 7, STORM DRAIN INLETS, is a tabulation for the various types and sizes of inlets and their prescribed uses.

The information in FIGURE 7 and the general requirements of beginning the storm drain conduit where the street gutter capacity is reached will furnish the information necessary to establish inlet locations.

FIGURES 8 through 21 shall be used to determine the 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, Type D

Pavement Width = 24 Feet

Gutter Slope = 1.00%

Pavement Cross Slope = 1/4-inch/1 Foot

Gutter Flow = 11 cfs

FIND : Length of Inlet Required (L_i)

SOLUTION: Refer to FIGURE 8

Enter Graph at cfs

Intersect Slope = 1.00%

Read L_i = 16.9 Feet

DO NOT USE 10-FOOT INLET IN COMBINATION WITH 8-FOOT INLET

Enter Graph at L_i = 10 Feet

Intersect Slope = 1.00%

Read Q = 5.8 cfs

Therefore, the two inlets have a total capacity of 10.0 cfs which is less than the gutter flow of 11 cfs.

USE TRIAL AND ERROR SOLUTION

Two 10-foot Inlets

Enter Graph at L_i = 10 Feet

Intersect Slope = 1.00%

Read Q = 5.8 cfs

2 * 5.8 = 11.6 cfs

The two inlets have a total capacity of 11.6 cfs which is more than the gutter flow.

USE EITHER TWO 10-FOOT INLETS OR OTHER SUITABLE COMBINATIONS WHICH WILL BEST FIT THE PHYSICAL CONDITIONS. CONSIDERATION SHOULD BE GIVEN TO ALTERNATE INLET LOCATIONS OR EXTENSION OF THE SYSTEM TO ALLEVIATE THE PROBLEM OF MULTIPLE INLETS AT A SINGLE LOCATION.

Inlets shall be sized to intercept all flow in the approaching gutter where possible.

3.14 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 the Section VIII together with an explanation of how to use the form. Inlet spacing shall be determined in accordance with Section 3.08 of this manual.

3.15 HYDRAULIC GRADIENT OF CONDUITS

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:

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

Beginning at the upper most inlet on the system a tentative hydraulic gradient for the selected conduit size is plotted approximately 2 feet below the gutter between each contributing inlet to insure that the selected conduit will carry the design flow at an elevation below the gutter 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 6 for VELOCITY HEAD COEFFICIENTS FOR CLOSED CONDUITS.)

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 t . storm drainage conduit and 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 by use of the HEC-2 Computer Program.

Concrete pipe conduit shall be used to carry the storm water. A flow chart, Figure 23, based on Manning's Equation may be used to determine the various hydraulic elements including the pipe size, the hydraulic gradient and the velocity. Special hydraulic calculators are also available for solution of Manning's Equation.

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 shall be plotted directly on the construction plan profile worksheet and adjusted as necessary.

3.16 VELOCITY IN CLOSED CONDUITS

TABLE 3 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 3 should be used in designing concrete pipe storm sewer systems.

TABLE 4 shows the maximum allowable velocities in closed conduits. The maximum desirable velocity should be limited to 15 fps in all pipe (including laterals), and only exceeded with written permission of the City Engineer.

3.17 ROUGHNESS COEFFICIENTS FOR CONDUITS

Recommended values for the roughness coefficient "n" are tabulated in TABLE 5. 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.

3.18 MINOR HEAD LOSSES

The values of K_j to be used are tabulated for various conditions in TABLE 6. 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.

3.19 PROCEDURE FOR HYDRAULIC DESIGN OF CLOSED CONDUITS

STORM SEWER CALCULATIONS, FORM C, has been included in the Section VIII, together with explanation for its use to facilitate the hydraulic design of a storm sewer.

3.20 OPEN CHANNELS

Open channels are to be used to convey storm waters 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 24, is an improved section recommended for use where larger storm flows are to be conveyed. The concrete flume in the channel bottom 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 24, 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 of Richardson.

The most efficient cross section of an open channel, from a hydraulic standpoint, 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, as well as present an aesthetically pleasing view. In areas where erosion must be controlled, the Engineer shall include in his plans the type of grass and placement to be furnished.

Design water surface shall be as shown on Figure 24 and as outlined in 3.06. Floodway or Drainage easements shall be provided as shown in 3.03(g).

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, PROCEDURE FOR HYDRAULIC DESIGN OF CULVERTS.

Where deemed necessary by the City Engineer, provisions to prevent erosion and sedimentation 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 Engineer. Construction and installation of all these items shall conform to Item 3.12 in the North Central Texas Council of Governments Standard Specifications for Public Works.

On all channels the water surface elevations, which may be assumed as coincident with the hydraulic gradient, shall be calculated and 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 Richardson in the preliminary stages of design.

Hydraulic calculations for Type I Channels, Figure 24, shall be made as outlined on FORM "D". 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".

In general, the use of existing channels in their natural condition or with a minimum of improvement and with reasonable safety factors is encouraged.

3.21 TYPES OF CHANNELS

FIGURE 24 illustrates the classifications and geometrics of various channel types which are to be used wherever possible.

Type I Channel is to be used when 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 storm waters, minimize public health hazards and make the floodplain usable for recreation purposes. In some instances it may be desirable to remove undergrowth.

3.22 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."

3.23 CHANNEL ALIGNMENT AND GRADE

While it is recognized that channel alignments must necessarily 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.

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.

Maximum allowable velocities are shown in TABLE 7. 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 unless otherwise authorized by the City Engineer. Velocities and outfalls less than eight feet per second will not require dissipation structures.

3.24 ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

Roughness coefficients to be used in solving Manning's Equation are shown in TABLE 7, together with maximum allowable velocities.

3.25 PROCEDURE FOR CALCULATION OF WATER SURFACE PROFILE FOR UNIMPROVED CHANNELS

FORM "D" included in Section VIII, together with the explanation for its use, shall be used for calculating a profile of the water surface along an unimproved channel. The HEC-2 Computer Program is an alternate method to the use of Form "D" and may be required by the City.

3.26 PROCEDURE FOR HYDRAULIC DESIGN OF OPEN CHANNELS

FORM "E", included in Section VIII, together with the explanation for its use, shall be used in the design for open channels.

3.27 HYDRAULIC DESIGN OF CULVERTS

The function of a culvert or bridge is to pass storm water 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 velocity of flow 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 two feet of freeboard shall be used when the structure is designed to pass a design storm frequency of 100 years calculated by Richardson's criteria. Surrounding physical conditions may be cause for an increase in this requirement. The City Engineering Department in their opinion of the surrounding area may require a greater freeboard and channel improvements for the safety of the general public.

3.28 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.

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.

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.

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.

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 of two levels to be calculated.

3.29 QUANTITY OF FLOW

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".

3.30 HEADWALLS AND ENTRANCE CONDITIONS

Headwalls are used to protect the embankment from erosion and the culvert from displacement. The headwalls, with or without wingwalls and aprons, shall be constructed in accordance with the State Department of Highways and Public Transportation standard drawings as required by the physical conditions of the particular installation.

In general, straight headwalls 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 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 to 15 feet per second. This requirements 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". The values of the entrance coefficient, Ke, 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.

3.31 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 8 is a tabulation of maximum allowable velocities based on downstream channel conditions.

3.32 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.

Bridge design and hydraulics shall be prepared utilizing the procedures outlined in the Texas Highway Department "Hydraulic Manual" or the Bureau of Public Roads "Hydraulics of Bridge Waterways".

A distance of 2 feet between the maximum design water surface and the lowest point of the bridge stringers shall be maintained as a minimum and may be increased as directed by the City of Richardson Engineering Department on behalf of the safety of the general public.

3.33 QUANTITY OF FLOW

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". The HEC-1 Computer Program is an alternate method to the use of Form "A" and may be required by the City.

3.34 PROCEDURE FOR HYDRAULIC DESIGN OF BRIDGES

FORM "G", included in the Section VIII, together with the explanation for its use, shall be used for the hydraulic design of bridges.

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.

3.35 DETENTION DESIGN

Detention basins are used to temporarily impound (detain) excess storm water, thereby reducing peak discharge rates. These basins can be used to provide detention required due to inadequate downstream storm drainage facilities, due to increased zoning resulting in a significant increase in runoff, or by downstream cities with detention requirements.

Detention basins within the City of Richardson shall be designed according to the following criteria:

Design Frequency - The 100-year frequency event is to be used in determining required detention volume.

In areas which are composed of highly erodible soils, the peak runoff rate from a 5-year frequency event as well as the 100-year event must be held to the pre-development rate. This will reduce the erosion which can result from the more frequent events.

Outflow Velocity - The outflow structure will discharge flows at a non-erosive rate.

Detention Storage - Basins with drainage areas greater than 500 acres are to be designed using the Modified Unit Hydrograph Method to determine the critical duration. This is the duration for the design frequency requiring the greatest detention storage volume. The design hydrograph routings through the detention basin are to be done using the Modified Puls Method.

Basins with drainage areas of 500 acres or less can be designed using the Modified Rational Method. This method estimates peak rates using the Rational Equation and storage requirements using inflow minus outflow hydrograph volume at the time of peak outflow.

Freeboard and Emergency Spillway - Where earth embankments are used to temporarily impound the required detention, the top of the embankment will be a minimum of 2.0 feet above the maximum 100-year pool level. In addition, an emergency spillway or overflow area will be provided at the maximum 100-year pool level to ensure that the 500-year frequency event does not overtop the embankment.

For detention basins with drainage areas of 500 acres or less, the chart on page 23 of this section can be used to estimate the required capacity for the emergency spillway. If the emergency spillway capacity is to be provided over the embankment, it will have to be structurally designed to prevent erosion and consequent loss of structural integrity. If the capacity is to be provided in a vegetated earth spillway separate from the embankment, the required width for a trapezoidal spillway with a control section can be estimated by the equation:

$B_w = \dfrac{0.36Q - 0.7ZD}{D^{3/2}}$, where ...*

... "B_w" = Bottom width

... "Q" = Emergency spillway capacity (cfs)

... "D" = Design depth above spillway crest (ft.)

... "Z" = Side slope, i.e., horizontal distance to 1 foot vertical

*USDA, SCS, Dimensions for Farm Pond Spillways Where No Exit Channel is Required.

The minimum width for the type of spillway is 4.0 feet.

Overflow Structure - Where the overflow structure conveys flow through the embankment in a conduit, the conduit shall be reinforced concrete design to support the external loads with an adequate factor of safety. It shall withstand the internal hydraulic pressures without leakage under full external load or settlement. It must convey water at the design velocity without damage to the interior surface of the conduit.

Earth Embankment Design - The steepest side slope permitted for a vegetated earth embankment is 4:1 and 2:1 for rock dam. The minimum crown width is as follows:

Total Height of Embankment (ft.)	Minimum Crown Width (ft.)
14 or less	8
15 - 19	10
20 - 24	12
25 - 34	14

Basin Grading - Detention basins to be excavated must provide positive drainage with a minimum grade of 0.3%. The steepest side slope permitted for an excavated slope not in rock is 4:1.

Earth Embankment Specifications - Earth embankments used to temporarily impound required detention volume must be constructed according to specifications to fill. These specifications should, at a minimum, be adequate for levee embankments and be based on the NCTCOG "Standard Specifications for Public Works Construction" for embankment, topsoil, sodding and seeding. Where permanent impoundment is to be provided, more stringent specifications are required based on geotechnical investigations of the site.

Fencing - Security fencing with a minimum height of 4 feet shall encompass the basin area, when required due to potential safety hazards created by prolonged storage of floodwaters. Design shall be such as not to restrict the inflow or outfall of the basin. Adequate access for maintenance equipment shall be provided. In basins to be used for recreation areas during dry periods, pedestrian access may be provided with the approval of Public Works.

Maintenance Provisions - Access must be provided in detention basin design for periodic desilting and debris removal. Basins with permanent storage must include de-watering facilities to provide for maintenance. Detention basins with a drainage area of 500 acres or more must include a desilting basin for the upstream pool area.

EXAMPLE

MODIFIED RATIONAL METHOD DETENTION BASIN DESIGN

GIVEN: A 10-acre site, currently agricultural use, is to be developed for townhouses. The entire Area is the drainage area of the proposed detention basin.

DETERMINE: Maximum release rate and required detention storage.

SOLUTION:

1. Determine 100-year peak runoff rate prior to site development. This is the maximum release rate from site after development.

   NOTE: Where a basin is being designed to provide detention for both its drainage area and a by-pass area; the maximum release rate is equal to the peak runoff rate prior to site development for the total of the areas minus the peak runoff rate after development for the by-pass area. This rate for the by-pass area will vary with the duration being considered.

2. Determine inflow hydrograph for storms of various durations in order to determine maximum volume required with release rate determined in Step 1.

   NOTE: Incrementally increase durations by 1- minutes to determine maximum required volume. The duration with a peak inflow less than maximum release rate or where required storage is less than storage for the prior duration is the last increment.

STEP 1. - Present Conditions

$Q = CIA$

$C = .30$
$T_c = 20$ minutes
$i100 = 7.0$ in./hr.
$Q100 = .30\ (7.0)\ 10 = 21.0$ cfs (Maximum release rate)

STEP 2. - Future Conditions (Townhouses)

$C = .80$
$T_c = 15$ minutes
$i100 = 7.7$ in./hr.
$Q100 = .80\ (7.7)\ 10 = 61.6$ cfs

Check various duration storms:

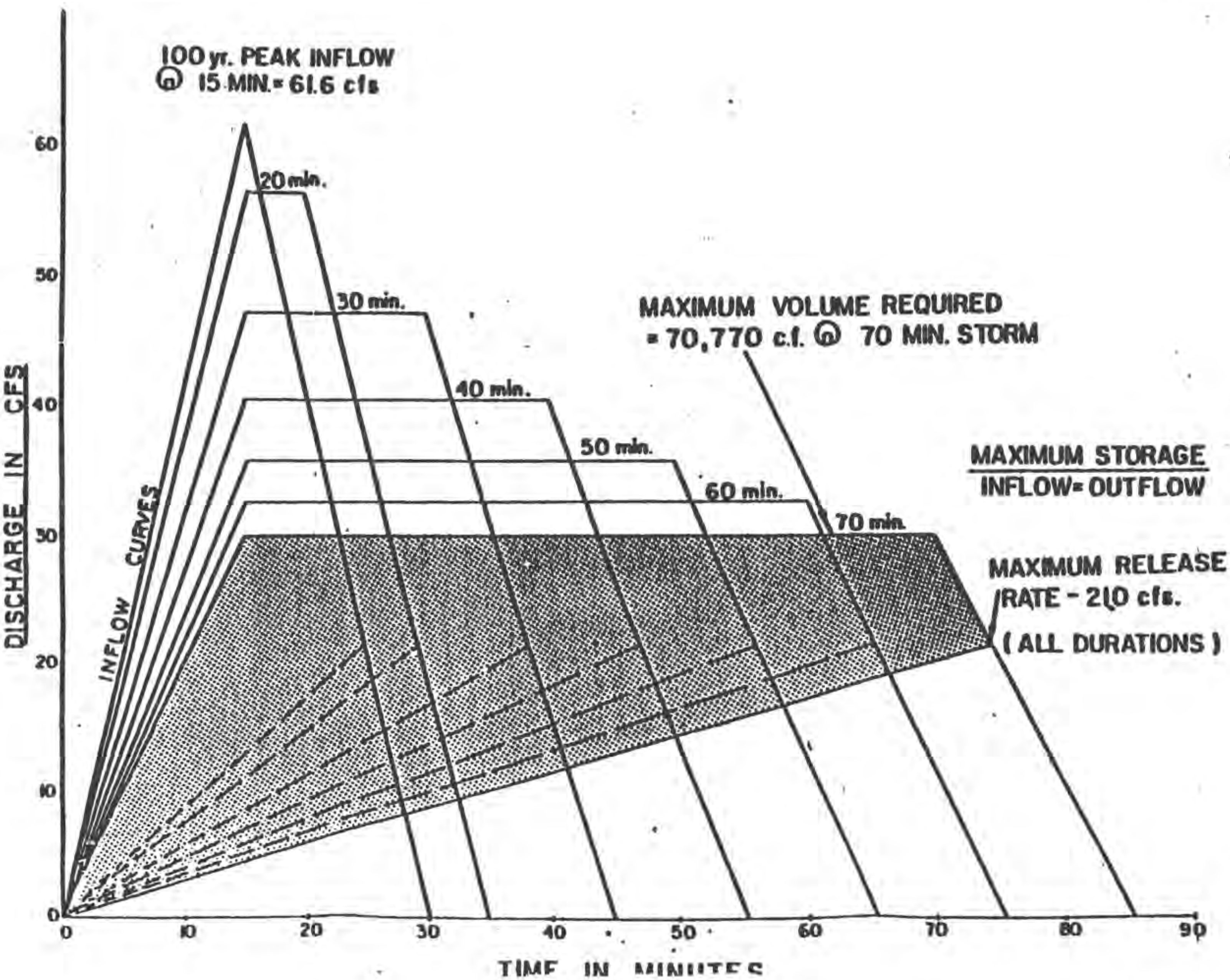
20 minutes	$i = 7.0$	$Q = .80\ (7.0)\ 10 = 56.0$ cfs
30 minutes	$i = 5.8$	$Q = .80\ (5.8)\ 10 = 46.4$ cfs
40 minutes	$i = 5.0$	$Q = .80\ (5.0)\ 10 = 40.0$ cfs

50 minutes	i = 4.4	Q = .80 (4.4) 10 = 35.2 cfs
60 minutes	i = 4.0	Q = .80 (4.0) 10 = 32.0 cfs
70 minutes	i = 3.7	Q = .80 (3.7) 10 = 29.6 cfs
80 minutes	i = 3.4	Q = .80 (3.4) 10 = 27.2 cfs
90 minutes	i = 3.1	Q = .80 (3.1) 10 = 24.8 cfs

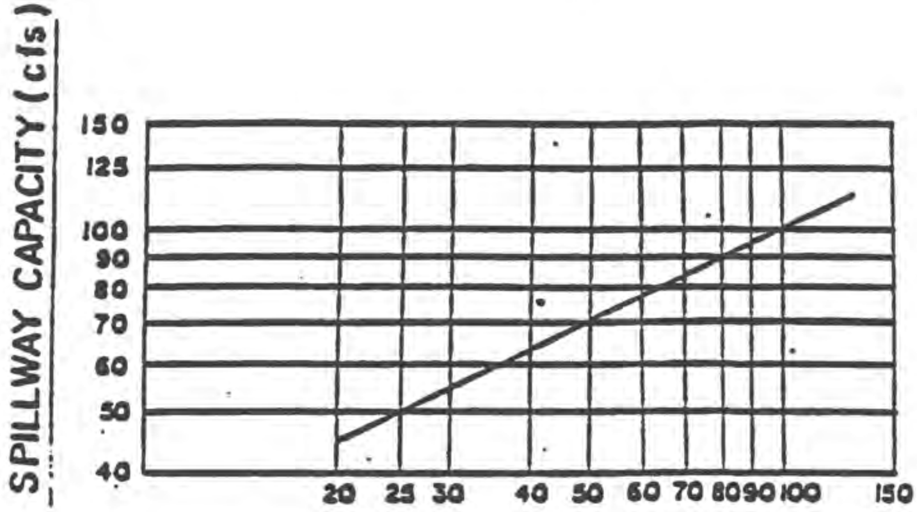
Maximum Storage Volume is determined by deducting the volume of runoff released during the time of inflow from the total inflow for each storm duration.

15 min. Storm	Inflow 15 (61.6) 60 sec/min	= 55,440 cf
	Outflow (0.5) 30 (21.0) 60 sec/min	= 18,900 cf
	Storage	= 36,540 cf
20 min. Storm	Inflow 20 (56.0) 60 sec/min	= 67,200 cf
	Outflow (0.5) 35 (21.0) 60 sec/min	= 22,050 cf
	Storage	= 45,150 cf
30 min. Storm	Inflow 30 (46.4) 60 sec/min	= 83,520 cf
	Outflow (0.5) 45 (21.0) 60 sec/min	= 28,350 cf
	Storage	= 55,170 cf
40 min. Storm	Inflow 40 (40.0) 60 sec/min	= 96,000 cf
	Outflow (0.5) 55 (21.0) 60 sec/min	= 34,650 cf
	Storage	= 61,350 cf
50 min. Storm	Inflow 50 (35.2) 60 sec/min	= 105,600 cf
	Outflow (0.5) 65 (21.0) 60 sec/min	= 40,950 cf
	Storage	= 64,650 cf
60 min. Storm	Inflow 60 (32.0) 60 sec/min	= 115,200 cf
	Outflow (0.5) 75 (21.0) 60 sec/min	= 47,250 cf
	Storage	= 67,950 cf
70 min. Storm	Inflow 70 (29.6) 60 sec/min	= 124.320 cf
	Outflow (0.5) 85 (21.0) 60 sec/min	= 53,550 cf
	Storage	= 70,770 cf
80 min. Storm	Inflow 80 (27.2) 60 sec/min	= 130,560 cf
	Outflow (0.5) 95 (21.0) 60 sec/min	= 59,850 cf
	Storage	= 70,710 cf
90 min. Storm	Inflow 90 (24.8) 60 sec/min	= 133,920 cf
	Outflow (0.5) 105 (21.0) 60 sec/min	= 66,150 cf
	Storage	= 67,770 cf

Maximum volume required is 70,770 cfs at the 70 min. storm duration.



100 yr. PEAK INFLOW
@ 15 MIN.= 61.6 cfs
20 min.
30 min.
40 min.
50 min.
60 min.
70 min.
MAXIMUM VOLUME REQUIRED
= 70,770 c.f. @ 70 MIN. STORM
MAXIMUM STORAGE
INFLOW= OUTFLOW
MAXIMUM RELEASE
RATE - 21.0 cfs.
(ALL DURATIONS)
INFLOW CURVES
DISCHARGE IN CFS
0
10
20
30
40
50
60
70
80
90
TIME IN MINUTES



DRAINAGE AREA (ACRES)

MINIMUM EMERGENCY SPILLWAY CAPACITY-cfs

FIGURE-2

SECTION IV

IV - CONSTRUCTION PLANS PREPARATION

4.01 GENERAL

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

4.02 PRELIMINARY DESIGN PHASE

Plans shall be submitted in accordance with the City of Richardson Developer's Checklist Preliminary Design Package (PDP). The preliminary design phase shall be complete in sufficient detail to allow review by the City of Richardson. To complete this phase, all topographic surveys should be furnished to allow establishment of alignment, grades and right-of-way requirements. All surveys shall be verified in the field.

Based upon the procedures and criteria outlined in SECTION III, CRITERIA AND DESIGN PROCEDURES, of this manual, the hydraulic design of the proposed facilities shall be accomplished. Calculations shall be made on appropriate forms and submitted with construction plans with the PDP submittal for review when required by the City Engineering Department.

These plans shall show the alignment, drainage areas, size of facilities, and grades.

Preliminary storm drainage plans shall be submitted in accordance with the developer's checklist and shall include drainage area map, plan-profile sheets and channel cross sections if applicable. The proposed improvements shall be drawn on 24" x 36" sheets.

(a) DRAINAGE AREA MAP

The drainage area map should have a scale of 1" = 200' showing the street right-of-way. When calculating runoff the drainage area map shall show the boundary of the drainage area contributing runoff into the

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.

Direction of flow within streets, alleys, natural and manmade drainage ways and at all system intersections shall be clearly shown on the drainage area map. Existing and proposed drainage inlets, storm sewer pipe systems and drainage channels shall be clearly shown and differentiated on the drainage area map. Plan-profile storm sewer or drainage improvement sheet limits and match lines shall be shown.

The Drainage Area Map should show enough topography to easily determine its location within the City.

(b) PLAN-PROFILE SHEETS

Inlets shall be given the same number designation as the area or sub-area contribution 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 radius and the inlet location shall also provide minimum interference with the use of adjacent property. Inlet locations directly above storm sewer lines 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 of inlet, type of inlet, number or designation, top of curb elevation and flow line of inlet as shown on the construction plans. Inlet laterals leading to storm sewers, where possible, shall enter the inlet at a 60 degree angle from the street side. Laterals shall be four and one-half feet from top of curb to flow line of inlet unless utilities or storm sewer depth requires otherwise. Laterals shall not enter the corners of inlets. Lateral profiles shall always be drawn showing appropriate information including the Hydraulic Gradient.

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 foot intervals and each sheet shall begin and end with even or fifty 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 profiles along the centerline of 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, capacity of pipe in cubic feet per second and velocity in feet per second. Also, the head loss at each interception point shall be shown. In partial flow, drawing shall show Q_{cap}, hydraulic slope and velocity.

Elevations of the flow line of the proposed storm sewer shall be shown at one hundred 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 unless otherwise approved by the City Engineer.

4.03 FINAL DESIGN PHASE

During the final design phase, the construction plans shall be prepared in final form with revisions and connections made as directed by the City. Plans shall be submitted as part of the Final Development Package (FDP) as required in the Developer's Checklist. All sheets shall be drawn in ink on 24" x 36" linen tracing cloth or mylar drafting film or equivalent and shall be clearly legible when sheets are reduced to half scale. All review comments shall be addressed, additional data incorporated, and final design and drafting of plans completed. All grades, elevations, pipe sizes, utility locations, items and quantities should be checked and each plan-profile sheet shall have a benchmark shown.

SECTION V

5.01 DEFINITIONS OF TERMS

Angle of Flare: Angle between direction of wingwall and center line of culvert or storm drain outlet.

Backwater Curve: The surface curve of a stream of water when backed up by a dam or other obstruction.

Conduit: Any closed device for conveying flowing water.

Control: The hydraulic characteristic which determined the stage-discharge relationship in a conduit.

Critical Flow: The state of flow for a given discharge at which the specific energy is a minimum with respect to the bottom of the conduit.

Entrance Head: The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head.

Entrance Loss: Head lost in eddies or friction at the inlet to a conduit, headwall or structure.

Flume: Any open conduit on a prepared grade, trestle or bridge.

Freeboard: 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.

Headwater: Depth of water in the stream channel measured from the invert of culvert.

HEC-1: Computer Program to analyze a Flood Hydrograph. This program is available from the U. S. Army Corps of Engineers.

HEC-2: Computer Program to analyze a Water Surface Profile. This program is available from the U. S. Army Corps of Engineers.

High water Elevation: The water surface elevation during the peak of the design storm.

Hydraulic Gradient: A line representing the friction head available at any given point within the system.

Invert: The flowline of pipe or box (inside bottom).

Manning's Equation: The uniform flow equation used to relate velocity, hydraulic radius and energy gradient slope.

Open Channel: A channel in which water flows with a free surface.

Rational Formula: The means of relating runoff with the area being drained and the intensity of the storm rainfall.

Soffit: The inside top of pipe or box.

Steady Flow: Constant discharge.

Surcharge: Height of water surface above the crown of a closed conduit at the upstream end.

Tailwater: Total depth of flow in the downstream channel measured from the invert at the culvert outlet.

Time of Concentration: 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.

Total Head Line (Energy Line): 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 friction head for conduits flowing under pressure.

Uniform Channel: A channel with a constant cross section and roughness coefficient.

Uniform Flow: 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.

Watershed: The area drained by a stream or drainage system.

5.02 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_s Sub-section area in square feet as used on unimproved channel calculations.

b Bottom width of channel in feet.

c Runoff Coefficient for use in Rational Formula representing the estimated ratio of runoff to rainfall which is dependent on the slope of the watershed, the land use and the character of soil.

C_o Street crown height in feet.

C_t A coefficient related to drainage basin characteristics and used in Unit Hydrograph calculations.

C_p640 A coefficient related to drainage basin characteristics and used in Hydrograph calculations.

cfs Cubic feet per second.

d Depth of flow in feet.

d_n Normal depth of flow in conduit in feet.

d_c Critical depth of flow in conduit in feet.

FL Flow line.

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_o 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.

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 Storm water 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 of cfs per square mile.

Q_p Peak rate of discharge of the Unit Hydrograph in cfs

R Hydraulic Radius = $\dfrac{\text{Cross section area of flow in sq. ft. (A)}}{\text{Wetted perimeter in ft. (P)}}$

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.

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.

$\frac{V^2}{2g}$ Velocity head. A measure, in feet, of the kinetic energy in 2g flowing water.

V_1 Upstream Velocity

V_2 Downstream Velocity

W Street width from face of curb in feet.

WP Wetted perimeter in feet.

Z Reciprocal of crown slope, $1/\theta_o$.

θ_o Crown slope of pavement in feet per foot.

Y Conveyance factor calculated for unimproved channels.

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SECTION VI

VI - LIST OF TABLES

TABLE 1

COEFFICIENTS OF RUNOFF AND MAXIMUM INLET TIMES

Zoning District	Runoff Coefficient C	Maximum Inlet Time In Minutes
Conceptual Planned Development	Variable	10 to 20
Duplex District	0.70	15
General Office District	0.90	10
General Retail	0.90	10
Heavy Commercial	0.90	10
Industrial District	0.90	10
Light Commercial	0.90	10
Limited Office District	0.90	10
Mid-range Office District	0.90	10
Multi-family	0.80	10
Multiple Family, High Rise	0.80	10
Neighborhood Office District	0.85	10
Neighborhood Service District	0.90	10
Office	0.90	10
Parking District	0.90	10
Residential 1 Acre	0.45	20
Residential 1/2 Acre	0.45	20
Residential 10,000 SF	0.65	15
Residential 13,000 SF	0.65	15
Residential 16,000 SF	0.65	15
Residential 5,000 SF	0.65	15
Residential 7,500 SF	0.65	15
Shopping Center	0.90	10
Townhouse 6 Units/Acre	0.80	15
Townhouse 9 Units/Acre	0.85	15
Townhouse 12 Units/Acre	0.90	10
Townhouse 15 Units/Acre	0.90	10

Non-Zoned Land Uses

Land Use	Runoff Coefficient C
Church	0.8
School	0.7
Park	0.4
Cemetery	0.4
Agricultural	0.3

TABLE 2

COEFFICIENTS "C_t" AND "C_p640"

Drainage Area Characteristics	Approximate Value of "C_t"	Approximate Value of "C_p640"
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 3

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. Slopes shown for each size pipe will produce a minimum velocity of 2.5 fps.

TABLE 4

VELOCITIES IN CLOSED CONDUITS

Type of Conduit	Min. Velocity	Max. Velocity
Culverts	2.5 fps	15 fps
Inlet Laterals	2.5 fps	15 fps
Storm Sewers	2.5 fps	12 fps

Storm sewers shall discharge into open channels at a maximum velocity of 8 feet per second.

TABLE 5

ROUGHNESS COEFFICIENTS FOR CLOSED CONDUITS

Material of New Construction	Recommended Roughness Coefficient "n"
Concrete Pipe Storm Sewer	0.013

Material of Existing Systems	
Concrete Pipe Storm Sewer	
Fair Alignment, Ordinary Joints	0.015
Poor Alignment, Poor Joints	0.017
Concrete Pipe Culverts	0.012
Monolithic Concrete Culverts	0.012

NOTE: Reinforced concrete pipe is the accepted material for construction of storm sewers. The use of other materials for the construction of storm sewers shall have prior approval from the City Engineering Department.

TABLE 6

VELOCITY HEAD LOSS COEFFICIENTS FOR CLOSED CONDUITS

MANHOLE AT CHANGE IN PIPE DIRECTION

DESCRIPTION	ANGLE	HEAD LOSS COEFFICIENT Kj
	90	1.00
	60	0.80
	45	0.65
Angle	30	0.50

BEND IN PIPES

DESCRIPTION	ANGLE	HEAD LOSS COEFFICIENT Kj
	* 90^{o}	0.80
	* 60^{o}	0.60
	** 45^{o}	0.50
Angle	** 30^{o}	0.45

ENLARGEMENTS IN PIPE SIZES WITH CONSTANT FLOW

DESCRIPTION	RATIO OF UPSTREAM DIAMETER TO DOWNSTREAM DIAMETER	HEAD LOSS COEFFICIENT Kj
	0.81	1.00
	0.82	0.90
	0.84	0.80
	0.85	0.70
	0.86	0.60
	0.88	0.50
	0.90	0.40
	0.92	0.30

* Only as authorized by City Engineer

** Horizontal curves are the accepted method of construction

TABLE 7

ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

Channel Description	Roughness Coefficient Minimum	Normal	Maximum	Maximum Velocity ft/sec
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	0.096	8
Flood Plain, Pasture				
Short Grass, No Brush	0.025	0.030	0.035	8
Tall Grass, No Brush	0.030	0.035	0.050	8
Flood Plain, Cultivated				
No Crops	0.025	0.030	0.035	8
Mature Crops	0.030	0.040	0.050	8
Flood Plain, 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 of similar description because banks offer less effective resistance.

Moderately Well Defined Channel	0.025	---	0.060	8
Irrigular 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

TABLE 7 (CONT'D)

ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

Channel Description	Roughness Coefficient Minimum	Normal	Maximum	Maximum Velocity ft/sec
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 8

DISCHARGE VELOCITIES

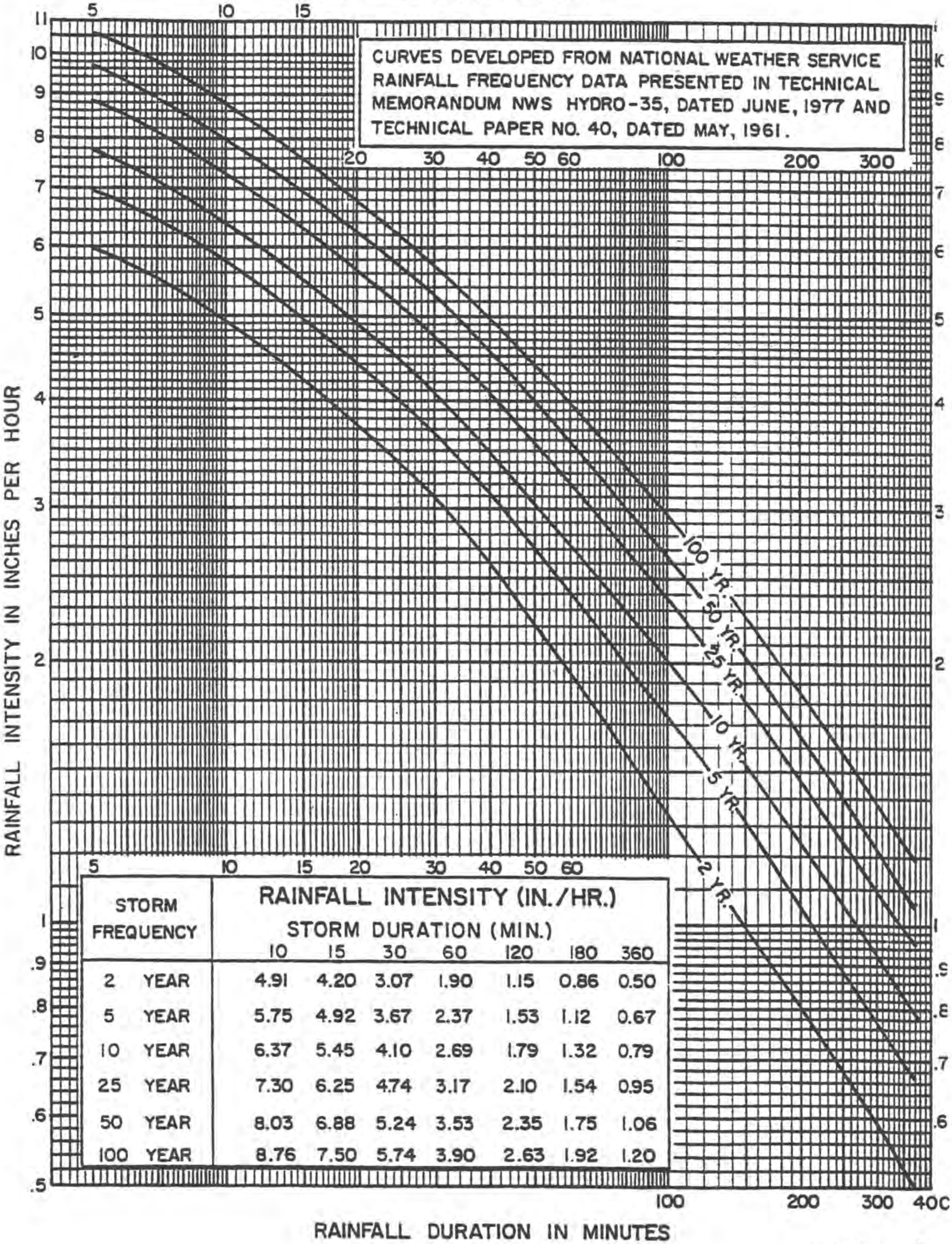
Culvert Discharges On	Maximum Allowable Velocity (fps)
Earth (Sandy)	6
Earth (Clay)	8
Sodded Earth	8
Concrete	15
Shale (Rock)	10

SECTION VII

VII - LIST OF FIGURES

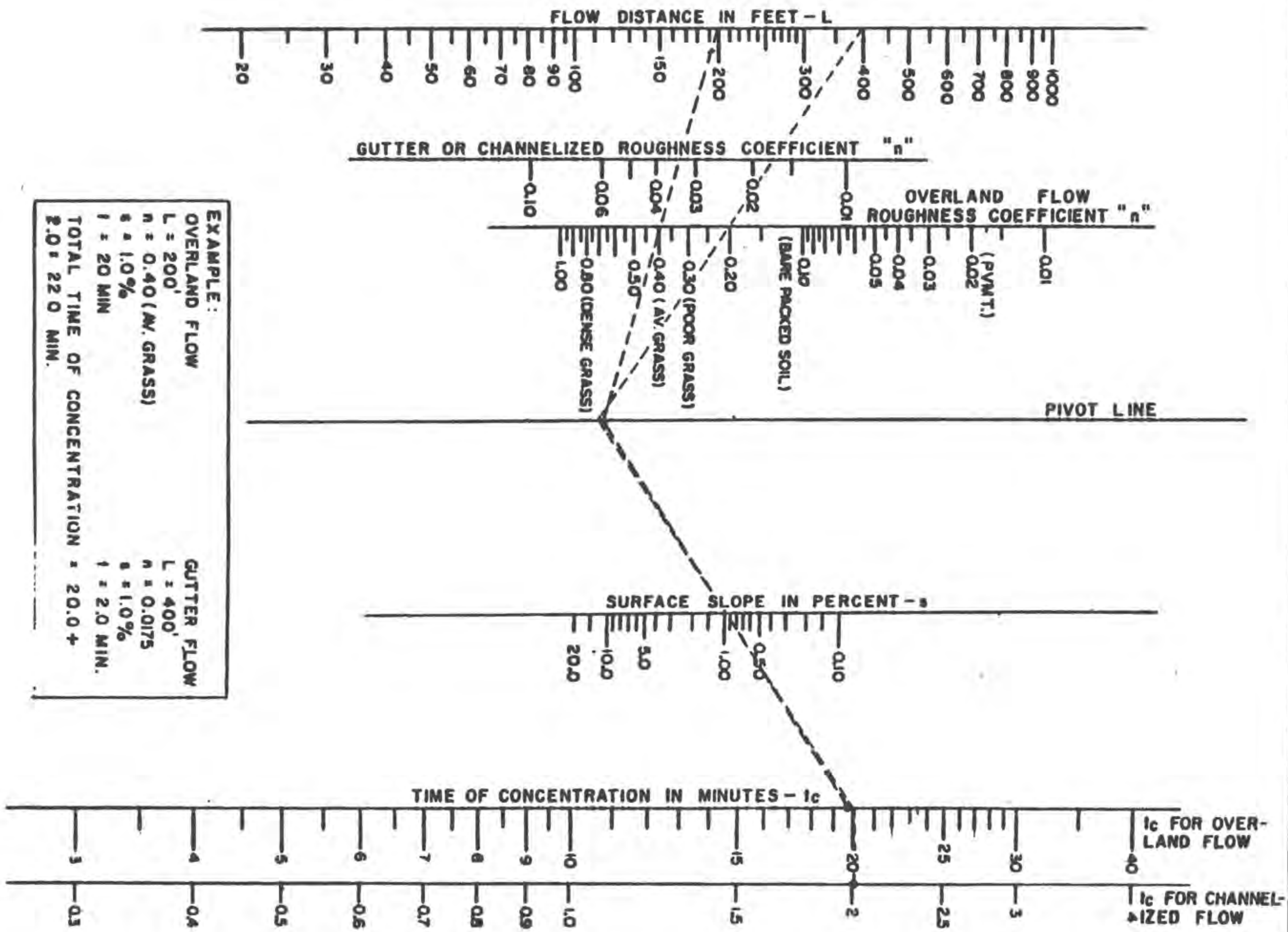
Figure No. Title

CITY OF RICHARDSON, TEXAS
RAINFALL INTENSITY CURVES



STORM FREQUENCY	RAINFALL INTENSITY (IN./HR.) STORM DURATION (MIN.) 10	15	30	60	120	180	360
2 YEAR	4.91	4.20	3.07	1.90	1.15	0.86	0.50
5 YEAR	5.75	4.92	3.67	2.37	1.53	1.12	0.67
10 YEAR	6.37	5.45	4.10	2.69	1.79	1.32	0.79
25 YEAR	7.30	6.25	4.74	3.17	2.10	1.54	0.95
50 YEAR	8.03	6.88	5.24	3.53	2.35	1.75	1.06
100 YEAR	8.76	7.50	5.74	3.90	2.63	1.92	1.20

FIGURE 1



TIME OF CONCENTRATION FOR SURFACE FLOW

EXAMPLE

Known:

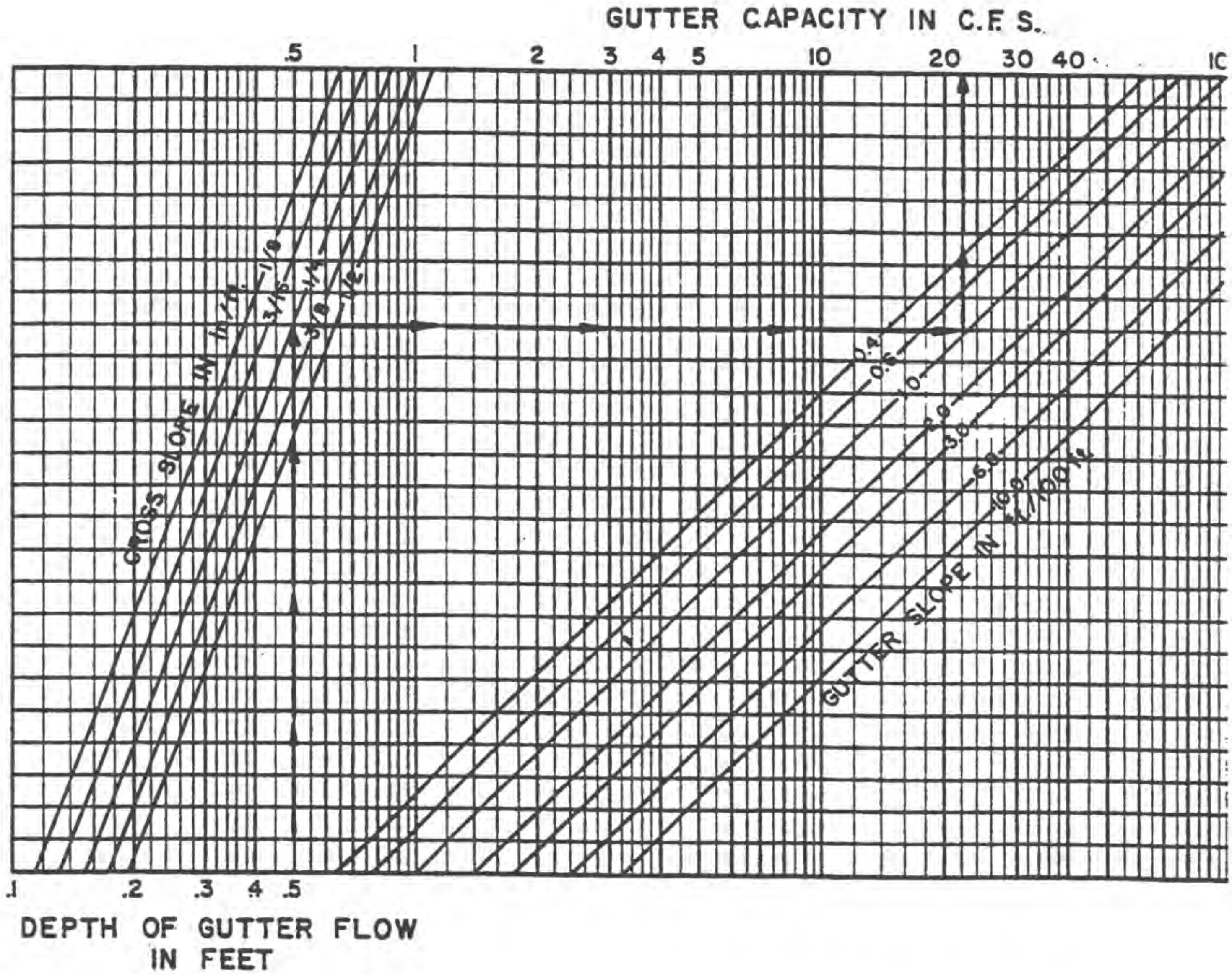
Major Thoroughfare,
Pavement Width = 33'
Gutter Slope = 1.0%
Pavement Cross Slope = 1/4"/1'
Depth of Gutter Flow = .5'

Find:

Gutter Capacity

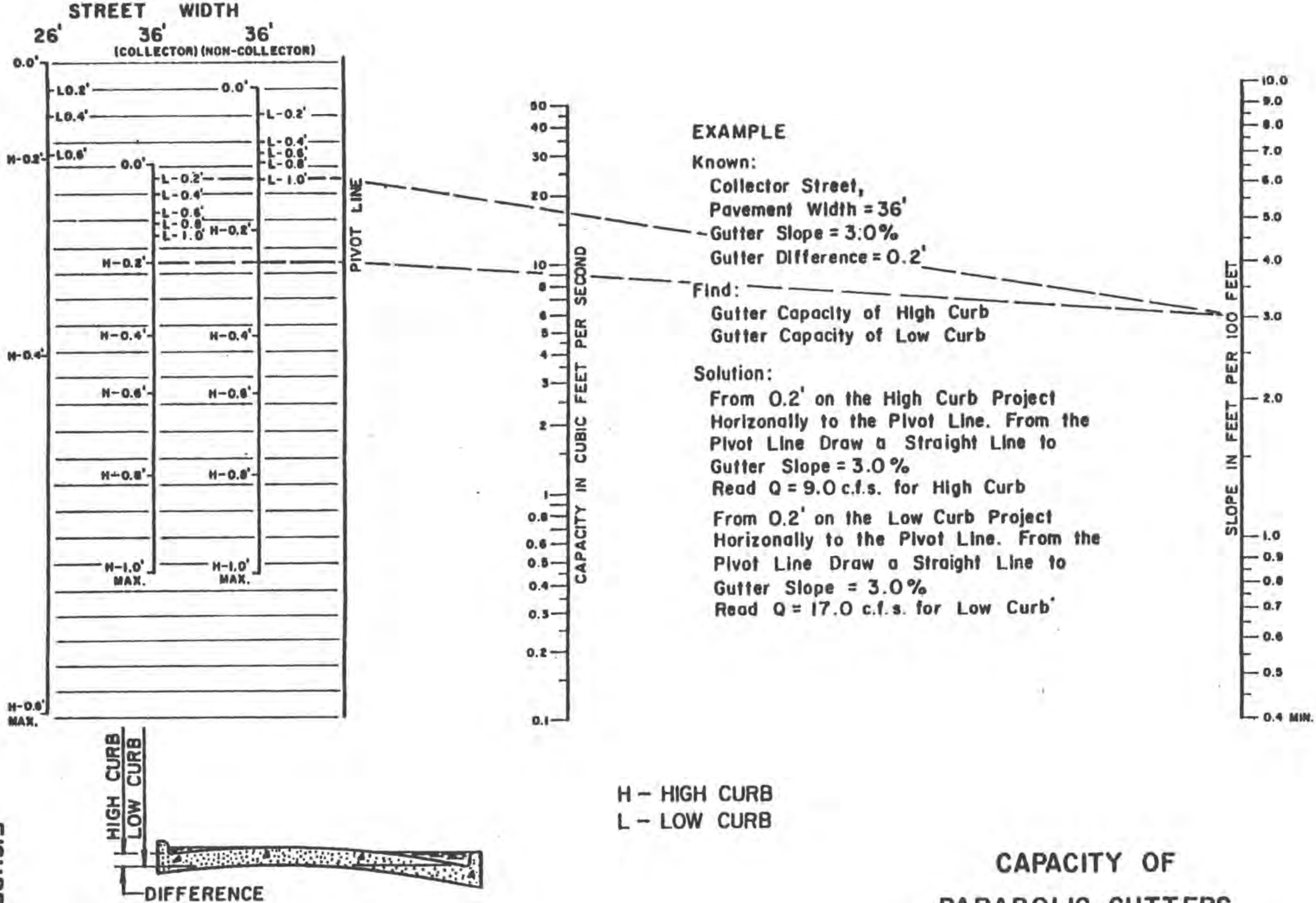
Solution:

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



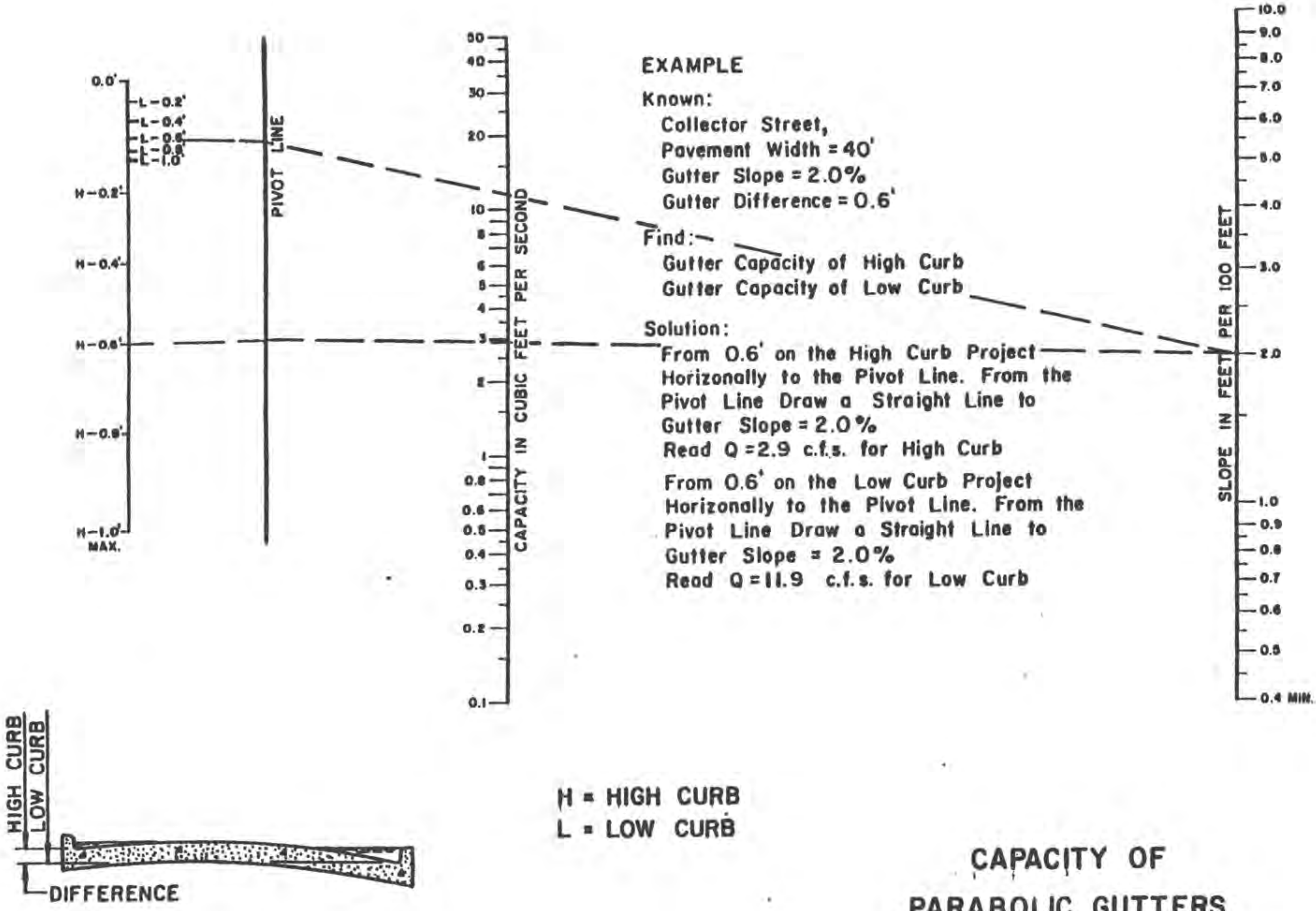
CAPACITY OF TRIANGULAR GUTTERS

(Rougness Coefficient n = .0175)



CAPACITY OF PARABOLIC GUTTERS (26' & 36' STREET WIDTHS)

FIGURE 4



EXAMPLE

Known:

Collector Street,
Pavement Width = 40'
Gutter Slope = 2.0%
Gutter Difference = 0.6'

Find:

Gutter Capacity of High Curb
Gutter Capacity of Low Curb

Solution:

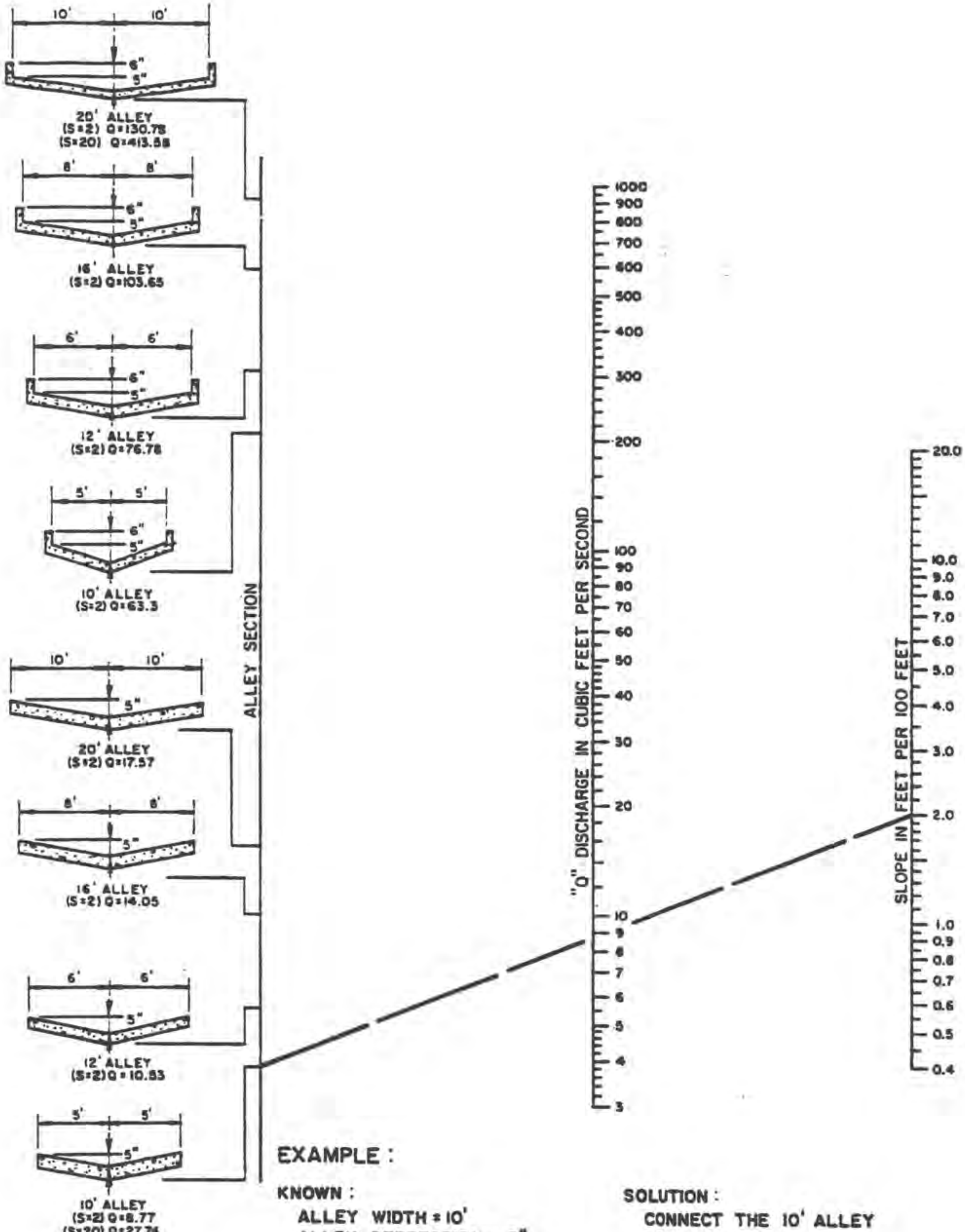
From 0.6' on the High Curb Project Horizonally to the Pivot Line. From the Pivot Line Draw a Straight Line to Gutter Slope = 2.0%
Read Q = 2.9 c.f.s. for High Curb

From 0.6' on the Low Curb Project Horizonally to the Pivot Line. From the Pivot Line Draw a Straight Line to Gutter Slope = 2.0%
Read Q = 11.9 c.f.s. for Low Curb

H = HIGH CURB
L = LOW CURB

FIGURE 5

CAPACITY OF PARABOLIC GUTTERS (40' F-F STREET WIDTH)



EXAMPLE :

KNOWN :

ALLEY WIDTH = 10'
ALLEY DEPRESSION = 5"
INVERT SLOPE = 2.2%

FIND :

INVERT FLOW (Q)

SOLUTION :

CONNECT THE 10' ALLEY SECTION WITH SLOPE = 2.2 % READ Q = 9.4 c.f.s.

Note :

The Capacities Obtained From This Nomograph are Based on a Straight Horizonal Alignment. Curved Alignments May Result In Reduced Capacity.

CAPACITY OF ALLEY SECTIONS
$n = 0.0175$

STORM DRAIN INLETS

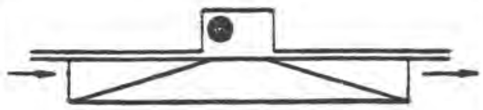
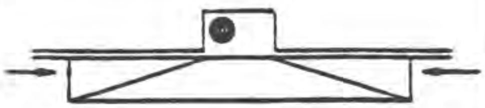
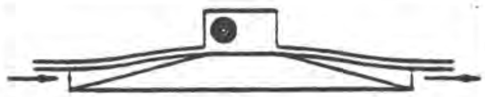
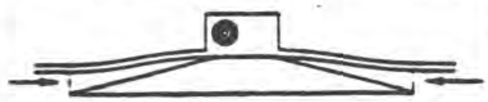
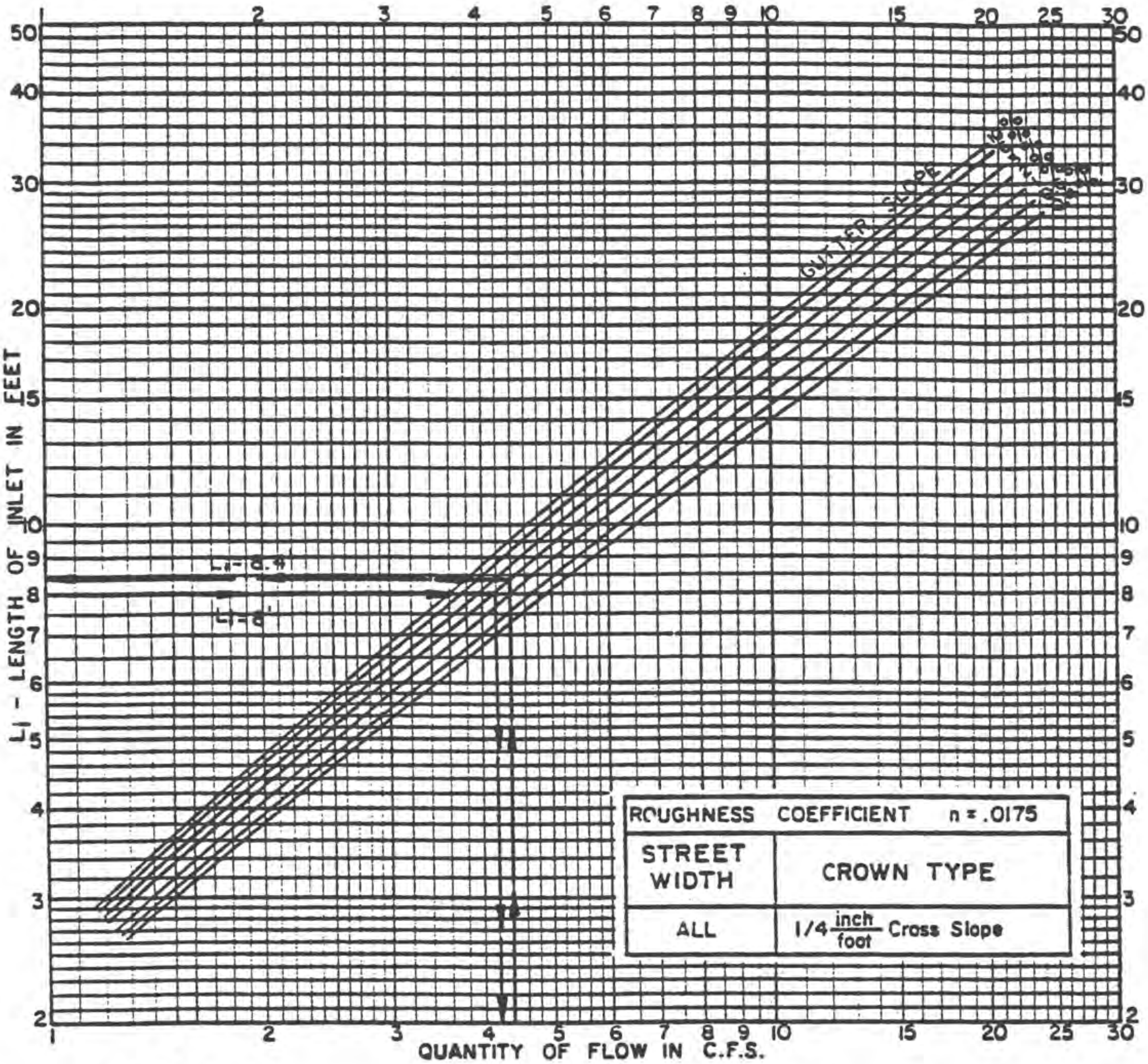
INLET TYPE	INLET DESCRIPTION	AVAIL. INLET SIZES	WHERE USED	DESIGN CURVES
I	 STANDARD CURB OPENING INLET ON GRADE	4' 6 8' 10'	26' LOCAL STREET 36' COLLECTOR STREET 40' COLLECTOR STREET ALLEYS	FIGURES 8 THROUGH 12
IA	 STANDARD CURB OPENING INLET AT LOW POINT	4' 6' 8' 10'	26' LOCAL STREET 36' COLLECTOR STREET ALLEY	FIGURE 13
II	 RECESSED CURB OPENING INLET ON GRADE	4' 6' 8 10'	40' COLLECTOR STREET 2-24' MAJOR STREET 2-33' MAJOR STREET 2-36' MAJOR STREET	FIGURES 8 THROUGH 12
IIA	 RECESSED CURB OPENING INLET AT LOW POINT	4' 6' 8' 10'	40' COLLECTOR STREET 2-24' MAJOR STREET 2-33' MAJOR STREET 2-36' MAJOR STREET	FIGURE 13
III	 COMBINATION INLET ON GRADE	4' 6' 8'	COMBINATION INLETS TO BE USED IN ALLEYS ONLY WITH WRITTEN APPROVAL FROM CITY ENGINEER.	FIGURES 14 THROUGH 16
IIIA	 COMBINATION INLET AT LOW POINT	4' 6' 8'	COMBINATION INLETS TO BE USED IN ALLEYS ONLY WITH WRITTEN APPROVAL FROM CITY ENGINEER.	FIGURE 20
IV	 GRATE INLETS	2 GRATE 3 GRATE 4 GRATE 6 GRATE	GRATE INLETS TO BE USED WHERE SPACE RESTRICTIONS PROHIBIT OTHER INLET TYPES OR AT LOCATIONS WITH NO CURB, AND WITH WRITTEN APPROVAL FROM THE CITY ENGINEER	FIGURES 16,17, 18,19 & 21
V	 TYPE 'Y' DROP INLET	2x2' 3'x3' 4'x4'	OPEN CHANNELS	FIGURE 22

FIGURE 7



EXAMPLE

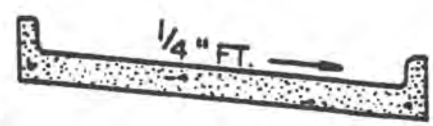
Known:
Pavement Width = 24'
Gutter Slope = 2.0 %
Pavement Cross Slope = 1/4" / 1'
Gutter Flow = 4.4 cfs

Find:
Length of Inlet Required (L_i)

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

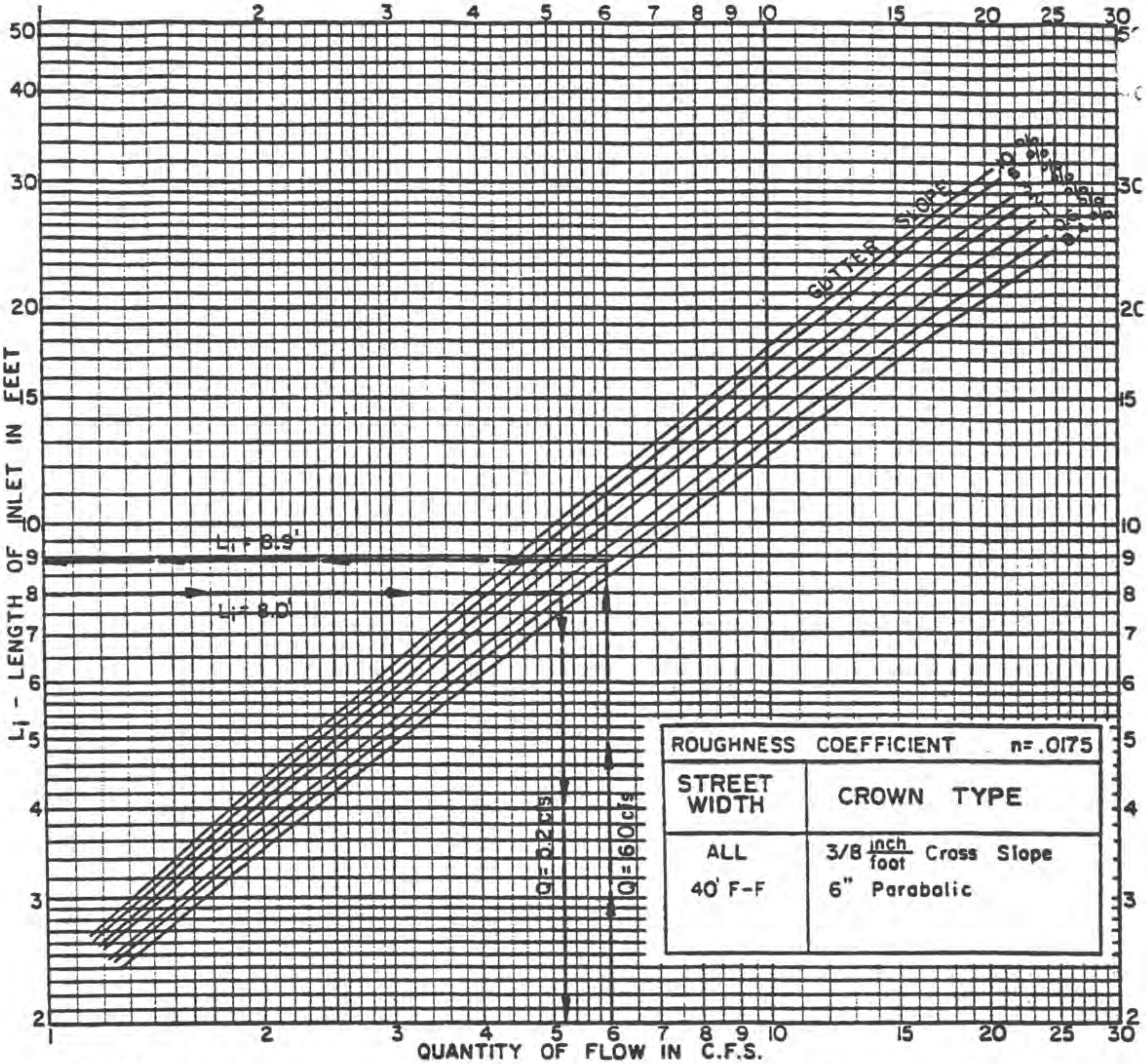
Decision:
1. Use 10' Inlet
No Flow Remains in Gutter
2. Use 8' Inlet
Intercept Only Part of Flow

Use 8' Inlet
Enter Graph at L_i = 8'
Intersect Slope = 2.0 %
Read Q = 4.2 cfs
Remaining Gutter Flow =
4.4 cfs − 4.2 cfs = 0.2 cfs



RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES ON GRADE

FIGURE 8



EXAMPLE

Known:

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

Find:

Length of Inlet Required (L_i)

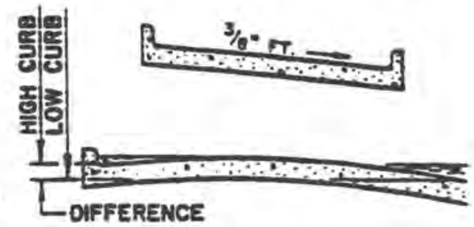
Solution:

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

Decision:

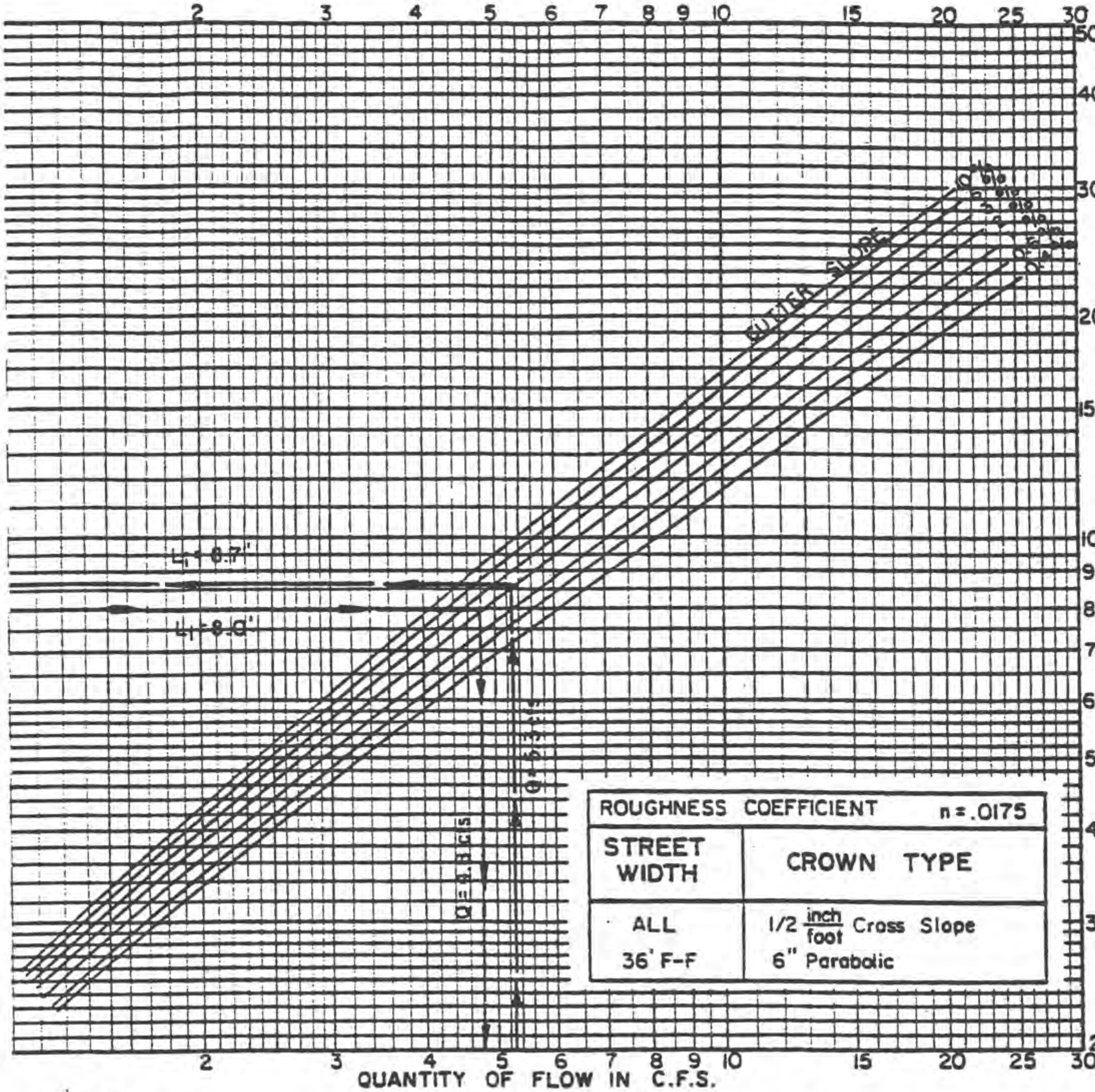
1. Use 10' Inlet
   No Flow Remains in Gutter
2. Use 8' Inlet
   Intercept Only Part of Flow

Use 8' Inlet
Enter Graph at L_i = 8'
Intersect Slope = 0.6 %
Read Q = 5.2 cfs
Remaining Gutter Flow =
6.0 cfs − 5.2 cfs = 0.8 cfs



RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES ON GRADE

FIGURE 9



EXAMPLE

Known:

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

Find:

Length of Inlet Required (L_i)

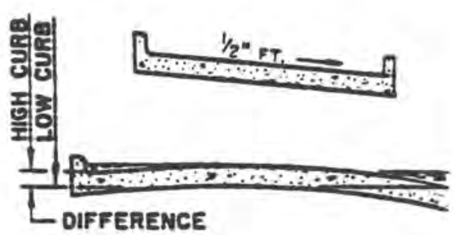
Solution:

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

Decision:

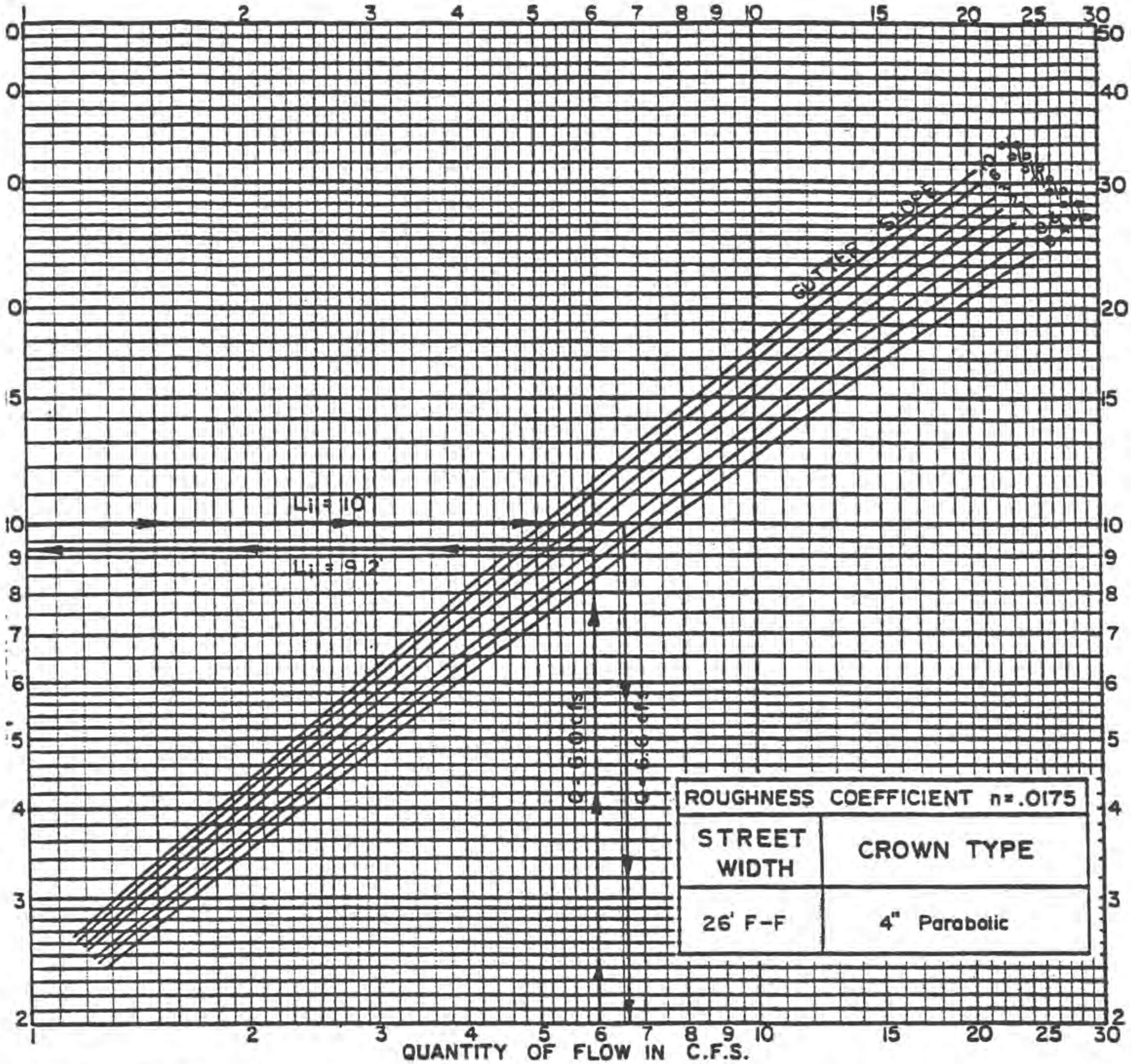
1. Use 10' Inlet
   No Flow Remains in Gutter
2. Use 8' Inlet
   Intercept Only Part of Flow

Use 8' Inlet
Enter Graph at L_i = 8'
Intersect Slope = 2%
Read Q = 4.8 cfs
Remaining Gutter Flow =
5.3 cfs − 4.8 cfs = 0.5 cfs



RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES ON GRADE

FIGURE 10



EXAMPLE

Known:

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

Find:

Length of Inlet Required (L_i)

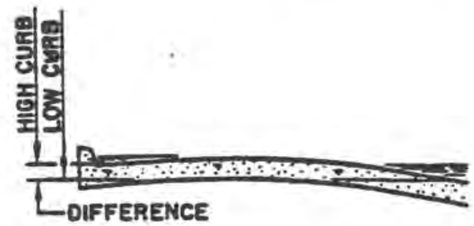
Solution:

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

Decision:

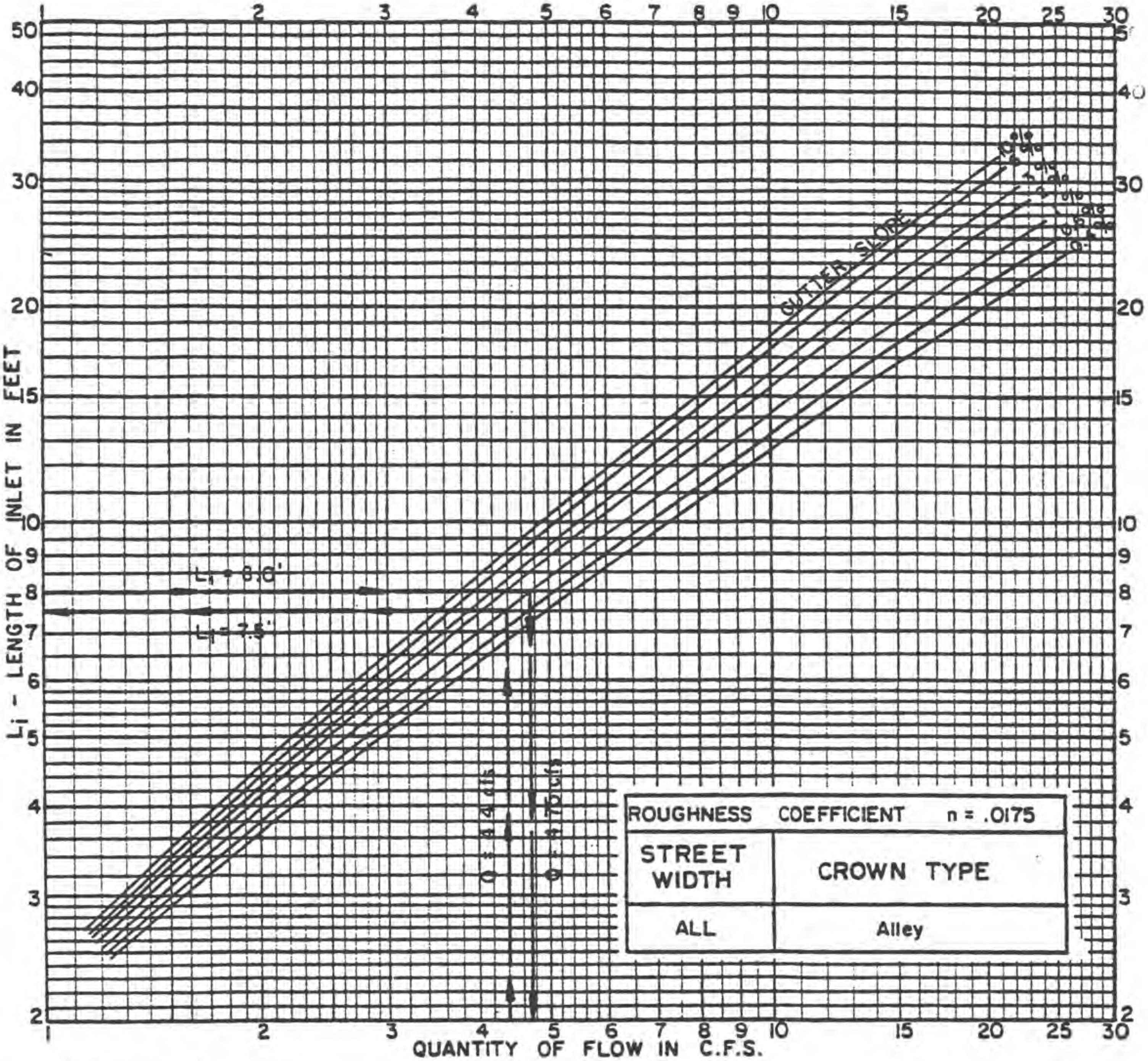
1. Use 10' Inlet
   No Flow Remains in Gutter
2. Use 8' Inlet
   Intercept Only Part of Flow

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

FIGURE 11



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:

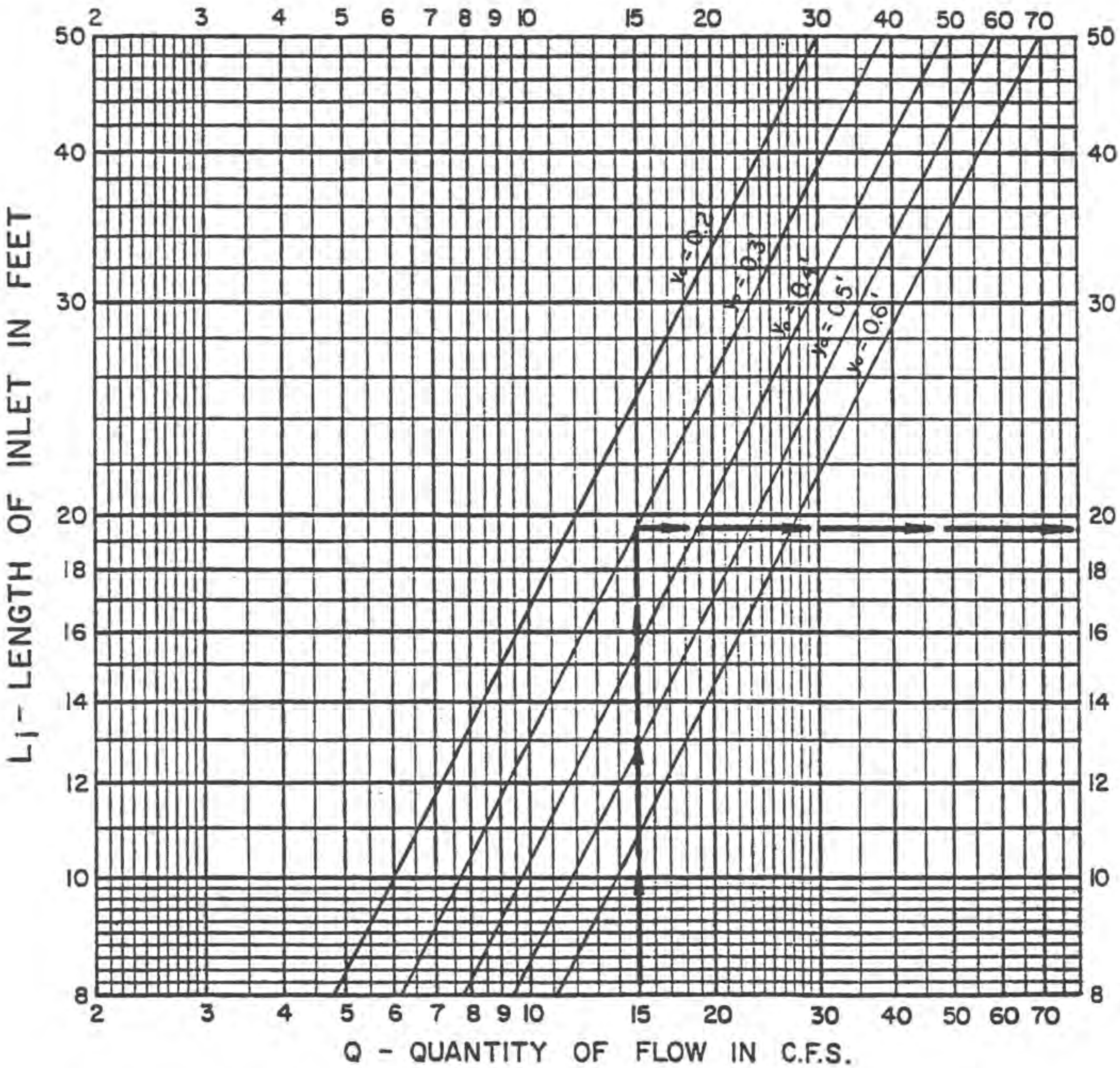
Quantity of Flow = 15.0 c.f.s.
Maximum Depth of Flow Desired in Gutter At Low Point (y_o) = 0.3'

Find:

Length of Inlet Required (L_i)

Solution:

Enter Graph at 15.0 c.f.s.
Intersect y_o = 0.3'
Read L_i = 19.5'
Use 20' Inlet



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

RECESSED AND STANDARD CURB OPENING INLET CAPACITY CURVES AT LOW POINT

FIGURE 13

EXAMPLE

Known:

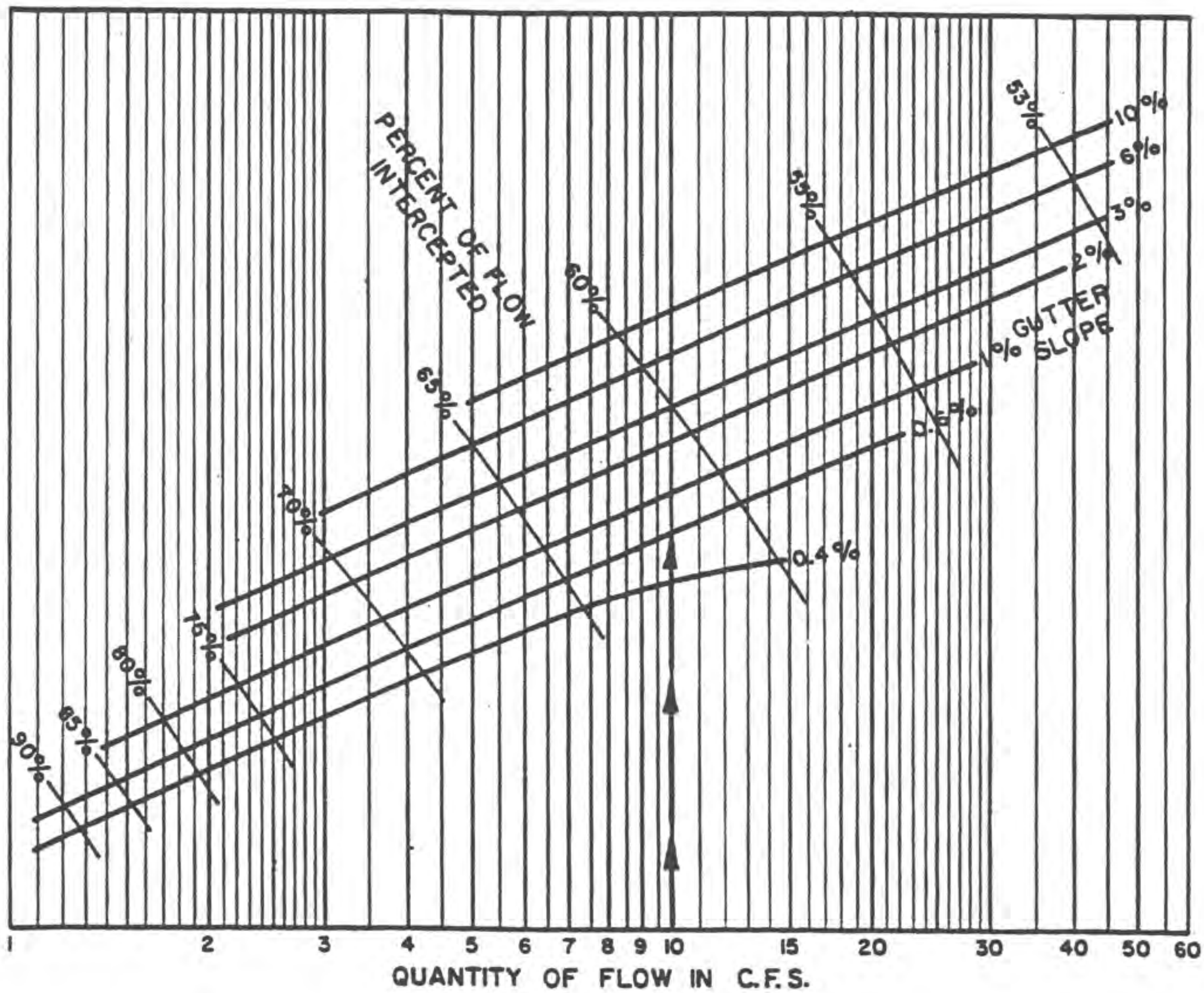
Quantity of Flow = 10.0 c.f.s.
Gutter Slope = 0.6 %

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 c.f.s − 6.2 c.f.s. = 3.8 c.f.s.



TWO GRATE COMBINATION INLET CAPACITY CURVES ON GRADE

EXAMPLE

Known:

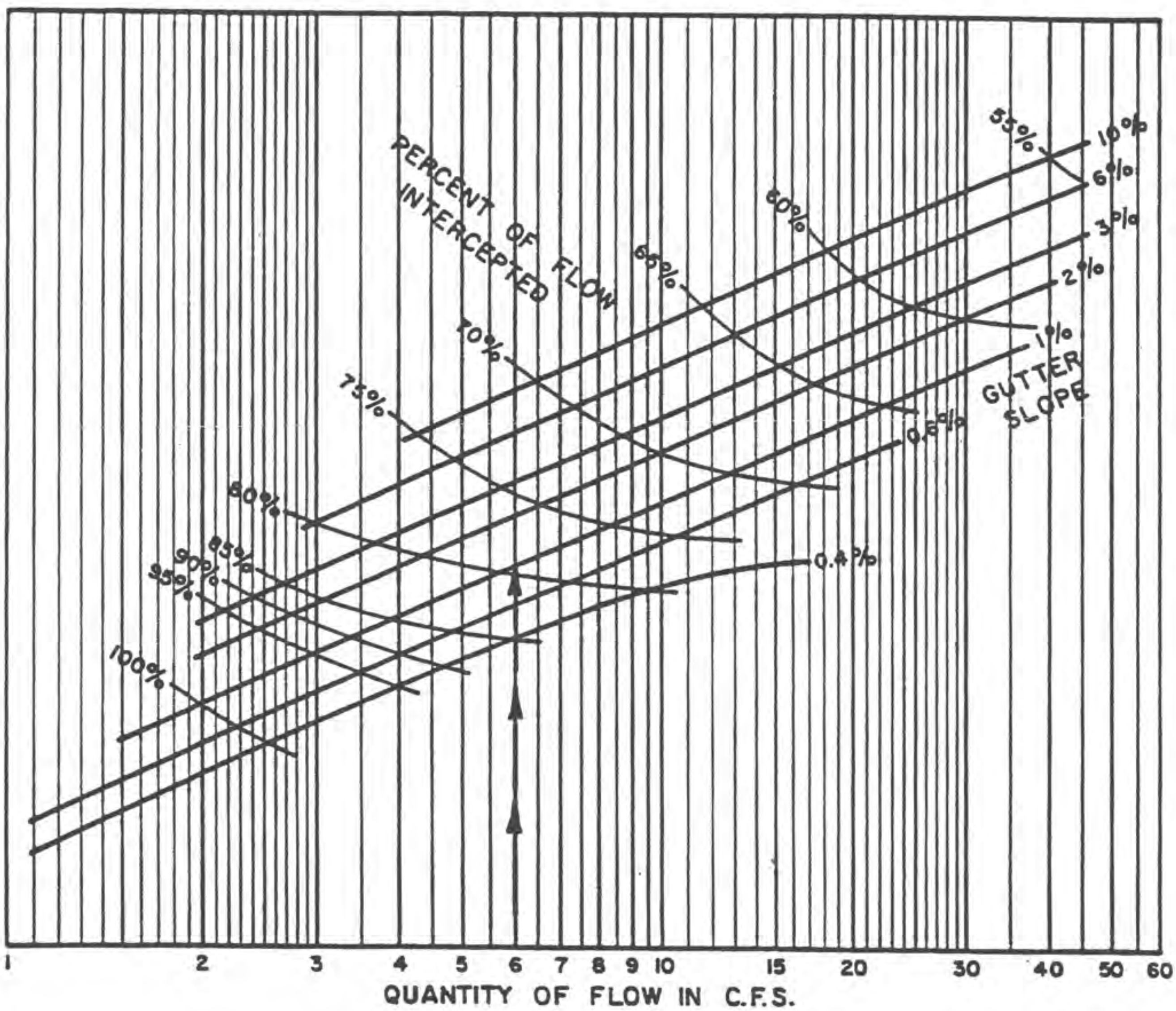
Quantity of Flow = 6.0 c.f.s.
Gutter Slope = 1.0 %

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

FIGURE 15

EXAMPLE

Known:

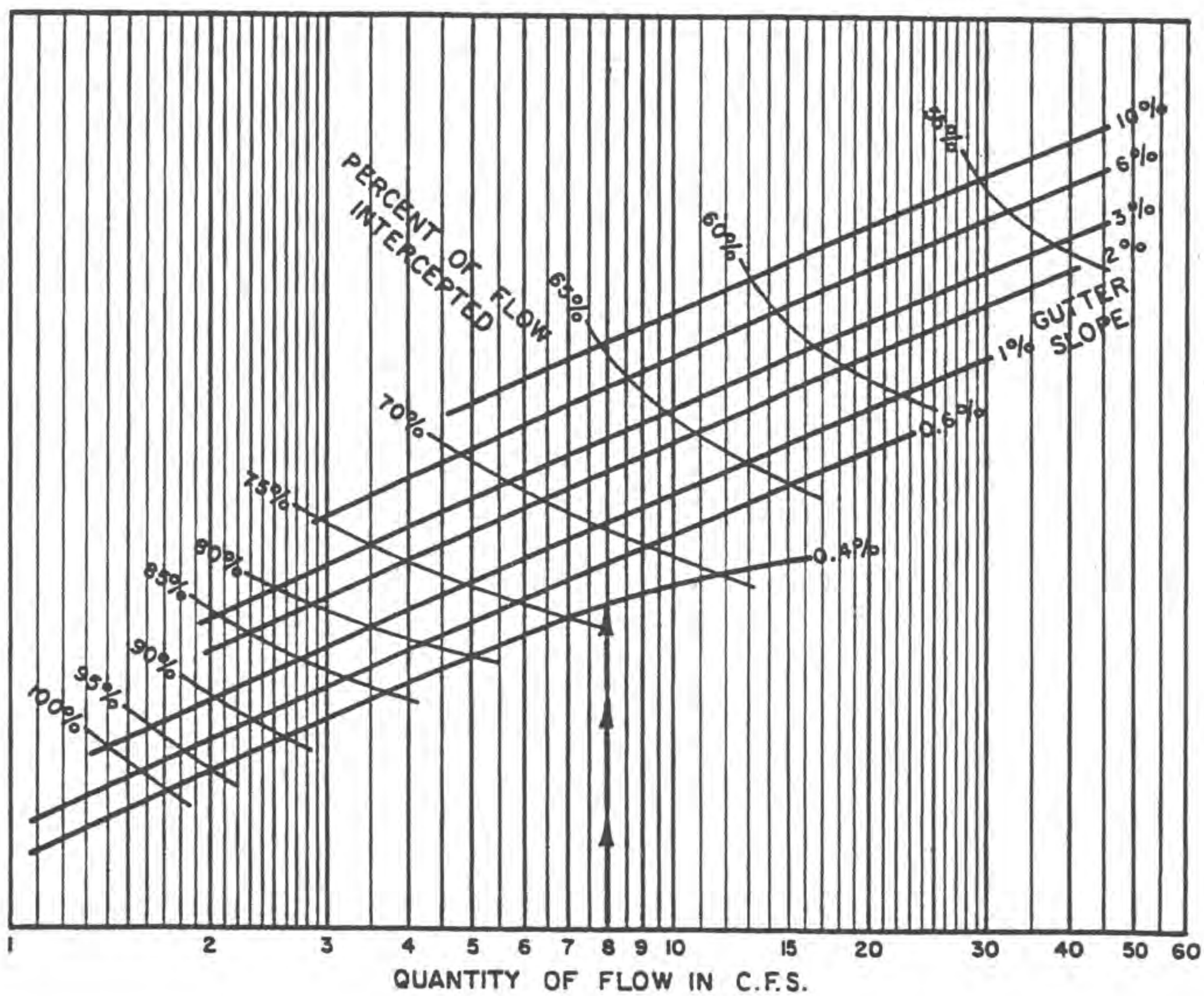
Quantity of Flow = 8.0 c.f.s.
Gutter Slope = 0.4%

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 INLET
CAPACITY CURVES
ON GRADE

EXAMPLE

Known:

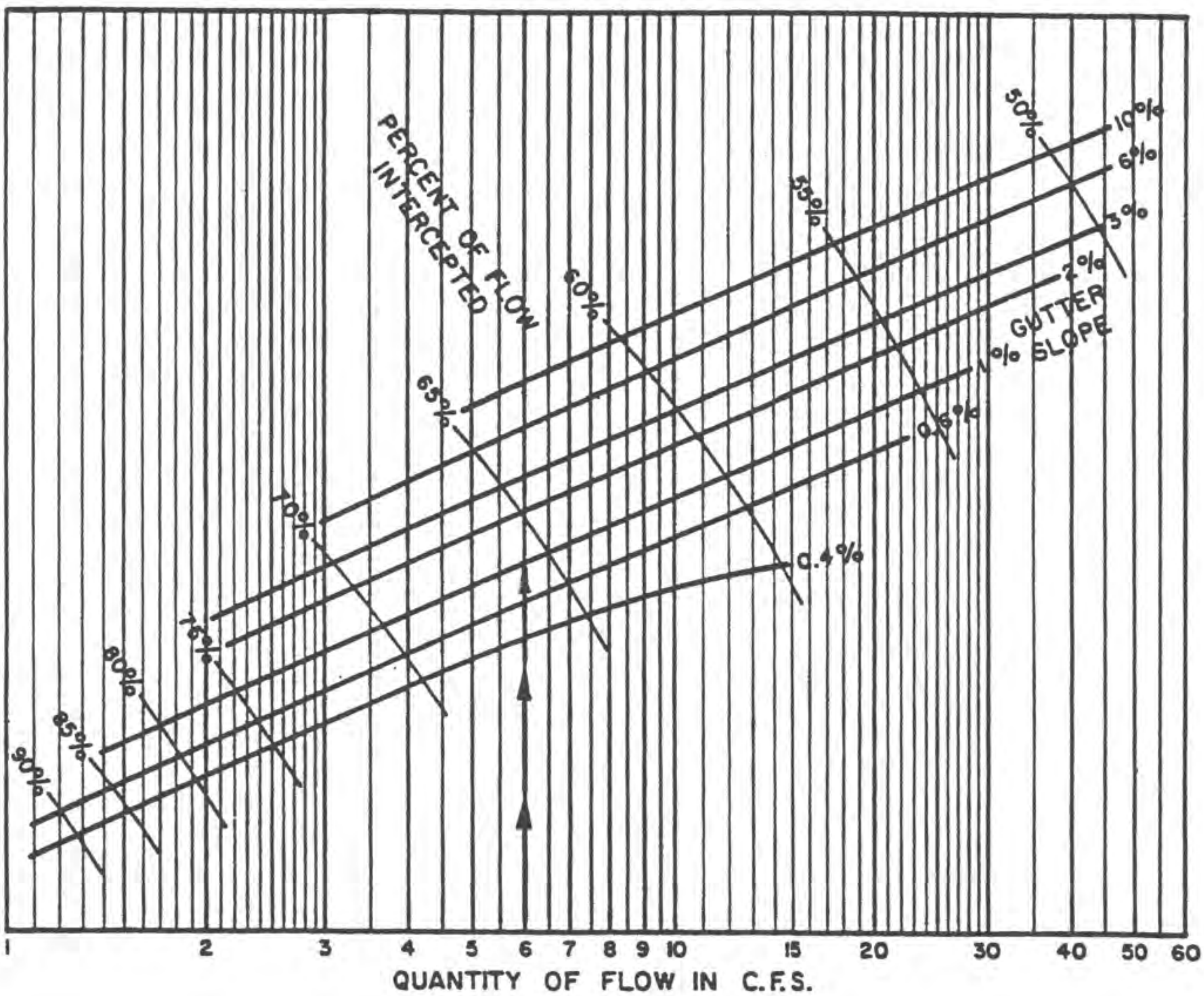
Quantity of Flow = 6.0 c.f.s.
Gutter Slope = 1.0%

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:

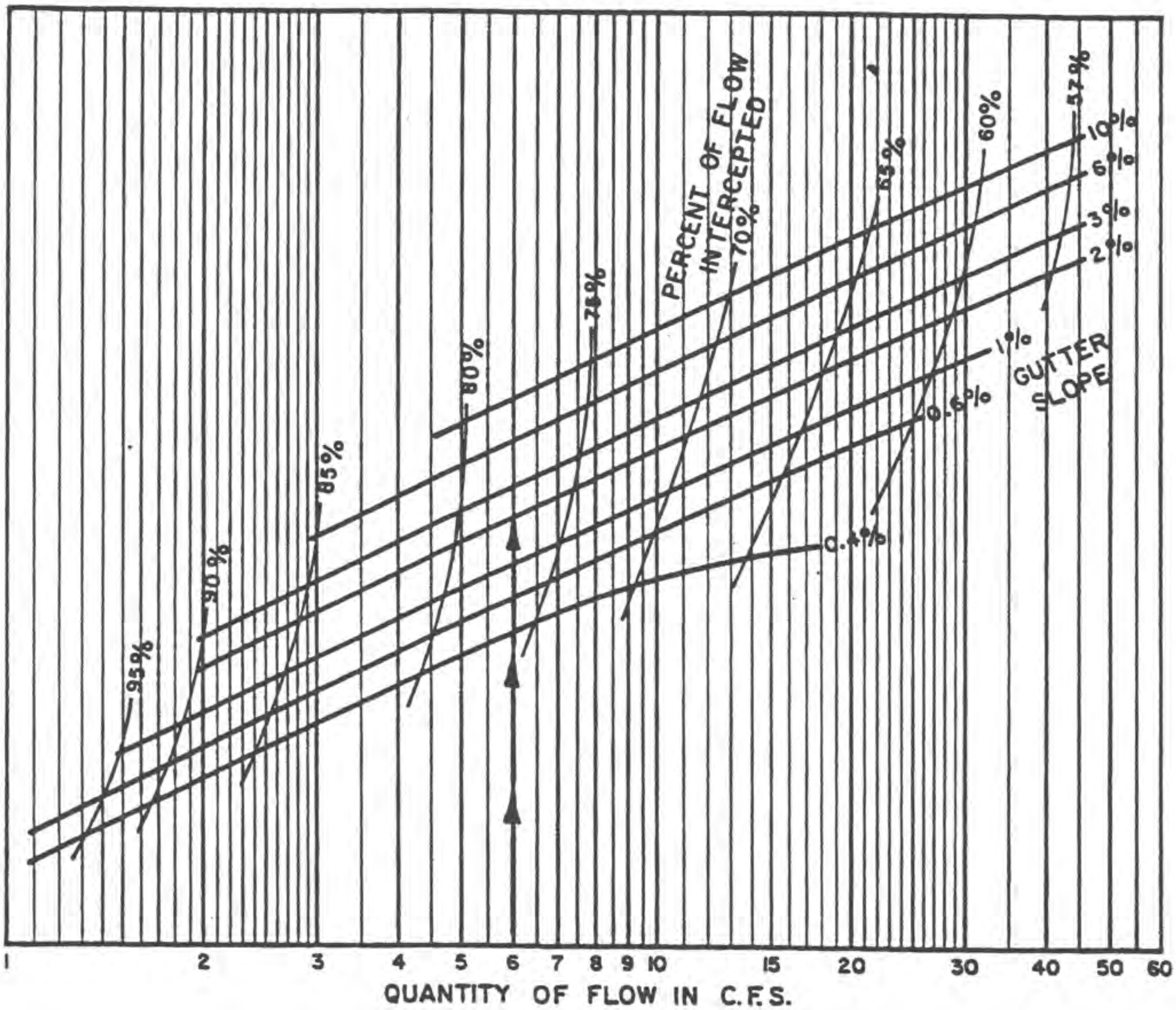
Quantity of Flow = 6.0 c.f.s.
Gutter Slope = 1.0 %

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

FIGURE 18

EXAMPLE

Known:

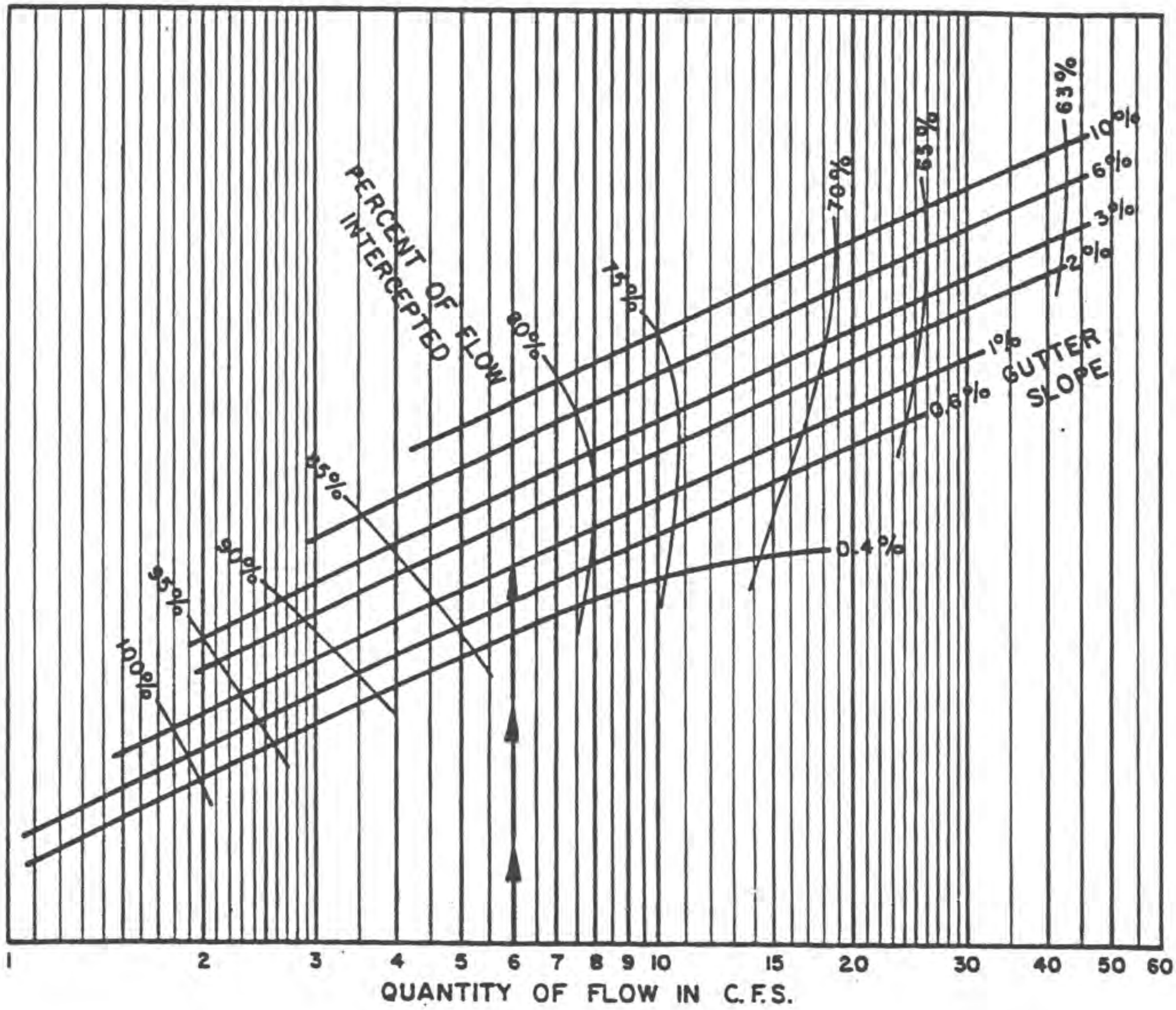
Quantity of Flow = 6.0 c.f.s.
Gutter Slope = 1.0%

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

FIGURE 19

EXAMPLE:

Known:

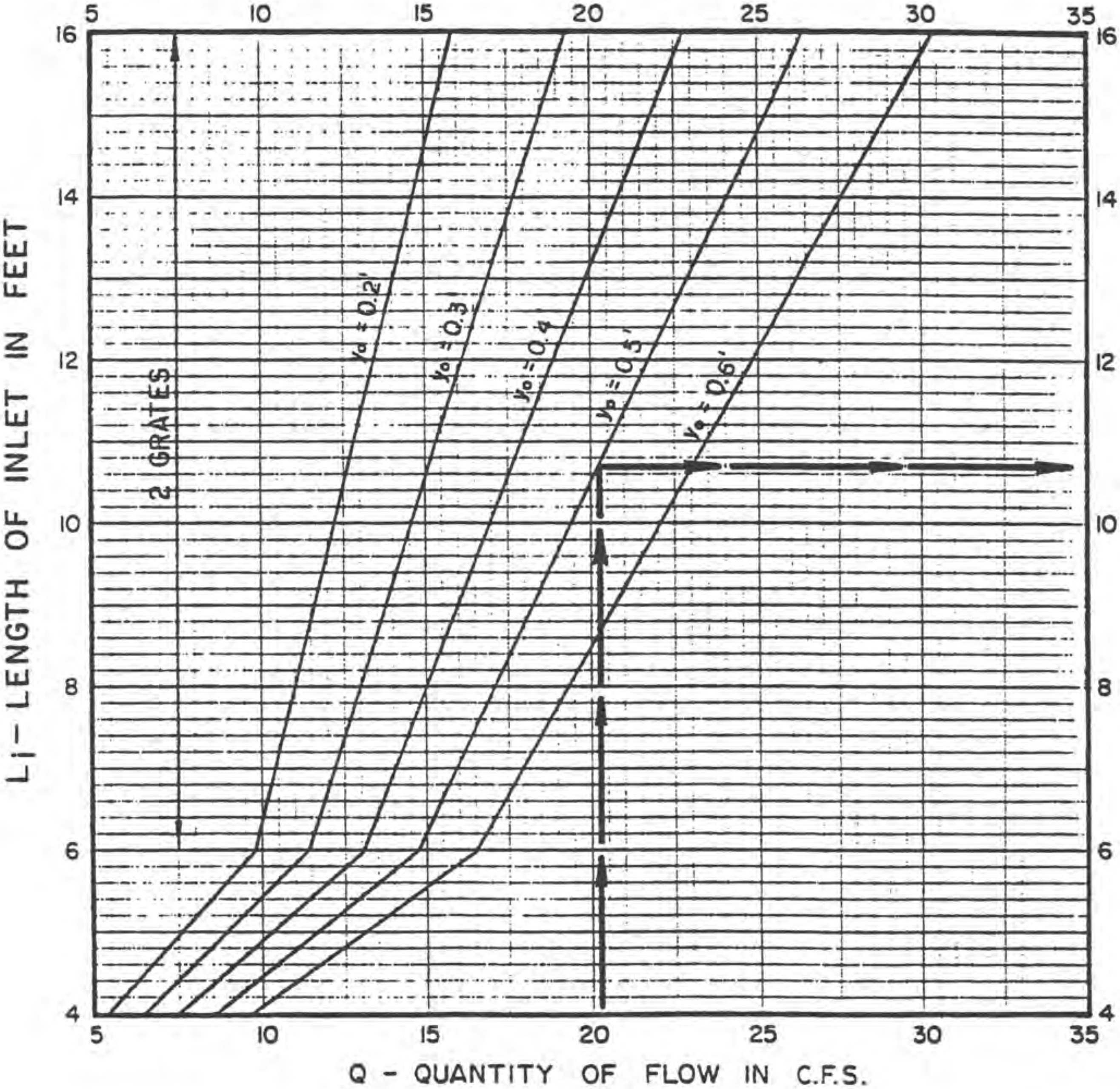
Quantity of Flow = 20.0c.f.s.
Maximum Depth of Flow Desired in Gutter At Low Point (y_o) = 0.5'

Find:

Length of Inlet Required (L_i)

Solution:

Enter Graph at 20.0c.f.s.
Intersect y_o = 0.5'.
Read L_i = 10.6'
Use 12' Inlet with 2 grates



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 :

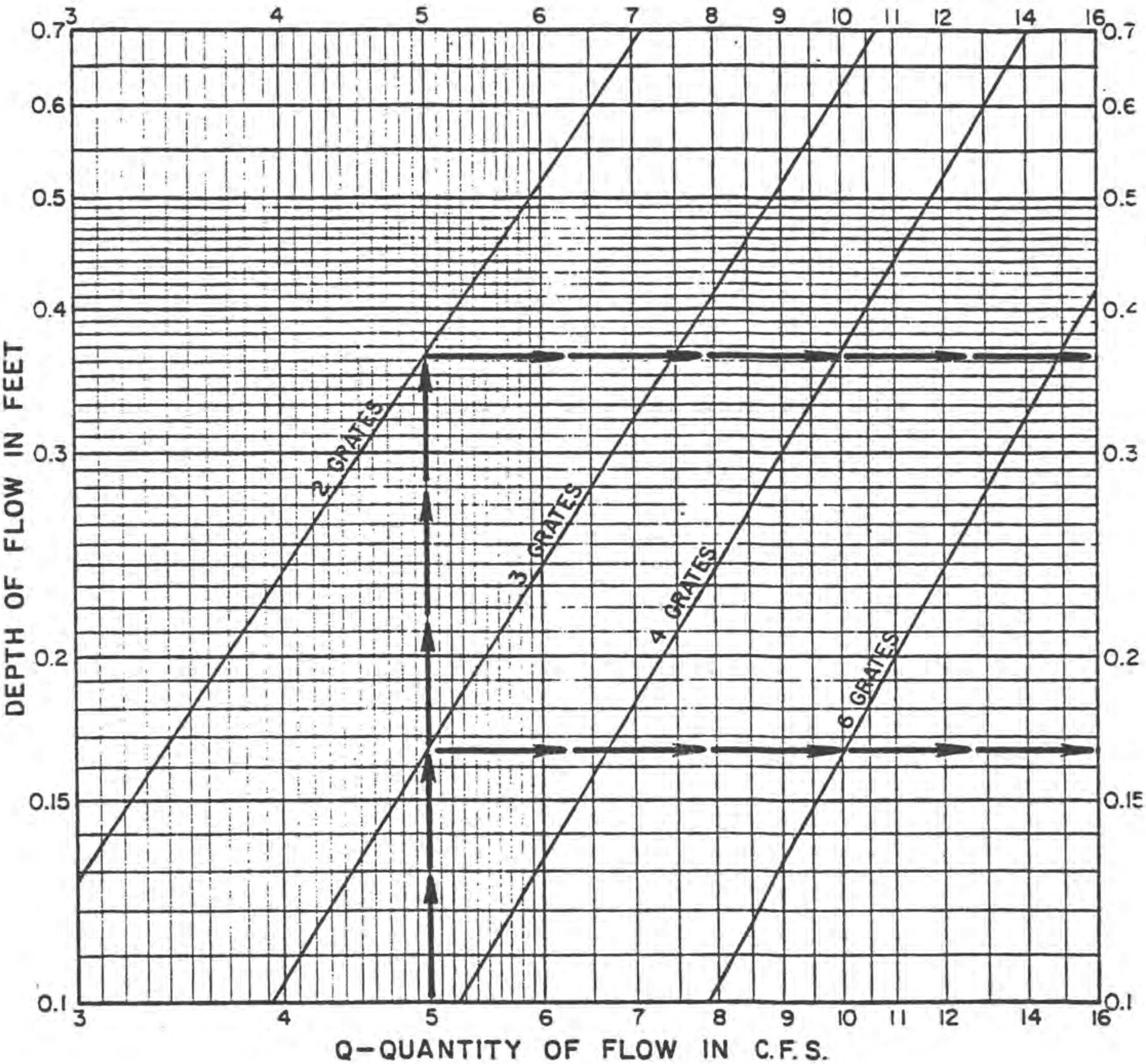
Quantity of Flow = 5.0 c.f.s.
Maximum Depth of Flow Desired at Low Point = 0.3

Find :

Inlet Required

Solution :

Enter Graph at 5.0 c.f.s.
Intersect 3 - Grate at 0.165
Intersect 2 - Grate at 0.365
Use 3 - Grate



GRATE INLET
CAPACITY CURVES
AT LOW POINT

EXAMPLE:

Known:

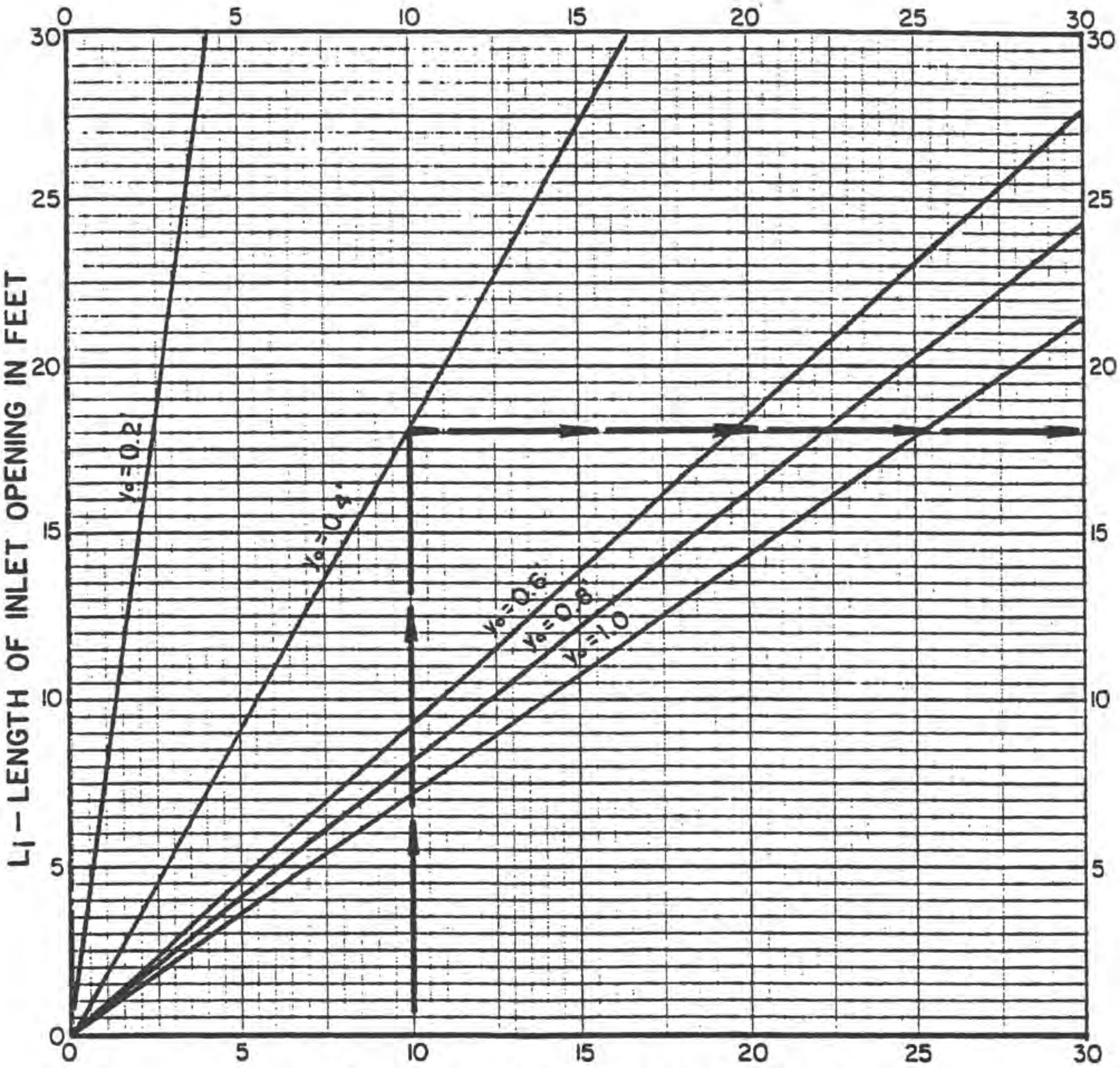
Quantity of Flow = 10.0c.f.s.
Maximum Depth of Flow Desired
(y_o) = 0.4'

Find:

Length of Inlet Opening Required (L_i)

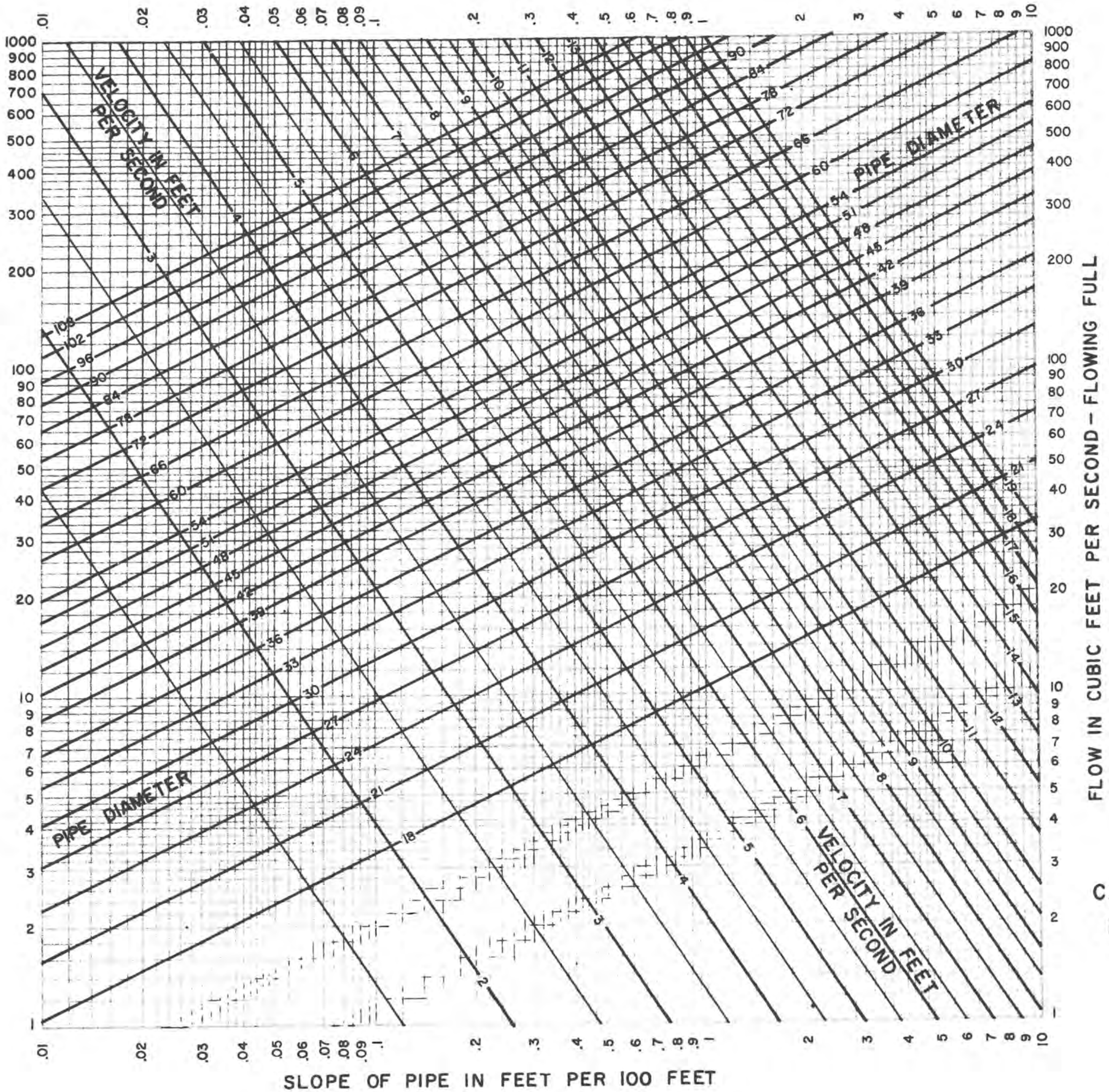
Solution:

Enter Graph at 10.0c.f.s.
Intersect y_o = 0.4'
Read L_i = 18.0'
Use 20' of Inlet ; 5' x 5'



Standard Drop Inlet Sizes:
2' x 2' ; Li = 8'
3' x 3' ; Li = 12'
4' x 4' ; Li = 16'

TYPE 'Y'
DROP INLET
CAPACITY CURVES
AT LOW POINT



A GRAPHICAL SOLUTION OF MANNING'S EQUATION

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

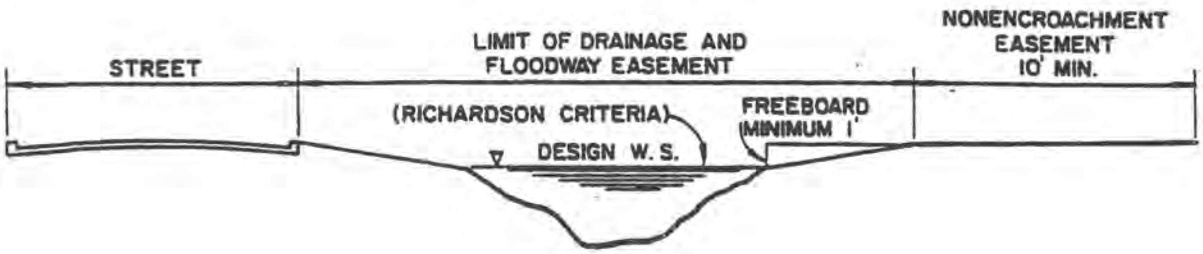
$n = 0.013$

CAPACITY OF CIRCULAR PIPES FLOWING FULL

FIGURE 23

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 RICHARDSON;
(3) UNSANITARY OR UNACCEPTABLE DRAINAGE CONDITIONS DO NOT EXIST IN THE CREEK;
(4) APPROVED BY THE CITY ENGINEER.



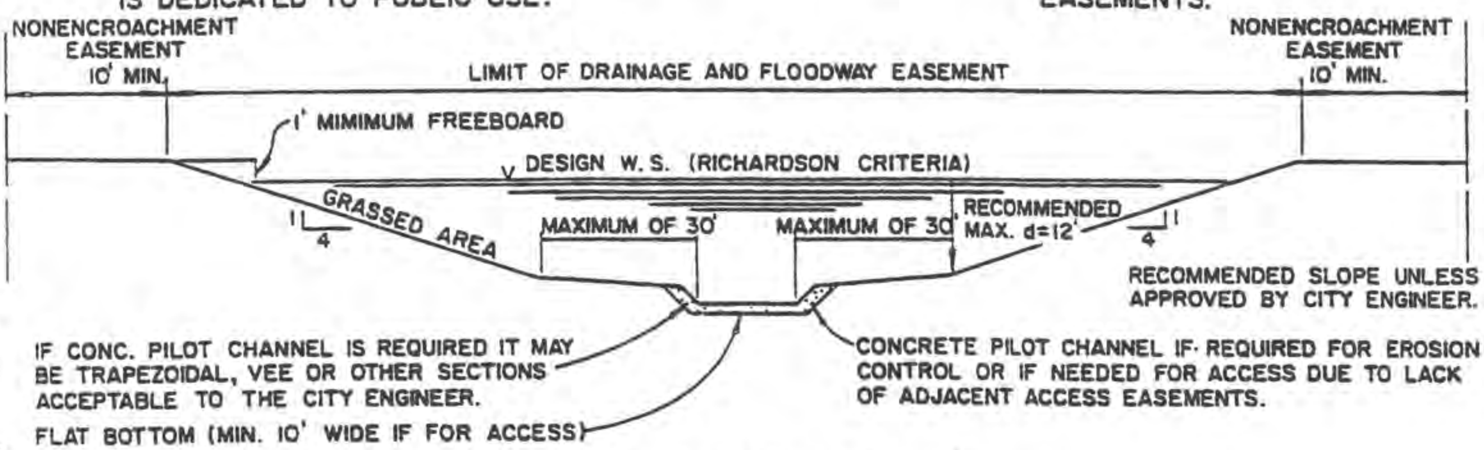
UNIMPROVED CHANNEL

TYPE I – NATURAL

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 ON AT LEAST ONE SIDE OF TYPE I CHANNELS IF THE DRAINAGE AND FLOODWAY IS DEDICATED TO PUBLIC USE.

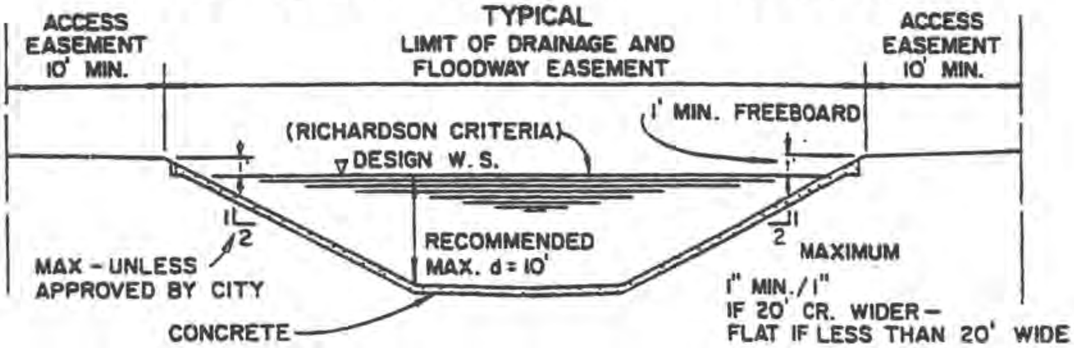
NOTE: NO ENCROACHMENTS SHALL BE BE PERMITTED IN ACCESS EASEMENTS.



UNLINED CHANNELS

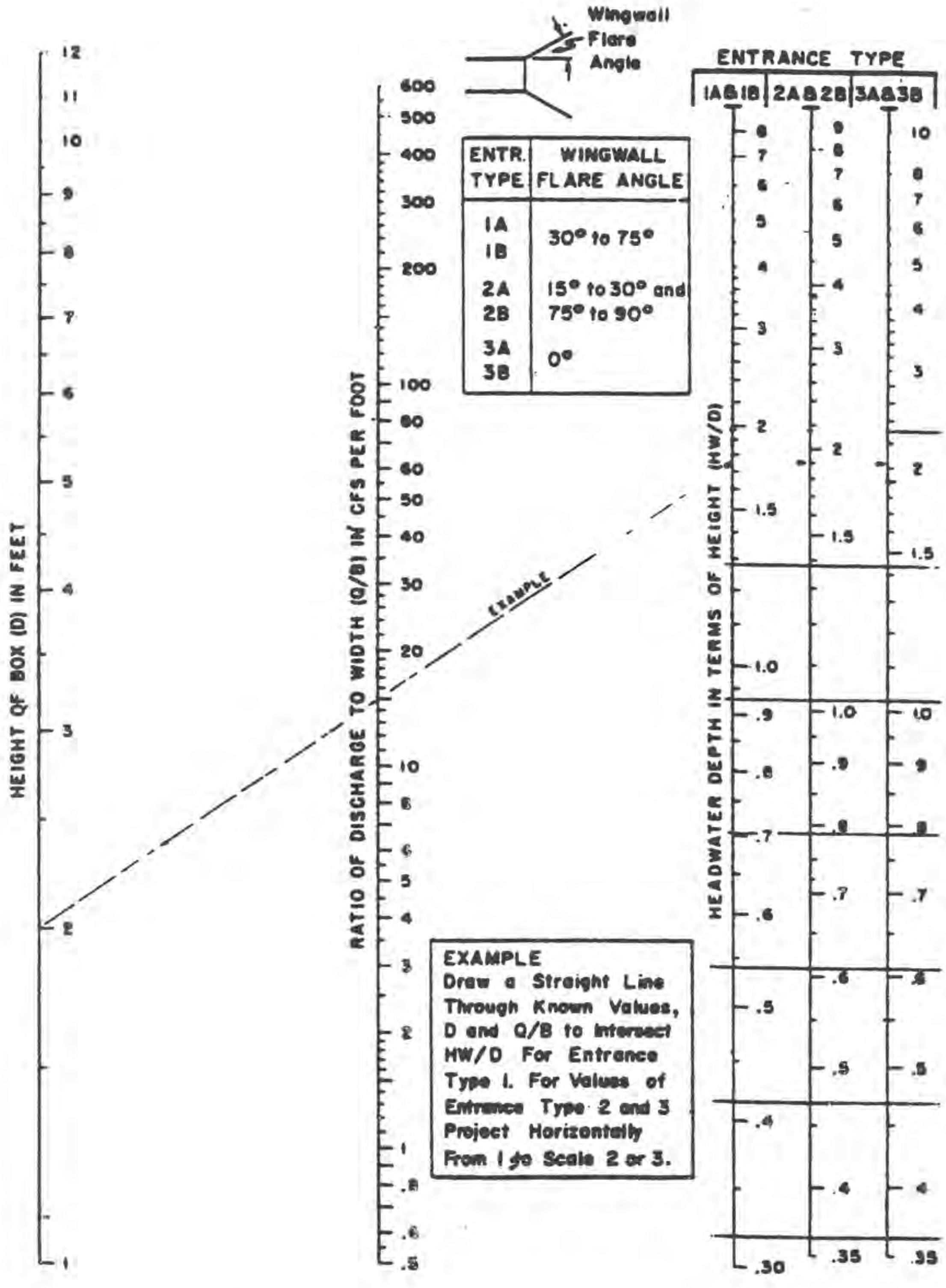
TYPE II – UNLINED WITH MAINTENANCE SECTION

TYPICAL



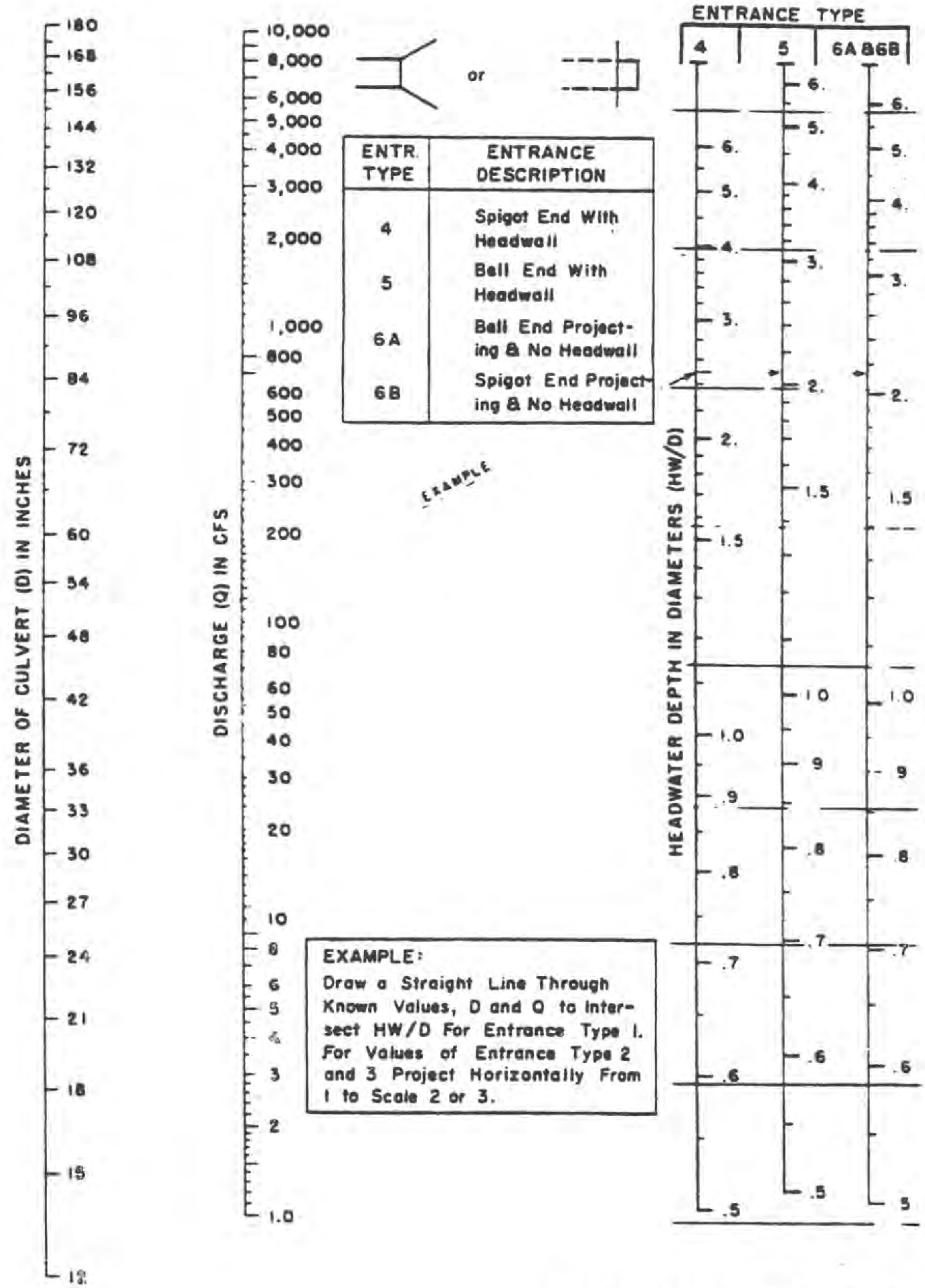
TYPE III – LINED

OPEN CHANNEL TYPES



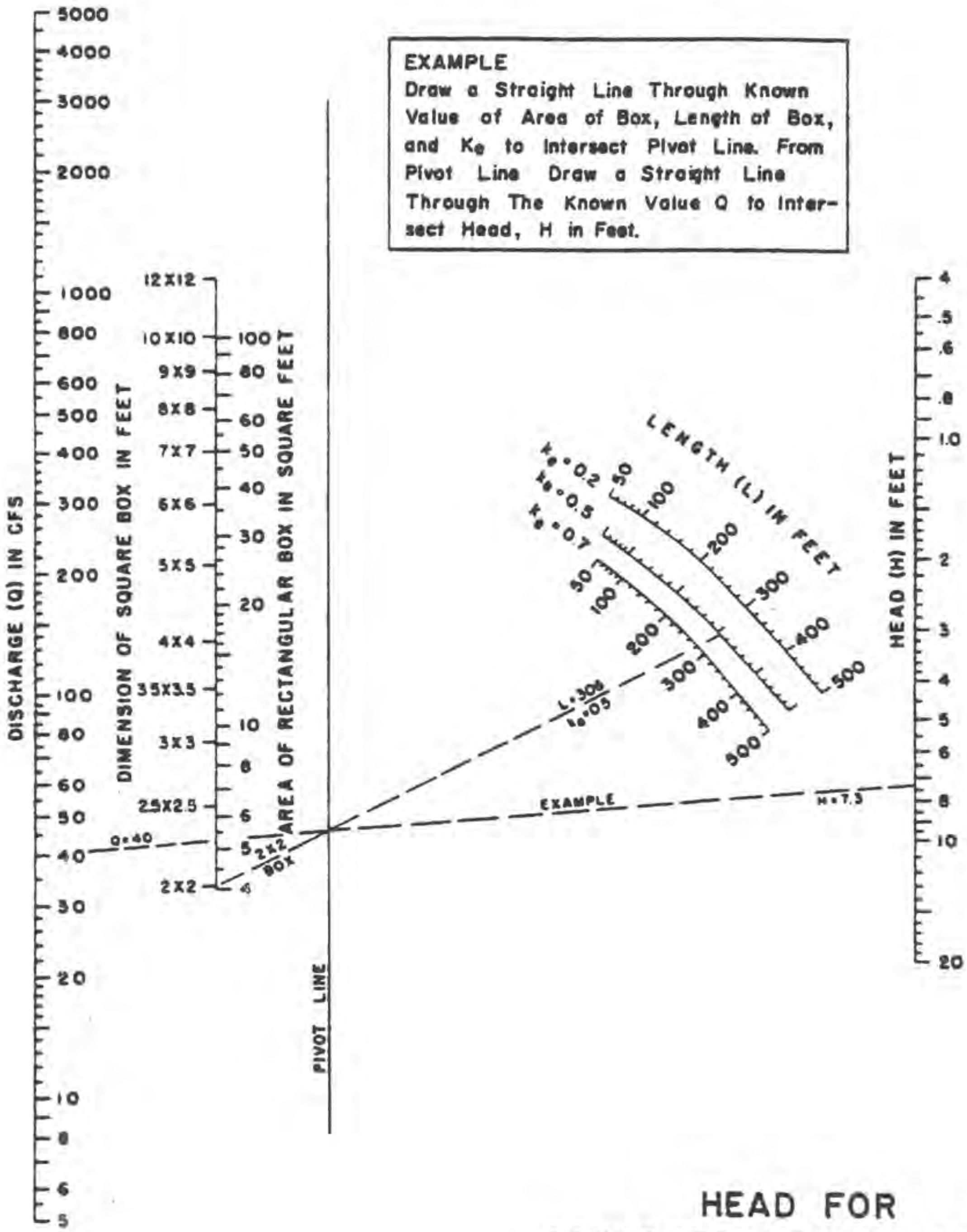
BUREAU OF PUBLIC ROADS JAN 1963

HEADWATER DEPTH FOR CONCRETE BOX CULVERT WITH INLET CONTROL



BUREAU OF PUBLIC ROADS JAN 1963

HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL



HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL

n = 0.012

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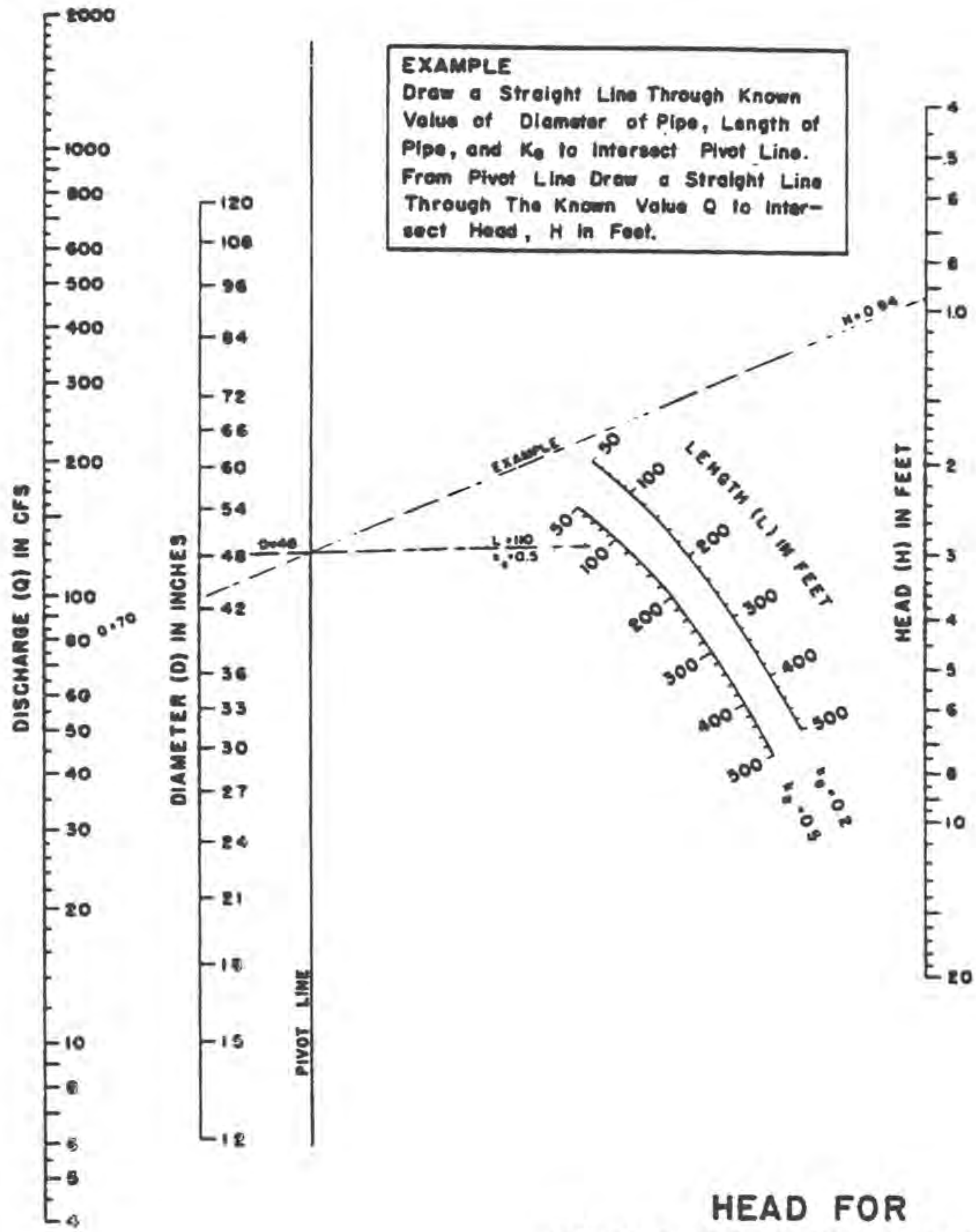


HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL

$n = 0.012$

BUREAU OF PUBLIC ROADS JAN 1963

EXAMPLE

Known:

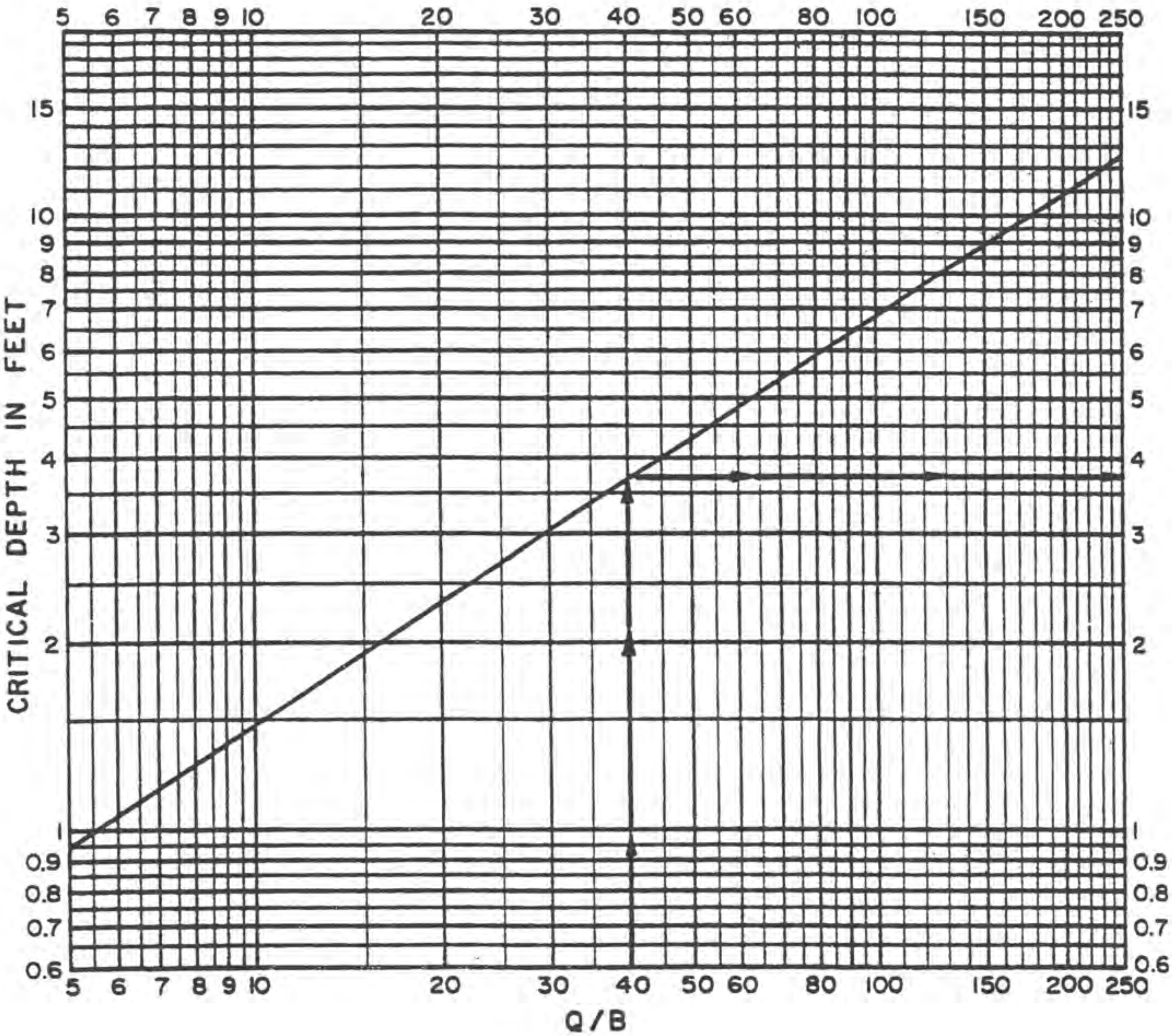
Discharge = 200 c.f.s.
Width of Conduit = 5'
Q/B = 40

Find:

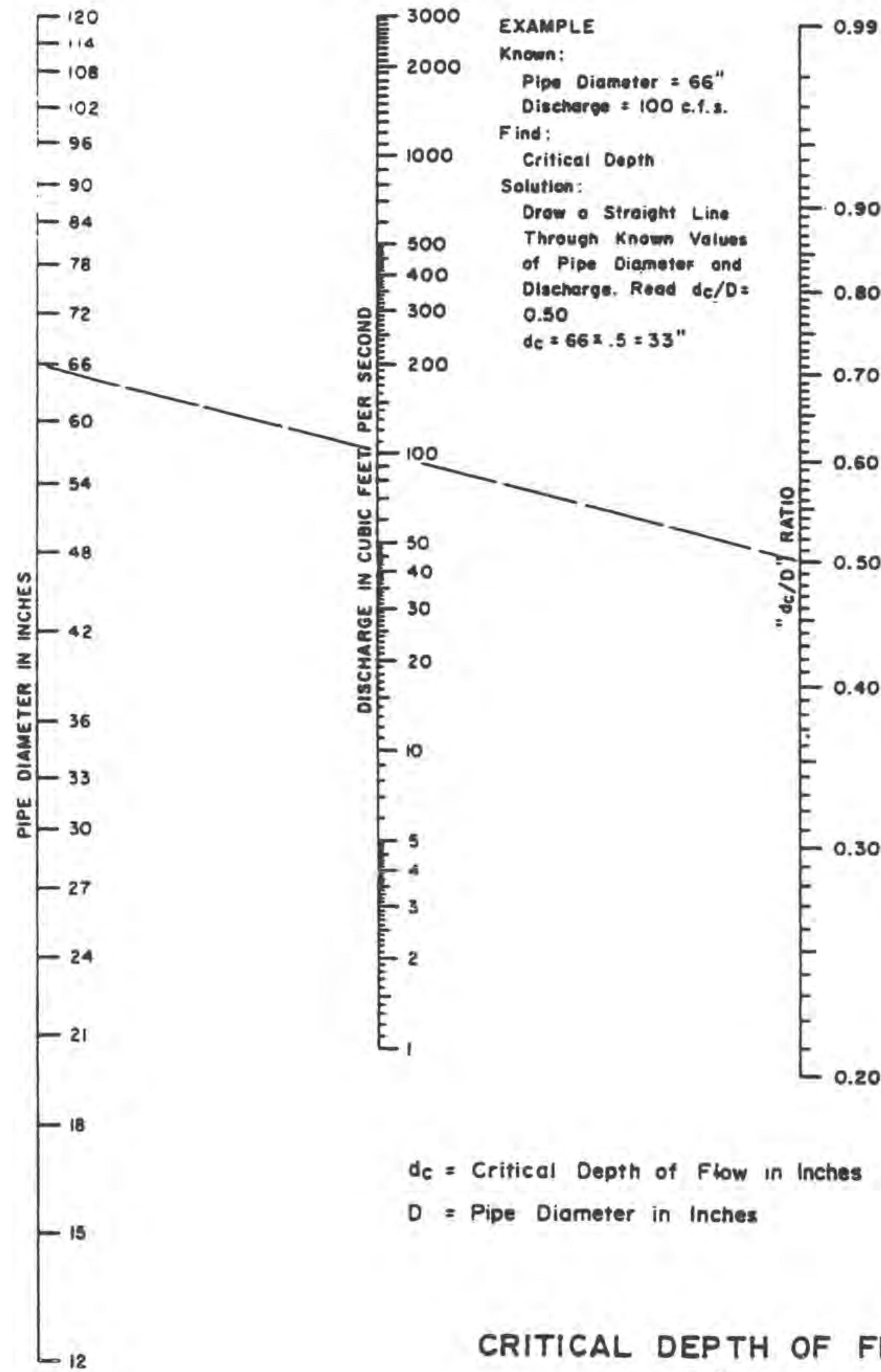
Critical Depth

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

TEXAS HIGHWAY DEPARTMENT

SECTION
VIII

STORM WATER RUNOFF CALCULATIONS - FORM "A"

Column 1 Location of the drainage structure for which the runoff calculation is being made or a design point on an open channel.

Columns 2 - 6 are to be used in calculating runoff by the Rational Method.

Column 2 Obtained from TABLE 1.

Column 3 Using the appropriate Design Storm Frequency, and the Time of Concentration in Column 2, Intensity is obtained from FIG. 1.

Column 4 Size of the drainage area tributary to the point of design shown in Column 1.

Column 5 Taken from TABLE 1 and is a weighted composite value if several different zoning districts fall within the drainage area.

Column 6 Column 3 multiplied by Columns 4 and 5.

Columns 7 - 19 are to be used in calculating runoff by the Unit Hydrograph Method.

Column 7 Taken from TABLE 2.

Column 8 Measured distance along the stream course from the uppermost limit of the drainage area to the point of design shown in Column 1.

Column 9 Measured distance along the stream course from the point of design shown in Column 1 to the measured center of gravity of the drainage area.

Column 10 A computed value using the values shown in Columns 7, 8 and 9.

Column 11 Taken from TABLE 2.

Column 12 Column 11 divided by Column 10.

Column 13 Size of the drainage area tributary to the point of design shown in Column 1.

Column 14 Column 12 multiplied by Column 13.

Column 15	Using the appropriate Design Storm Frequency and a duration of two hours, this value is obtained from FIG. 1.
Column 16	Obtained by multiplying the value in Column 15 times two.
Column 17	Constant value of 1.11 inches for the Richardson geographic area.
Column 18	Result of subtracting Column 17 from Column 16.
Column 19	Column 14 multiplied by Column 18.
Column 20	The flow used for design depends on the size of the drainage area. If the size of the drainage area is less than 500 acres, Q_R should be entered. If the drainage area is larger than 500 acres, Q_u should be entered.

CITY OF RICHARDSON

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DESIGN STORM FREQUENCY ______________

STORM WATER RUNOFF CALCULATIONS

BY ______________

DATE ______________

LOCATION	DESIGN FLOW (Q_R) BY RATIONAL METHOD					DESIGN FLOW (Q_u) BY UNIT HYDROGRAPH METHOD													FLOW USED FOR DESIGN
	Time of Concen-tration (minutes)	Intensity "I" (in./hr.)	Drainage Area "A" (Acres)	Runoff Coefficient "C"	Design Flow $Q_R = C \times I \times A$	Coefficient "C_t"	Length "L" (miles)	Length "L_{ca}" (miles)	Lag Time $t_p = C_t(L \times L_{ca})^{.3}$ (hours)	Coefficient C_p640	Peak Unit Flow $q_p = \frac{C_p640}{t_p}$ (c.f.s./sq.mi.)	Drainage Area "A" (sq. miles)	Peak Flow $Q_p = q_p \times A$ (c.f.s.)	Intensity "I" at 2 Hours (in./hr.)	Design Storm "S_D"="I"x2 (inches)	Initial and Subsequent Losses "L_{IS}" (inches)	Total Runoff $R_T = S_D - L_{IS}$ (inches)	Design Flow $Q_u = R_T \times Q_p$ (c.f.s.)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

FORM A

<u>INLET DESIGN CALCULATIONS - FORM "B"</u>

Column 1   Inlet number or designation. The first inlet shown is the most upstream.

Column 2   Construction plan station of the inlet.

Column 3   Design Storm Frequency is same as the Design Storm Frequency of the storm sewer.

Column 4   Time of concentration for each inlet is taken from TABLE 1.

Column 5   Using the time of concentration and the Design Storm Frequency, rainfall intensity is taken from FIG. 1.

Column 6   Runoff Coefficient is taken from TABLE 1 according to the zoning of the drair area.

Column 7   Area drained by the specific inlet. Care should be taken to keep the drainage area flow separate into the appropriate street gutters.

Column 8   Product of Column 5 multiplied by Columns 6 and 7.

Column 9   If there is any flow which was not fully intercepted by an upstream inlet, it should be entered here.

Column 10   Sum of Columns 8 and 9.

Column 11   Capacity of the street in which the inlet is located, from either FIGS. 3, 4, 5 or 6. If the total gutter flow shown in Column 10 is in excess of the value in Column 11, the inlet should be moved upstream. If it is substantially less than the value in Column 11, an investigation should be made to see if the inlet can be moved downstream.

Column 12   Street gutter slope to be used in selecting the proper size inlet.

Column 13   Crown type of the street on which the inlet is located.

Column 14   Selected size of the inlet taken from FIGS. 8 - 22.

Column 15   Inlet type taken from FIG. 7.

Column 16   If the selected inlet does not intercept all of the gutter flow, the difference between the two values should be entered here and in Column 9 of the inlet which will intercept the flow.

CALCULATIONS

BY ______________________

DATE ______________________

INLET		Design Storm Frequency (yrs.)	AREA RUNOFF Q = CIA					Carry-Over From Upstream Inlet (c.f.s.)	Total Gutter Flow (c.f.s.)	Gutter Capacity (c.f.s.)	Gutter Slope (ft./100 ft.)	Crown Type	SELECTED INLET		Carry-Over To Downstream Inlet (c.f.s.)
No.	Location		Time Of Conc. (min.)	Intensity I (in./hr.)	Runoff Coeff. "C"	Area (Ac.)	"Q" (c.f.s.)						Length "LI" (Feet)	Type	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16

FORM B

STORM SEWER CALCULATIONS - FORM "C"

Column 1	Upstream station of the section of conduit being designed. Normally, this would be the point of a change in quantity of flow, such as an inlet, or a change in grade.
Column 2	Downstream station of the section of conduit being designed.
Column 3	Distance in feet between the upstream and downstream stations.
Column 4	Drainage sub-area designation from which flow enters the conduit at the upstream station.
Column 5	Area in acres of the drainage sub-area entering the conduit.
Column 6	Runoff coefficient, obtained from TABLE 1, based on the characteristics of the subdrainage area.
Column 7	Column 5 multiplied by Column 6.
Column 8	Obtained by adding the value shown in Column 7 to the value shown immediately above in Column 8.
Column 9	This time in minutes is transposed from Column 19 on the previous line of calculations. The original time shall be equal to the time of concentration as shown on TABLE 1
Column 10	Design Storm Frequency.
Column 11	Using the time at the upstream station shown in Column 10, this value is taken from FIG. 1.
Column 12	Column 8 multiplied by Column 11.
Column 13	This slope should be computed from the profile of the ground surface. Normally, the hydraulic gradient will have a slope approximately the same as the proposed conduit and will be located above the inside crown of the conduit.
Column 14	Utilizing the values in Columns 12 and 13, a conduit size should be selected. In the case of concrete pipe, FIG. 23 may be used.
Column 15	Velocity in the selected conduit based on the values in Columns 12, 13 and 14. Taken from FIG. 23 for concrete pipe.
Column 16	Friction head loss is the product of Column 3 times Column 13.
Column 17	Calculation is made utilizing the values of Columns 15 and 16.

Column 18	Calculation is based on the values of Columns 3 and 15.
Column 19	Sum of Columns 9 and 18.
Column 20	Special design comments may be entered here.

CITY OF RICHARDSON

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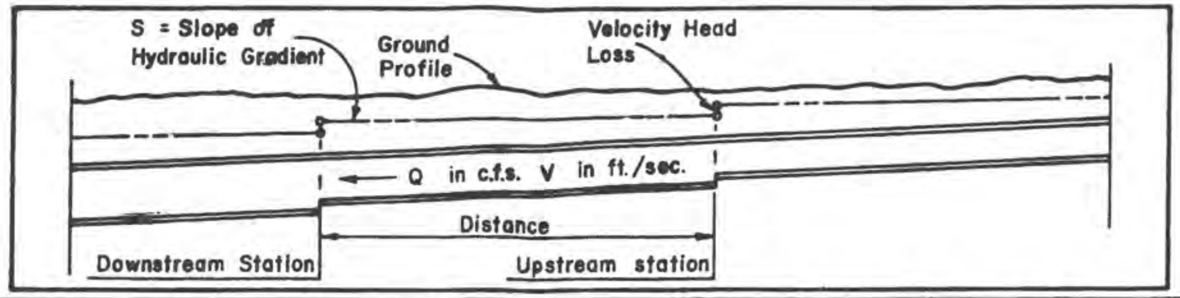
STORM SEWER LINE ______________________

INITIAL INLET TIME_____MINUTES



STORM SEWER CALCULATIONS

BY ______________________

DATE ______________________

RUNOFF COLLECTION POINT (Inlet or Manhole)		Distance Between Collection Points	INCREMENTAL DRAINAGE AREA				Accum-ulated "CA"	Time at Upstream Station (minutes)	Design Storm Frequency (yrs.)	Intensity "I" (inches/hr)	Storm Water Runoff "Q" (c.f.s.)	Slope of Hydraulic Gradient "S" (ft./ft.)	Selected Storm Sewer Size	Velocity in Sewer Between Collection Points "V" (f.p.s.)	Head Loss Coeff. Kj	Velocity Head Loss at Upstream Station $K_j \frac{V^2}{2g}$ (feet)	Flow Time in Sewer $\frac{\text{Distance}}{V \times 60}$ (minutes)	Time at Downstream Station (minutes)	REMARKS
UPSTREAM STATION	DOWNSTREAM STATION		Area No.	Drainage Area "A" (Acres)	Runoff Coeff. "C"	Incremental "CA"													
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

FORM C

WATER SURFACE PROFILE CALCULATIONS - FORM "D"

Column 1	At each point where a water surface elevation is desired, a cross section must be obtained. The sections are numbered and subdivided according to the assigned roughness coefficient.
Column 2	Know or assumed water surface elevation at the particular section.
Column 3	Distance along the channel between sections.
Column 4	Area of sub-section calculated from plotted cross sections.
Column 5	Wetted perimeter of each sub-section exclusive of the water interfaces between adjacent sub-sections.
Column 6	Column 4 divided by Column 5 (Hydraulic Radius).
Column 7	Column 6 raised to 2/3 power.
Column 8	Roughness coefficient for Manning's formula from TABLE 7.
Column 9	Column 4 multiplied by 1.486 and the product divided by Column 8.
Column 10	Column 9 multiplied by Column 7.
Column 11	The total flow shown in the upper left of the calculation form divided by Column 10 and squared, which is the friction slope.
Column 12	Average friction slope between sections.
Column 13	Column 12 multiplied by Column 3.
Column 14	Flow in each individual sub-section. Varies directly with conveyance factor shown in Column 10. The sum of the values must equal the total flow.
Column 15	Column 14 divided by Column 4.
Column 16	Column 15 squared.
Column 17	Column 16 multiplied by Column 14.
Column 18	Sum of the values in Column 17 of a particular section divided by twice the acceleration of gravity and multiplied by the total flow.
Column 19	Algebraic difference in velocity heads between sections.

Column 20 Eddy losses are calculated as 10% of the value of Column 19 when such value is positive and 50% of the absolute value of Column 19 when such value is negative.

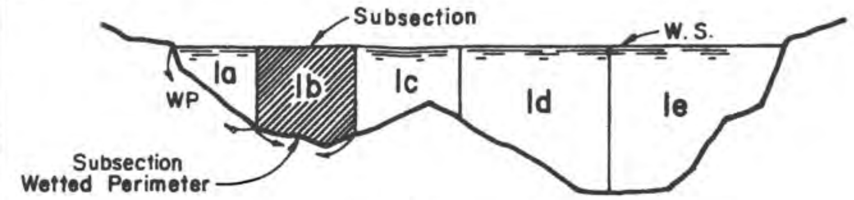
Column 21 Sum of Columns 13, 19 and 20.

Column 22 The sum of the value shown in Column 2 for the previous section and the value in Column 21. If the elevations calculated for subsequent sections do not agree within a reasonable limit with the assumed elevations shown in Column 2 for that particular section, the assumed elevations for such section must be revised and the section properties recomputed until the desired accuracy is obtained. An accuracy of +/- 0.3 feet is considered a reasonable limit.

CITY OF RICHARSON

OPEN CHANNEL

TOTAL FLOW, Q_T= c.f.s.



TYPICAL SECTION

WATER SURFACE PROFILE CALCULATIONS

FOR UNIMPROVED CHANNELS

BY

DATE

Section and Subsection	Known or Assumed W.S. Elevation	Distance Between Sections "L" (feet)	Subsection Area "A_s" (sq./ft.)	Subsection Wetted Perimeter "WP" (feet)	Subsection Hydraulic Radius "R"= $\frac{A}{WP}$	$R^{2/3}$	Subsection Roughness Coefficient "n"	$\frac{1.486A}{n}$	Conveyance Factor "Y"= $\frac{1.486A}{n}(R^{2/3})$	Friction Slope "S_f"=$\left(\frac{Q_T}{Y}\right)^2$ (ft./ft.)	Avg. S_f Between Sections (ft./ft.)	Friction Head Loss Between Sections "h_f"= Avg. S_f(L) (feet)	Subsection Flow "Q_s" (c.f.s.)	Subsection Velocity "V_s"= $\frac{Q_s}{A_s}$ (f.p.s.)	V_s^2	$V_s^2(Q_s)$	Weighted Velocity Head "h_v"= $\frac{\Sigma(V_s^2Q_s)}{2gQ_T}$ (feet)	Velocity Head Loss "h_{va}"= $h_{v1}-h_{v2}$ (feet)	Eddy Losses "E_l"= .10(+h_{va}) or .50(−h_{va}) (feet)	Total Loss "ΔH"= $(h_f)+(h_{va})+(E_l)$ (feet)	WATER SURFACE ELEVATION
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

FORM D

OPEN CHANNEL CALCULATIONS - FORM "E"†

Column 1	Downstream limit of the section of channel under consideration.
Column 2	Upstream limit of the section of channel under consideration.
Column 3	Type of channel as shown in FIGURE 24 is entered here.
Column 4	Flow in the section of channel under consideration.
Column 5	Roughness coefficient of the channel cross-section taken from TABLE 7.
Column 6	Slope of the channel which is most often parallel to slope of the hydraulic gradient.
Column 7	Square root of Column 6.
Column 8	Calculation is made using the values in Columns 4, 5 and 7.
Column 9	Assumed width of the bottom width of the channel.
Column 10	Assumed depth of flow.
Column 11	Assumed slope of the sides of the channel.
Column 12	Areas of flow which is calculated based on Columns 9, 10 and 11.
Column 13	Wetted perimeter calculated from Columns 9, 10 and 11.
Column 14	Value is calculated from Columns 12 and 13.
Column 15	Column 14 raised to 2/3 power.
Column 16	Product of Column 13 times Column 15.

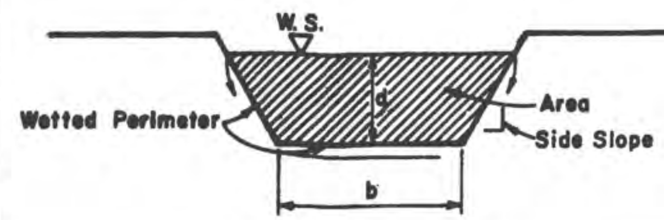
When the value of Column 16 equals the value of Column 8, the channel has been adequately sized. When the value of Column 16 exceeds the value of Column 8 by more than 5%, then the channel width or depth should be decreased and another trial section analyzed.

Column 17	Calculation is based on the values of Columns 4 and 12.
Column 18	Calculation is based on Column 17.
Column 19	Remarks concerning the channel section analyzed may be entered.

† NOTE: Form "E" should be used only to size open channels. Form "D" should be used to calculate stream profile.

CITY OF RICHARDSON

OPEN CHANNEL



OPEN CHANNEL CALCULATIONS

BY

DATE

CHANNEL STATION		Channel Type	Flow "Q" (c. f. s.)	Roughness Coeff. "n"	Slope "S" (ft./ft.)	"$S^{1/2}$"	$\frac{Q \times n}{1.486 \times S^{1/2}}$	Width "b" (feet)	Depth "d" (feet)	Side Slope	Area "A" (sq. ft.)	Wetted Perimeter "WP" (feet)	Hydraulic Radius "R" = $\frac{A}{WP}$ (feet)	$R^{2/3}$	$A \times R^{2/3}$	Velocity $V = \frac{Q}{A}$ (f. p. s.)	Velocity Head $\frac{V^2}{2g}$ (ft.)	REMARKS
From	To																	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

FORM E

FORM E

HYDRAULIC DESIGN OF CULVERTS, FORM "F"

INFORMATION IN UPPER RIGHT OF SHEET:

Culvert Location

This is a word description of the physical location.

Length

The actual length of the culvert.

Total Discharge, QT

This is the flow computed on FORM "A."

Design Storm Frequency

Obtained from TABLE 1 and used on FORM "A."

Roughness Coefficient, n

Obtained from TABLE 5.

Maximum Velocity

Obtained from TABLE 4.

Tailwater

This is the design depth of water in the downstream channel and is obtained in connection with the channel design performed on FORM "D" or FORM "E."

D. S. Channel Width

This is the bottom width of the downstream channel obtained from the calculations on FORM "E." The culvert should be sized to approximate this width whenever possible.

Entrance Description

This is a listing of the actual condition as shown in the "Culvert Entrance Data" shown on the calculation sheet.

Roadway Elevation

The elevation of the top of curb at the upstream end of culvert.

U. S. Culvert F. L.

The flow line of the culvert at the upstream end.

Difference

The difference in elevations of the roadway and the upstream flow line.

Required Freeboard

The vertical distance required for safety between the upstream design water surface and the roadway elevation or such other requirements which may occur because of particular physical conditions.

Allowable Headwater

This is obtained by subtracting the freeboard from the difference shown immediately above.

D. S. Culvert F. L.

The flow line elevation of the downstream end of the culvert.

Culvert Slope, So

This is the physical slope of the structure calculated as indicated.

Columns 1 - 10 deal with selection of trial culvert size and are explained as follows:

Column 1	Total design discharge, Q, passing through the culvert divided by the allowable maximum velocity gives trial total area of culvert opening.
Column 2	Culvert width should be reasonably close to the channel bottom width, W, downstream of the culvert.
Column 3	Lower range for choosing culvert depth is trial area of culvert opening, Column 1, divided by channel width, Column 2.
Column 4	Allowable headwater obtained from upper right of sheet.
Column 5	Trial depth, D, of culvert corresponding to available standard sizes and between the numerical values of Columns 3 and 4.

Columns 6, 7 and 8 are solved simultaneously based on providing a total area equivalent to the trial area of opening in Column 1.

Column 6	Number of culvert openings.
Column 7	Inside width of one opening.

Column 8 Inside depth of one opening if culvert is box structure or diameter if culvert is pipe.

Column 9 Column 6 multiplied by Column 7 and Column 8.

Columns 11 - 15 (Inlet Control) and 16 - 27 (Outlet Control) deal with Headwater Calculations which verify hydraulics of trial culvert selected and are explained as follows:

Column 11 Obtained from upper right of sheet.

Column 12 When the allowable headwater is equal to or less than the value in Column 8, enter Case I. When the allowable headwater is more than the value in Column 8, enter Case II.

Column 13 Column 10 divided by Column 7.

Column 14 Obtained from FIGURE 25 for box culverts or FIGURE 26 for pipe culverts.

Column 15 Column 14 multiplied by Column 8.

Column 16 Obtained from upper part of sheet.

Column 17 Obtained from FIGURE 27 for box culverts and FIGURE 28 for pipe culverts.

Column 18 Tailwater depth from upper right of sheet.

Column 19 So, culvert slope, multiplied by culvert length, both obtained from upper right of sheet.

Column 20 Sum of Columns 17 and 18 minus Column 19.

Column 21 Obtained from FIGURE 27 for box culverts and FIGURE 28 for pipe culverts.

Column 22 Critical depth obtained from FIGURE 29 for box culverts and FIGURE 30 for pipe culverts.

Column 23 Sum of Columns 22 and 8 divided by two.

Column 24 Tailwater depth from upper right of sheet.

Column 25 Enter the larger of the two values shown in Column 23 or Column 24.

Column 26 Previously calculated in Column 19 and may be transposed.

Column 27 The sum of Columns 21 an 25 minus Column 26.

Column 28 Enter the larger of the values from either Column 15, 20 or 27. This determines the controlling hydraulic conditions of the particular size culvert investigated.

Column 29 When the Engineer is satisfied with the hydraulic investigations of various culverts and has determined which would be the most economical selection, the description should be entered.

CULVERT DESIGN CALCULATIONS

CITY OF RICHARDSON

CULVERT LOCATION: ____________________

____________________ LENGTH, L ____________________

TOTAL DISCHARGE, Q ________ DESIGN STORM FREQ. ________

TOTAL DISCHARGE + FACTOR OF SAFTEY OF 25% Q_{Design} = ________

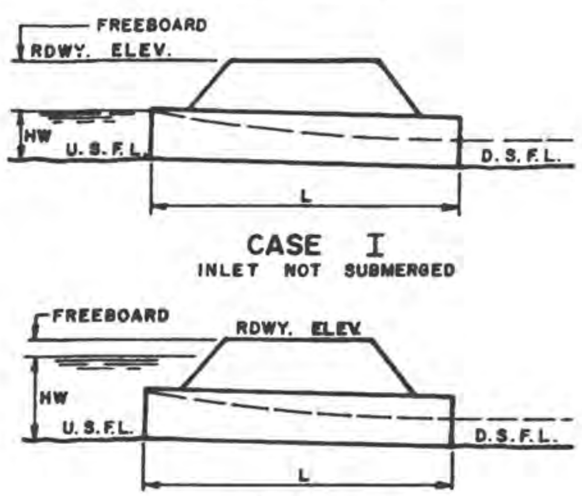
ROUGHNESS COEFF., n ________ MAX. VEL. ________

TAILWATER ________ D. S. CHANNEL WIDTH ________

ENTRANCE DESCRIPTION ____________________

RDWY. ELEV. ________	U.S. CULV. F.L. ________
U.S. CULV. F.L. ________	D.S. CULV. F.L. ________
DIFFERENCE ________	DIFFERENCE ________
REQ'D. FREEBOARD ________ FT.	CULV. SLOPE, $S_o = \frac{\text{DIFF. FT.}}{\text{LENGTH FT.}}$
ALLOW. HEADWATER ________ FT.	S_o = ________

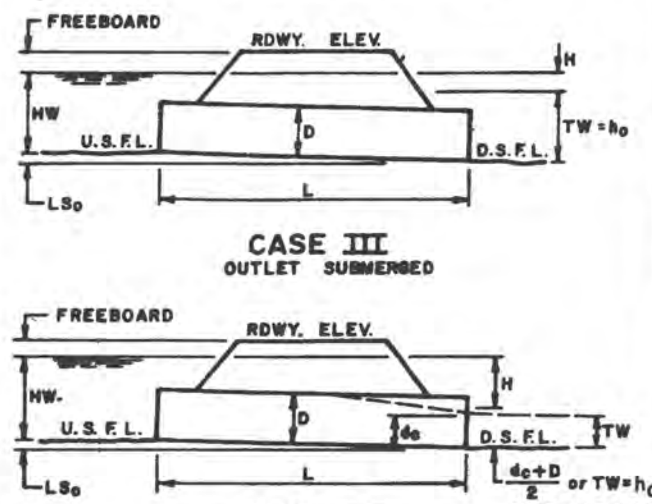
INLET CONTROL



CASE I
INLET NOT SUBMERGED

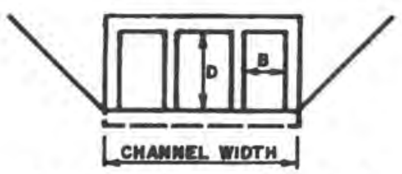
CASE II
INLET SUBMERGED

OUTLET CONTROL



CASE III
OUTLET SUBMERGED

CASE IV
OUTLET NOT SUBMERGED



TYPICAL BOX CULVERT

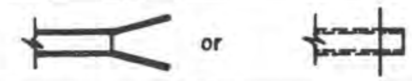
TYPICAL PIPE CULVERT

CULVERT ENTRANCE DATA

CONCRETE BOX CULVERT

TYPE	FLARE ANGLE	ENTRANCE EDGE	K_e
1A	30° to 75°	Square	0.4
1B	30° to 75°	Round	0.3
2A	15° to 30° & 75° to 90°	Square	0.5
2B	15° to 30° & 75° to 90°	Round	0.3
3A	0° (Extension of Sides)	Square	0.7
3B	0° (Extension of Sides)	Round	0.5

CONCRETE PIPE



TYPE	ENTRANCE DESCRIPTION	K_e
4	Spigot End With Headwall	0.5
5	Bell End With Headwall	0.2
6A	Bell End Projecting With No Headwall	0.3
6B	Spigot End Projecting With No Headwall	0.6

TRIAL CULVERT										HEADWATER CALCULATION																	The Greater Controlling Head Water (Inlet or Outlet) (feet)	SELECTED CONDUIT SIZE
		DEPTH RANGE D. R.			POSSIBLE CULVERT SIZES					INLET CONTROL (See Figure 25 & 26)					OUTLET CONTROL (See Figure 27, 28, 29, & 30)													
																CASE III HW = H + TW − L x S_o (feet)				CASE IV HW = H + h_o − L x S_o (feet)	$h_o = \frac{d_c+D}{2}$ or h_o = TW (use larger)							
Trial Area of Opening $T{\cdot}A_c = \frac{Q_{Design}}{V_{max.}}$ (sq. ft.)	Channel Width "W" (feet)	$\frac{T{\cdot}A_c}{W}$ (feet)	AHW (feet)	Try Depth "D" (feet)	No. Openings	Width of Box "B" (feet)	Box Depth or Pipe Dia. "D" (feet)	Total Culvert Area "Ac" (sq. ft.)	"Q" (Design) Each Opening (c.f.s.)	Entrance Type	Case No.	$\frac{Q_{Design}}{B}$ (c.f.s.)	$\frac{HW}{D}$ (figure 25 & 26)	HW	Entrance Coeff. K_e	"H" (feet) (figure 27 & 28)	"TW" (feet)	L x S_o (feet)	"HW" (feet)	"H" (feet) (figure) 27 & 28)	d_c (figure) 29 & 30 (feet)	$\frac{d_c+D}{2}$ (feet)	TW (feet)	h_o (feet)	L x S_o (feet)	"HW" (feet)		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29

FORM F

Q_{Design} = CALCULATED FLOW Q + FACTOR OF SAFTEY OF 25%. Q_{Design} = Q x 1.25

FORM F

BRIDGE DESIGN CALCULATIONS - FORM "G"

Column 1 & 2	Obtained from calculations on Form "A"
Column 3	Assume an average velocity that is less than the maximum allowable velocity and more than 4 feet per second. Maximum velocities are equal to those specified for open channels.
Column 4	Total flow as shown on upper part of sheet divided by Column 3.
Column 5	Column 4 divided by Column 2.
Column 6	Selected bridge length utilizing standard span lengths.
Column 7	Calculated from bridge and channel geometrics.
Column 8	Total flow through bridge divided by Column 7.
Column 9	Selected head loss coefficient based upon specific conditions.
Column 10	Calculated utilizing values in Columns 8 and 9.

CITY OF RICHARDSON

BRIDGE DESIGN CALCULATIONS

Q= ____________________

BY ____________________

DATE ____________________

Velocity in Downstream Channel "V_c" (f.p.s.)	Depth of Flow in Downstream Channel "D_c" (feet)	Trial Velocity through Bridge "V_T" (f.p.s.)	Trial Opening Area "A_T" = $\frac{Q}{V_T}$ (sq./ft.)	Trial Length of Opening "L_T" = $\frac{A_T}{D_c}$ (feet)	Actual Length of Opening "L" (feet)	Actual Area of Opening Below Highwater "A" (sq./ft.)	Actual Velocity through Bridge "V" = $\frac{Q}{A}$ (f.p.s.)	Head Loss Coefficient "K_b"	Head Loss through Bridge "h_1" = $K_b\left(\frac{V^2}{2g}\right)$ (feet)
1	2	3	4	5	6	7	8	9	10

FORM G