DEPARTMENT OF ENGINEERING
PARISH OF JEFFERSON
STATE OF LOUISIANA

STORM DRAINAGE DESIGN MANUAL

Prepared by Engineering Division
Department of Public Utilities
1981

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INTRODUCTION
SECTION 1

1.01 PURPOSE

The purpose of this drainage manual is to establish standard principles and practices for the design and construction of surface drainage systems within the Parish of Jefferson, Louisiana and within its extra-territorial jurisdiction. The design factors, formulae, graphs and procedures are intended for use as engineering guides in the solution of drainage problems involving determination of the quantity, rate of flow, method of collection, storage, conveyance and disposal of storm water.

Methods of design other than those indicated herein may be considered in difficult cases where experience clearly indicates they are preferable. However, there should be no extensive variations from the practices established herein without the express approval of the Department of Public Utilities.

1.02 SCOPE

The manual represents the application of accepted principles of surface drainage engineering and is a working supplement to basic information obtainable from standard drainage handbooks and other publications on drainage. It is presented in a format that gives logical development of solutions to the problems of storm drainage.

DRAINAGE POLICY
FOR THE
PARISH OF JEFFERSON, LOUISIANA

1.03 APPLICATION

This policy shall govern the planning, design, construction and operation of storm drainage facilities within the Parish of Jefferson and within all areas subject to it's extraterritorial jurisdiction. Definitions, formulae, criteria, procedures and data in this manual have been developed to support this policy. The policy shall take precedence over the manual in cases of conflict.

1.04 GENERAL

A. All drainage facilities including street curbs, gutters and inlets shall be designed to intercept and transport run-off from a 10-year frequency storm.

B. The drainage system shall be designed to carry and/or store the runoff from a 10-year frequency storm.

C. No improvements shall be constructed which will increase the frequency of flooding or the depth of inundation of un-protected structures. This requirement may be waived in the case of structures erected in violation of Parish of Jefferson floodproofing requirements.

D. Peak flows shall not be increased at any location for any

storm frequency, (more frequent than the 10-year storm), which will result in the inundation of unprotected structures not previously subject to inundation as a result of that same frequency storm.

E. Regulation of peak flows to allowable level as determined by the provisions of this policy shall be achieved by storage on-site or off-site. The Storage Section, (Section 9), of the Drainage Criteria Manual provides a guide to acceptable methods but does not limit the designer to the methods presented therein.

1.05 STREET DRAINAGE

A. No lowering of the standard height of street crown shall be allowed for the purposes of hydraulic design, unless approved by the Chief Engineer, Department of Public Utilities. In no case will it be allowed on thoroughfares or expressways.

B. Where additional hydraulic capacity is required on the street and is not obtainable by modification of crossfall, the gradient must be increased, or curb inlets and storm sewers installed to remove a portion of the flow.

1.06 STORM SEWERS

Pipe shall be reinforced concrete, except as otherwise approved or required by the Department of Public Utilities. Concrete pipe shall be manufactured and installed in compliance with the Standard Specifications for the Department of Public Utilities.

1.07 BRIDGES AND CULVERTS

A. Construction plans for proposed reinforced concrete box culverts, bridges and related structure may be adaptations of Louisiana Highway Department Standards.

B. For drainage structures designed to carry residential streets, water overtopping the structure from 100-year frequency storms shall not flood the roadway to a depth greater than eighteen inches (18") above the roadway crown.

C. For drainage structures designed to carry any street other than a residential street, water overtopping the structure from a 100-year frequency storm shall not flood the roadway to a depth greater than eighteen (18") either above the roadway crown.

1.08 COMPUTATIONS

Computations to support all drainage designs shall be submitted to the Department of Public Utilities for re-

view. The computations shall be in such form as to provide the basis for timely and consistent review and will be made a part of the permanent record for future evaluation. The computation shall be accompanied by the certification of a Registered Professional Engineer that the design procedure is in full compliance with the requirements of this manual.

1.09 DRAINAGE PLAN PREPARATION

A. Plan and profile shall be drawn on sheets 23" x 36" to a horizontal scale of 1" to 20' or 1" to 50' and a vertical scale of 1" to 2' or 1" to 5' (except that scales may vary on special projects, such as culverts and channel cross-sections).

B. Good quality reproducibles of the original drawings shall be presented to the Department of Public Utilities prior to the receipt of final approval and shall remain the permanent property of the Parish of Jefferson.

C. Stationing shall proceed upstream with the North arrow pointing to the top of the sheet, or to the right.

D. Plans for the proposed drainage system shall include property lines, lot and block numbers, dimensions, right-of-way and easement lines, flood plains, street names, paved surfaces (existing or proposed); location, size and type of inlets, manholes, culverts, pipes, channels and related

structures; contract limits, outfall details, miscellaneous riprap construction, contour lines and title block.

E. Profiles shall indicate the proposed system (size and material) with elevations, flow-lines, gradients, left and right bank channel profiles, station numbers, inlets, manholes, ground line and curb line elevations, typical sections, riprap construction, filling details, minimum permissible slab elevations within 10-year flood plains and adjacent to open drainage features, pipe corssings. design flow capacities, and title block.

DETERMINATION OF STORM RUNOFF
SECTION 2

2.01 General

2.02 Effect Of Urbanization

2.03 Rational Method

- A. Runoff Coefficient
- B. Rainfall Intensity
- C. Drainage Area
- D. Example of Rational Method Calculation

2.04 Hydrograph Method

- A. General
- B. Unit Hydrograph
- C. Rational Method Unit Hydrograph
- D. Example of Rational Method Unit Hydrograph Calculations
- E. SCS Curvilinear Method Unit Hydrograph
- F. Example of SCS Curvilinear Unit Hydrograph Calculations
- G. SCS Triangular Method Unit Hydrograph
- H. Example of SCS Triangular Method Hydrograph Calculations

DETERMINATION OF STORM RUNOFF
SECTION 2

2.01 GENERAL

If continuous records of the amounts and rates of runoff in, through and from urban areas were as readily available as are records of precipitation, they would provide the best source of data on which to base the design of storm drainage and flood protection systems. Unfortunately, such records are available in very few areas in sufficient quantity to permit an experience based prediction of the probable frequencies and amounts of runoff. The accepted practice, therefore, is to relate runoff to rainfall, thereby providing a means of predicting the rates and amounts of runoff to be expected from urban watersheds at given recurrence intervals.

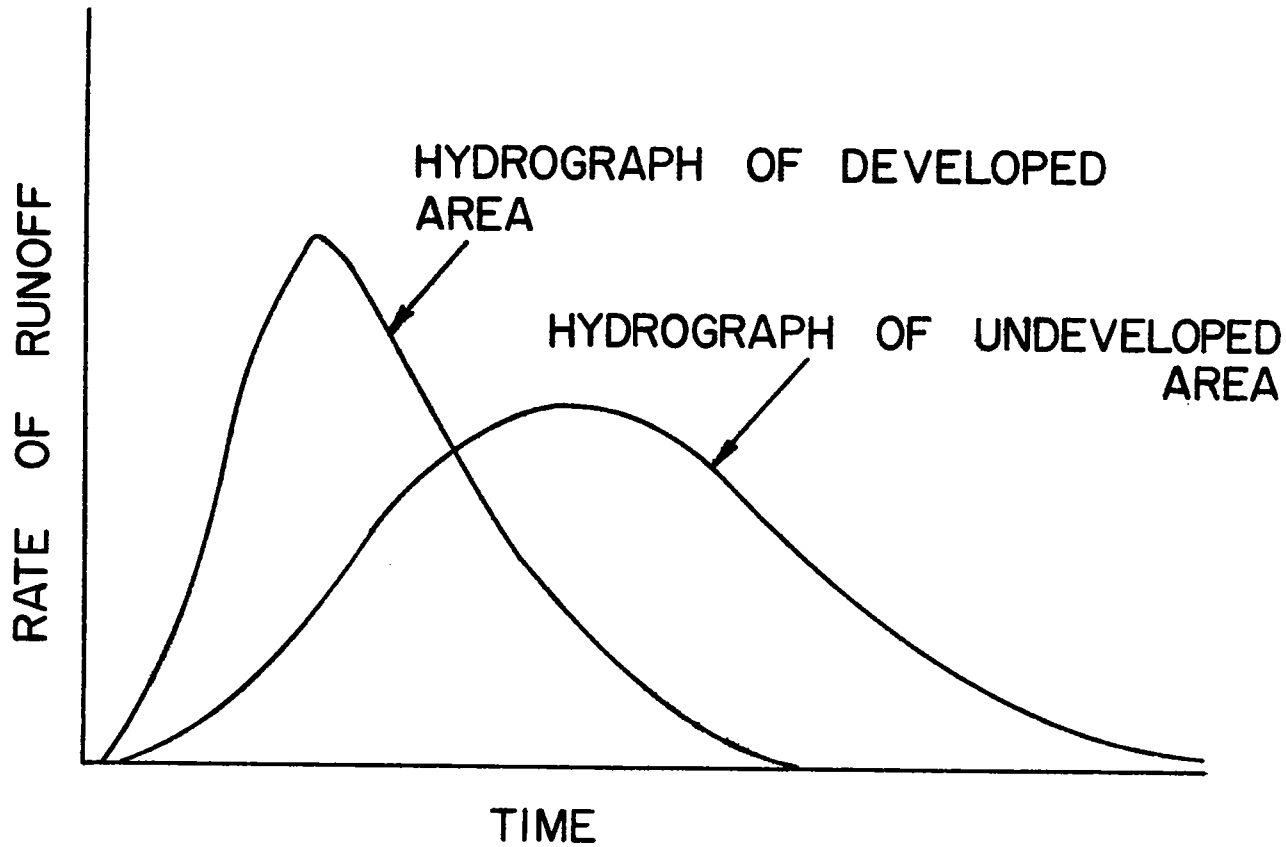
Numerous methods of runoff computation are available on which the design of storm drainage and flood control systems may be based. The Rational Method is generally accepted as adequate for drainage areas up to 400 acres. For larger drainage basins, the use of local recorded flood data in conjunction with unit hydrograph techniques will produce reliable results.

2.02 EFFECT OF URBANIZATION

It has long been recognized that urban development has a pronounced effect on the rate of runoff from a given rainfall. The hydraulic efficiency of a drainage area is

generally improved by urbanization which in effect reduces the storage capacity of a watershed. This reduction of a watershed's storage capacity is a direct result of the elimination of porous surfaces, small ponds, and holding areas. This comes about by the grading and paving of building sites, streets, drives, parking lots, and sidewalks and by construction of buildings and other facilities characteristic of urban development. The result of the improved hydraulic efficiency is illustrated graphically in Figure 2-1, which is a plot of the runoff rate versus time for the same storm with two different stages of watershed development.

When analyzing an area for design purposes, urbanization of the full watershed shall be assumed. Zoning maps, future land use maps, and master plans should be used as aids in establishing the anticipated surface character of the ultimate development. The selection of design runoff coefficients and/or percent impervious cover factors, which are explained in the following discussions of runoff calculation, must be based upon the assumed future urbanization of the complete watershed.



EFFECTS OF WATERSHED DEVELOPMENT
ON STORM HYDROGRAPH

Figure 2-1

2.03 RATIONAL METHOD

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

$$Q = CiA$$

where,

Q is defined as the peak rate of runoff in cubic feet per second. Actually, Q is in units of inches per hour

per acre. Since this rate of in/hr/ac differs from cubic feet/second by less than one percent, the more common cfs is used.

C is the coefficient of runoff representing the ratio of peak runoff to rainfall.

i is the average intensity of rainfall in inches per hour for a period of time equal to the critical time of flow of the drainage area to the point under consideration.

A is the area in acres contributing runoff to the point of design.

Basic assumptions associated with the Rational Method:

1. The computed peak rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.

2. The time of concentration is the critical time of concentration, to be discussed under paragraph "B" this Section.

3. The ratio of runoff to rainfall, C, is uniform during the storm duration.

4. Rainfall intensity is uniform during the storm duration.

A. Runoff Coefficient (C)

Nature of Surface. The proportion of the total rainfall that will reach the storm drains depends on the relative porosity or imperviousness of the surface, and the slope and ponding character of the surface. Impervious surfaces, such as asphalt pavements and roofs of buildings, will be subject to nearly 100 percent runoff, regardless of the slope, after the surfaces have become thoroughly wet. On site inspections and aerial photographs may prove valuable in estimating the nature of the surface within the drainage area.

Soil. The runoff coefficient "C" in the Rational Formula is also dependent on the character of the soil. The type and condition of the soil determines its ability to absorb precipitation. The rate at which a soil absorbs precipitation generally decreases as and if the rainfall continues for an extended period of time. The soil absorption or infiltration rate is also influenced by the presence of soil moisture before a rain (antecedent precipitation), the rainfall intensity, the proximity of the ground water table, the degree of soil compaction, the porosity of the subsoil, vegetation, ground slopes, depressions and storage.

Runoff Coefficient. It should be noted that the runoff coefficient "C" is the variable of the Rational Method which is least susceptible to precise determination. Proper use requires judgment and experience on the part of the engineer, and it's use in the formula implies a fixed ratio for any

given drainage area, which in reality is not the case. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff.

Table 2-1 presents recommended ranges for "C" values.

TABLE 2-1

RATIONAL METHOD RUNOFF COEFFICIENTS
BY LAND USE TYPES
FOR USE IN Q = CiA

Land Use Type	Runoff Coefficients (C)
Single-Family Dewelling District, 1 acre	0.38
Single-Family Dwelling District, ½ acre	0.40
Single-Family Dwelling District, 16,000 sq. ft.	0.45
Single-Family Dwelling District, 13,000 sq. ft.	0.45
Single-Family Dwelling District, 10,000 sq. ft.	0.45
Single-Family Dwelling District, 7,500 sq. ft.	0.45
Single-Family Dwelling District, 5,000 sq. ft.	0.45
Duplex Dwelling District	0.60
Townhouse Dwelling District	0.80
Multiple-Family Dwelling District	0.80
Mobile Home District	0.55
Agricultural District	Variable

Parking District	0.90
Office District	0.90
Neighborhood Service District	0.90
Shopping Center District	0.90
General Retail District	0.90
Light Commercial District	0.90
Central Area District	0.90
Heavy Commercial District	0.90
Industrial District	0.70 to 0.90
Churches	0.70 to 0.90
Schools	0.50 to 0.90
Park & Greenbelt;	0.30 to 0.70
Slope less than 2%	0.40
Slope 2% - 7%	0.45
Slope > 7%	0.50
Cemetery	0.3 to 0.5

B. Rainfall Intensity (i)

Rainfall intensity (i) is the average rainfall rate in inches per hour which is considered for a particular drainage basin or sub-basin and is selected on the basis of design rainfall duration and design of frequency of occurrence. The design duration is equal to the time of concentration for the drainage area under consideration. The frequency of occurrence is a statistical variable which is established by design standards or chosen by the engineer as a design parameter.

Time of Concentration. The time of concentration used in the rational equation is the critical time of concentration for the point of interest. The critical time of concentration is the time associated with the peak runoff from the watershed to the point of interest. Runoff from a watershed usually reaches a peak at the time when the entire watershed is contributing, in which case, the time of concentration is the time for a drop of water to flow from the remotest point in the watershed to the point of interest. Runoff may reach a peak prior to the time the entire watershed is contributing. A trial and error procedure can be used to determine the critical time of concentration. The time of concentration to any point in a storm drainage system is a combination of the "inlet time" and the "time of flow in the conduit".

The inlet is the time for water to flow over the surface of the ground to the storm sewer inlet. Because the areas contributing to most storm sewer inlets are relatively small, it is customary in practice to determine the inlet time on the basis of experience under similar conditions. Inlet time decreases as the slope and the imperviousness of the surface increases, and it increases as the distance over which the water has to travel increases and as retention by the contact surfaces increases. Table 2-2 gives usual values for inlet times of concentration.

TABLE 2-2

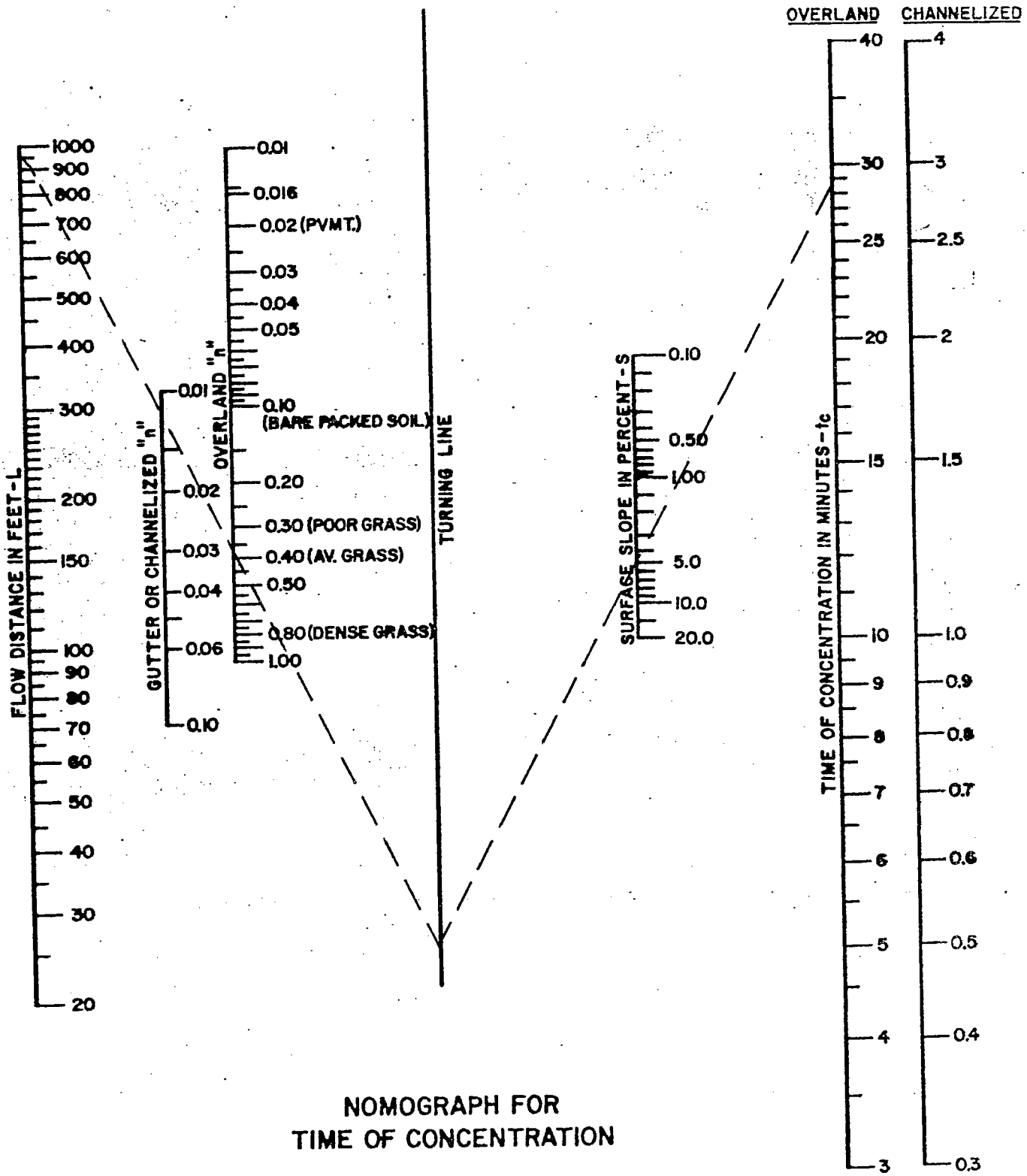
INLET TIME OF CONCENTRATION

Type of Area	Usual Inlet Time
Parks and Open Areas	20 min.
Residential	15 min.
Industrial	10 min.
Commercial	10 min.
Suburban Business	10 min.

Although it is customary to determine inlet time on the basis of experience, all inlet times shall be verified by direct overland flow computations. The overland flow computations shall be made as follows:

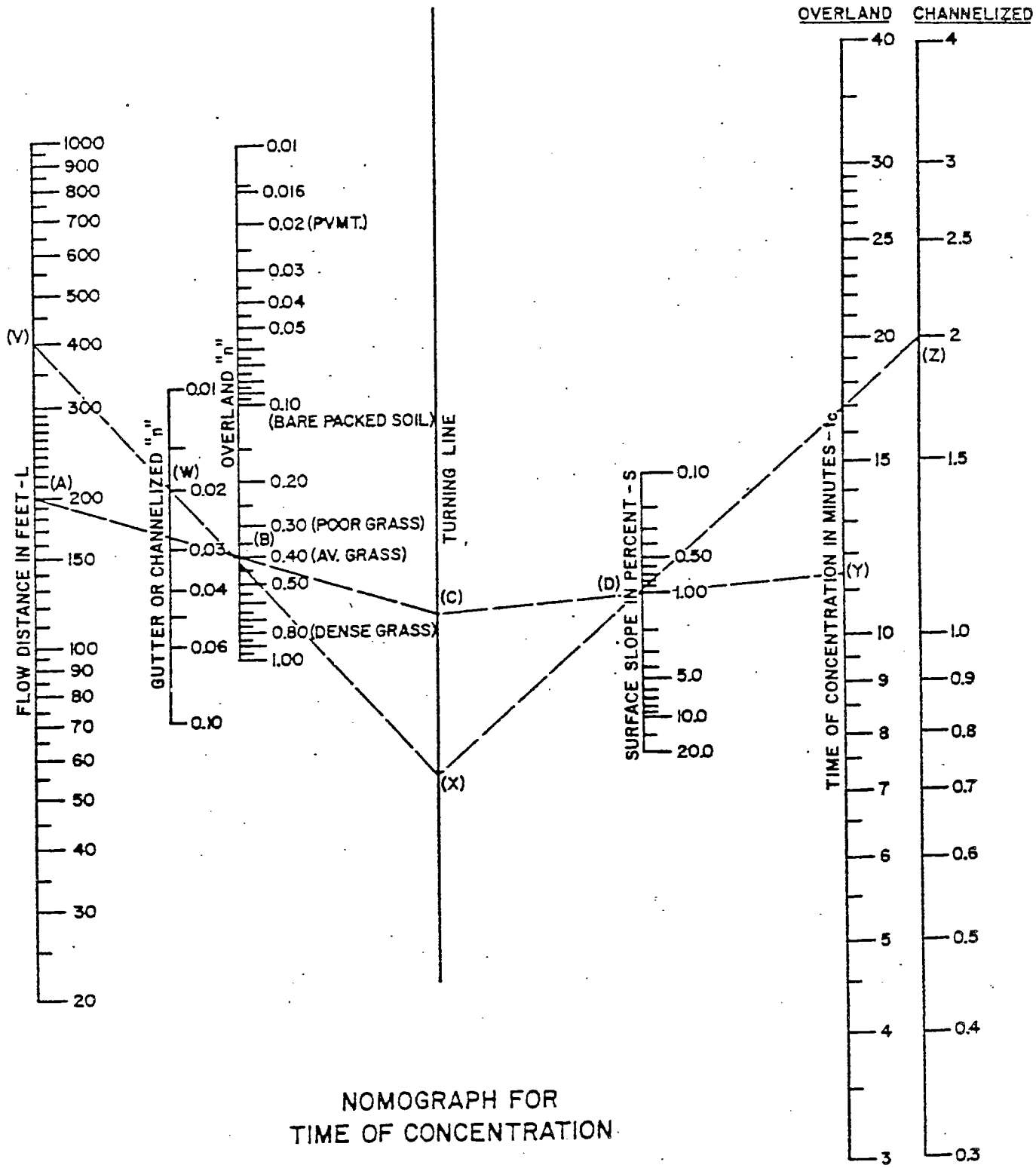
1) For a runoff flow distance of 1,000 feet or less, the longest inlet time shall be determined by using the street, gutter, and overland grades and roughness according to Figure 2-2A. This nomograph is used to calculate the time of flow for both overland flow and gutter or channelized flow. The term "channelized flow" refers to flow through constructed and eroded swales and other small channels. For flow in large drainage channels the designer is referred to Section VII, "Flow in Ditches and Channels". Typical values of the roughness coefficient "n" for overland flow cross areas of pavement, poor grass, dense grass, and other covers are given directly in Figure 2-2A. For gutter or channelized flow, suggested values for the roughness coefficient "n" are: for concrete gutter, n = 0.02; for average grass, n = 0.40; and for dense grass, n = 0.80. The following example is given to illustrate the use of Figure 2-2-A.

<u>Example:</u> A value for the inlet time of concentration is needed for the flow across 200 feet of an open lot and 400 feet in a street to an inlet. The open lot is covered with an average growth of grass and has a ground slope of one per-



NOMOGRAPH FOR
TIME OF CONCENTRATION

Figure 2-2



NOMOGRAPH FOR
TIME OF CONCENTRATION

Figure 2-2-A

cent.

The overland flow across the open lot is determined as follows: A straightedge is placed so that it passes through the point for Flow Distance = 200' (Point A) and the point for overland flow roughness coefficient n = 0.40 (av. Grass) (Point B). The designer should note that there are separate lines for the overland flow roughness "n" and the gutter or channelized roughness "n". With the straightedge in place a point is marked on the pivot line (Point C). The straightedge is then positioned so that it passes through the selected point on the pivot line and the point for surface slope in percent = 1.0 (Point D). With the straightedge in place the time of concentration for overland flow is determined to be 11.5 minutes (Point Y). The designer should note that there are separate lines for overland flow time of concentration and for channelized flow time of concentration.

The channelized flow down the street is determined as follows: A straightedge is placed so that it passes through the point for flow distance = 400' (Point V) and the point for gutter roughness n = 0.02 (Point W). With the straightedge in place a point is marked on the pivot line (Point X). The straightedge is then positioned so that is passes through the selected point on the pivot line and the point for surface slope in percent = 1.0 (Point D). The time of concentration for flow in the street is then read on the channelized flow line (Point Z). The gutter flow time is 2.0 minutes. The total

inlet time of concentration is the sum of the overland flow time and the gutter flow time, or 11.5 + 2.0 = 13.5 minutes.

2) For flow distances in excess of 1,000 feet, the inlet time shall be the sum of the time determined from Figure 2-2 for the first 1,000 feet and the time determined by direct computation using the following formula for the remaining distance.

$$T = \frac{D_F}{60V}$$

T = overland flow time (minutes)

D_F = Flow Distance (ft.) over 1,000 ft.

V = average velocity of runoff flow (ft/sec)

The value of V is dependent upon the type of land and the slop of the land over which the runoff flows. Design values of V for varied land types and slopes are in Table 2-3.

TABLE 2-3

RUNOFF VELOCITY
(in ft/sec)

Land Type	Surface Slope		
	0-2%	3-6%	7-10%
Woodlands	0.5	0.7	0.9
Pastures	0.5	0.6	0.8
Grassy (Parks)	0.5	0.7	0.9
Pavements	2.40	4.4	4.5

Overland flow distances will rarely exceed 400' in developed areas. If the overland flow time is calculated to be in excess of 20 minutes, the designer should verify that the time is reasonable considering the projected ultimate development of the area.

The time of flow in the conduit is the quotient of the length of the conduit and the velocity of flow as computed using the hydraulic factors of the conduit. The time of concentration within a conduit is usually less than the actual time for the flood crest to reach a given point by an amount equal to the time required to fill the conduit. The time required to fill the conduit is defined as the time of storage. The time of storage shall be neglected in the design of storm runoff conduits even though it may represent an appreciable percentage to the total time of concentration. This procedure will not substantially affect the precision of the calculations and will contribute to a conservative design.

Frequency of Occurrence. A key design consideration is the selection of the frequency criteria. It is seldom economical to design storm drainage facilities to handle the maximum runoff that may occur. Storm drainage improvements in Jefferson Parish shall be designed to intercept and carry the runoff from a 10 year frequency storm.

Intensity Coefficient. Rainfall intensity curves for various

design storms are available from U.S. Weather Bureau Technical Paper No. 25 for New Orleans Metropolitan Area and are shown in Figure 2-3.

The curves are applicable for design frequencies up to the 100-year storm and durations from 5 minutes to 34 hours, and should prove adequate for use in determining peak flows by the Rational Method or other appropriate methods.

C. Drainage Area (A)

The size and shape of the watershed must be determined. The area may be determined through the use of planimetric-topographic maps, supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. A drainage area map shall be provided for each project. The drainage area contributing to the system being designed and the drainage sub-area contributing to each inlet point shall be identified. The outlines of the drainage divides must follow actual lines rather than the articical land divisions as used in the design of sanitary sewers. The drainage divide lines are determined by the pavement slopes, locations of downspouts, paved and unpaved yards, grading of lawns and many other features that are introduced by the urbanization process.

D. Example of Rational Method Calculation

The designer is to determine the peak rate of runoff from a 10-year frequency storm, given the following conditions: The area drained is 35 acres. The land surface is open, and

is expected to remain as a greenbelt area. The land slope is five percent (5%), and the distance from the upper watershed boundary to the design point is 1,200 ft.

Step 1. The value for the runoff coefficient (C) is chosen from Table 2-1 for the appropriate land use. The value for parks and greenbelt is assumed as, C = 0.55.

Step 2. To determine the intensity (i), the time of concentration must first be determined. The flow distance is greater than 1,000 ft., therefore the nomograph for time of concentration (Figure 2-2) is used to determine the time of concentration for the first 1,000 feet. The time of concentration for the remaining 200 feet is calculated by using the flow velocity, given in Table 2-3, for grassy parkland at a 5% slope. The time of concentration for the first 1,000 feet from Figure 2-2 is 28 minutes. The remaining flow distance is 200 ft. and the flow velocity is 0.7 ft/sec; therefore, the time of concentration is 200 ft/(0.7 x 60) = 5 minutes. The sum of the two times of concentration is 33 minutes. The intensity is then obtained from Figure 2-3. Using a duration (or time of concentration) of 33 minutes and the

10-year frequency curve, the intensity is found to be i = 4.25 inches/hr.

Step 3. The drainage area is given as 35 acres. The value of runoff is therefore:

$$Q_{10} = C\ i\ A$$

$$Q_{10} = (0.55)\ (4.25)\ (35.0)$$

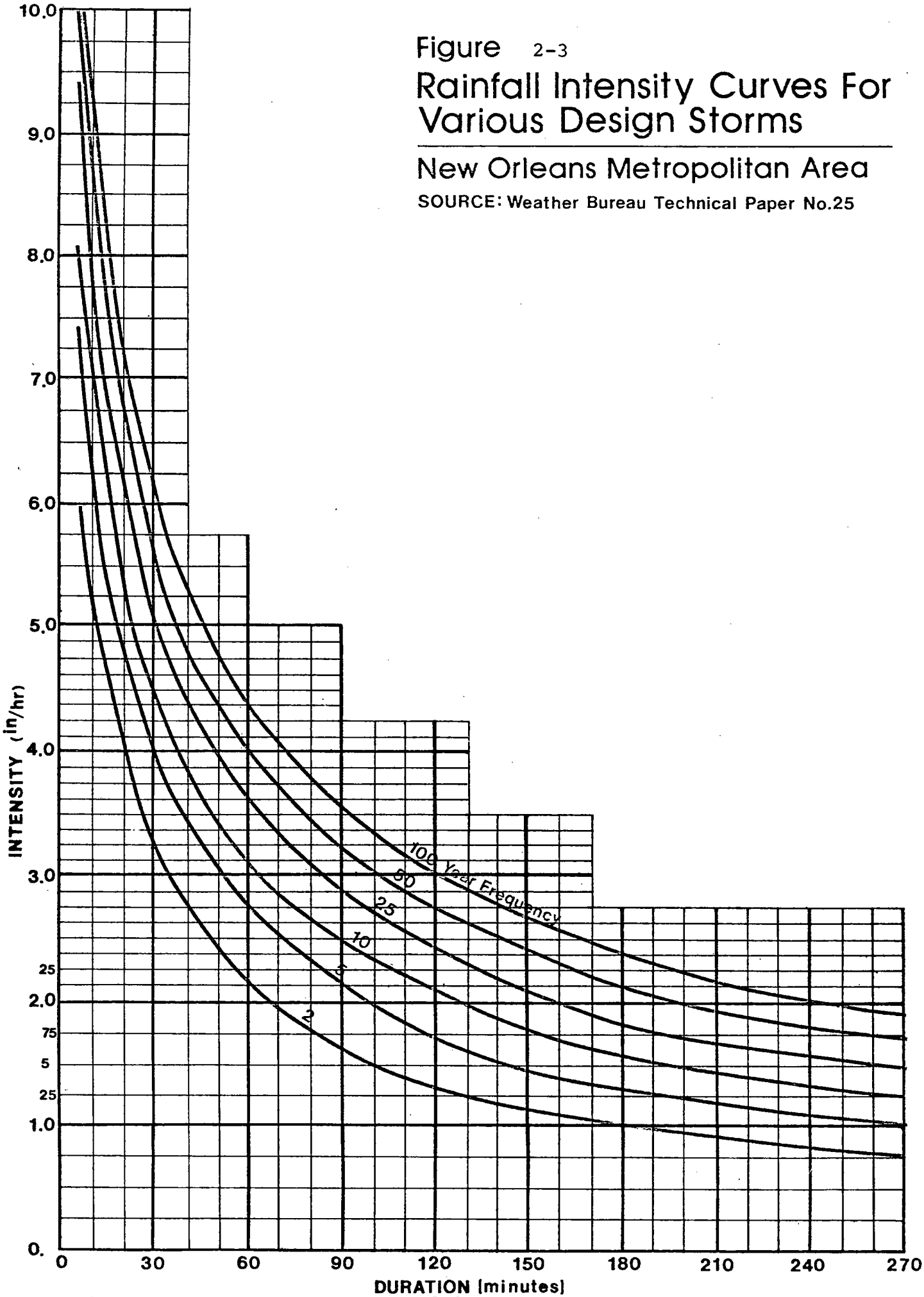
$$Q_{10} = 81.8 \text{ cfs}$$

Figure 2-3

Rainfall Intensity Curves For Various Design Storms

New Orleans Metropolitan Area

SOURCE: Weather Bureau Technical Paper No.25



2.04 HYDROGRAPH METHOD

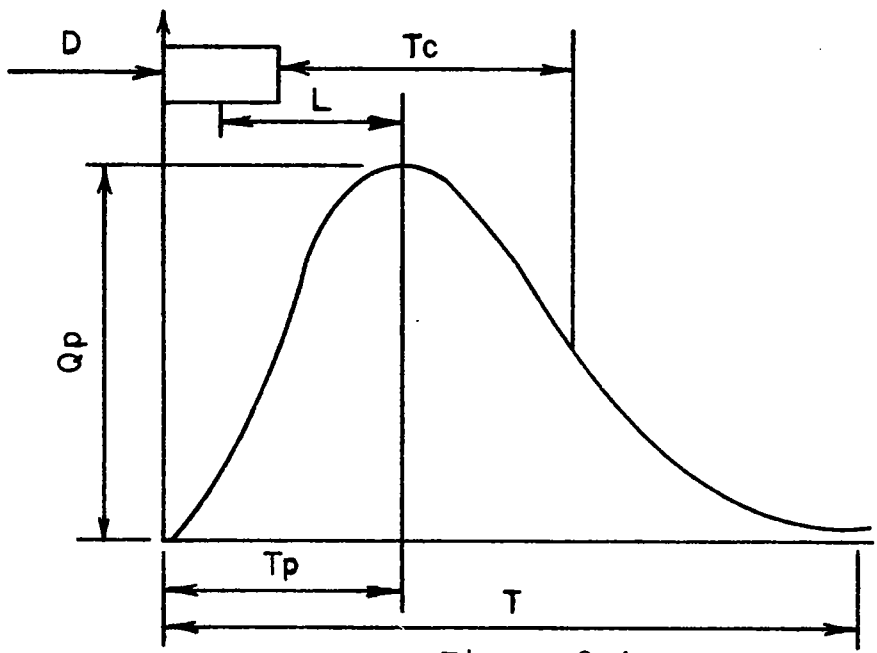
A. GENERAL

The Rational Method of design is a very common, often used design method. Basically, this method produces a peak flow at a particular point in time depending on the area, intensity of the storm, the runoff factor and the time of concentration. The flows produced at times other than the time corresponding to the peak flows are unknown. This is a major deficiency when attempting to route flows in a major storm carrier. Tributaries discharging flows at different intervals of time will produce peaks with a time lag equal to their related flow times to the design point. The use of Hydrograph Method is to account for these stagering peaks at any given design point.

B. UNIT HYDROGRAPH

If two identical rainstorms could occur over a drainage basin with identical conditions prior to the rain, the hydrographs of runoff from the two storms would be expected to be the same. This is the basis of the unit-hydrograph concept. Actually the occurrence of identical storms is very rare. Storms may vary in duration, amount, and areal distribution of rainfall. A unit hydrograph is a hydrograph with a volume of 1 in. (25 mm) of runoff resulting from a rainstorm of specified duration and areal pattern. In other words, the unit hydrograph describes the lagging and smoothing associated with routing the one inch volume of

excess precipitation, (i.e. precipitation - losses through infiltration, etc.) through the watershed. A unit hydrograph with major time and flow rate parameters is illustrated in Figure 2-4.



where =

D = duration of unit excess precipitation

L = lag

T_c = time of concentration

T = time base

T_p = time to peak

Q_p = peak flow rate

Figure 2-4

To calculate the direct runoff hydrograph from a particular excess precipitation (rainfall - minus losses through infiltration, etc.) hyteograph requires the use of unit hydrograph and the assumption of linearity. Linearity implies that an increment of excess precipitation will result in direct runoff which is proportional to its volume and that the separate effects of each increment can be superimposed.

For gaged watersheds the unit hydrograph can be developed by using observed direct runoffs and applying statistical procedures.

For an ungaged watershed the unit hydrograph is developed

from empirical equations which are based on physical characteristics of the watershed such as size, slope, land use and channel characteristics. These procedures are also referred to as synthetic unit hydrograph methods. There are numerous procedures to develop a synthetic unit hydrograph as follows:

1 - Rational Method Unit Hydrograph

2 - Soil Conservation Services (SCS) Curvilinear Unit Hydrograph

3 - Soil Conservation Services (SCS) Triangular Unit Hydrograph

C. RATIONAL METHOD UNIT HYDROGRAPH

The concept of the rational method is that if a constant rate of excess precipitation is placed over a watershed for a duration equal to or greater than the time of concentration, the entire watershed will be contributing to the stream flow at the mouth of the watershed. Therefore, the flow rate out of the watershed will equal the excess precipitation rate onto the watershed. This procedure also assumes; that the time for direct runoff to decrease to zero would be equal to the time of concentration and that the rise to peak and the recession to zero are straight lines.

Therefore, the unit hydrograph for a duration equal to the time of concentration, according to the rational method is one which rises linearly to a peak of $\frac{1\ (in)}{tc\ (hrs)} \times A$ (acres) at $t = tc$ then falls linearly to zero at $t = 2tc$.

D. EXAMPLE OF RATIONAL METHOD UNIT HYDROGRAPH CALCULATIONS

Using the Rational Unit Hydrograph Method, find the direct runoff hydrograph for an urban watershed with the following characteristics.

Area (A) = 200 acres

Runoff Coefficient C = 0.50

Time of Concentration (tc) = 20 min.

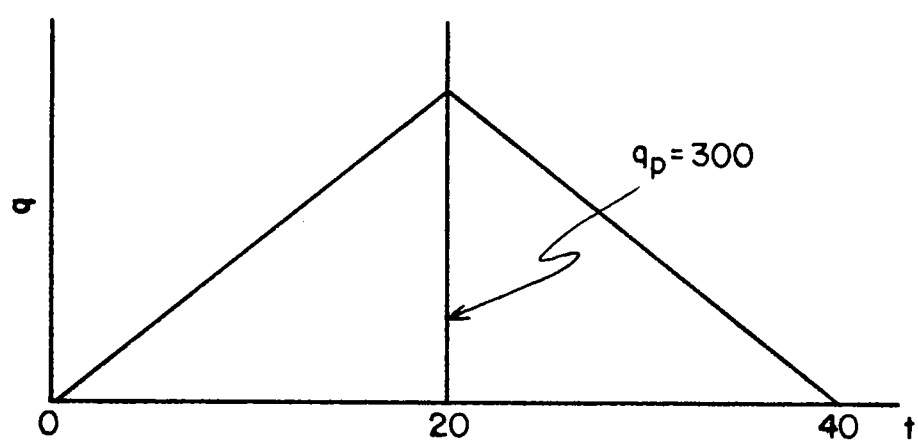
10 Year Frequency Storm

Establish Unit Hydrograph and calculate the direct runoff hydrograph.

The unit hydrograph for a duration tc = 20 is q = 0 @ t = 0

$$q_p = \frac{C \times 1 \times A \text{ (acres)}}{tc \text{ hrs}} = \frac{0.50 \times 1 \times 200}{1/3 \text{ hrs}} = 300 \text{ cfs @ } t = 20 \text{ min. and}$$

q = 0 @ t = 40 min. as shown in Figure 2-5



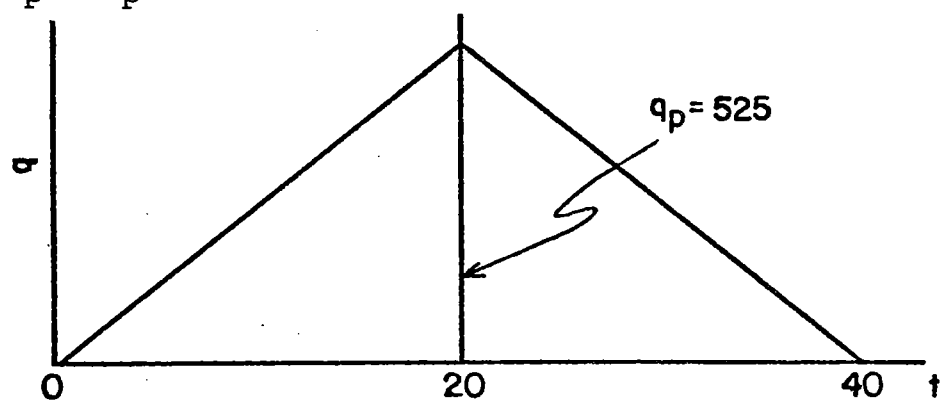
Unit Hydrograph
Figure 2-5

Unit Hydrograph represent the hydrograph of one inch runoff from a 20 min. storm, therefore, the implied rate of runoff is 3 in/hr. The intensity of rainfall (i) for a concentration time (tc) from Figure 2-3 is 5.25 in.

Direct Runoff Hydrograph is developed by multiplying the ordinates of the unit hydrograph by ratio = $\frac{5.25}{3}$ = 1.75

therefore

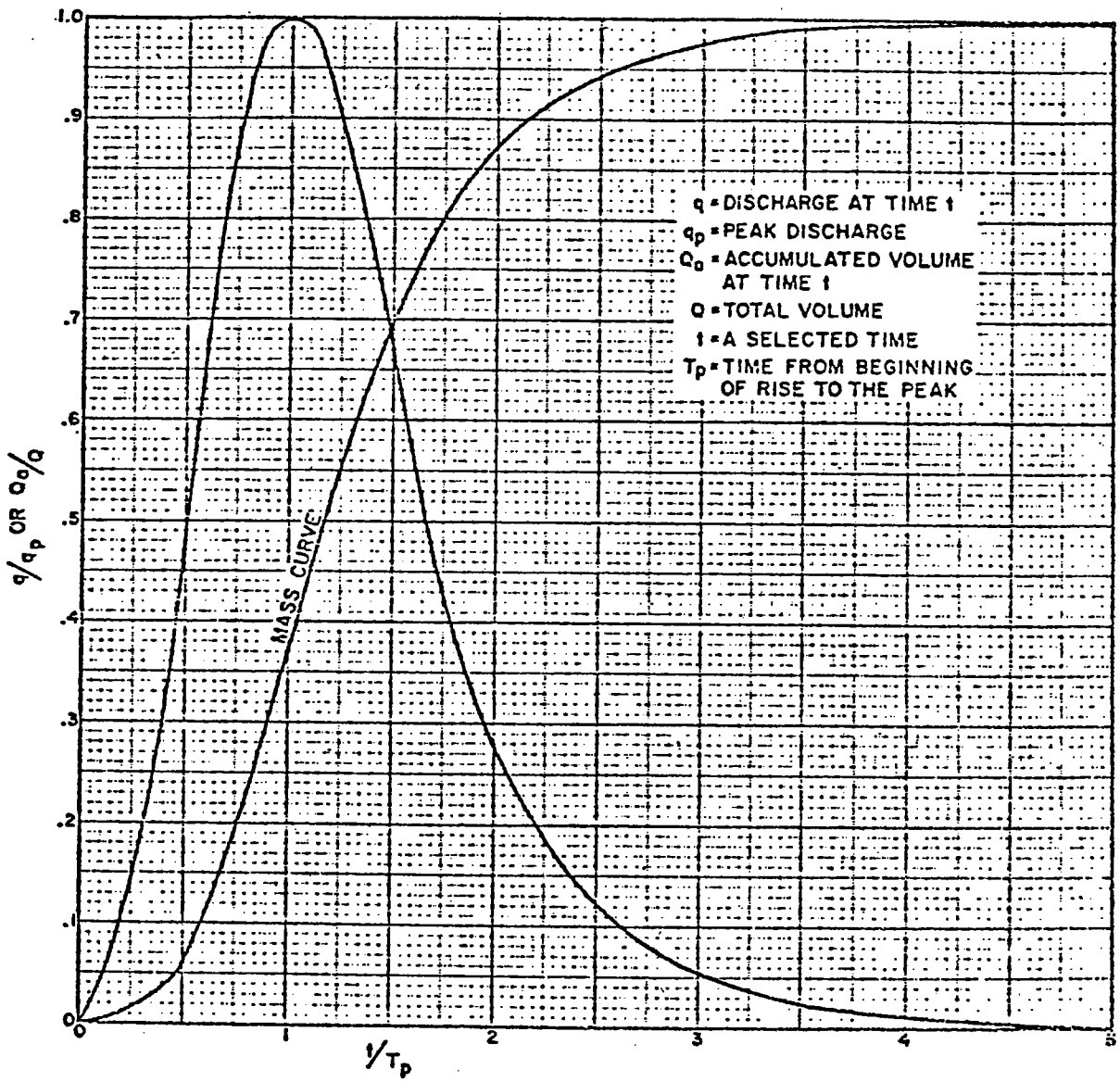
$Q_p = q_p \times 1.75 = 5.25$ as shown in Figure 2-6



Direct Runoff Hydrograph
Figure 2-6

E. SCS CURVILINEAR METHOD UNIT HYDROGRAPH

In this procedure the results of numerous unit hydrographs have been compiled and finally utilized into dimensionless form by dividing flow rate q by peak flow rate q_p and time t by time to peak T_p. The average shape of the derived dimensionless unit hydrograph is shown in Figure 2-7. (See Page 2-25).



Dimensionless unit hydrograph and mass curve
Figure 2-7

SOURCE: USDA Soil Conservation Service, National Engineering Handbook, Section 4 (NEH-4), Hydrology, Aug. 1972.

The characteristics of the dimensionless unit hydrograph are as follows:

$$L = 0.6\ TC$$

$$D = 2/9\ L$$

$$Tp = 5\ D$$

$$T = 5\ Tp$$

$$q_p = \frac{484A}{Tp}$$

where,

L = lag = time from the centroid of the inch of rainfall to the peak of unit hydrograph.

D = Duration of the rainfall.

Tp = Time from the beginning of excess to the peak of unit hydrograph.

T = Total time of direct runoff.

Tc = Time of concentration.

q_p = is in cfs.

A = is in square mile.

Tp = is in hours.

F. EXAMPLE OF SCS CURVILINEAR METHOD UNIT HYDROGRAPH CALCULATIONS

Using the SCS Curvinilinear Method Unit Hydrograph find the direct runoff hydrograph for an urban watershed with the following characteristics.

$Tc = 20$ min $= 1/3$ hrs.

$A = 200$ acres $= 0.3125$ mi^2

10 Year Frequency Storm.

Establish Unit Hydrograph:

$L = 0.6\ TC = 0.20$ hrs.

$D = 2/9\ L = 0.044$ hrs.

$Tp = 5\ D = 0.22$ hrs.

$T = 5\ Tp = 1.11$ hrs.

$$q_p = \frac{484A}{TP} = \frac{484(0.3125)}{0.22} = 681 \text{ cfs}$$

Complete the calculations as follows:

1. List t in increments of D
2. Find t/T_p
3. Find corresponding q/q_p, Figure (2-7)
4. Unit hydrograph ordinate $q = q/q_p \times q_p$

1 t(hrs)	2 t/T_p	3 q/q_p	4 q(cfs)
0	0	0	0
.04	.20	.100	68
.09	.40	.310	211
.13	.60	.660	449
.18	.80	.930	633
.22	1.00	1.000	681
.27	1.20	.930	633
.31	1.40	.780	531
.36	1.60	.560	381
.40	1.80	.390	265

.44	2.00	.280	191
.49	2.20	.207	141
.53	2 40	.147	100
.58	2.60	.107	73
.62	2.80	.077	52
.67	3.00	.055	37
.71	3.20	.040	27
.76	3.40	.029	20
.80	3.60	.021	14
.84	3.80	.015	10
.89	4.00	.011	7
.93	4.20	.009	6
.98	4.40	.006	4
1.02	4.60	.004	3
1.07	4.80	.002	1
1.11	5.00	.000	0

Implied rate of runoff from Unit Hydrograph is $1^{in}/_{1/3\ hr}$ = 3 in/hr.

Rainfall intensity (I) for tc = 20 min from Figure 2-3 is 5.25 in/hr

Direct Runoff Hydrograph is developed by multiplying the ordinate of the Unit Hydrograph by $\frac{5.25}{3}$ = 1.75

G. EXAMPLE OF SCS TRIANGULAR METHOD UNIT HYDROGRAPH CALCULATIONS

The SCS has developed a triangular unit hydrograph which preserves the most important characteristics of their curvilinear one. The relationship for this Unit Hydrograph are as follows:

$$L = 0.6\ TC$$

$$D = 0.4\ L$$

$$T_p = 3\ D$$

$$T = 8\ D$$

$$q_p = \frac{484A}{T_p}$$

H. EXAMPLE OF SCS TRIANGULAR METHOD UNIT HYDROGRAPH CALCULATIONS

Using the SCS Triangular Unit Hydrograph find the direct runoff hydrograph from an urban watershed of the example 2-04-D:

$$L = 0.6\ TC = 0.6 \times 1/3 = 0.20 \text{ hrs.}$$

$$D = 0.4\ L = 0.4 \times 0.2 = 0.08 \text{ hrs.}$$

$$T_p = 3\ D = 3 \times .08 = 0.24 \text{ hrs.}$$

$$T = 8\ D = 0.64 \text{ hrs.}$$

$$q_p = \frac{484A}{T_p} = \frac{484(.3125)}{.24} = 630 \text{ cfs}$$

Complete the calculation:

1. List t in increments of D
2. Find t/T_p
3. Find corresponding q/q_p
4. Unit hydrograph ordinate $q = q/q_p \times q_p$

1 t(hrs)	2 t/T_p	3 q/q_p	4 q(cfs)
0	0	0	0
.08	1/3	1/3	210
.16	2/3	2/3	420
.24	1	1	630
.32	4/3	.8	504

.40	5/3	.6	378
.48	.2	.4	252
.56	7/3	.2	126
.64	8/3	0	0

Direct runoff hydrograph is developed by multiplying the ordinates of the Unit Hydrograph by $\frac{5.25}{3} = 1.75$

FLOW IN STREETS
SECTION 3

3.01 General

- A. Interference Due to Flow in Streets
- B. Interference Due to Ponding
- C. Interference Due to Water Flowing Across Traffic Lane
- D. Effect on Pedestrians
- E. Reduction of Allowable Carrying Capacity

3.02 Permissible Spread of Water

- A. Industrial Streets and Arterials
- B. Neighborhood Collector Streets
- C. Residential Collector Streets
- D. Residential Streets

3.03 Minimum and Maximum Velocities

3.04 Design Method

- A. Straight Crowns
- B. Parabolic Crowns

FLOW IN STREETS
SECTION 3

3.01 GENERAL

The location of inlets and permissible flow of water in the streets should be related to the extent and frequency of interference to traffic and the likelihood of flood damage to surrounding property. Interference to traffic is regulated by design limits of the spread of water into traffic lanes. especially in regard to arterials. Flooding of surrounding property from streets is controlled by limiting curb buildup to the top of curb for a 100-year storm.

A. Interference Due to Flow in Streets

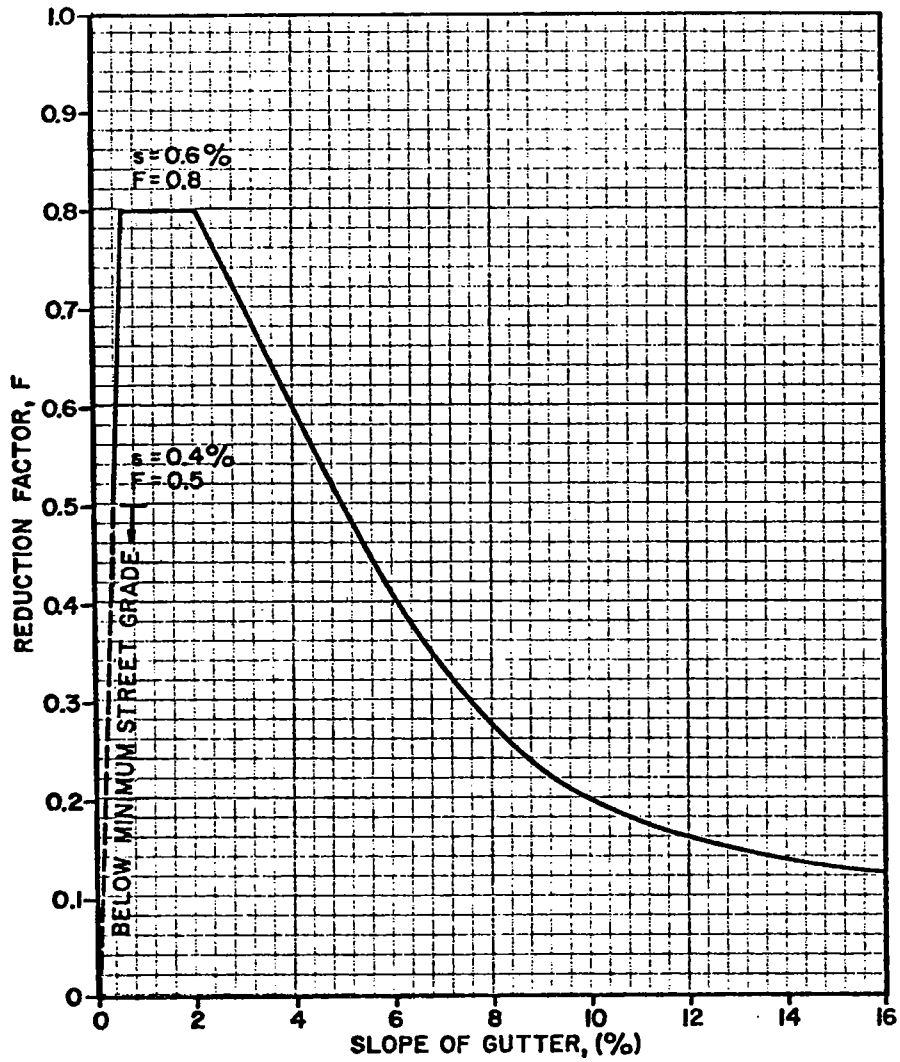
Water which flows in a street, whether from rainfall directly onto the pavement surface or overland flow entering from adjacent land areas, will flow in the gutters of the street until it reaches an overflow point or some outlet, such as a storm sewer inlet. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively enroach into the traffic lane. On streets where parking is not permitted, as with many arterial streets, flow widths exceeding a few feet become a traffic hazard. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow.

As the width of flow increases further it becomes impossible for vehicles to operate without moving through water and they

again use the now inundated lane. Splash from vehicles traveling in the inundated lane obscures the vision of drivers of vehicles moving at a higher rate of speed in the open lane. Eventually, if width and depth of flow become great enough, the street loses it's effectiveness as a traffic-carrier. During these periods it is imperative that emergency vehicles such as fire trucks, ambulances, and police cars be able to traverse the street by moving along the crown of the roadway.

B. Interference Due to Ponding

Storm runoff ponded on the street surface because of grade changes or the crown slope of intersecting streets has a substantial effect on the street-carrying capacity. Because of the localized nature of ponding, vehicles moving at a relatively high speed may enter a pond. The manner in which ponded water affects traffic is essentially the same as for curb flow, that is, the width of spread into the traffic lane is critical. Ponded water will often completely halt all traffic. Ponding in streets has the added hazard of surprise to drivers of moving vehicles, producing erratic and dangerous response.



REDUCTION FACTOR FOR
ALLOWABLE GUTTER CAPACITY

Figure 3-1

C. Interference Due to Water Flowing Across Traffic Lane

Whenever storm runoff, other than limited sheet flow, moves across a traffic lane, a serious and dangerous impediment to traffic flow occurs. The cross-flow may be caused by superelevation of a curve, a street intersection, overflow from the higher gutter on a street with crossfall, or simply poor street design. The problem associated with this type

of flow is the same as for ponding in that it is localized in nature. Vehicles may be travelling at high speed when they reach the location. If vehicular movement is slow and the street is lightly travelled. as on residential streets, limited cross flows do not cause sufficient interference to be unacceptable.

The depth and velocity of cross flows shall be maintained within such limits that they will not have sufficient force to threaten moving traffic.

D. Effect on Pedestrains

In areas with heavily used sidewalks, splash due to vehicles moving through water adjacent to the curb is a serious problem.

Streets should be classified with respect to pedestrian traffic as well as vehicular traffic. As an example, streets which are classified as residential for vehicles and located adjacent to a school are arterials for pedestrian traffic. Allowable width of gutter flow and extent of ponding should reflect this fact.

E. Reduction of Allowable Carrying Capacity

As the storm water flow approaches an arterial street, tee intersection, or cul-de-sac, the allowable carrying capacity shall be calculated by multiplying the reduction factor from Figure 3-1 times the theoretical gutter capacity. The grade used to determine the reduction factor shall be the same

effective grade used to calculate the theoretical capacity.

3.02 PERMISSIBLE SPREAD OF WATER

A. Industrial Streets and Areterials

Inlets shall be spaced at such an interval as to provide one clear traffic lane in each direction during the peak flows of the design storm. The design storm will have a 10-year frequency.

Example: Street width 60'; two 12' traffic lanes to remain clear.

Therefore: Street flow in each gutter shall not exceed

$$\frac{60 - 24}{2} = 18 \text{ feet}$$

B. Neighborhood Collector Streets

The flow of water in gutters of a neighborhood collector street shall be limited so that one standard lane will remain clear during the peak runoff from the design storm. Inlets shall be located at low points or wherever the flow exceeds the one standard lane requirement. Gutter depression at the inlet is discouraged. The design storm will have a 10-year return frequency.

Example: Street width 44'; one 12' traffic lane to remain clear.

Therefore: Street flow in each gutter shall not exceed

$$\frac{44 - 12}{2} = 16 \text{ feet}$$

C. Residential Collector Streets

The flow of water in gutters of a residential collector street shall be limited so that one standard lane will remain clear during the peak runoff from the design storm. Inlets shall be located at low points or wherever the flow exceeds the one standard lane requirement. Gutter depression at the inlet is discouraged. The design storm will have a 10-year return frequency.

Example: Street width 36'; one 12' traffic lane to remain clear.

Therefore: Street flow in each gutter shall not exceed

$$\frac{36 - 12}{2} = 12 \text{ feet}$$

D. Residential Streets

The flow of water in gutters of a residential street shall be limited to a flow at the curb of 6 inches or wherever the street is just covered, whichever is the least depth. Inlets shall be located at low points, or wherever the gutter flow exceeds the permissible spread of water. The design storm will have a 10-year return frequency.

3.03 MINIMUM AND MAXIMUM VELOCITIES

To insure cleaning velocities at very low flows, the gutter shall have a minimum slope of 0.0004 ft/ft (0.4%). The maximum velocity of curb flow shall be 10 fps. Along sharp horizontal curves peak flows tend to jump behind the curb-

line and driveways and other curb breaks. Water running behind the curbline can result in considerable damage due to erosion and flooding. In a gutter which has a slope greater than 0.10 ft/ft (10%) and a radius of 400 feet or less, design flow shall not exceed 4 inches at the curb.

3.04 DESIGN METHOD

A. Straight Crowns

Flow in gutters which are on straight crown pavements is normally calculated by using Manning's Equation for various hydraulic properties for uniform flow in pavement gutters or triangular channels. The equation is:

$$Q_o = 0.56z/n \; S_o^{\frac{1}{2}} \; Y_o^{8/3}$$

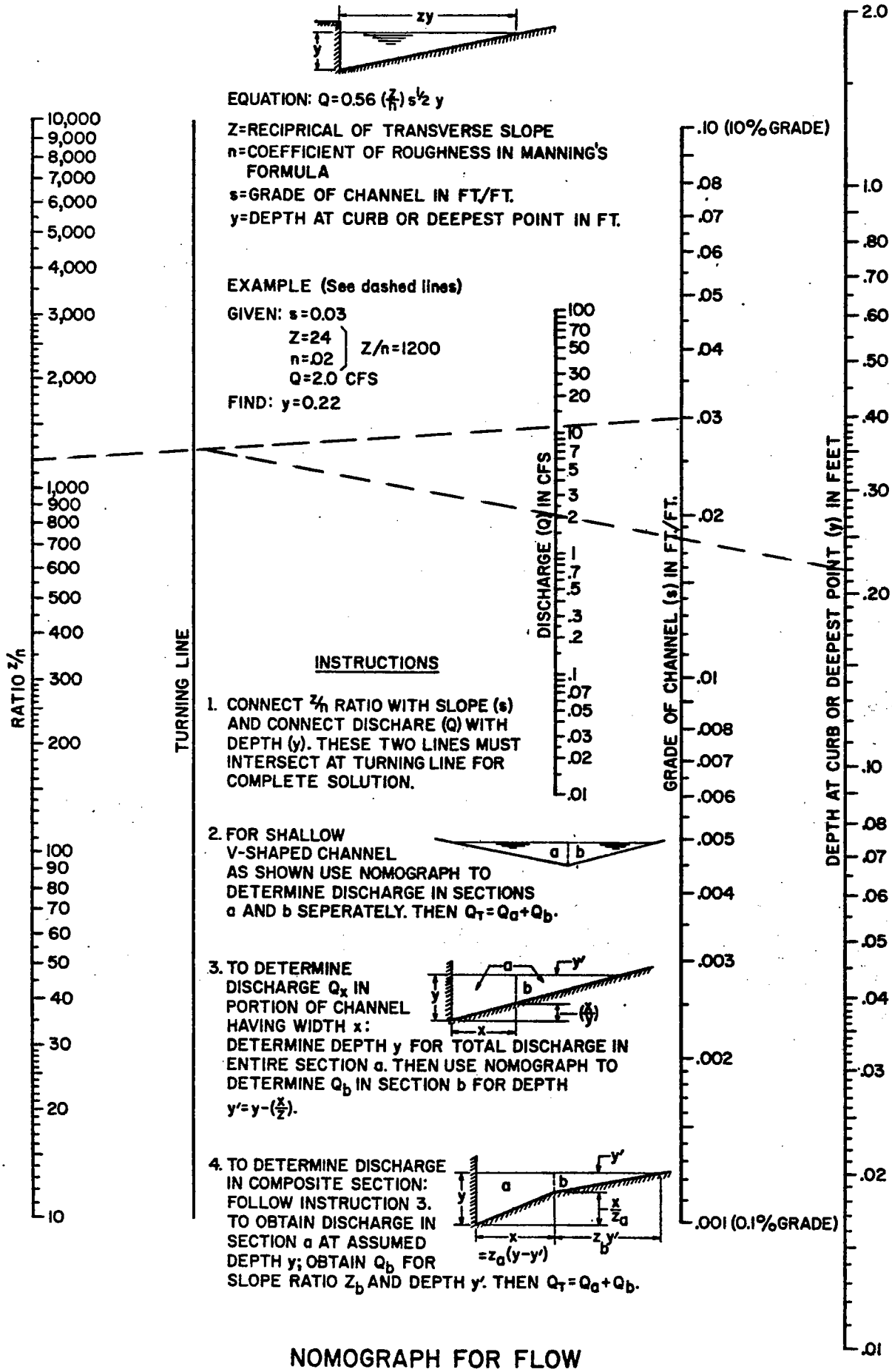
z = Reciprocal of the crown slope (ft/ft)

S_o = Street or gutter slope (ft/ft)

n = Roughness coefficient

Y_o = Depth of flow in gutter (ft)

The nomograph in Figure 3-2 provides for direct solution of flow conditions for triangular channels most frequently encountered in urban drainage design. For the usual concrete gutter a value of 0.016 for "n" is recommended.



NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS

Figure 3 - 2

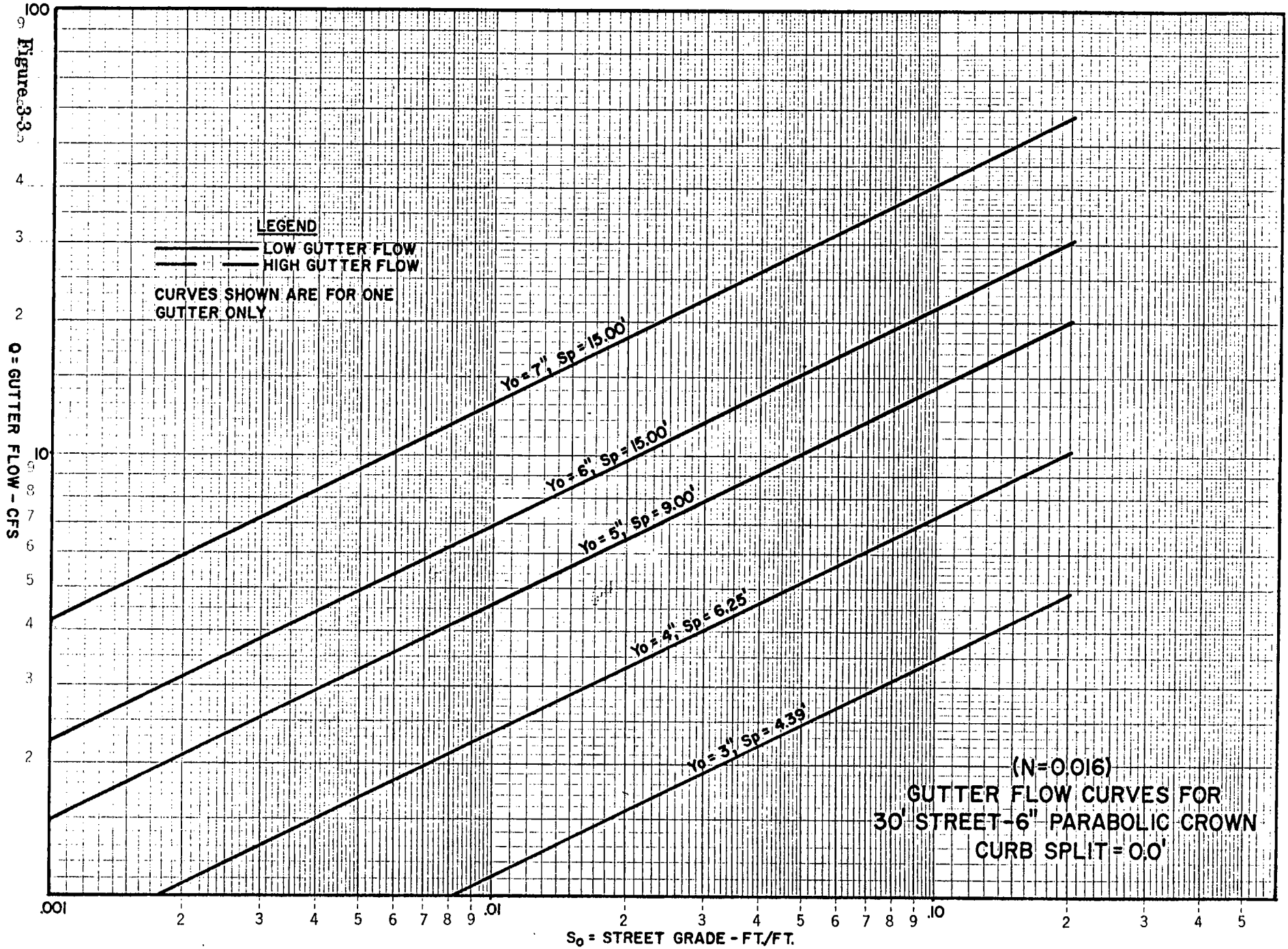
B. Parabolic Crowns

Flow in gutters which are on parabolic crown pavements is calculated from a variation of Manning's Equation for steady flow in a prismatic open channel. However, this equation becomes complicated and difficult to solve for each design case. To provide a means of readily determining flow in the gutter and the spread of water into the traffic lane, gutterflow curves have been prepared. Figure 3-3 and 3-8 present flow curves for various combinations of street widths and parabolic crown heights.

Special consideration should be taken when working with cross sections which have the pavement crown above the top curb. When the crown exceeds the height of the top of curb the maximum depth of water at the curb (Y_o) is the height of curb, not the crown height.

For street sections where water is allowed to cover the street, the crown height should be chosen with care when designing the parabolic section. A parabolic section which has the crown equal to the top of curb will carry more water than will a section which has the crown 1" above the top of curb.

Figure 3-3


LEGEND
LOW GUTTER FLOW
HIGH GUTTER FLOW
CURVES SHOWN ARE FOR ONE GUTTER ONLY
Yo = 7", Sp = 15.00'
Yo = 6", Sp = 15.00'
Yo = 5", Sp = 9.00'
Yo = 4", Sp = 6.25'
Yo = 3", Sp = 4.39'
(N=0.016)
GUTTER FLOW CURVES FOR
30' STREET-6" PARABOLIC CROWN
CURB SPLIT = 0.0'
Q = GUTTER FLOW - CFS
So = STREET GRADE - FT./FT.
100
10
.001
.01
.10

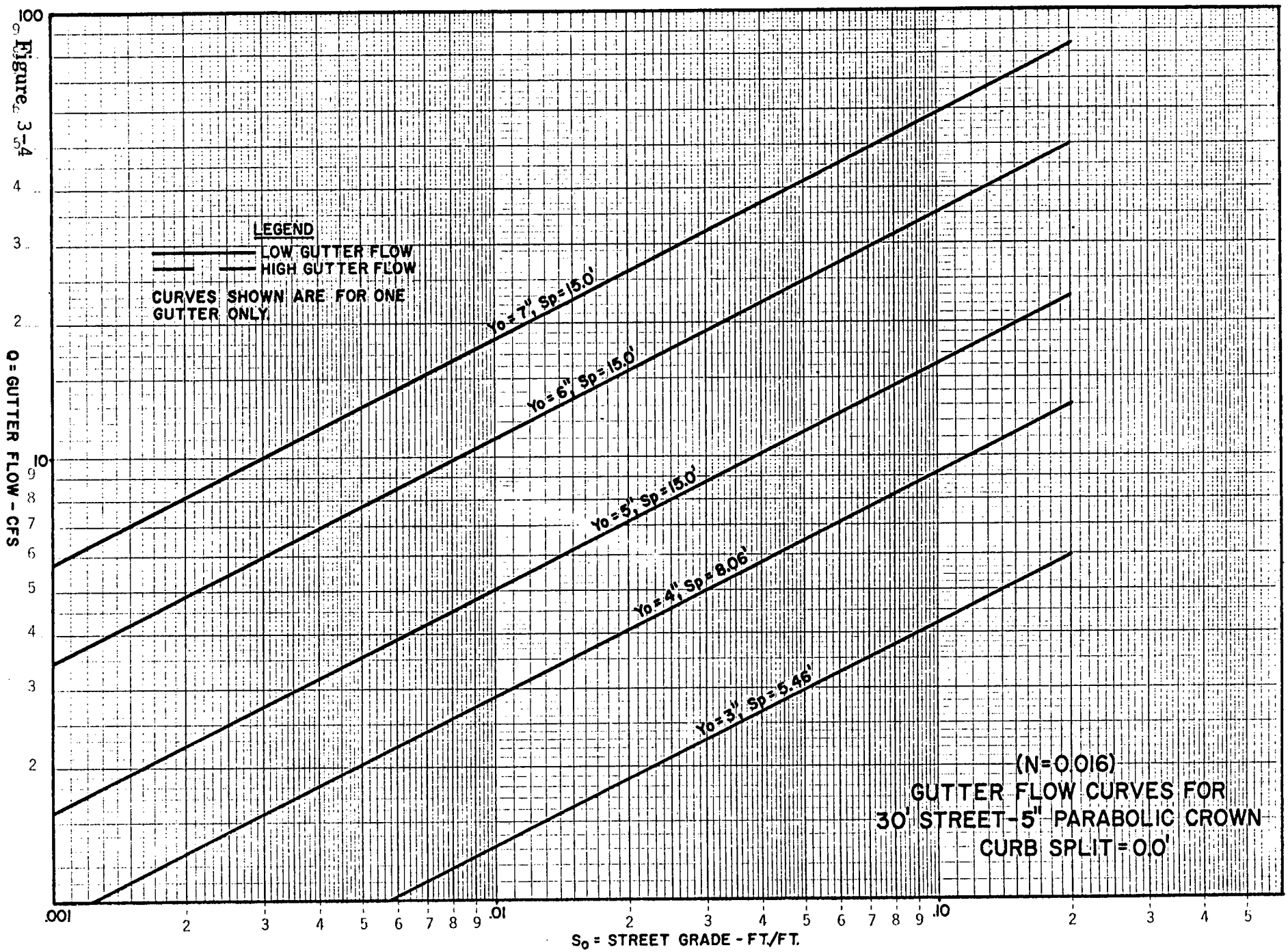

LEGEND
LOW GUTTER FLOW
HIGH GUTTER FLOW
CURVES SHOWN ARE FOR ONE GUTTER ONLY.
Yo = 7", Sp = 15.0'
Yo = 6", Sp = 15.0'
Yo = 5", Sp = 15.0'
Yo = 4", Sp = 8.06'
Yo = 3", Sp = 5.46'
(N=0.016)
GUTTER FLOW CURVES FOR
30' STREET-5" PARABOLIC CROWN
CURB SPLIT = 0.0'
Q = GUTTER FLOW - CFS
So = STREET GRADE - FT./FT.
100
10
.001
.01
.10

Figure 3-4

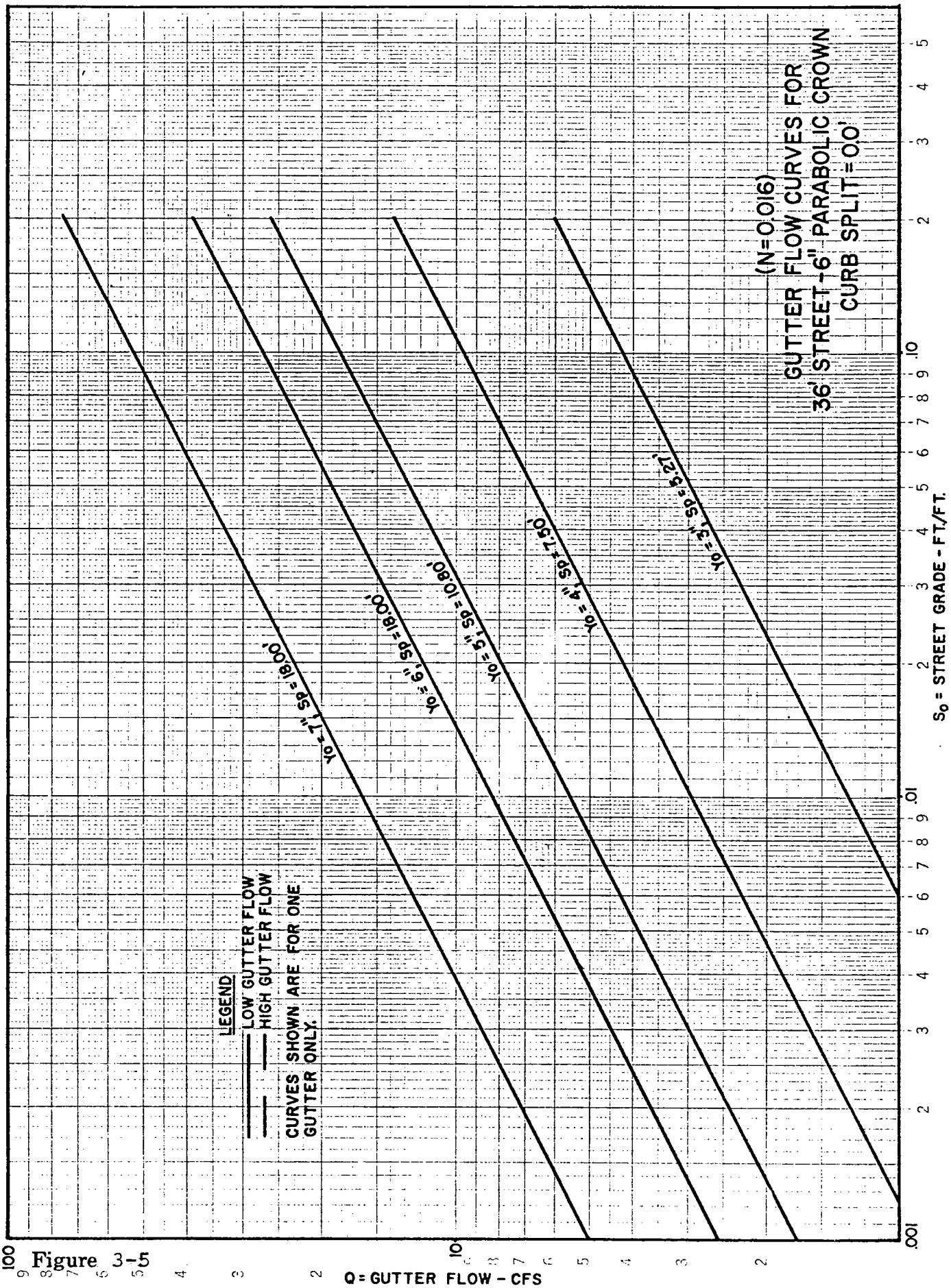

GUTTER FLOW CURVES FOR
36' STREET-6" PARABOLIC CROWN
CURB SPLIT = 0.0'
(N=0.016)
LEGEND
LOW GUTTER FLOW
HIGH GUTTER FLOW
CURVES SHOWN ARE FOR ONE GUTTER ONLY.
Yo = 7.1" Sp = 18.00'
Yo = 6.1" Sp = 18.00'
Yo = 5.1" Sp = 10.80'
Yo = 4.1" Sp = 7.50'
Yo = 3.1" Sp = 5.27'
S0 = STREET GRADE - FT./FT.
Q = GUTTER FLOW - CFS

Figure 3-5

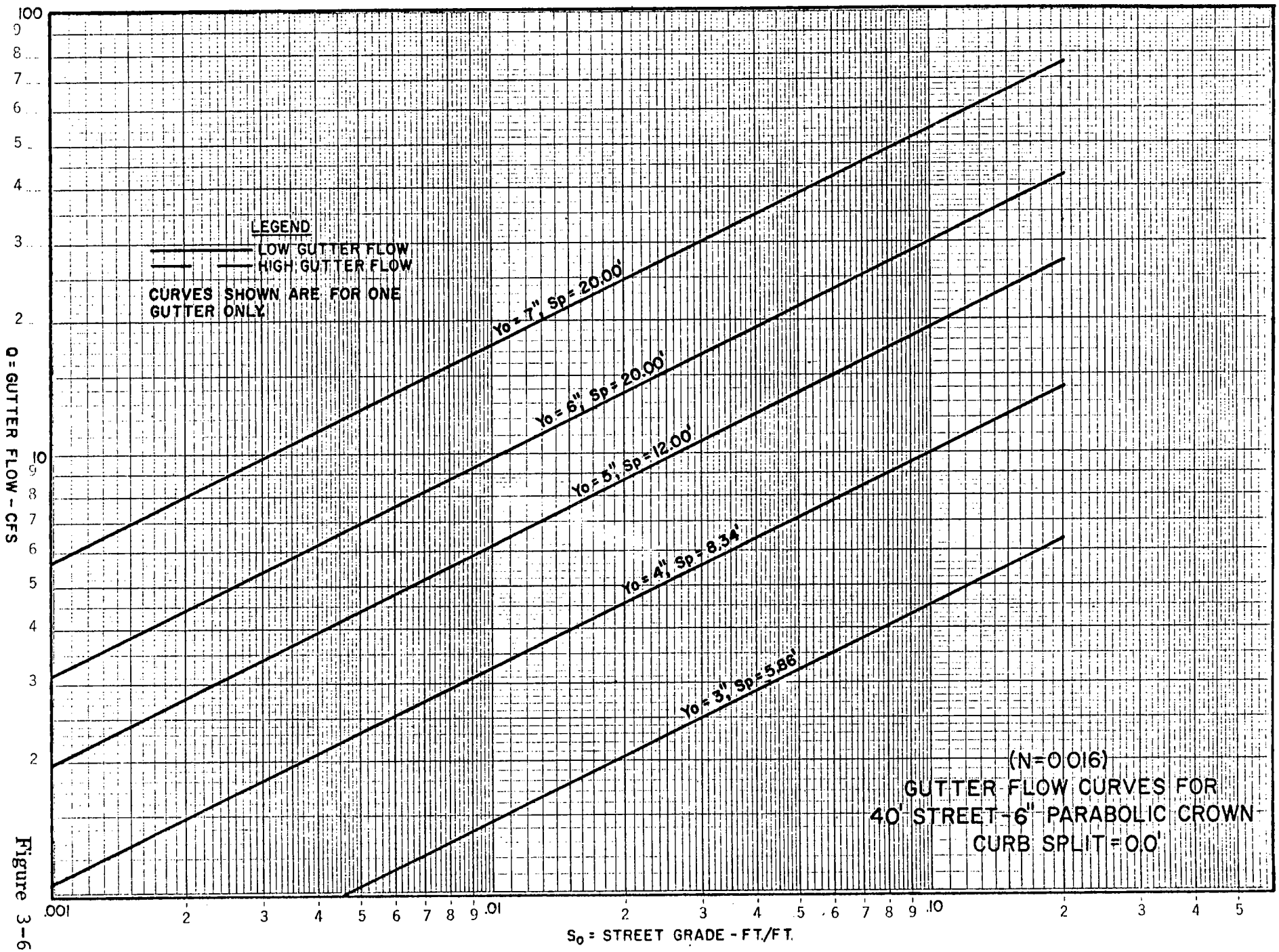

LEGEND
LOW GUTTER FLOW
HIGH GUTTER FLOW
CURVES SHOWN ARE FOR ONE GUTTER ONLY
Yo = 7", Sp = 20.00'
Yo = 6", Sp = 20.00'
Yo = 5", Sp = 12.00'
Yo = 4", Sp = 8.34'
Yo = 3", Sp = 5.86'
(N=0.016)
GUTTER FLOW CURVES FOR
40' STREET-6" PARABOLIC CROWN
CURB SPLIT = 0.0'
Q = GUTTER FLOW - CFS
So = STREET GRADE - FT./FT.
100
10
.001
.01
.10

Figure 3-6

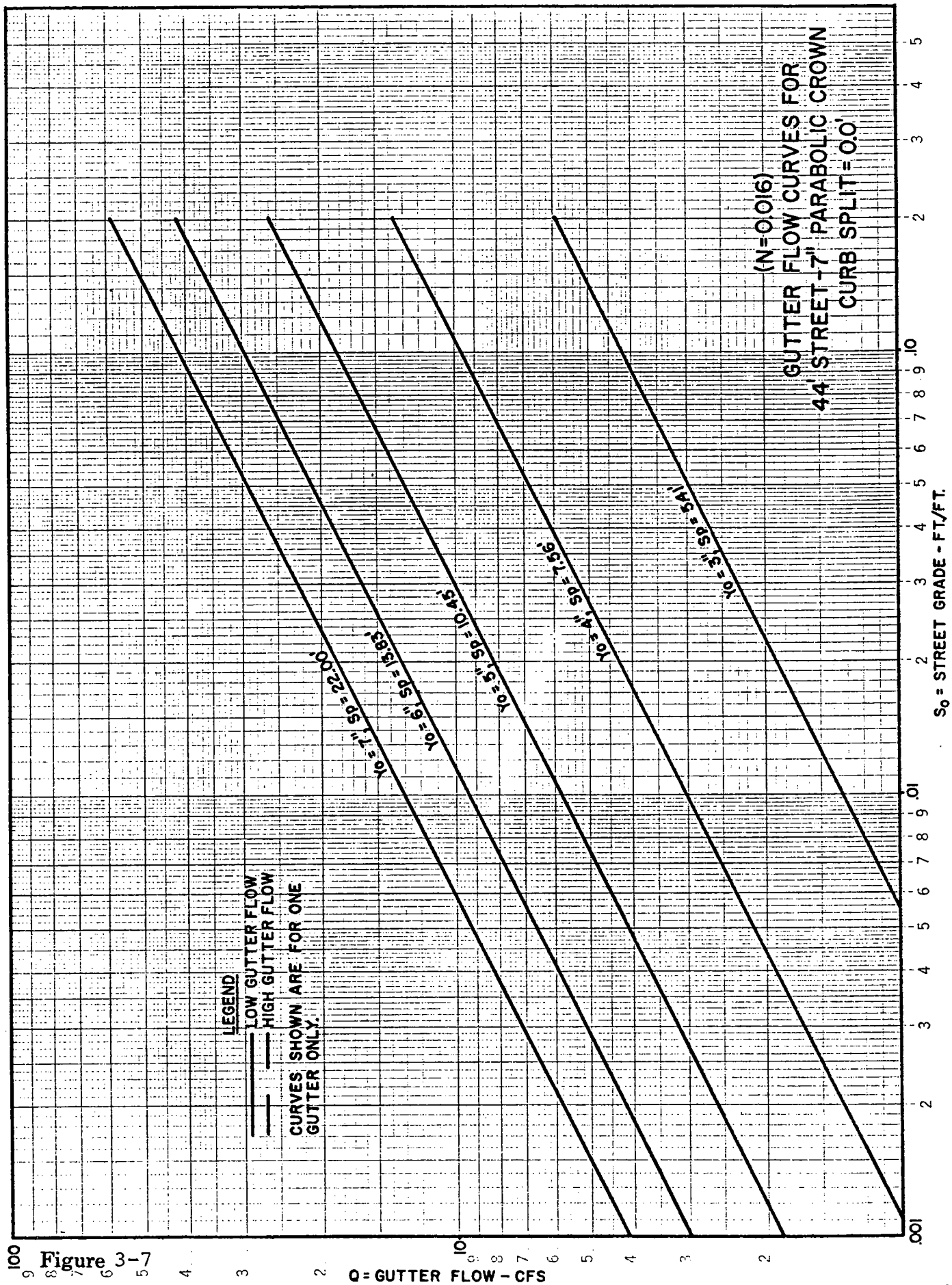

GUTTER FLOW CURVES FOR
44' STREET -7" PARABOLIC CROWN
CURB SPLIT = 0.0'
(N=0.016)
Yo = 3" Sp = 5.4'
Yo = 4" Sp = 7.56'
Yo = 5" Sp = 10.45'
Yo = 6" Sp = 13.83'
Yo = 7" Sp = 22.00'
LEGEND
LOW GUTTER FLOW
HIGH GUTTER FLOW
CURVES SHOWN ARE FOR ONE GUTTER ONLY.
Q = GUTTER FLOW - CFS
So = STREET GRADE - FT./FT.

Figure 3-7

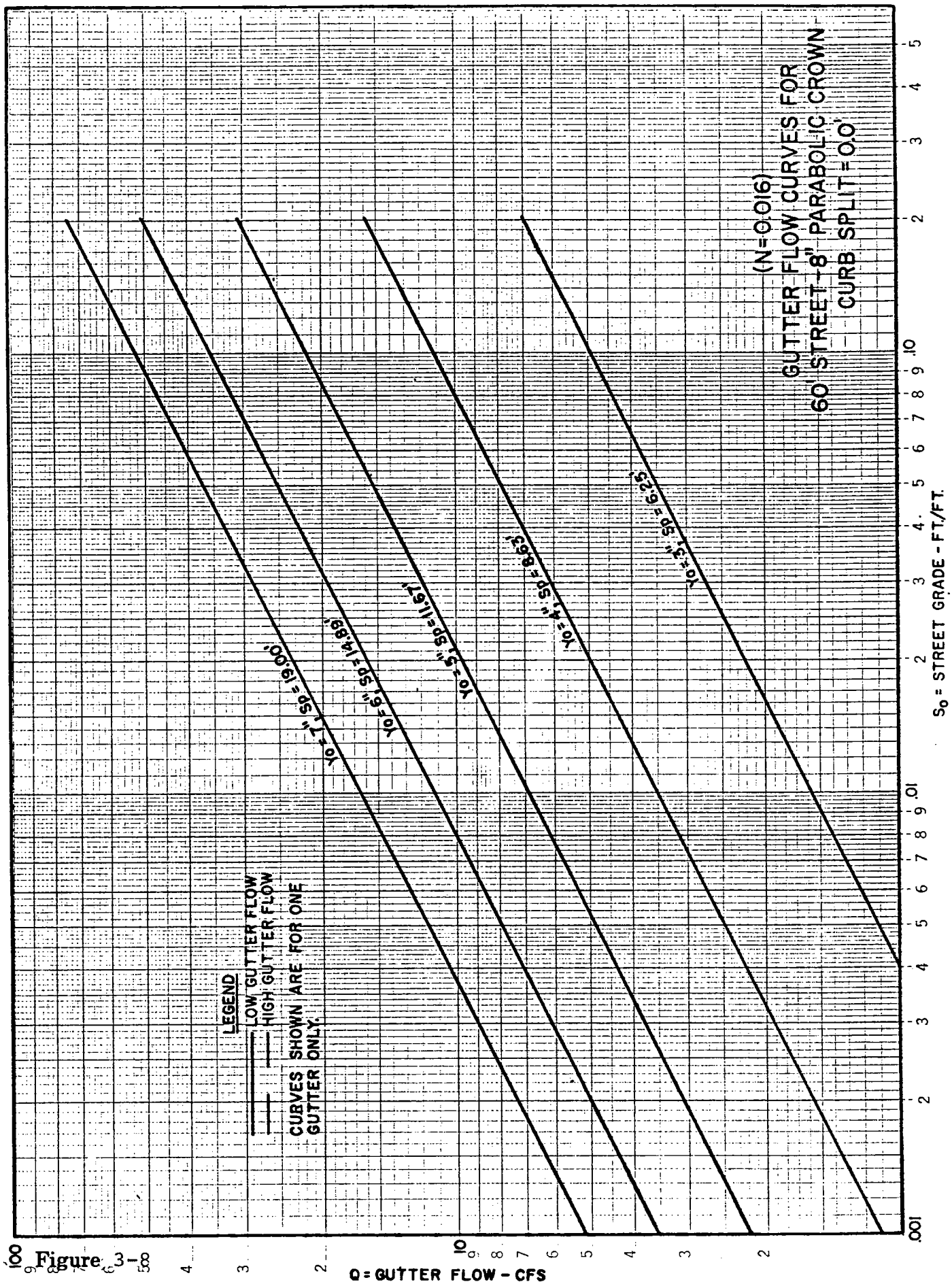

GUTTER FLOW CURVES FOR
60' STREET - 8" PARABOLIC CROWN
CURB SPLIT = 0.0'
(N=0.016)
LEGEND
LOW GUTTER FLOW
HIGH GUTTER FLOW
CURVES SHOWN ARE FOR ONE GUTTER ONLY.
Yo = 7" SP = 6.61'
Yo = 6" SP = 14.89'
Yo = 5" SP = 11.67'
Yo = 4" SP = 8.63'
Yo = 3" SP = 6.25'
So = STREET GRADE - FT./FT.
Q = GUTTER FLOW - CFS

Figure 3-8

STORM DRAIN INLETS
SECTION 4

4.01 General

4.02 Classification

4.03 Inlets In Sumps

- A. Curb Opening Inlets (Type No. 1)
- B. Grate Inlets (Type 2, 3 & 4)
- C. Combination Inlets (Type No. 5)

4.04 Inlets On Grade

- A. Curb Opening Inlets On Grade (Type No. 1)
- B. Grate Inlets On Grade (Type 2, 3 & 4)
- C. Combination Inlets On Grade (Type No. 5)

4.05 Use of Figures 4-7 and 4-8

STORM DRAIN INLETS
SECTION 4

4.01 GENERAL

The primary purpose of storm drain inlets is to intercept excess surface runoff and deposit it in a drainage system, thereby reducing the possibility of surface flooding.

The most common location for inlets is in streets which collect and channelize surface flow making it convenient to intercept. Because the primary purpose of streets is to carry vehicular traffic, inlets must be designed so as not to conflict with that purpose.

The following guidelines shall be used in the design of inlets to be located in streets:

1. Design and location of inlets shall take into consideration pedestrian and bicycle traffic.

2. Inlet design and location must be compatible with the criteria established in Section 3 of this manual.

4.02 CLASSIFICATION

The most common inlets used in Jefferson Parish areas are classified into two major groups, namely: inlets in sumps and inlets on grade without gutter depression. Each of the two major classes includes several varieties. The following are presented herein and are likely to find reasonable wide use. (See Figures 4-1 through 4-4).

Inlets in Sumps	Available Standards
(1) Curb Opening	Type No. 1
(2) Grate	Nos. 2, 3 and 4
(3) Combination (Grate and Curb Opening)	Type No. 5
(4) Drop (Round Grate Covering)	Type 12"Ø
(5) Drop (Square Grate Covering)	14"x14", 17½"x17½", 24"x24" 24"x36", 36"x36"

Inlets on Grade	
(1) Curb Opening	Type No. 1
(2) Grate	Nos. 2, 3 & 4
(3) Combination (Grate and Curb Opening)	Type No. 5

Drainage Department review of proposed drainage plans shall include an examination of the supporting computations. Computations must be submitted either as a part of the plans or on separate tabulation sheets convenient for review and permanent record.

4.03 INLETS IN SUMPS

Inlets in sumps are inlets in low points of surface drainage to relieve ponding. The capacity of inlets in sumps must be known in order to determine the depth and width of ponding for a given discharge. The charts in this section may be used in the design of any inlet in a sump.

A. Curb Opening Inlets (Type No. 1)

General. Unsubmerged curb opening inlets (Type No. 1) in a sump or low point are considered to function as rectangular weirs with a coefficient of discharge of 3.0. Their capacity shall be based on the following equation:

$$Q = 3.0y^{3/2}L$$

Q = Capacity in cfs of curb opening inlet or capacity in cfs of drop inlet.

y = Head at the inlet in feet.

L = Length of opening through which water enters the inlet.

Figure 4-5 provides for direct solution of the above equation.

Curb opening inlets and drop inlets in sumps have a tendency to collect debris at their entrances. For this reason, the calculated inlet capacity shall be reduced by 10 percent to allow for clogging.

B. Grate Inlets

General. A grate inlet Type 2, 3 and 4 in a sump can be considered an orifice with a coefficient of discharge of 0.60. The capacity shall be based on the following equation:

$$Q = 4.82A_g y^{\frac{1}{2}} ;$$

Q = Capacity in cfs

A_g = Area of clear opening in sq. ft.

y = Depth of flow at inlet or head at sump in feet.

The curves shown in Figure 4-6 provides for direct solution of the above equation.

Grate inlets in sumps have a tendency to clog when flows carry debric such as leaves and papers. For this reason, the calculated inlet capacity of a grate inlet shall be reduced by 25 percent to allow for clogging.

C. Combination Inlets (Type No. 5)

The capacity of a combination inlet Type No. 5 consisting of a grate and curb opening inlet in a sump shall be considered to be the sum of the capacities obtained from Figures 4-8 and 4-9.

4.04 INLETS ON GRADE

A. Curb Opening Inlets On Grade (Type No. 1)

General. The capacity of a curb inlet. like any weir, depends upon the head and length of overfall. In the case of a curb opening inlet, the head at the upstream end of the opening is the depth of flow in the gutter. In streets where grades are greater than 1 percent the velocities are high and the depths of flow are usually small as there is little time to develop cross flow into the curb openings; therefore, undepressed inlets are inefficient when used in streets of appreciable slope, but may be used satisfactorily where the grade is low and the crown slope high or the gutter channelized. Undepressed inlets do not interfere with traffic and usually are not

susceptible to clogging. Inlets on grade should be designed and spaced so that 5 to 15 percent of gutter flow reaching each inlet will carry over to the next inlet downstream, provided the carry over is not objectionable to pedestrian or vehicular traffic.

The capacity of an undepressed inlet shall be determined by use of Figures 4-7 and 4-8. An example of the use of Figures 4-7 and 4-8 is included at the end of this section.

B. Grate Inlets On Grade (Types 2, 3 and 4)

General. Grate inlets on grade have a greater hydraulic capacity than curb inlets of the same length so long as they remain unclogged. Grate inlets on grade are inefficient in comparision to grate inlets in sumps. Their capacity shall be the capacity determined from Figure 4-6 reduced by 15 percent. Grate inlets should be so designed and spaced so that 5 to 15 percent of the gutter flow reaching each inlet will carry over to the next downstream inlet, provided the carry over is not objectionable to pedestrian or vehicular traffic.

Grates with bars parallel to the curb should always be used for the above described installations because transverse framing bars create splash which causes the water to jump or ride over the grate. Grates used shall be certified by the manufacturer as bicycle-safe. For flows on streets with grades less than one percent, little or no splashing

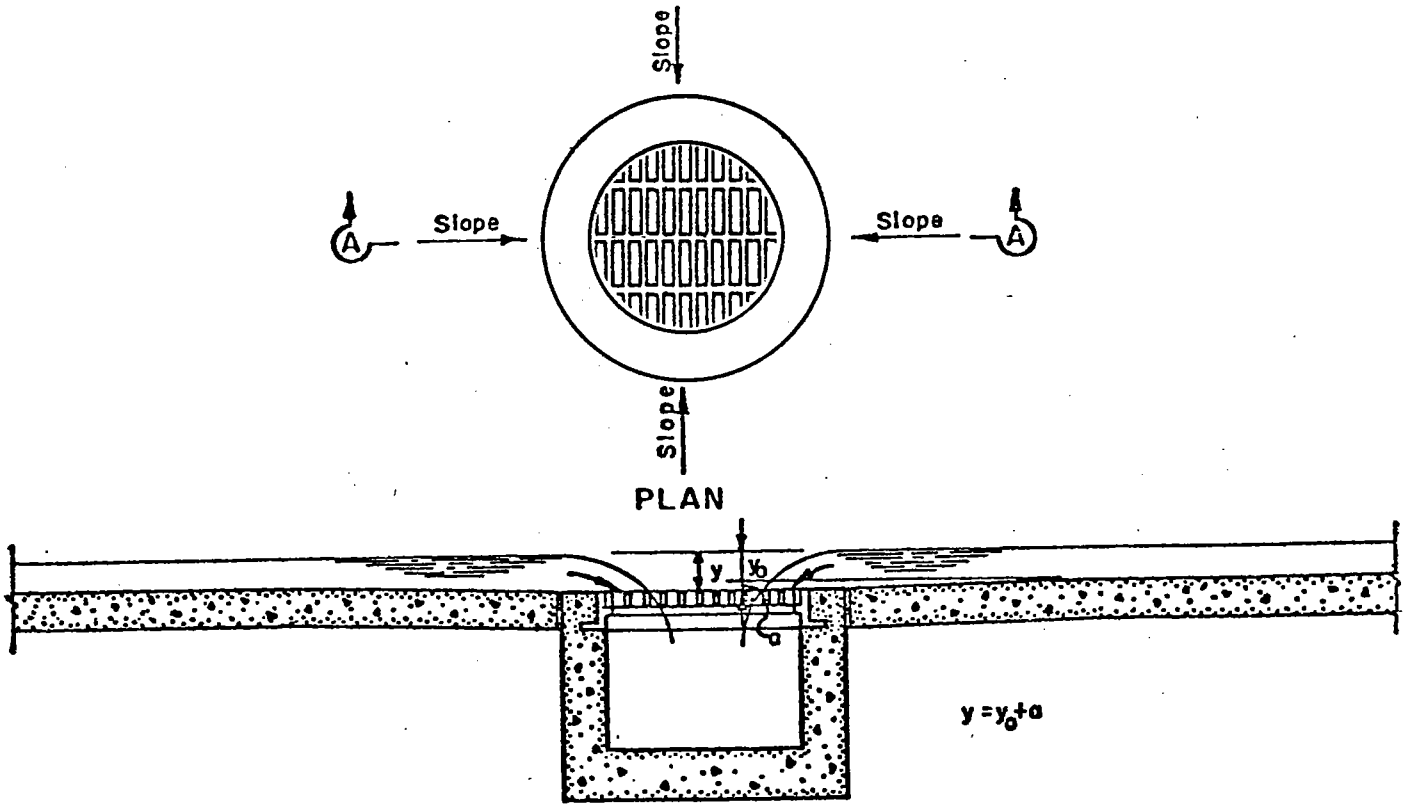
occurs regardless of the direction of bars.

The calculated capacity for a grate inlet shall be reduced by 25 percent to allow for clogging.

C. Combination Inlets On Grade (Type No. 5)

General. Combination (curb opening and grate) inlets on grade have greater hydraulic capacity than curb inlets or grate inlets of the same length. Generally speaking, combination inlets are the most efficient of the three types of the inlets presented in this manual. Grates with bars parallel to the curb should always be used.

The capacity of a Type No. 5 inlet shall be considered to be the sum of the cpacities as determined for a Type No. 1 inlet and a Type 2, 3 or 4 inlet (allowing for reduction due to clogging).

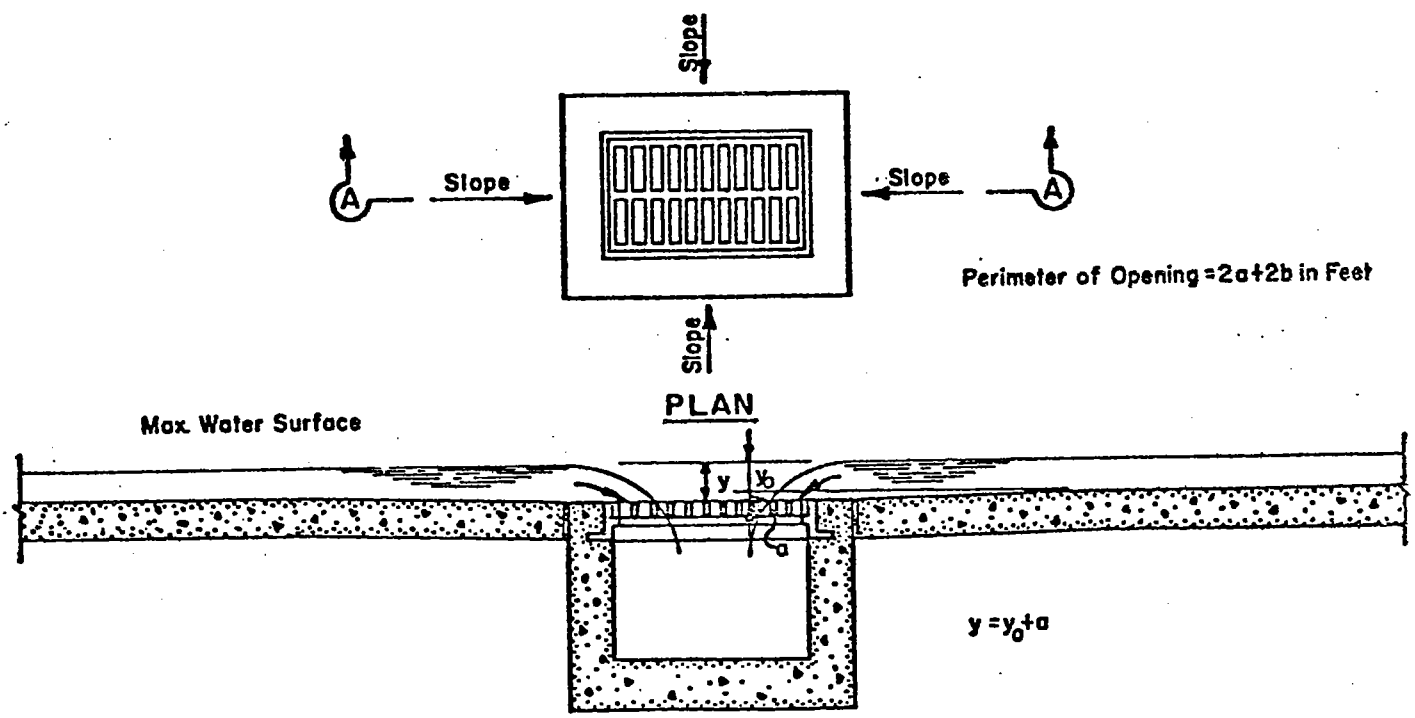


SECTION A-A

ROUND

DROP INLET

(Grate covering)



SECTION A-A

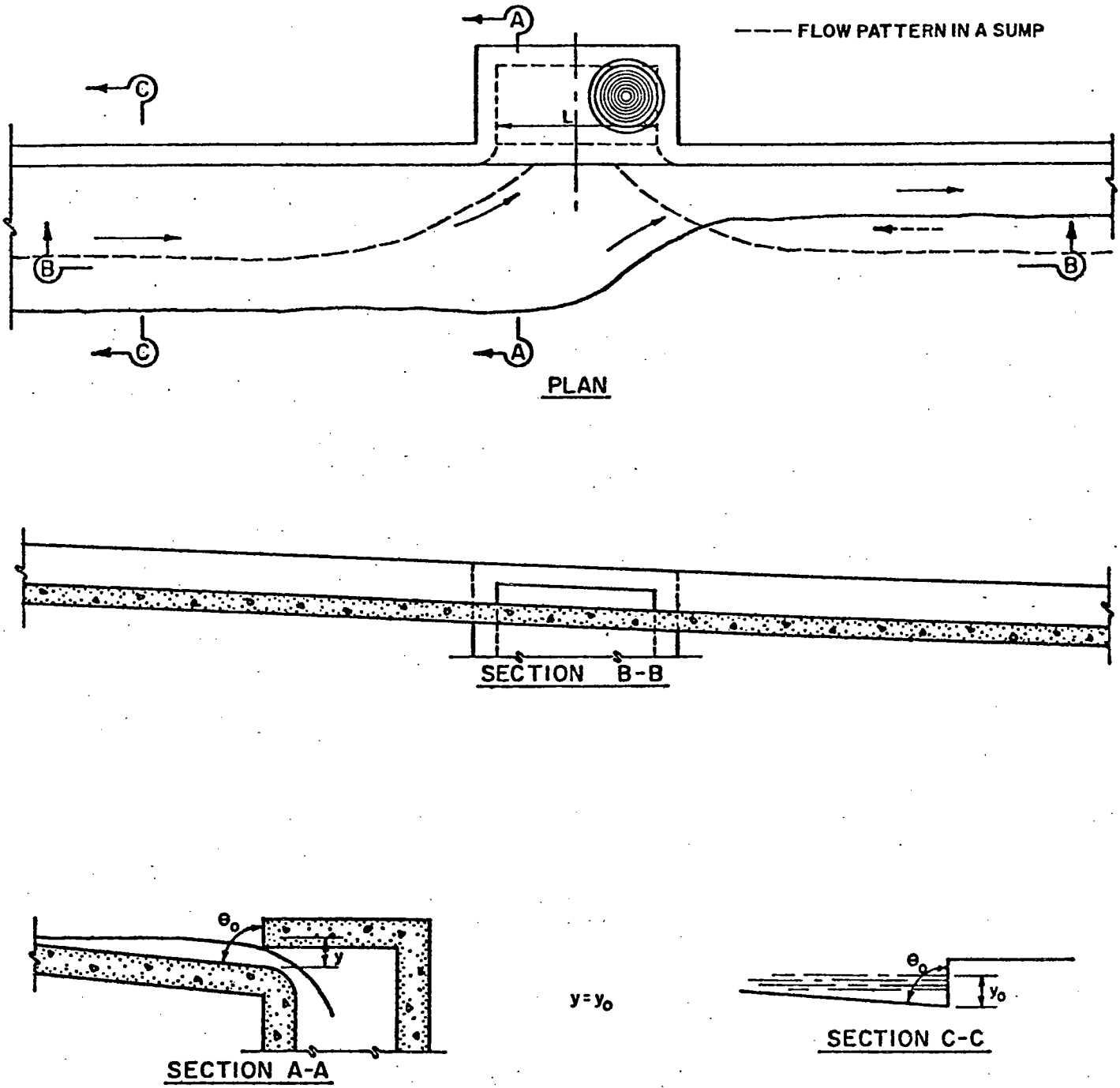
SQUARE

DROP INLET

(Grate covering)

Figure- 4-1

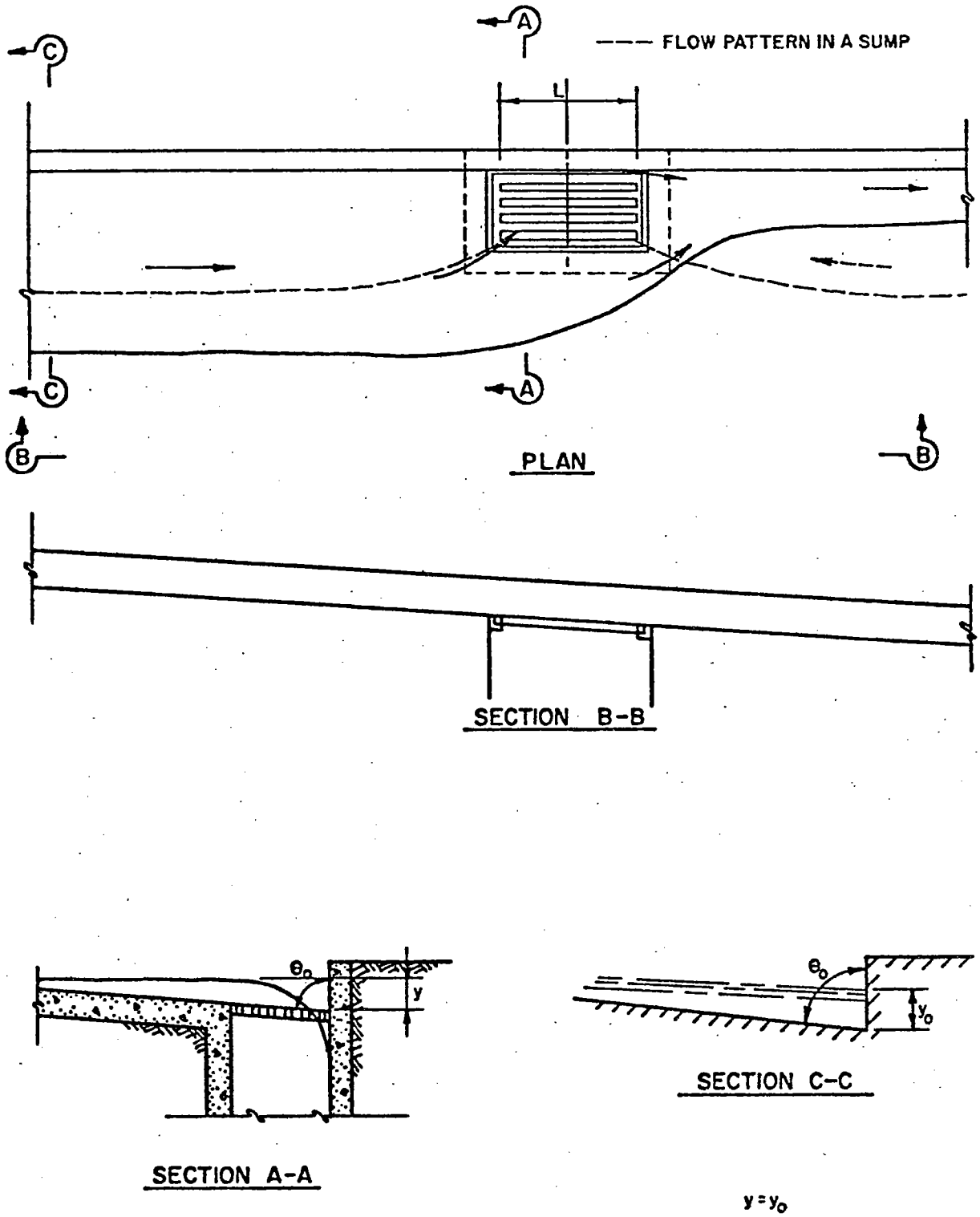
REFER TO DEPT OF PUBLIC WORKS
STANDARD INLET DETAILS



CURB-OPENING INLET TYPE I

Figure 4-2

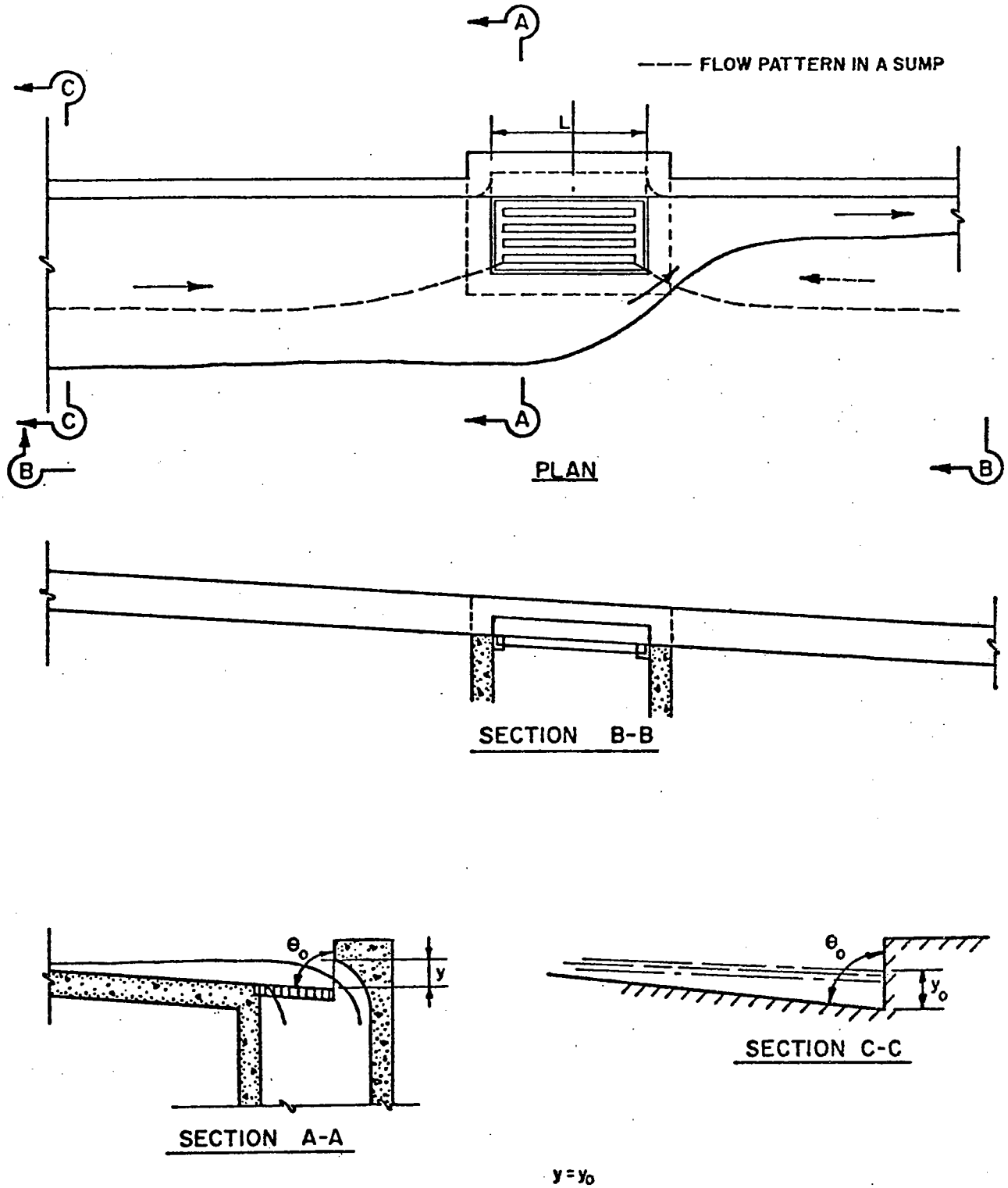
REFER TO DEPT. OF PUBLIC WORKS
STANDARD INLET DETAILS



GRATE INLET TYPES 2, 3, & 4

Figure 4-3

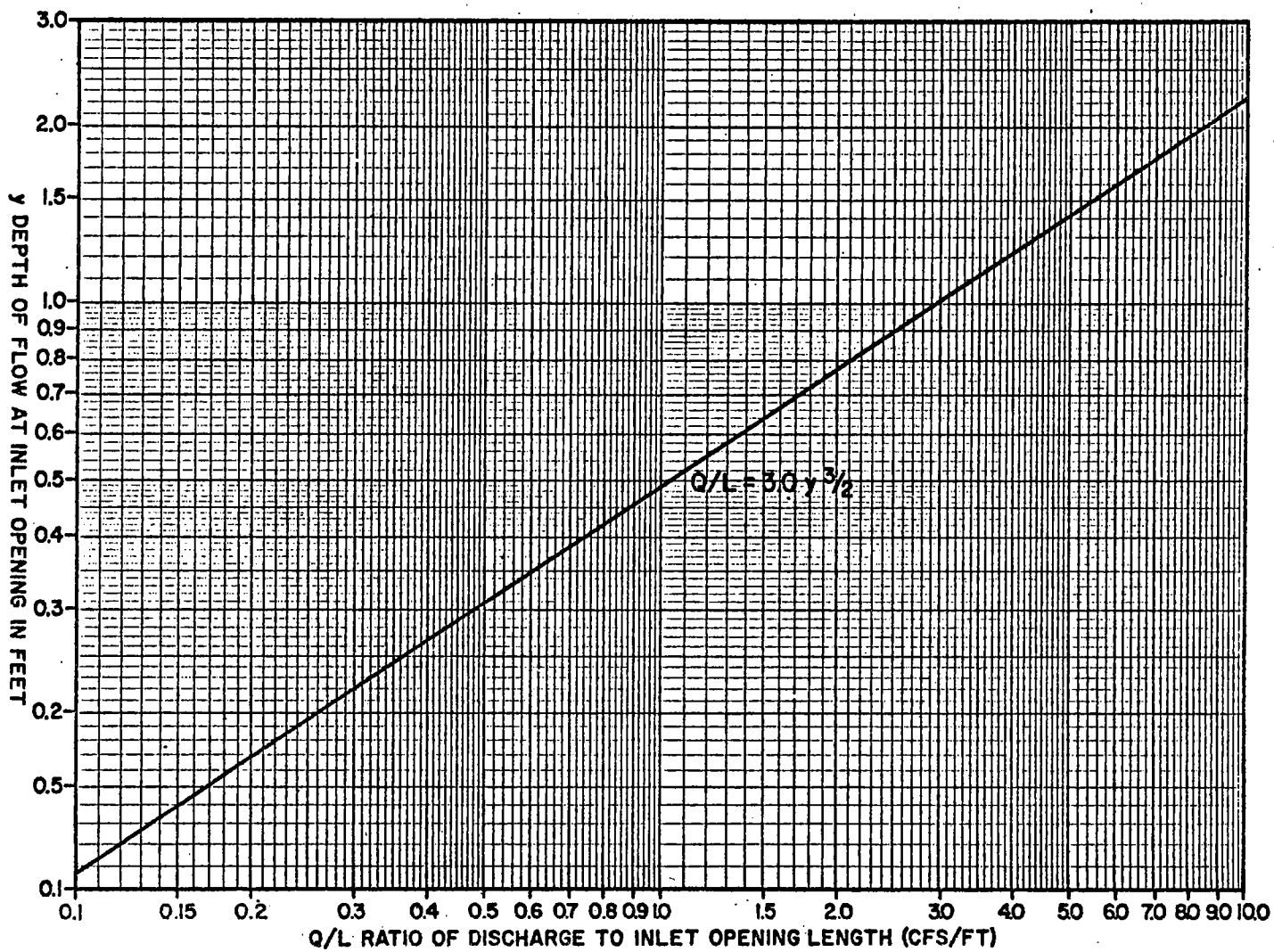
REFER TO DEPT. OF PUBLIC WORKS STANDARD INLET DETAILS



COMBINATION OR OFFSET INLET TYPE NO. 5

Figure 4-4

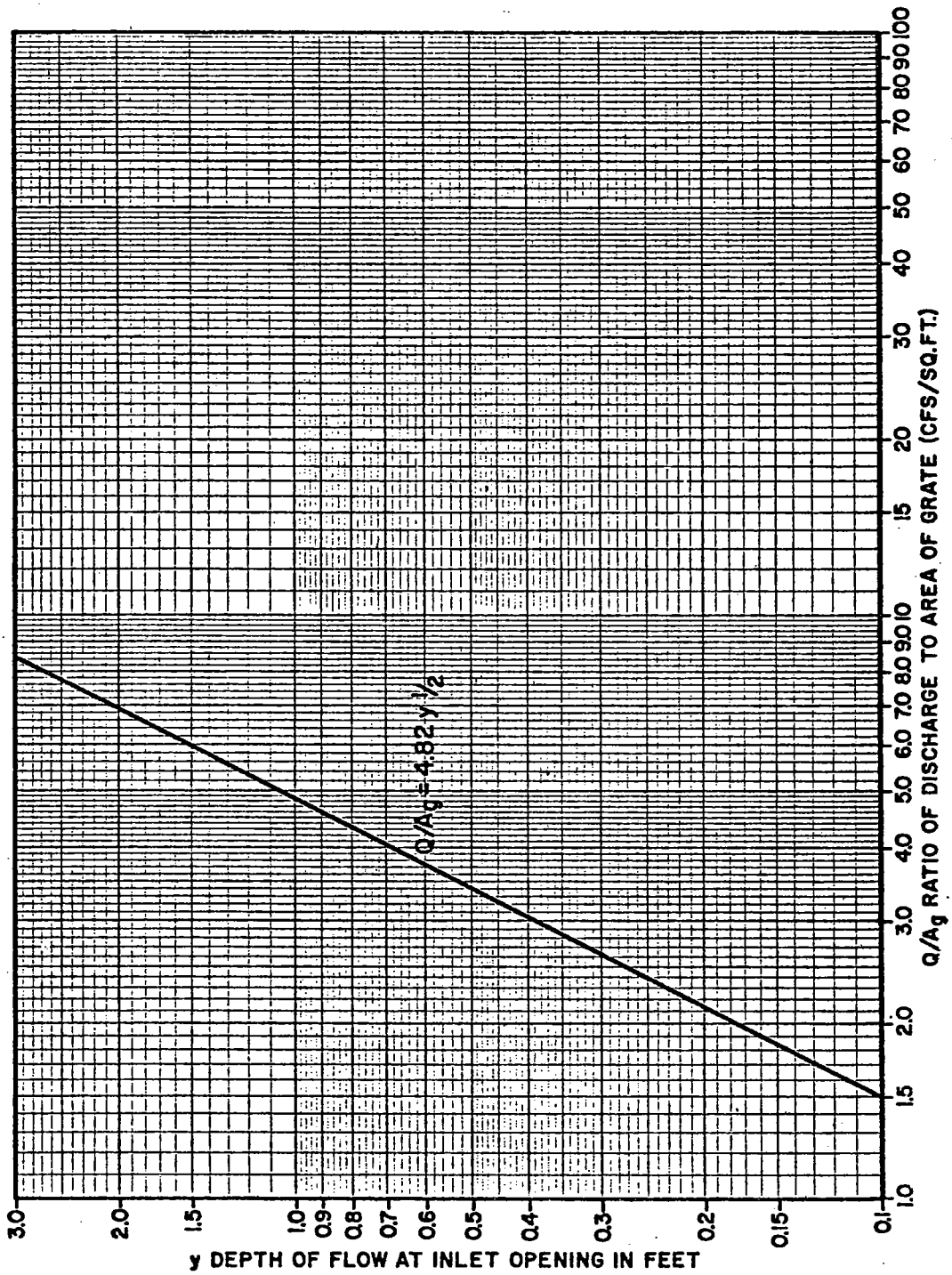
REFER TO DEPT OF PUBLIC WORKS
STANDARD INLET DETAILS



INLET CAPACITY TYPE NO. 1

(CURB OPENING)

Figure 4-5



DROP INLET CAPACITY TYPES 2, 3, & 4

Figure 4-6

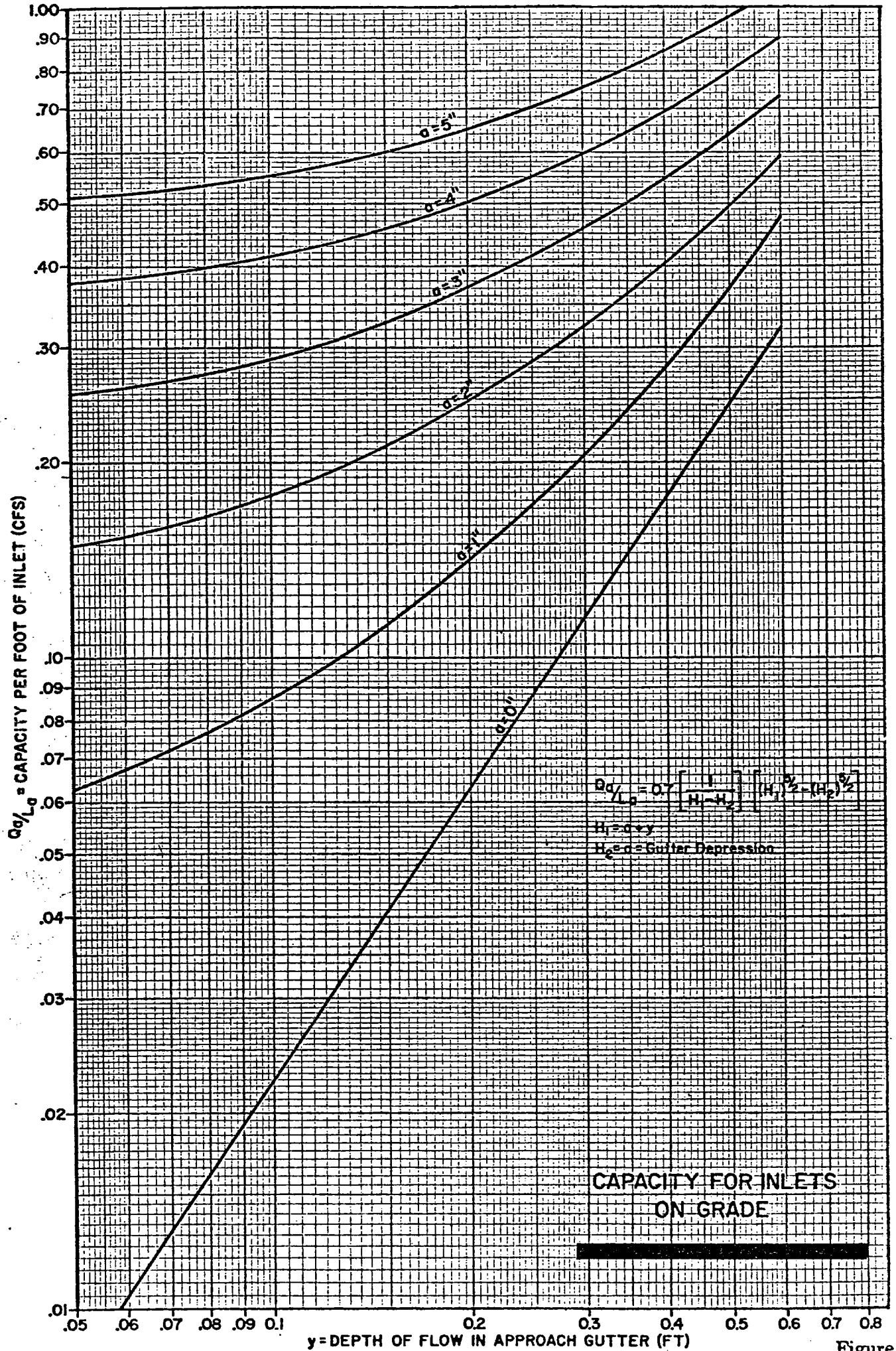
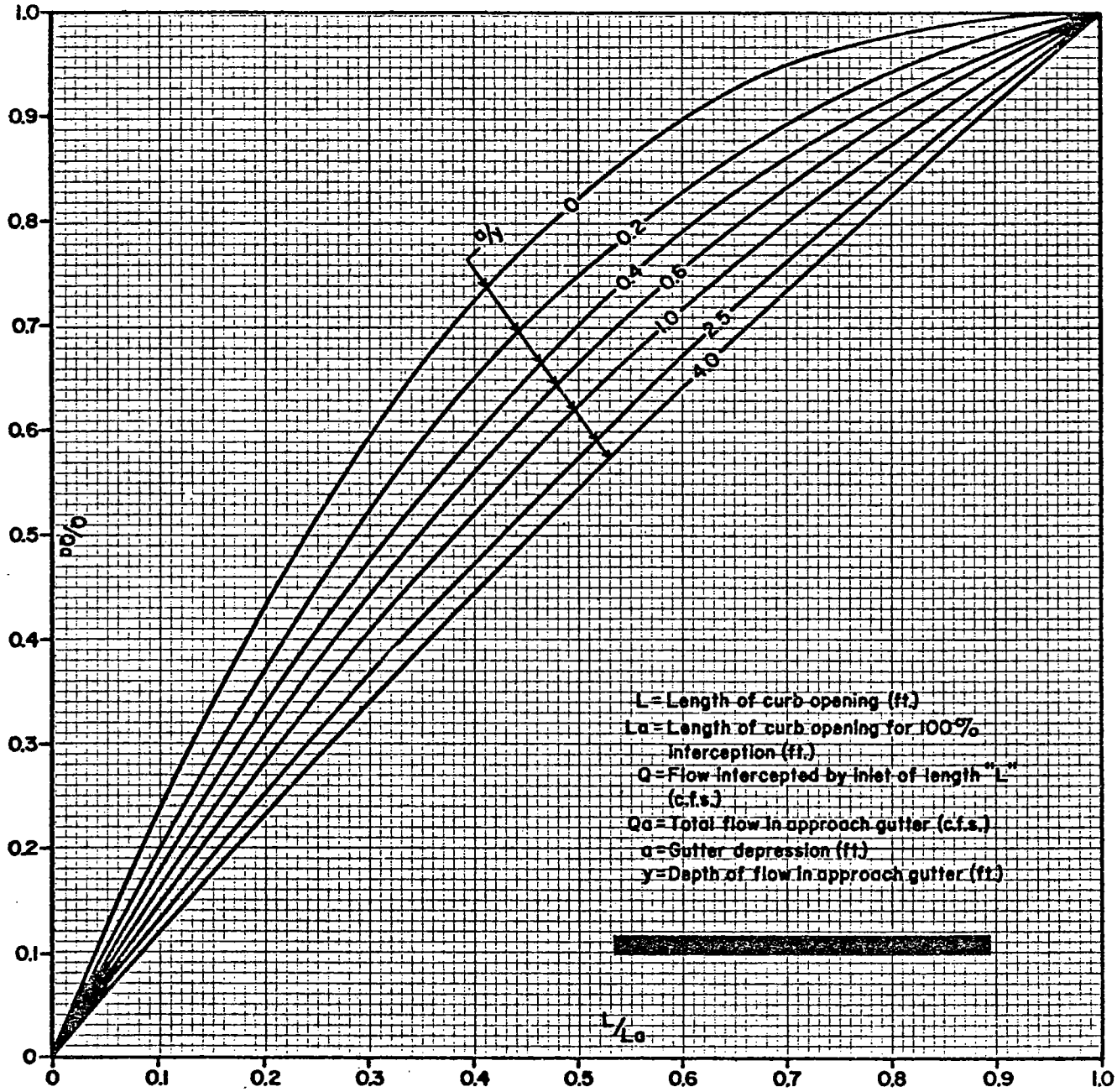

1.00
.90
.80
.70
.60
.50
.40
.30
.20
.10
.09
.08
.07
.06
.05
.04
.03
.02
.01
a=5"
a=4"
a=3"
a=2"
a=1"
a=0"
Q_a/L_a = CAPACITY PER FOOT OF INLET (CFS)
$Q_a/L_a = 0.7\left[\frac{1}{H_1-H_2}\right]\left[(H_1)^{5/2}-(H_2)^{5/2}\right]$
$H_1 = a + y$
$H_2 = a$ = Gutter Depression
CAPACITY FOR INLETS ON GRADE
.05
.06
.07
.08
.09
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
y = DEPTH OF FLOW IN APPROACH GUTTER (FT)

Figure 4-7



RATIO OF INTERCEPTED TO TOTAL FLOW FOR INLETS ON GRADE

Figure 4-8

4.05 USE OF FIGURES 4-7 AND 4-8

Example 1.

Given: Street Width = 30'

Cross Slope = 0.0'

Street Grade = 1.0%

Q_a in one gutter = 8 cfs

Determine: Capacity of a Type No. 1 inlet (4' curb opening).

Step 1. From Figure 3-3 (Section 3) depth of flow in gutter is y - 6.2" or 0.51'.

Step 2. Enter Figure 4-7 with y = 0.51' find corresponding $\frac{Q}{L} = 0.25$

Step 3. Compute L_a ; $L_a = \frac{8}{0.25} = 32.0$

Step 4. Compute $\frac{L}{L_a} = \frac{4}{32} = 0.125$

Step 5. Enter Figure 4-8 with $\frac{L}{L_a} = 0.125$ find corresponding $\frac{Q}{Q_a} = 0.290$

Step 6. Determine Q from $\frac{Q}{Q_a} = 0.29$

$Q = 0.29(8) = 2.40$ cfs

Example 2.

Given: Street Width = 44'

Cross Slope = 0.0'

Street Grade = 0.6%

Q_a in gutter = 8 cfs

Determine: Length of undepressed curb inlet required to intercept 80% of gutter flow (Q = 6.4).

Step 1. From Figure 3-7, Section 3, depth of flow in gutter (y) = 6.25" or 0.52'

Step 2. Entering Figure 4-7 with y = 0.52 find corresponding $\frac{Q_a}{L_a} = 0.27$.

Step 3. Determine L_a ; $L_a = \frac{8}{0.27} = 29.6$.

Step 4. Entering Figure 4-8 with $\frac{Q}{Q_a} = 0.8$ find corresponding $\frac{L}{L_a} = 0.48$.

Step 5. Compute L from $\frac{L}{L_a} = 0.48$, L = 29.6 (0.48) = 14.2'

Use L = 15'.

Step 6. Compute $\frac{L}{L_a} = \frac{15}{29.6} = 0.50$.

Step 7. Entering Figure 4-8 with $\frac{L}{L_a} = 0.50$ find corresponding $\frac{Q}{Q_a} = 0.82$.

Step 8. Compute Q from $\frac{Q}{Q_a} = 0.82$.

Q = 8(0.82) = 6.56 cfs $\geqslant$ 6.4 cfs o.k.

Step 9. If using standard Type 1 inlets the spacing of the inlets should be adjusted to intercept gutter flow equal to their inlet capacities.

FLOW IN STORM DRAINS AND THEIR APPURTENANCES
SECTION 5

5.01 General

5.02 Velocities and Grades

- A. Minimum Grades
- B. Maximum Velocities
- C. Minimum Diameter

5.03 Materials

5.04 Full or Part Full Flow in Storm Drains

- A. General
- B. Pipe Flow Charts

5.05 Hydraulic Gradient and Profile of Storm Drain

5.06 Manhole Location

5.07 Pipe Connections

5.08 Minor Head Losses at Structures

5.09 Utilities

FLOW IN STORM DRAINS AND THEIR APPURTENANCES
SECTION 5

5.01 GENERAL

A general description of storm drainage systems and quantities of storm runoff is in Section 2 of this manual. It is the purpose of this section to consider the significance of the hydraulic elements of storm drains and their appurtenances to a storm drainage system.

Hydraulically, storm drainage systems are conduits (open or enclosed) in which unsteady and non-uniform free flow exists. Storm drains accordingly are designed for open-channel flow to satisfy as well as possible the requirements for unsteady and non-uniform flow. Steady flow conditions may or may not be uniform.

5.02 VELOCITIES AND GRADES

A. Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent excessive deposition of solid material, otherwise objectionable clogging may result. The controlling velocity is near the bottom of the conduit and considerably less than the mean velocity. Storm drains shall be designed to have a minimum mean velocity flowing full of 2.5 fps. Table 5-1 indicates the grades for both concrete pipe (n = 0.012) and for corrugated metal pipe (n = 0.024) to produce a velocity

of 2.5 fps, which is considered to be the lower limit of scouring velocity. The minimum slope for standard construction procedures shall be 0.30% when possible. Any variance must be approved by the Director of Public Utilities.

B. Maximum Velocities

Maximum velocities in conduits are important mainly because of the possibilities of excessive erosion on the storm drain inverts. Table 5-2 shows the limits of maximum velocity.

C. Minimum Diameter

Pipes which are to become an integral part of the public storm sewer system shall have a minimum diameter of 15 inches.

TABLE 5-1

MINIMUM SLOPE REQUIRED
TO PRODUCE SCOURING VELOCITY

Pipe Size (Inches)	Concrete Pipe Slope ft./ft.	Corrugated Metal Pipe Slope ft./ft.
15	0.0020	0.0070
18	0.0015	0.0060
21	0.0013	0.0049
24	0.0011	0.0041
27	0.0009	0.0035
30	0.0008	0.0031
36	0.0006	0.0024
42	0.0005	0.0020

48	0.0004	0.0016
54	0.0003	0.0014
60	0.0003	0.0012
66	0.0003	0.0011
72	0.0002	0.0010
78	0.0002	0.0009
84	0.0002	0.0008
96	0.0002	0.0007

TABLE 5-2

MAXIMUM VELOCITY IN STORM DRAINS

Description	Maximum Permissible Velocity
Culverts (all types)	15 fps
Storm Drains (inlet laterals)	10 fps
Storm Drains (collectors)	15 fps
Storm Drains (mains)	12 fps

5.03 MATERIALS

In selecting a roughness coefficient for concrete pipe, between 0.011 and 0.015, consideration will be given to the average conditions during the useful life of the structure. An "n" value of 0.017 for concrete pipe shall be used primarily in analyzing old conduits where alignment is poor and joints have become rough. If, for example, concrete pipe is being designed at a location where it is considered suitable, and there is reason to

believe that the roughness would increase through erosion or corrosion of the interior surface, slight displacement of joints, or entrance of foreign materials, a roughness coefficient will be selected which, in the judgment of the designer, will represent the average condition. Any selection of "n" values below the minimum or above the maximum, either for monolithic concrete structures, concrete pipe or corrugated metal pipe, must have the written approval of the Department of Public Utilities.

The coefficient of roughness listed in Table 5-3 are for use in the nomographs contained herein, or for direct solution of Manning's Equation.

TABLE 5-3

ROUGHNESS COEFFICIENTS "n" FOR STORM DRAINS

Materials of Construction	Design Coefficient	Range of Manning Coefficient
Concrete Pipe	0.012	0.011-0.015
Corrugated-metal pipe		
Plain or Coated	0.024	0.022-0.026
Paved Invert	0.020	0.018-0.022

5.04 FULL OR PART FULL FLOW IN STORM DRAINS

A. General

All storm drains shall be designed by the application of the continuity equation and Mannings Equation either through the

appropriate charts and nomographs or by direct solutions of the equation as follows:

$$Q = AV, \text{ and}$$

$$Q = \frac{1.49}{n} AR^{2/3} S_f^{1/2}$$

Q = Pipe Flow (cfs)

A = Cross-sectional area of pipe (ft.^{2})

V = Velocity of flow (fps)

n = Coefficient of roughness of pipe

R = Hydraulic radius = A/W_p (ft.)

S_f = Friction slope in pipe (ft./ft.)

W_p = Wetted perimeter (ft.)

There are several general rules to be observed when designing storm sewer runs. When followed they will tend to alleviate or eliminate the common mistakes made in storm sewer design. These rules are as follows:

1. Select pipe size and slope so that the velocity of flow will increase progressively, or at least will not appreciably decreases, at inlets, bends, or other changes in geometry or configuration.

2. Do not discharge the contents of a larger pipe into a smaller one, even though the capacity of the smaller pipe may be greater due to steeper slope.

3. At changes in pipe size match the soffits of the two pipes at the same level rather than matching the

flow lines.

4. Conduits are to be checked at the time of their design with reference to critical slope. If the slope of the line is greater than critical slope, the unit will likely be operating under entrance control instead of the originally assumed normal flow. Conduit slope should be kept below critical slope if at all possible. This also removes the possibility of a hydraulic jump within the line.

B. Pipe Flow Charts

Figures 5-1 through 5-9 are nomographs for determining flow properties in circular pipe, elliptical pipe, and pipe-arches. The nomographs are based upon a value of "n" of 0.012 for concrete and 0.024 for corrugated metal. The charts are self-explanatory, and their use is demonstrated by the example in Figure 5-1.

For values of "n" other than 0.012, the value of Q should be modified by using the formula below:

$$Q_c = \frac{Q_n\ (0.012)}{n_c}, \text{ where}$$

Q_c = FLow based upon n_c

n_c = Value of "n" other than 0.012

Q_n = Flow from nomograph based on n = 0.012

This formula is used in two ways. If n_c = 0.015 and Q_c is unknown, use the known properties to find Q_n from the nomograph,

and then use the formula to convert Q_n to the required Q_c. If Q_c is one of known properties, you must use the formula to convert Q_c (based on n_c) to Q_n (based on n = 0.012) first, and then use Q_n and the other known properties to find the unknown value on the nomograph.

Example 1:

Given: Slope = 0.005, depth of flow (d) = 1.8' diameter D = 36", n = 0.018.

Find: Discharge (Q).

First determine d/D - 1.8'/3.0' = 0.6. Then enter Figure 5-1 to read Q_n = 34 cfs. Using the formula Q_c = 34 (0.012/0.018) = 22.7 cfs (Answer).

Example 2:

Given: Slope = 0.005; diameter D = 36", Q = 22.7 cfs, n = 0.018

Find: Velocity of flow (fps)

First convert Q_c to Q_n, so that nomograph can be used. Using the formula Q_n = 22.7 (0.018)/(0.012) = 34 cfs, enter Figure 5-1 to determine d/D = 0.6. Now enter Figure 5-3 to determine V = 7.5 fps (Answer).

5.05 HYDRAULIC GRADIENT AND PROFILE OF STORM DRAIN

In storm drain systems flowing full, all losses of energy

through resistance of flow in pipes, by changes of momentum or by interference with flow patterns at junctions, must be accounted for by the accumlative head losses along the system from its initial upstream inlet to its outlet. The purpose of accurate determinations of head losses at junctions is to include these values in a progressive calculation of the hydraulic gradient along the storm drain system. In this way, it is possible to determine the water surface elevation which will exist at each structure.

The Department of Public Utilities does not require the hydraulic grade line to be established for all storm drainage design. It is not necessary to compute the hydraulic grade line of a conduit run if the slope and the pipe sizes are chosen so that the slope is equal to or greater than friction slope, the inside top surfaces rather than the flow lines of successive pipes are lined up at changes in size, and the surface of the water at the point of discharge will not rise above the top of the outlet. In such cases the pipe will not operate under pressure and the slope of the water surface under capacity discharge will approximately parallel the slope of the invert of the pipe.

Lacking these conditions or when it is desired to check the system against a larger flood than that used in sizing the pipes, and if the tailwater is known, then the hydraulic grade line and energy shall be computed and plotted. The

friction head loss shall be determined by direct application of Manning's Equation or by appropriate nomographs in this section. Minor losses due to turbulence at structures shall be determined by the procedure of Paragraph 5.08. The hydraulic grade line shall in no case be closer to the surface of the ground or street than 2 feet unless otherwise approved by the Department of Public Utilities. If the storm sewer system is to be extended at some future date, present and future operation of the system must be considered.

5.06 MANHOLE LOCATION

Manholes shall be located at intervals not to exceed 600 feet for pipe 30 inches in diameter or smaller. Manholes shall preferably be located at street intersections, conduit junctions, changes of grade and changes of alignment.

Manholes for pipe greater than 30 inches in diameter shall be located at points where design indicates entrance into the conduit is desirable; however, in no case shall the distance between openings or entrances be greater than 1,200 feet.

5.07 PIPE CONNECTIONS

Prefabricated wye and tee connections are available up to and including 24" x 24". Connections larger than 24 inches will be made by field connections. This recommendation is based primarily on the fact that field connections are

more easily fitted to a given alignment than are precast connections. Regardless of the amount of care exercised by the Contractor in laying the pipe, gain in footage invariably throws precast connections slightly out of alignment. This error increases in magnitude as the size of pipe increases.

5.08 MINOR HEAD LOSSES AT STRUCTURES

The following total energy head losses at structures shall be determined for inlets, manholes, wye branches or bends in the design of closed conduits. See Figures 5-10 and 5-11 for details of each case. Minimum head loss used at any structure shall be 0.10 foot, unless otherwise approved.

The basic equation for most cases, where there is both upstream and downstream velocity, takes the form as set forth below with the various conditions of the coefficient K_j shown in Tables 5-4, 5-5, and 5-6.

$$h_j = K_j \left(\frac{v_2^2 - v_1^2}{2g} \right)$$

h_j = Junction or structure head loss in feet.

v_1 = Velocity in upstream pipe in fps.

v_2 = Velocity in downstream pipe in fps.

K_j = Junction or structure coefficient of loss.

In the case where the initial velocity is negligible the equation for head loss becomes:

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

Short radius bends may be used on 24" and larger pipes when flow must undergo a direction change at a junction or bend. Reductions in head loss at manholes may be realized in this way. A manhole shall always be located at the end of such short radius bends.

TABLE 5-4

JUNCTION OR STRUCTURE
COEFFICIENT OF LOSS

Case No.	Reference Figure	Description of Condition	Coefficient Kj
I	5-10	Inlet on Main Line	0.50
II	5-10	Inlet on Main Line with Branch Lateral	0.25
III	5-10	Manhole on Main Line with 45° Branch Lateral	0.50
IV	5-10	Manole on Main Line with 90° Branch Lateral	0.25
V	5-11	45 Wye Connection or cut-in	0.75
VI	5-11	Inlet or Manhole at Beginning of Line	1.25
VII	5-11	Conduit on Curves for 90° *	
		Curve radius = diameter	0.50
		Curve radius = (2 to 8) diameter	0.40
		Curve radius = (8 to 20) diameter	0.25
VIII	5-11	Bends where radius is equal to diameter	
		90° Bend	0.50
		60° Bend	0.48
		45° Bend	0.35

22-½° Bend	0.20
Manhole on line with 60° Lateral	0.35
Manhole on line with 22-½° Lateral	0.75

*Where bends other than 90° are used, the 90° bend coefficient can be used with the following percentage factor applied:

60° Bend - 85%; 45° Bend - 70%; 22-½° Bend - 40%

The values of the coefficient "Kj" for determining the loss of head due to obstructions in pipes are shown in Table 5-5 and the coefficient are used in the following equation to calculate the head loss at the obstruction"

$$hj = Kj \left(\frac{v_2^2}{2g} \right)$$

TABLE 5-5

HEAD LOSS COEFFICIENTS DUE TO OBSTRUCTIONS

$\frac{A^*}{A.}$	Kj	$\frac{A^*}{A.}$	Kj
1.05	0.10	3.0	15.0
1.1	0.21	4.0	27.3
1.2	0.50	5.0	42.0
1.4	1.15	6.0	57.0
1.6	2.40	7.0	72.5
1.8	4.00	8.0	88.0
2.0	5.55	9.0	104.0

2.2	7.05	10.00	121.0
2.5	9.70		

*$\frac{A}{A}$. = Ratio of area of pipe to area of opening at obstruction.

The values of the coefficient "Kj" for determining the loss of head due to sudden enlargements and sudden contractions in pipes are shown in Table 5-6 and the coefficients are used in the following equation to calculate the head loss at the change in section:

$$h_j = K_j \frac{V^2}{2g}$$ where v = velocity in smaller pipe

TABLE 5-6

HEAD LOSS COEFFICIENTS DUE TO SUDDEN ENLARGEMENTS AND CONTRACTIONS

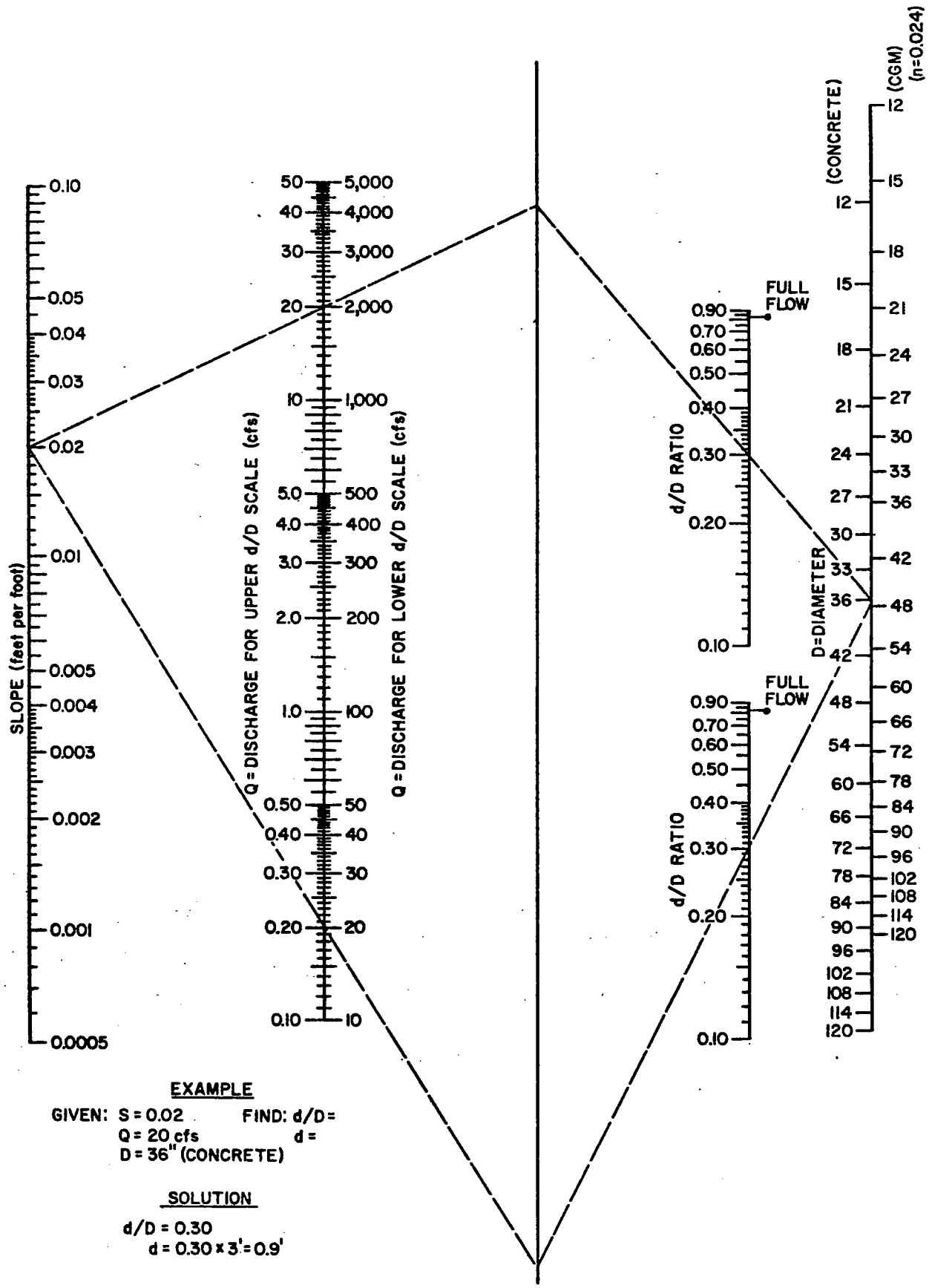
$\frac{D_2}{D1}$*	Sudden Enlargements K_j	Sudden Contractions K_j
1.2	0.10	0.08
1.4	0.23	0.18
1.6	0.35	0.25
1.8	0.44	0.33
2.0	0.52	0.36
2.5	0.65	0.40
3.0	0.72	0.42
4.0	0.80	0.44
5.0	0.84	0.45
10.0	0.89	0.46
	0.91	0.47

$**\frac{D_2}{D_1}$ = Ratio of larger to smaller diameter.

5.09 UTILITIES

In the design of a storm drainage system, the engineer is frequently confronted with the problem of grade conflict between the proposed storm drain and existing utilities such as water, gas and sanitary sewer lines.

When conflicts arise between a proposed drainage system and a utility system, the owner of the utility system shall be contacted and made aware of the conflict. Any adjustments necessary to either the drainage system or the utility can then be determined.



EXAMPLE

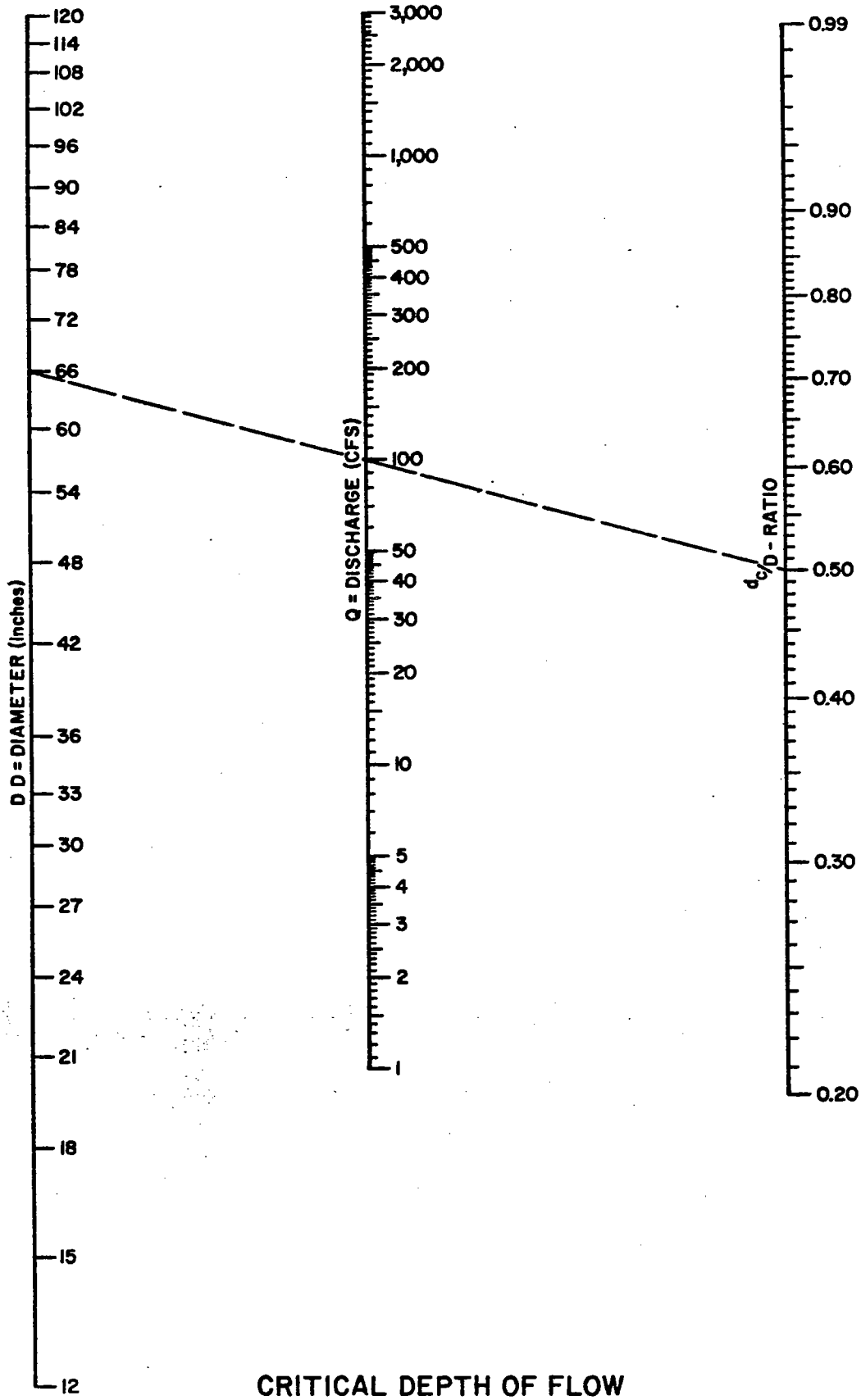
GIVEN: S = 0.02 FIND: d/D =
Q = 20 cfs d =
D = 36" (CONCRETE)

SOLUTION

d/D = 0.30
d = 0.30 x 3' = 0.9'

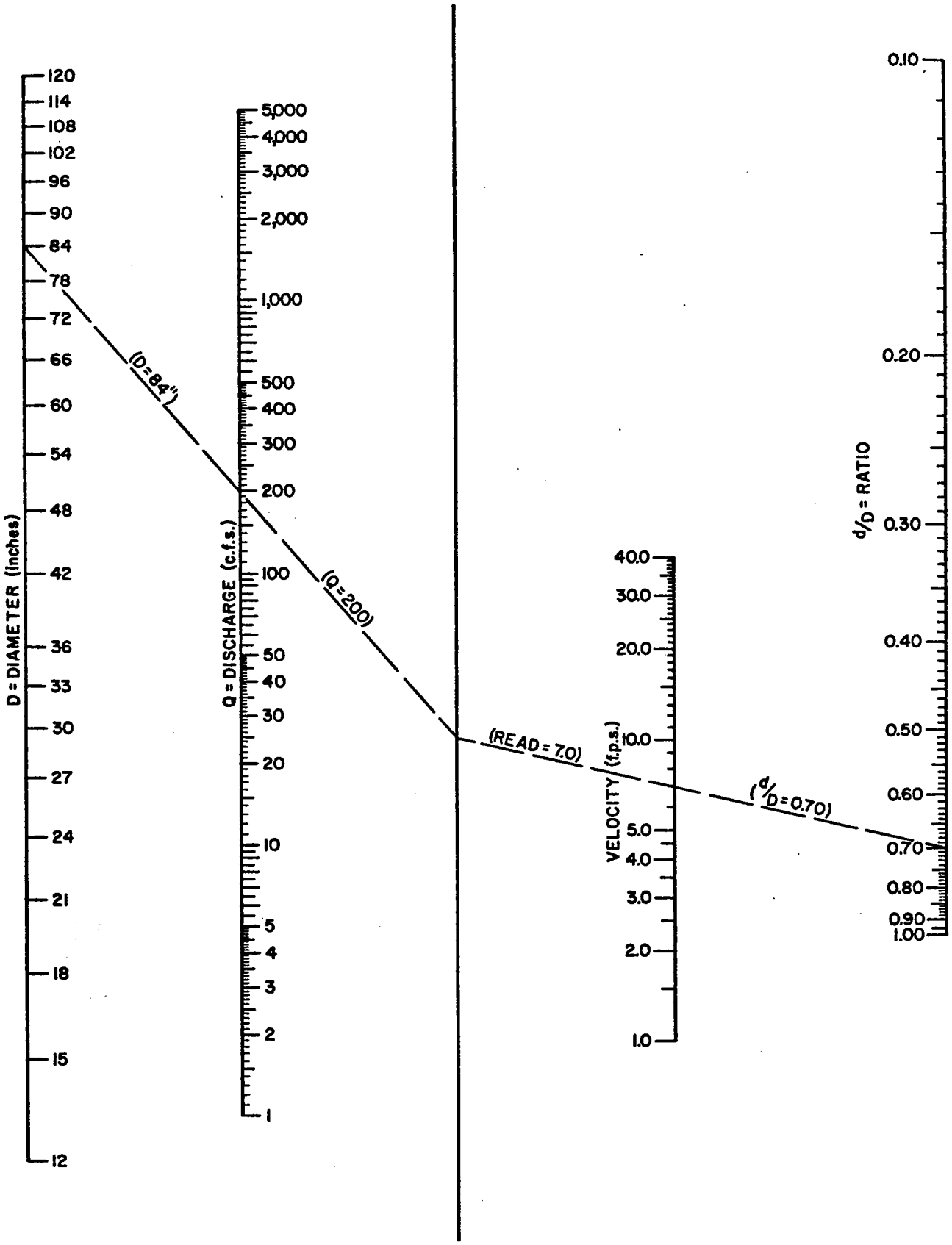
UNIFORM FLOW FOR
PIPE CULVERTS

Figure 5-1



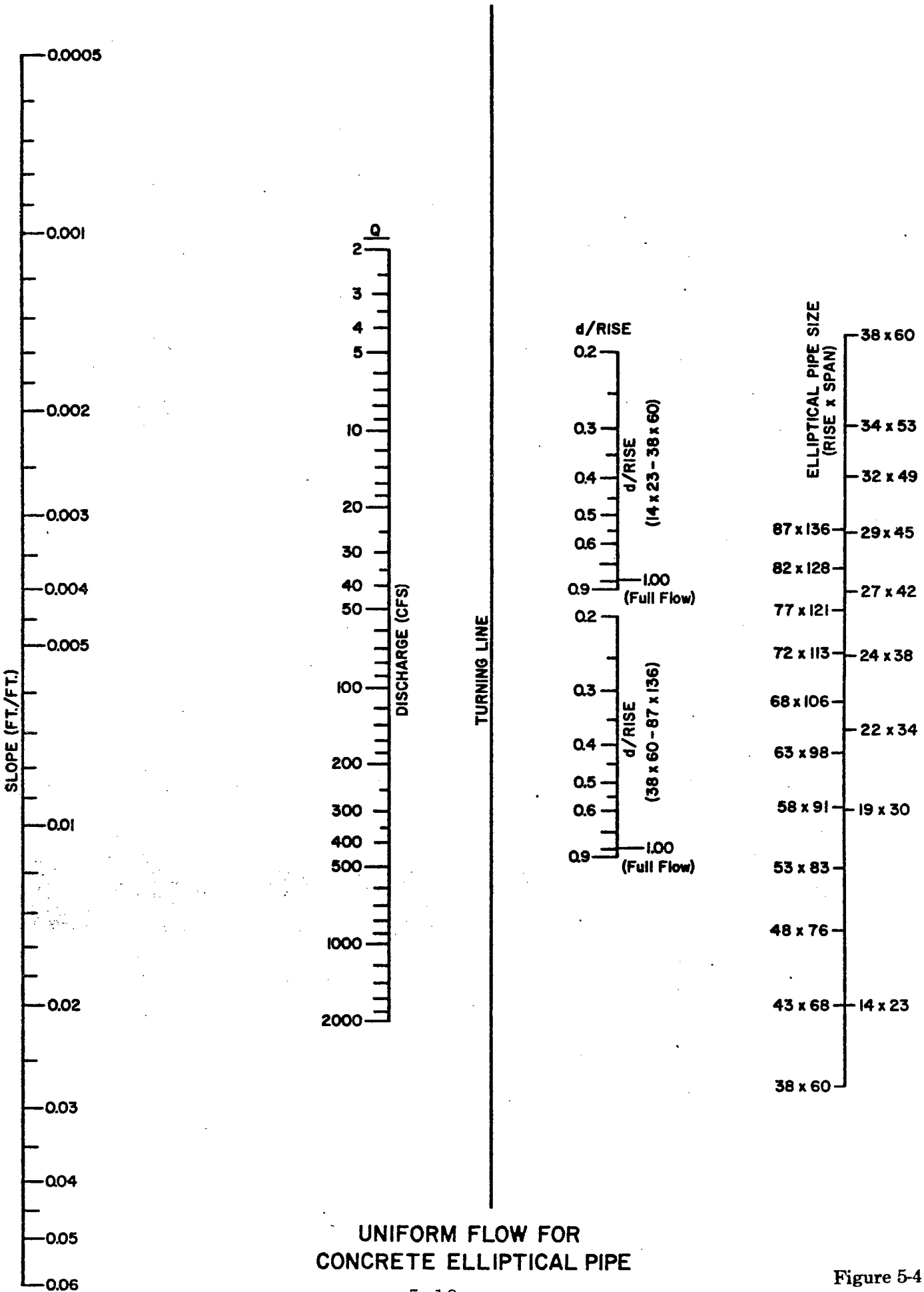
CRITICAL DEPTH OF FLOW
FOR CIRCULAR CONDUITS

Figure 5-2



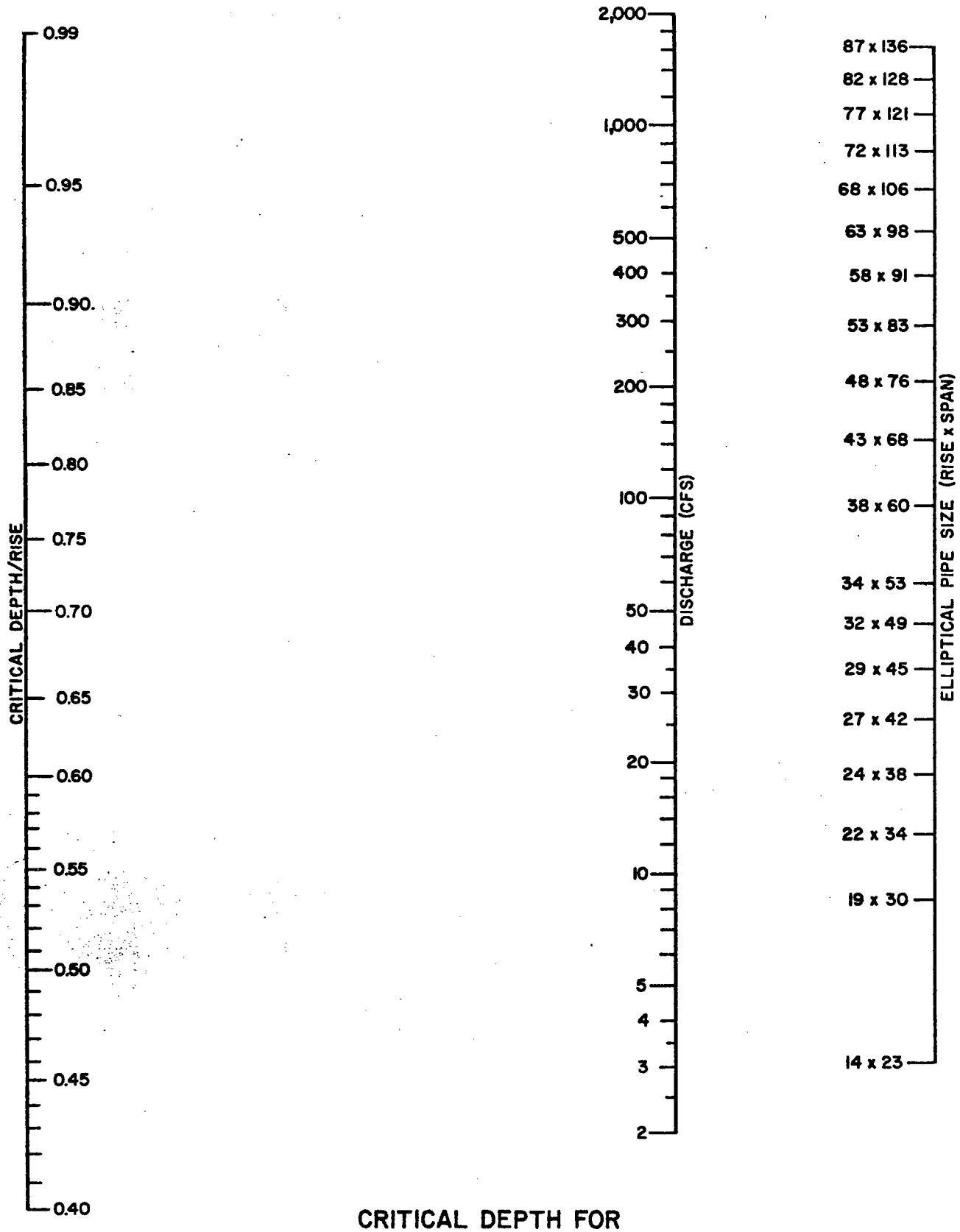
VELOCITY IN PIPE CONDUITS

Figure 5-3



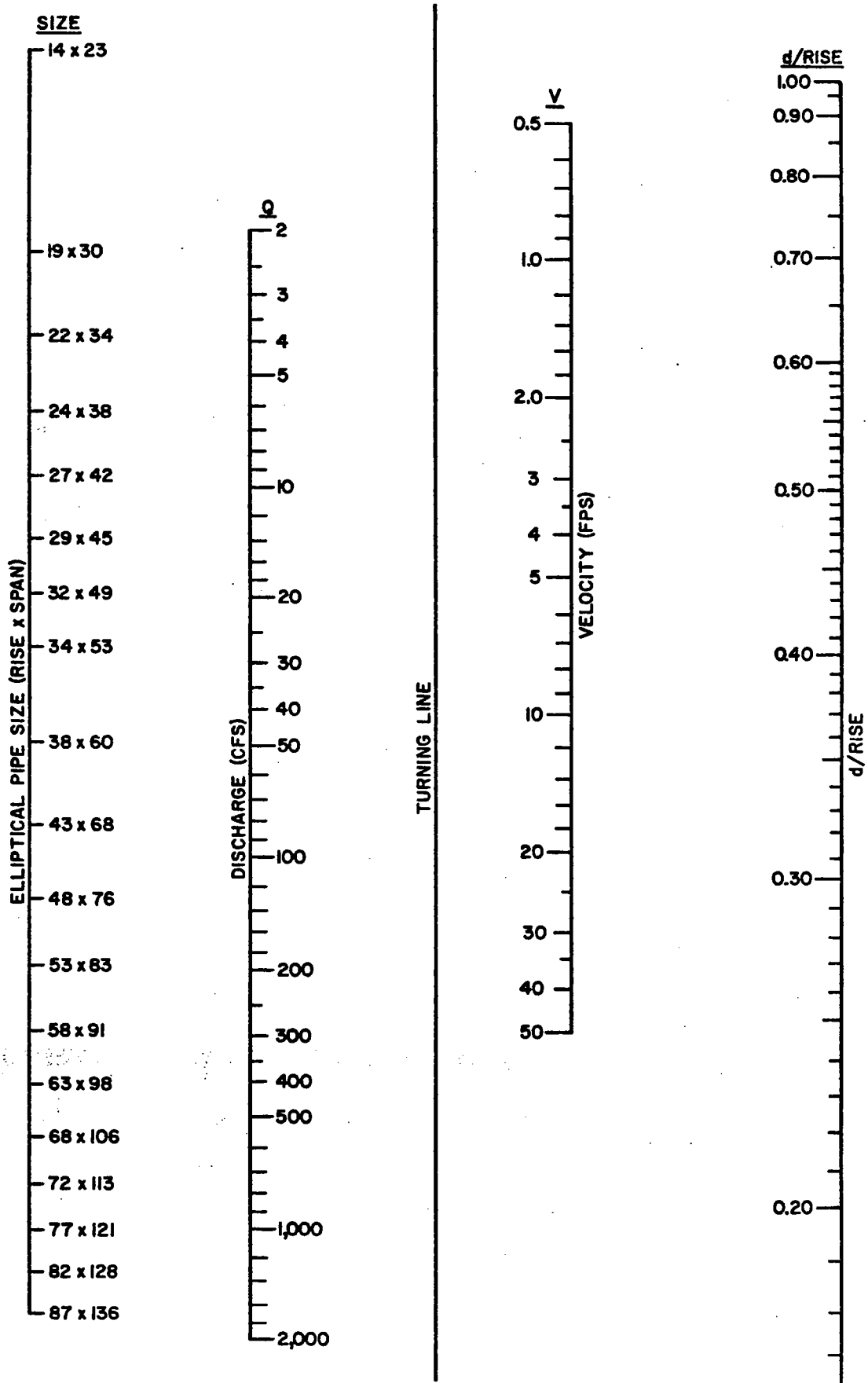
UNIFORM FLOW FOR
CONCRETE ELLIPTICAL PIPE

Figure 5-4



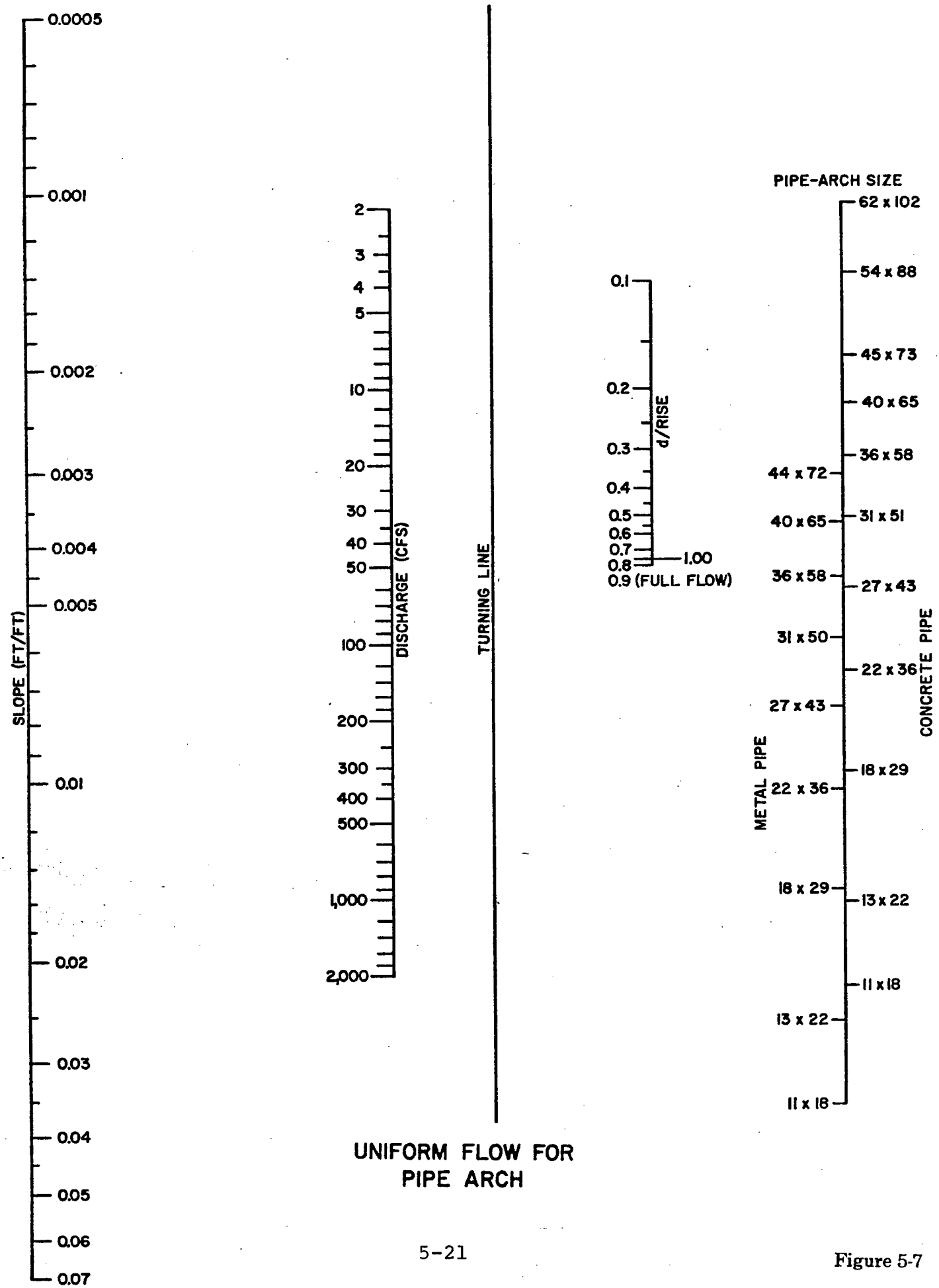
CRITICAL DEPTH FOR
ELLIPTICAL PIPE

Figure 5-5



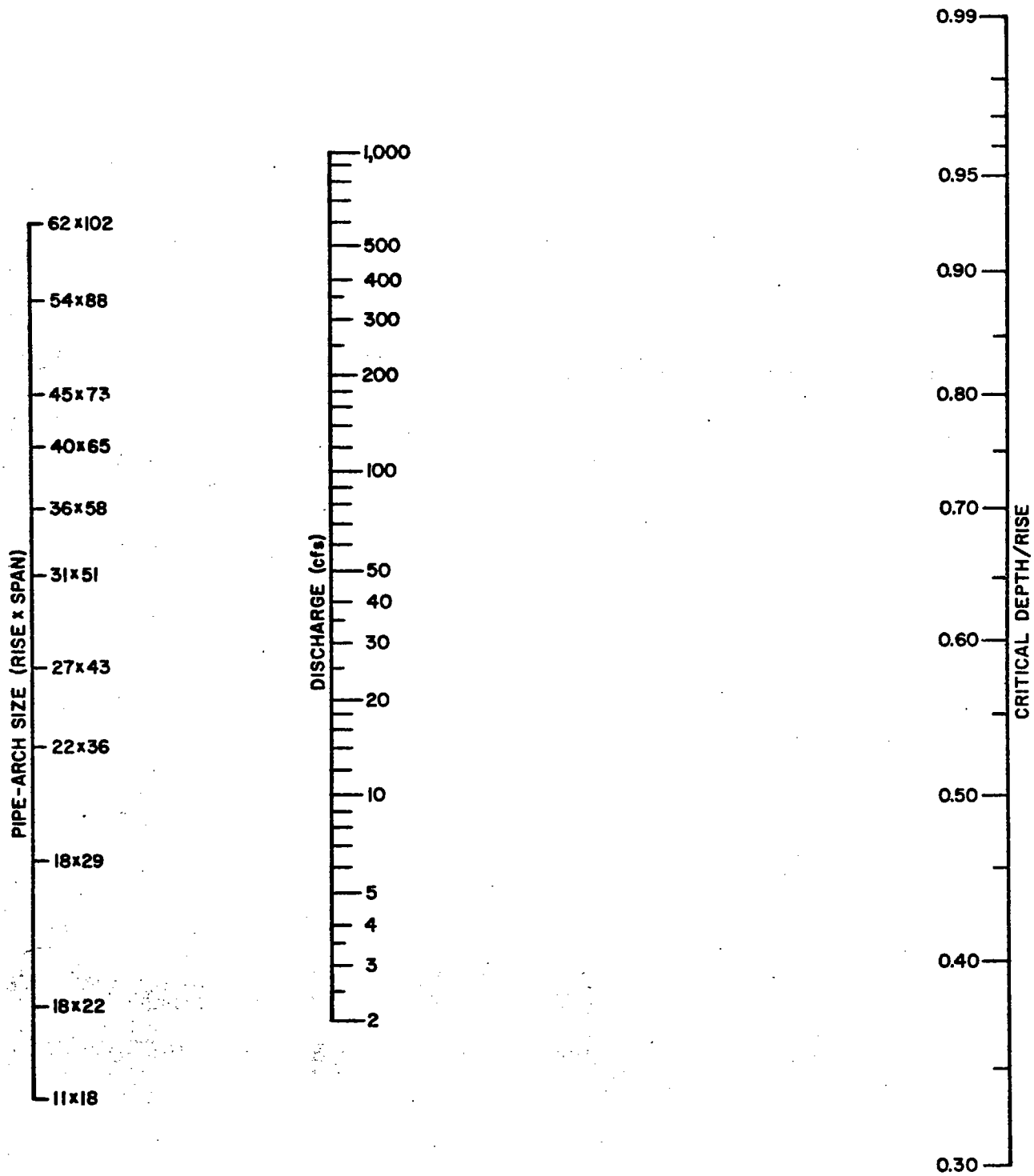
VELOCITY IN
ELLIPTICAL PIPE

Figure 5-6



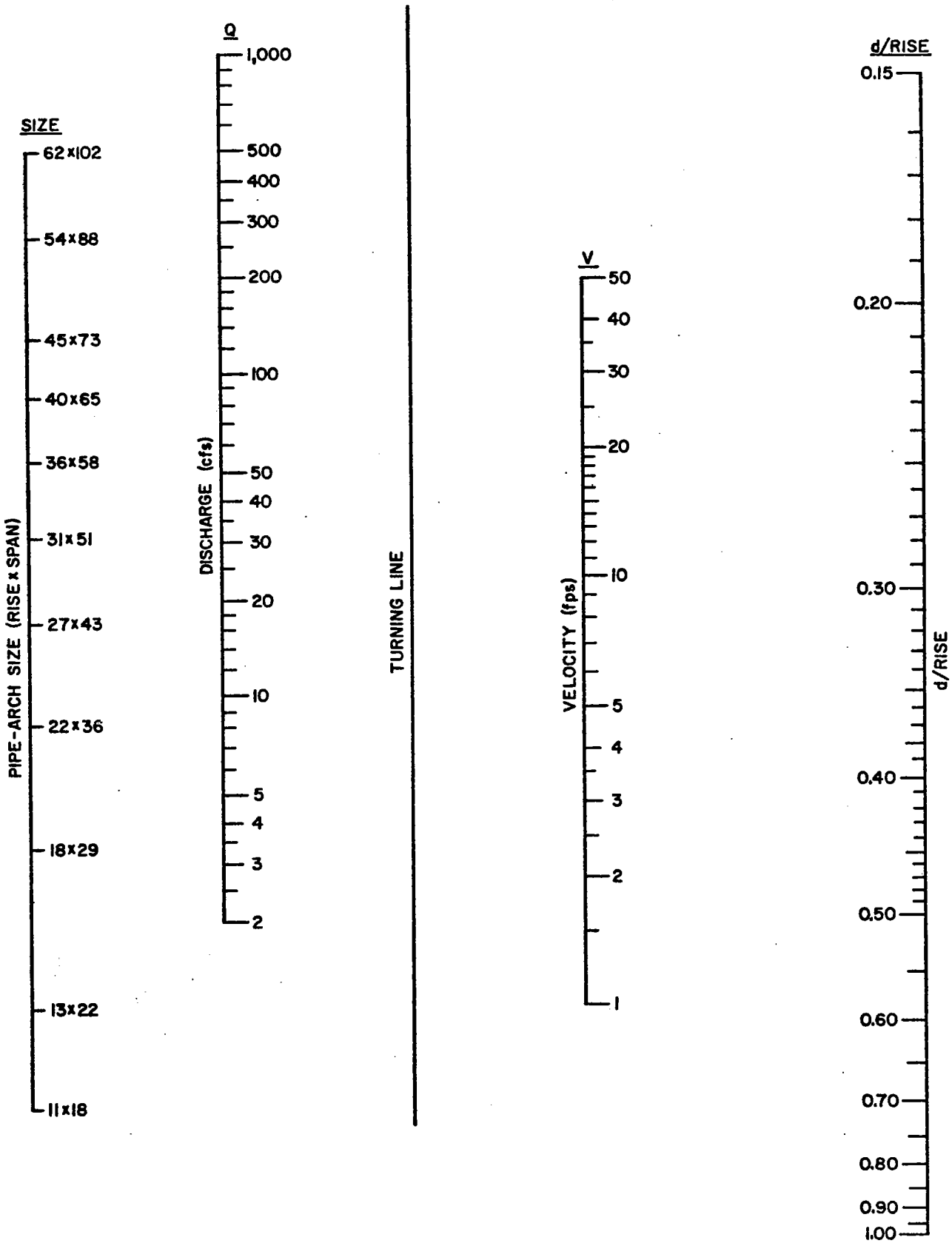
UNIFORM FLOW FOR PIPE ARCH

Figure 5-7



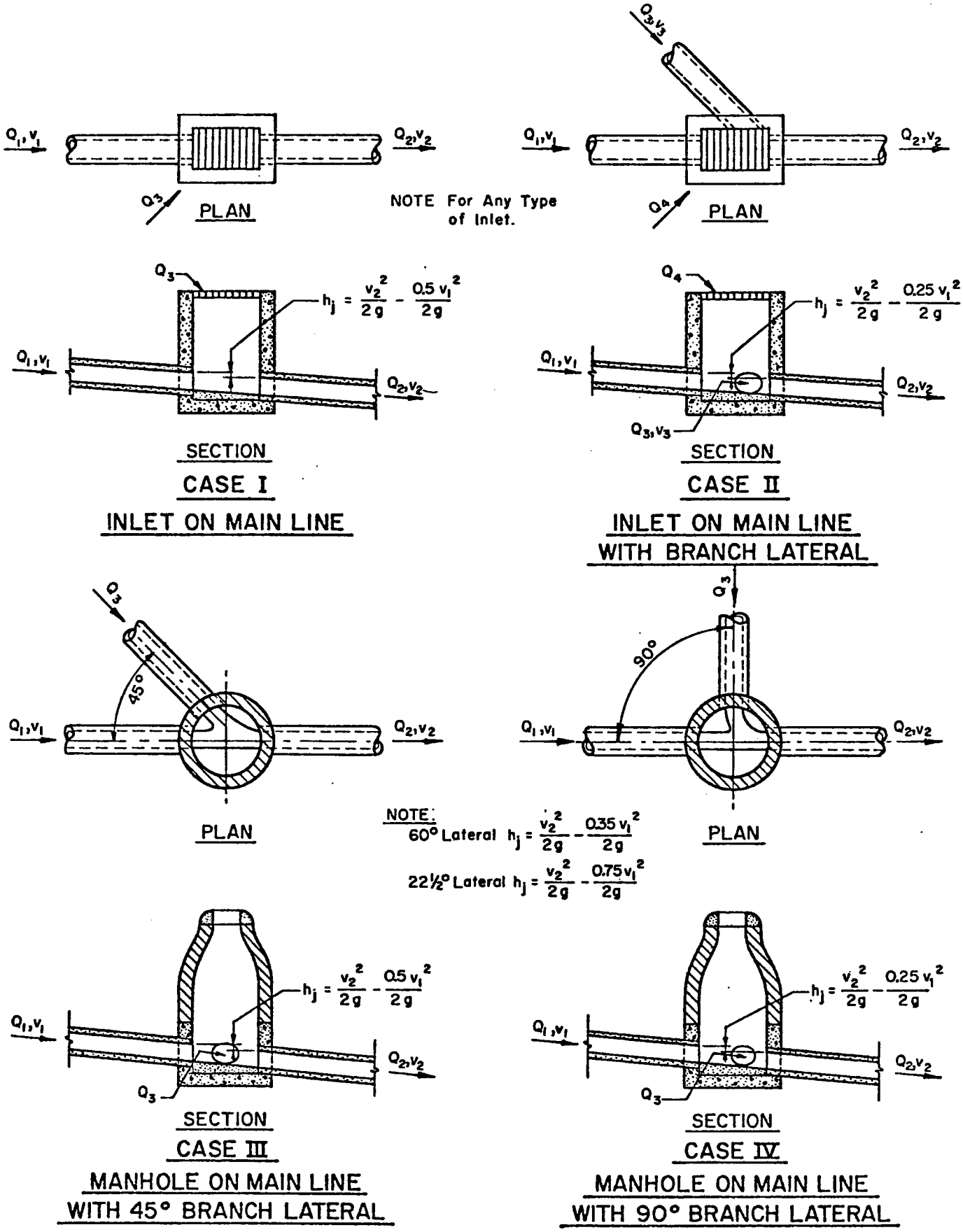
CRITICAL DEPTH OF FLOW
FOR PIPE-ARCH

Figure 5-8



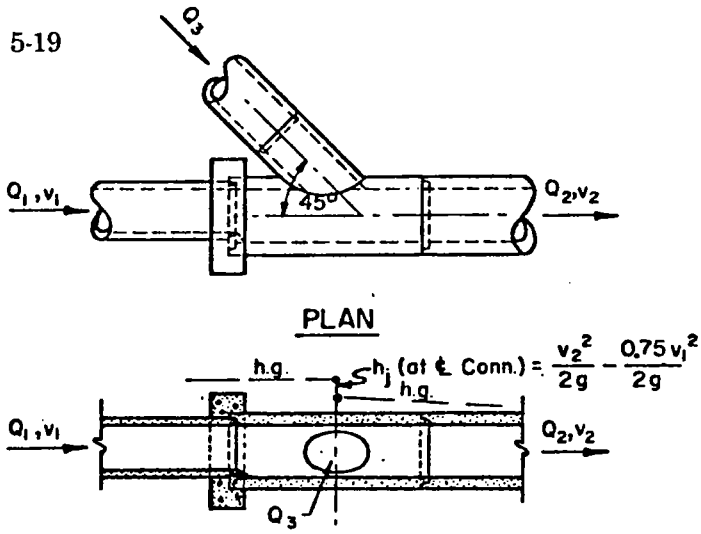
VELOCITY IN
PIPE-ARCH

Figure 5-9



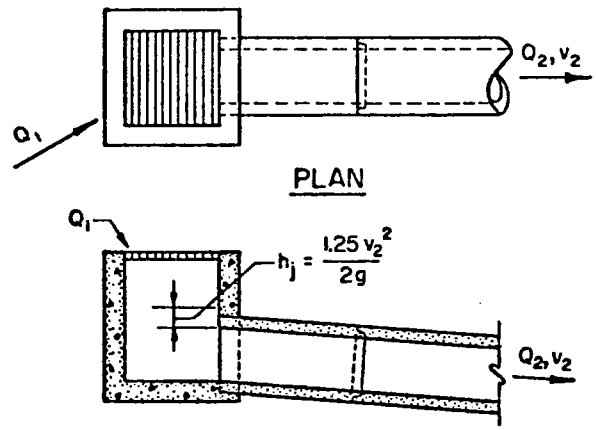
MINOR HEAD LOSSES DUE TO TURBULENCE AT STRUCTURES

Figure 5-10



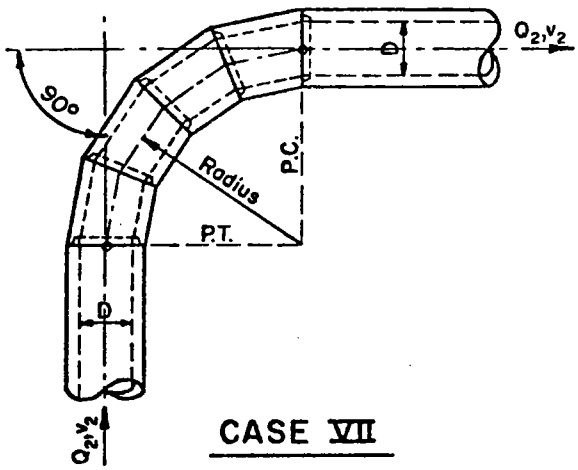
CASE V

45° WYE CONNECTION OR CUT IN



CASE VI

INLET OR MANHOLE AT BEGINNING OF LINE



CASE VII

CONDUIT ON 90° CURVES*

NOTE: Head loss applied at P.C. for length of curve.

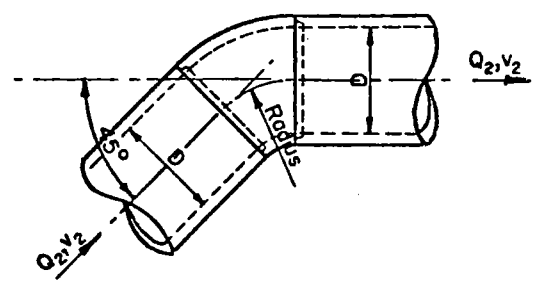
Radius = Dia. of Pipe $h_j = 0.50 \frac{v_2^2}{2g}$

Radius = (2-8) Dia. of Pipe $h_j = 0.25 \frac{v_2^2}{2g}$

Radius = (8-20) Dia. of Pipe $h_j = 0.40 \frac{v_2^2}{2g}$

Radius = Greater than 20 Dia. of Pipe $h_j = 0$

*When curves other than 90° are used, apply the following factors to 90° curves.
60° curve 85%
45° curve 70%
22½° curve 40%



CASE VIII

BENDS WHERE RADIUS IS EQUAL TO DIAMETER OF PIPE

NOTE: Head loss applied at begining of bend

90° Bend $h_j = 0.50 \frac{v_2^2}{2g}$

60° Bend $h_j = 0.43 \frac{v_2^2}{2g}$

45° Bend $h_j = 0.35 \frac{v_2^2}{2g}$

22½° Bend $h_j = 0.20 \frac{v_2^2}{2g}$

Figure 5-11

MINOR HEAD LOSSES DUE TO TURBULENCE AT STRUCTURES

DESIGN OF ENCLOSED STORM DRAINAGE SYSTEMS
SECTION 6

6.01 General

6.02 Preliminary Design Considerations

6.03 Inlet System

6.04 Storm Sewer System

A. Storm Sewer Pipes

B. Junctions, Inlets and Manholes

DESIGN OF ENCLOSED STORM DRAINAGE SYSTEMS
SECTION 6

6.01 GENERAL

All storm drains shall be designed by the application of the Manning Equation either directly or through appropriate charts or nomographs. In the preparation of hydraulic designs, a thorough investigation shall be made of all existing structures and their performance on the waterway in question.

The design of a storm drainage system should be governed by the following six conditions:

(1) The system must accommodate all surface runoff resulting from the selected design storm without serious damage to physical facilities or substantial interruption of normal traffic.

(2) Runoff resulting from storms exceeding the design storm must be anticipated and disposed of with minimum damage to physical facilities and minimum interruption of normal traffic.

(3) The storm drainage system must have a maximum reliability of operation.

(4) The construction costs of the system must be reasonable with relationship to the importance of the facilities it protects.

(5) The storm drainage system must require minimum maintenance and must be accessible for maintenance operations.

(6) The storm drainage system must be adaptable to future expansion with minimum additional cost.

An example of the design of a storm drainage system is outlined in Paragraphs 6.03 and 6.04. The design theory has been presented in the preceding sections with corresponding tables and graphs of information.

6.02 PRELIMINARY DESIGN CONSIDERATIONS

A. Prepare a drainage map of the entire area to be drained by proposed improvements. Contour maps serve as excellent drainage area maps when supplemented by field reconnaissance.

B. Make a tentative layout of the proposed storm drainage system, locating all inlets, manholes, mains, laterals, ditches, culverts, etc.

C. Outline the drainage area for each inlet in accordance with present and future street development.

D. Indicate on each drainage area the code identification number, the size of area, the direction of surface runoff by small arrows, and the coefficient of runoff for the area.

E. Show all existing underground utilities.

F. Establish design rainfall frequency.

G. Establish minimum inlet time of concentration.

H. Establish the typical cross section of each street.

I. Establish permissible spread of water on all streets within the drainage area.

J. Include A. through I. with plans submitted to the Department of Public Utilities for review. The drainage map submitted shall be suitable for permanent filing in the Engineering Department and shall be a good quality reproducible.

6.03 INLET SYSTEM

Determining the size and location of inlets is largely trial and error procedure. Using criteria outlined in Sections 2, 3 and 4 of this manual, the following steps will serve as a guide to the procedure to be used.

A. Beginning at the upstream end of the project drainage basin, outline a trial subarea and calculate the runoff from it.

B. Compare the calculated runoff to allowable street capacity. If the calculated runoff is greater than the allowable street capacity reduce the size of the trial subarea. If the calculated runoff is less than street capacity, increase the size of the trial subarea. Repeat this procedure until the calculated runoff equals the allowable street capacity. This is the first point at which a portion of the flow must be removed from the street. The percentage of flow to be removed will depend on street capacities versus runoff entering the street downstream.

C. Record the drainage area, time of concentration, runoff coefficient and calculated runoff for the subarea. This information shall be recorded on the plans or in tabular form convenient for review.

D. If an inlet is to be used to remove water from the street, size the inlet (inlets) and record the inlet size, amount of intercepted flow, and amount of flow carried over (bypassing the inlet).

E. Continue the above procedure for other subarea until a complete system of inlets has been established. Remember to account for carry-over from one inlet to the next.

F. After a complete system of inlets has been established, modification should be made to accommodate special situations

such as point sources of large quantities of runoff, and variation of street alignments and grades.

G. Record information as in C. and D. for all inlets.

H. After the inlets have been located and sized the inlet pipes can be designed.

I. Inlet pipes are sized to carry the volume of water intercepted by the inlet Inlet pipe capacities may be controlled by the gradient available, or by entry condition into the pipe at the inlet. Inlet pipe sizes should be determined for both conditions and the larger size thus determined used.

6.04 STORM SEWER SYSTEM

After the computation of the quantity of storm runoff entering each inlet, the storm sewer system required to carry off the runoff is designed. It should be borne in mind that the quantity of flow to be carried by any particular section of the storm sewer system is not the sum of the inlet design quantities of all inlets above that section of the system, but is less than the straight total. This situation is due to the fact that as the time of concentration increases the rainfall intensity decreases.

A. Storm Sewer Pipe

The ground line profile is now used in conjunction with the previous runoff calculations. The elevation of the hydraulic gradient is arbitrarily established approximately two feet (2') below the ground surface. When this tentative gradient is set and the design discharge is determined, a Manning flow chart may be used to determine the pipe size and velocity.

It is probable that the tentative gradient will have to be adjusted at this point since the intersection of the discharge and the slope on the chart will likely occur between standard pipe sizes. The smaller pipe should be used if the design discharge and corresponding slope does not result in an encroachment on the two foot (2') criteria below the ground surface. If there is encroachment, use the larger pipe which will establish a capacity somewhat in excess of the design discharge. Velocities can be read directly from a Manning Flow Chart based on a given discharge, pipe size and gradient slope.

B. Junctions, Inlets and Manholes

A. Determine the hydraulic gradient elevations at the upstream end and downstream end of the pipe section in question. The elevation of the hydraulic gradient of the upstream end of pipe is equal to the elevation of the downstream (hydraulic gradient) plus the product of the length of pipe and the pipe gradient.

B. Determine the velocity of flow for incoming pipe (main line) at junction, inlet or manhole at design point.

C. Determine the velocity of flow for outgoing pipe (main line) at junction, inlet or manhole at design point.

D. Compute velocity head for outgoing velocity (main line) at junction, inlet, or manhole at design point.

E. Compute velocity head for incoming velocity (main line) at junction, inlet, or manhole at design point.

F. Compute head loss at junction, inlet, or manhole.

G. Compute head loss at junction, inlet, or manhole.

$$h_j = K_j \left(\frac{v_2^{\,2} - v_1^{\,2}}{2g} \right)$$

H. Compute hydraulic gradient at upstream end of junction as if junction were not there.

I. Add head loss to hydraulic gradient elevation determined to obtain hydraulic gradient elevation at upstream end of junction.

All information shall be recorded on the plans or in tabular form convenient for review.

FLOW IN OPEN CHANNELS
SECTION 7

7.01 General

7.02 Channel Discharge

- A. Manning's Equation
- B. Uniform Flow
- C. Normal Depth

7.03 Water Surface Profiles

7.04 Design Considerations

7.05 Channel Cross Sections

- A. Side Slopes
- B. Depth
- C. Bottom Width
- D. Trickle Channels
- E. Freeboard

7.06 Channel Drops

7.07 Baffle Chutes

7.08 Structure Aesthetics

FLOW IN OPEN CHANNELS
SECTION 7

7.01 GENERAL

Open channels for use in the major drainage system have significant advantage in regard to cost, capacity, multiple use for recreational and aesthetic purposes, and potential for detention storage. Disadvantages include right-of-way needs and maintenance costs. Careful planning and design are needed to minimize the disadvantages, and to increase the benefits.

The ideal channel is a natural one carved by nature over a long period of time. The benefits of such a channel are that:

A. Velocities are usually low, resulting in longer concentration times and lower downstream peak flows.

B. Channel storage tends to decrease peak flows.

C. Maintenance needs are usually low because the channel is somewhat stabilized.

D. The channel provides a desirable green belt and recreational area adding significant social benefits.

Generally speaking, the natural channel or the man-made channel which most nearly conforms to the character of a natural channel is the most efficient and the most

desirable.

In many areas facing urbanization, the runoff has been so minimal that natural channels do not exist. However, small trickle paths nearly always exist which provide an excellent basis for location and construction of channels. Good land planning should reflect even these minimal trickle channels to reduce development costs and minimize drainage problems. In some cases the prudent utilization of natural water routes in the development of a major drainage system will reduce the requirements for an underground storm sewer system.

Channel stability is a well recognized problem in urban hydrology because of the significant increase in low flows and peak storm runoff flows. A natural channel must be studied to determine the measures needed to avoid future bottom scour and bank cutting. Erosion control measures can be taken at reasonable cost which will preserve the natural appearance without sacrificing hydraulic efficiency.

7.02 CHANNEL DISCHARGE

A. Manning's Equation

Careful attention must be given to the design of drainage channels to assure adequate capacity and minimum maintenance to overcome the results of erosion and silting. The hydraulic characteristics of channels shall be determined by Manning's equation.

$$Q = \frac{1.49}{n} A R^{2/3} S^{\frac{1}{2}}$$

Q = Total discharge in cfs

n = Coefficient of roughness

A = Cross-sectional area of channel in sq. ft.

R = Hydraulic radius of channel in feet.

S = Slope of the frictional gradient in feet per foot.

B. Uniform Flow

For a given channel condition of roughness, discharge, and slope, there is only one possible depth for maintaining a uniform flow. This depth is the normal depth. When roughness, depth, and slope are known at a channel section, there can only be one discharge for maintaining a uniform flow through the section. This discharge is the normal discharge.

If the channel is uniform and resistance and gravity forces are in exact balance, the water surface will be parallel to the bottom of the channel. This is the condition of uniform flow.

Uniform flow is more often a theoretical abstraction than an actuality. True uniform flow is difficult to find in the field or to obtain in the laboratory. Channels are sometimes designed on the assumption that they will carry uniform flow at the normal depths, but because of conditions difficult if not impossible to evaluate and hence not taken into account, the flow will actually have depths considerably different

from uniform depth. The engineer must be aware of the fact that uniform flow computation provides only an approximation of what will occur; however, such computations are useful and necessary for planning.

C. Normal Depth

The normal depth is computed so frequently that it is convenient to use nomographs for various types of cross sections to eliminate the need for trial and error solutions, which are time consuming. A nomograph for uniform flow is given in Figure 7-1.

7.03 WATER SURFACE PROFILES

Open channel flow in urban drainage systems is usually non-uniform because of bridge openings, curves and structures. This necessitates the use of backwater computations for all final channel design work.

A water surface profile must be computed for all channels and shown on all final drawings. Computation of the water surface profile should utilize standard backwater methods or acceptable computer routines, taking into consideration all losses due to changes in velocity, drops, bridge openings and other obstructions.

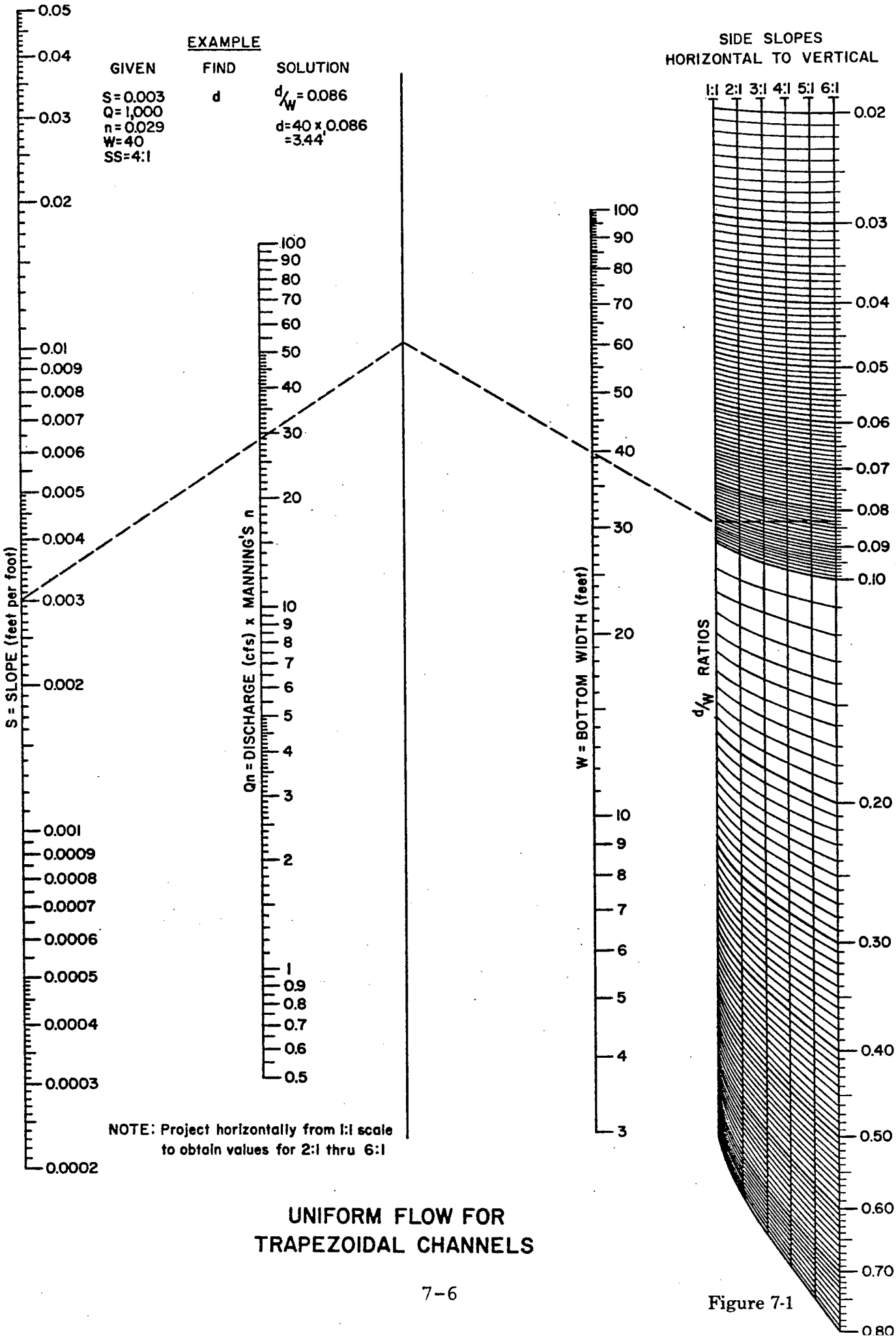
7.04 DESIGN CONSIDERATIONS

Channels should have trapezoidal sections of adequate cross-sectional areas to take care of uncertainties in

runoff estimates, changes in channel coefficients, channel obstructions and silt accumulations.

Accurate determination of the "n" value is critical in the analysis of the hydraulic characteristics of a channel. The "n" value for each channel reach should be based on experience and judgment with regard to the individual channel characteristics. Table 7-1 gives a method of determining the composite roughness coefficient based on actual channel conditions.

Where practicable, unlined channels should have sufficient gradient, depending upon the type of soil, to provide velocities that will be self-cleaning but will not be so great as to create erosion. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from the high velocities of large volumes of water. Unless approved otherwise by the Department of Public Utilities, channel velocities in man-made channels shall not exceed 6 fps.



UNIFORM FLOW FOR
TRAPEZOIDAL CHANNELS

Figure 7-1

TABLE 7-1

COMPUTATION OF COMPOSITE ROUGHNESS COEFFICIENT
FOR EXCAVATED AND NATURAL CHANNELS

$$^{*}n = (n_o + n_1 + n_2 + n_4)\ m$$

	Channel Conditions	Value
Material Involved n_0	Earth	0.020
	Rockcut	0.025
	Fine Gravel	0.024
	Coarse Gravel	0.028
Degree of Irregularity n_1	Smooth	0.000
	Minor	0.005
	Moderate	0.010
	Severe	0.020
Variation of Channel Cross Section n_2	Gradual	0.000
	Alternating Occasionally	0.005
	Alternating Frequently	0.010-0.015
Relative Effect Of Obstructions n_3	Negligible	0.000
	Minor	0.010-0.015
	Appreciable	0.020-0.030
	Severe	0.040-0.060
Vegetation n_4	Low	0.005-0.010
	Medium	0.010-0.025
	High	0.025-0.050
	Very High	0.050-0.100
Degree of Meandering m	Minor	1.000-1.200
	Appreciable	1.200-1.500
	Severe	1.500

Roughness Coefficient For Line Channels

Concrete Lined - n = 0.017
Rubble RipRap - n = 0.022

*Open Channel Hydraulics
Ven Te Chow, Ph.D.

7.05 CHANNEL CROSS SECTIONS

The channel shape may be almost any type suitable to the location and to the environmental conditions. Often the shape can be chosen to suit open space and recreational needs to create additional sociological benefits.

A. Side Slope

Except in horizontal curves the flatter the side slope, the better. Normally slopes shall be no steeper than 3:1, which is also the practical limit for mowing equipment. Rock or concrete lined channels or those which for other reasons do not require slope maintenance may have slopes as steep as 1:1/2:1.

B. Depth

Deep channels are difficult to maintain and can be hazardous. Constructed channels should therefore be as shallow as practical.

C. Bottom Width

Channels with narrow bottoms are difficult to maintain and are conductive to high velocities during high flows. It is desirable to design open channels such that the bottom width is at least twice the depth.

D. Trickle Channels

The low flows, and sometimes base flows, from urban areas

must be given specific attention. If erosion of the bottom of the channel appears to be a problem, low flows shall be carried in a paved trickle channel which has a capacity of 5.0 percent of the design peak flow. Care must be taken to insure that low flows enter the trickle channel without the attendant problem of the flow paralleling the trickle channel.

E. Freeboard

For channels with flow at high velocities, the surface roughness, wave action, air bulking, and splash and spray are quite erosive along the top of the flow. Freeboard height should be chosen to provide a suitable safety margin. The height of freeboard shall be a minimum of one foot, or provide an additional capacity of approximately one-third of the design flow. For deep flows with high velocities one may use the formula:

$$\text{Freeboard (in feet)} = 1.0 + 0.025 \; v\sqrt[3]{d}, \text{ where}$$

v = velocity of flow

d = depth of flow

For the freeboard of a channel on a sharp curve, extra height must be added to the outside bank or wall in the amount:

$$H = \frac{v^2 \quad (T + B)}{2gR}$$

H = additional height on outside edge of channel (ft.)

V = velocity of flow in channel (fps)

T = width of flow at water surface (ft.)

B = bottom width of channel (ft.)

R = centerline radius of turn (ft.)

g = acceleration of gravity (32.2 ft/sec.^{2})

If R is equal or greater than 3 x B, additional freedoard is not required.

7.06 CHANNEL DROPS

The use of channel drops permits adjustment of channel gradients which are too steep for the design conditions. In urban drainage work it is often desirable to use several low head drops in lieu of a few higher drops. Special attention must be given to protecting the channel from erosion in the area of channel drops.

The use of sloped drops will generally result in lower cost installations. Sloped drops can easily be designed to fit the channel topography.

Sloped drops shall have roughened faces and shall be no steeper than 2:1. They shall be adequately protected from scour, and shall not cause an upstream water surface drop which will result in high velocities upstream. Side cutting just downstream from the drop is a common problem which must be protected against.

The length L will depend upon the hydraulic characteristics of the channel and drop. For a q of 30 cfs/ft., L would be about 15 feet, that is, about ½ of the q value. The L should not be less than 10 feet, even for low q values.

In addition, followup riprapping will often be necessary at most drops to more fully protect the banks and channel bottom. The criteria given is minimal, based on the philosophy that it is less costly to initially under-protect with riprap, and to place additional protection later after erosional tendencies are determined in the field. Project approvals are to be based on provisions for such follow-up construction.

7.07 BAFFLE CHUTES

Baffle chutes are used to dissipate the energy in the flow at a larger drop. They require no tailwater to be effective. They are particularly useful where the water surface upstream is held at a higher elevation to provide head for filling a side storage pond during peak flows.

Baffle chutes are used in channels where water is to be lowered from one level to another. The baffle piers prevent undue acceleration of the flow as it passes down the chute. Since the flow velocities entering the downstream channel are low, no stilling basin is needed. The chute, on a 2:1 slope or flatter, may be designed to discharge up to 60 cfs per foot of width, and the drop may be as high as structurally feasible. The lower end of the chute is constructed to below stream bed level and backfilled as necessary. Degradation of the stream bed does not, therefore, adversely affect the performance of the structure. In urban drainage design, the lower end should be protected

from the scouring action.

The baffled apron shall be designed for the full design discharge. Baffle chutes shall be designed using acceptable methods such as those presented by A.S. Peterka of the United States Bureau of Reclamation in Engineering monograph No. 25.

7.08 STRUCTURE AESTHETICS

The use of hydraulic structures in the urban environment requires an approach not encountered elsewhere in the design of such structures. The appearance of these structures is very important. The treatment of the exterior should not be considered of minor importance. Apperance must be an integral part of design.

Parks. It must be remembered that structures are often the only above-ground indication of the underground works involved in an expensive project. Furthermore, parks and green belts may later be developed in the area in which the structure will play a dominant environmental role.

Play Areas. An additional consideration is that drainage structures offer excellent opportunities for neighborhood children to play. It is almost impossible to make drainage works inaccessible to children and therefore what is constructed should be made as safe as is reasonably possible. Safety hazards should be minimized and vertical drops protected with decorative fencing or rails.

Concrete Surface Treatment. The use of textured concrete presents a pleasing appearance and removes form marks. Exposed aggregate concrete is also attractive by may require special control of the aggregate used in the concrete.

Rails and Fences. The use of rails and fences along concrete walls provides a pleasing topping to an otherwise stark wall, and yet gives a degree of protection against someone inadvertently falling over the wall.

DESIGN OF CULVERTS
SECTION 8

8.01 General

8.02 Quantity Of Flow

8.03 Headwalls And Endwalls

A. General

B. Conditions at Entrance

C. Type of Headwall or Endwall

8.04 Culvert Discharge Velocities

8.05 Selection Of Culvert Size And Type

A. Culvert Types

B. Examples of Culvert Sizing Computations

DESIGN OF CULVERTS
SECTION 8

8.01 GENERAL

The function of a drainage culvert is to pass the design storm flow under a roadway or railroad without causing excessive backwater and without creating excessive downstream velocities. The designer shall keep energy losses and discharge velocities within reasonable limits when selecting a structure which will meet these requirements.

8.02 QUANTITY OF FLOW

The design storm flow shall be determined by the Rational Method or Hydrograph Method as set forth in Section 2 of this manual. The system shall accommodate a 10 year frequency storm including provision for limited overflows at bridges and culverts without exceeding minimum floor elevations of adjoining properties.

The designer shall consult the "Master Drainage Plan" before proceeding with the design of any storm drainage improvements. In the event the particular watershed or waterway is not covered by the "Master Drainage Plan", then the designer shall proceed with the design from the nearest downstream control, making allowances for future expansion of the drainage system.

8.03 HEADWALLS AND ENDWALLS

A. General

The normal functions of properly designed headwalls and endwalls are to anchor the culvert to prevent movement due to lateral pressures, to control erosion and scour resulting from excessive velocities and turbulence, and to prevent adjacent soil from sloughing into the waterway opening. All headwalls shall be constructed of reinforced concrete and may be either straight parallel headwalls, flared headwalls, or warped headwalls with or without aprons as may be required by site conditions.

B. Conditions at Entrance

It is important to recognize that the operation characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Design of culverts must involve consideration of energy losses that may occur at the entrance. The entrance head losses may be determined by the following equation.

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

h_e = Entrance head loss in feet.

V_2 = Velocity of flow in culvert.

V_1 = Velocity of approach in fps.

K_e = Entrance loss coefficient as shown in Table 8-1.

TABLE 8-1

VALUES OF ENTRANCE LOSS COEFFICIENTS "K_e"

Type of Structure and Entrance Design	Value of K_e
Box, Reinforced Concrete	
Submerged Entrance	
Parallel Wingwalls	0.5
Flared Wingwalls	0.4
Free Surface Flow	
Parallel Wingwalls	0.5
Flared Wingwalls	0.15
Pipe, Concrete	
Projecting from fill, socket end	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe	0.2
Square-edge	0.5
End-Section conforming to fill slope	0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no Headwall)	0.9
Headwall or headwall and wingwalls	
Square-edge	0.5
End-section conforming to fill slope	0.5

In order to compensate for the retarding effect on the velocity of approach in channels producted by the creation of the headwater pools at culvert entrances, the velocity of approach in the channel (v_a) shall be reduced by the factors as shown in Table 8-2.

TABLE 8-2

REDUCTION FACTORS FOR VELOCITY OF APPROACH

Velocity of Approach "Va" (fps)	Description of Conditions	V_1 to be used in formula for h_e
0-6	All culverts	$V_1 = V_a$
Above 6	Good alignment of the approach channel; head-water pool permissible within the right-of-way.	$V_1 = 0.5\ V_a$
Above 6	Good alignment of the approach channel; channel slopes have been lined; limited back-water pool permissible within the right-of-way.	$V_1 = 0$

C. Type of Headwall or Endwall

In general the following guidelines should be used in the selection of the type of headwall or endwalls.

Parallel Headwall and Endwall

1. Approach velocities are low (below 6 fps).
2. Backwater pools may be permitted.
3. Approach channel is undefined.
4. Ample right-of-way or easement is available.
5. Downstream channel protection is not required.

Flared Headwall and Endwall

1. Channel is well defined.
2. Approach velocities are between 6 and 10 fps.
3 Medium amounts of debris exist.

The wings of flared walls should be located with respect to the direction of the approaching flow instead of the culvert axis.

Warped Headwall and Endwall.

1. Channel is well defined and concrete lined.
2. Approach velocities are between 8 and 20 fps.
3. Medium amounts of debris exist.

These headwalls are effective with drop down aprons to accelerate flow through culvert, and are effective endwalls for transitioning flow from closed conduit flow to open channel flow. This type of headwall should be used only where the drainage structure is large and right-of-way or easement is limited.

8.04 CULVERT DISCHARGE VELOCITIES

The velocity of discharge from culverts should be limited as shown in Table 8-3. Consideration must be given to the effect of high velocities, eddies or other turbulence on the natural channel, downstream property and roadway embankment.

TABLE 8-3

CULVERT DISCHARGE-VELOCITY LIMITATIONS

Downstream Condition	Maximum Allowable Discharge Velocity (fps)
Earth	6 fps
Sod Earth	8 fps
Paved or Riprap Apron	15 fps
Shale	10 fps
Rock	15 fps

8.05 SELECTION OF CULVERT SIZE AND TYPE

A. Culvert Types

Culverts shall be selected based on hydraulic principles, economy of size and shape, and with a resulting headwater depth which will not cause damage to adjacent property. It is essential to the proper design of a culvert that the conditions under which the culvert will operate are known. Five types of operating conditions are listed below with a discussion of each following.

Type I Flowing Part Full with Outlet Control and Tailwater Depth Below Critical Depth. (Figure 8-1)

Type II Flowing Part Full with Outlet Control and Tailwater Depth Above Critical Depth. (Figure 8-2)

Type III FLowing Part Full with Inlet Control.

(Figure 8-3)

Type IVA Flowing Full with Submerged Outlet.

(Figure 8-4)

Type IVB Flowing Full with Partially Submerged

Outlet. (Figure 8-5)

TYPE I

CULVERT FLOWING PART FULL
WITH OUTLET CONTROL AND TAILWATER DEPTH
BELOW CRITICAL DEPTH

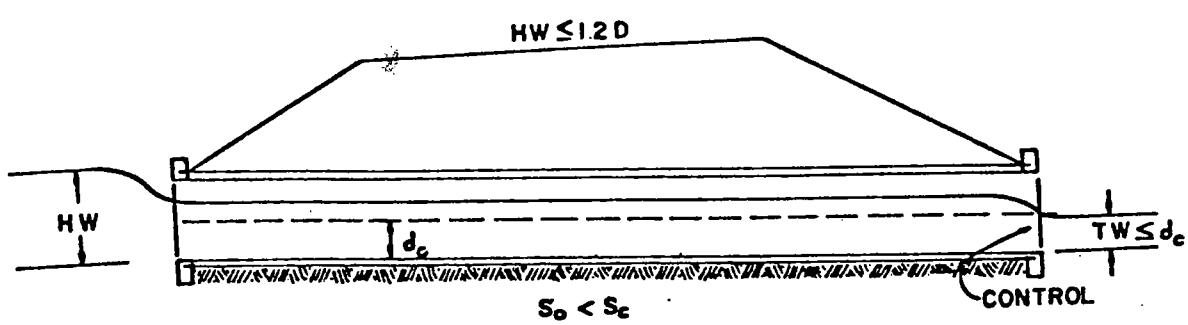


Figure 8-1

Conditions

The entrance is unsubmerged (HW $\leq$ 1.2D), the slope at design discharge is sub-critical ($S_o < S_c$), and the tailwater is below critical depth (TW $\leq d_c$).

The above condition is a common occurrence where the natural channels are on flat grades and have wide, flat flood plains. The control is critical depth at the outlet.

In culvert design, it is generally considered that the headwater pool maintains a constant level during the design storm. If this level does not submerge the culvert inlet, the culvert flows part full.

If critical flow occurs at the outlet the culvert is said to have "Outlet Control." A culvert flowing part full with outlet control will require a depth of flow in the barrel of the culvert greater than critical depth while passing through critical depth at the outlet.

The capacity of a culvert flowing part full with outlet control and tailwater depth below critical depth shall be governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_oL$$

HW = Headwater depth above the invert of the upstream end of the culvert in feet. Headwater must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

d_c = Critical depth of flow in feet = $\sqrt[3]{\frac{q^2}{32.2}}$

D = Diameter of pipe or height of box.

q = Discharge in cfs per foot.

V_c = Critical velocity in feet per second occurring at critical depth.

h_e = Entrance head loss in feet.

$$h_e = K_e \left(\frac{v_c^2}{2g} \right)$$

K_e = Entrance loss coefficient (See Table 8-1)

h_f = Friction head loss in feet = $S_f L$.

S_f = Friction slope or slope that will produce uniform flow. For Type I operation the friction slope is based upon 1.1 d_c (See Figures 8-7 and 8-11)

S_o = Slope of culvert in feet per foot.

L = Length of culvert in feet.

TYPE II

CULVERT FLOWING PART FULL
WITH OUTLET CONTROL AND TAILWATER DEPTH
ABOVE CRITICAL DEPTH

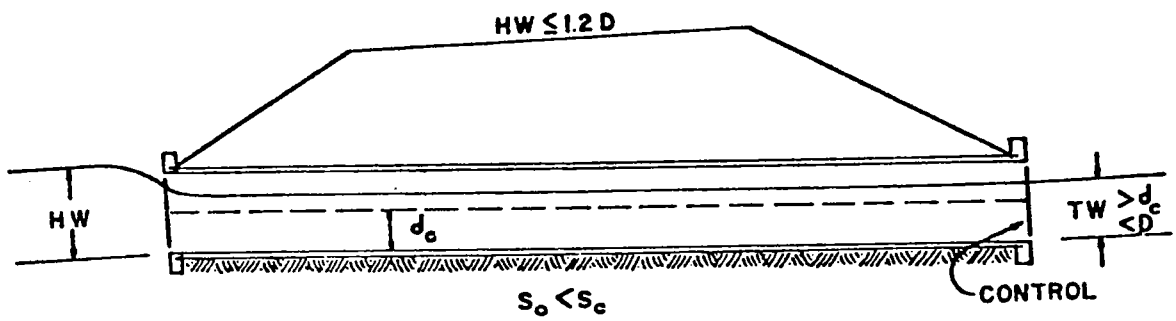


Figure 8-2

Conditions

The entrance is submerged (HW ≤ 1.2D), the slope at design discharge is sub-critical ($S_o < S_c$), and the tailwater is above critical depth (TW > d_c).

The above condition is a common occurrence where the channel is deep, narrow and well defined.

If the headwater pool elevation does not submerge the culvert inlet, the slope at design discharge is subcritical, and the tailwater depth is above critical depth the control is said to occur at the outlet; and the capacity of the culvert shall be

governed by the following equation when the approach velocity is considered zero.

$$HW = TW + \frac{V_{TW}^{2}}{2g} + h_e + h_f - S_oL$$

HW = Headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

TW = Tailwater depth above the invert of the downstream end of the culvert in feet.

V_{TM} = Culvert discharge velocity in feet per second at tailwater depth.

h_e = Entrance head loss in feet.

$$h_e = K_e \left(\frac{V_{TM}^{2}}{2g}\right)$$

E_e = Entrance loss coefficient (See Table 8-1).

h_f = Friction head loss in feet = S_fL

S_f = Friction slope or slope that will produce uniform flow. For Type II operation the friction slope is based upon TW depth.

S_o = Slope culvert in feet per foot.

L = Length of culvert in feet.

TYPE III

CULVERT FLOWING PART FULL WITH INLET CONTROL

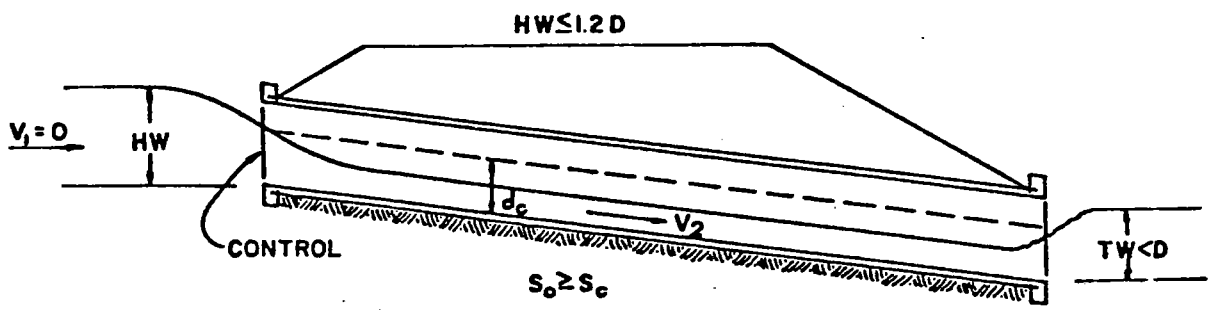


Figure 8-3

Conditions

The entrance is unsubmerged ($HW \leq 1.2D$) and the slope at design discharge is equal to or greater than critical (Supercritical) ($S_o \geq S_c$).

This condition is a common occurrence for culverts in rolling or mountainous country where the flow does not submerge the entrance. The control is critical depth at the entrance.

If critical flow occurs near the inlet, the culvert is said to have "Inlet Control". The maximum discharge through a culvert flowing part full occurs when flow is at critical depth for a given energy head. To assure that flow passes through critical depth near the inlet, the culvert must be laid on a slope equal to or greater than critical slope for the design

discharge. Placing culverts which are to flow part full on slopes greater than critical slope will increase the outlet velocities but will not increase the discharge. The discharge is limited by the section near the inlet at which critical flow occurs.

The capacity of a culvert flowing part full with control at the inlet shall be governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + \frac{V_2^2}{2g} + K_e \frac{V_2^2}{2g}$$

HW = Headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D or entrance is submerged and Type IV operation will result.

d_c = Critical depth of flow in feet = $\sqrt[3]{\frac{q^2}{32.2}}$

q = Discharge in cfs per foot.

V_2 = Velocity of flow in the culvert in feet per second. The velocity of flow varies from critical velocity at the entrance to uniform velocity at the outlet provided the culvert is sufficiently long. Therefore, the outlet velocity is the discharge divided by the area of flow in the culvert.

K_e = Entrance loss coefficient (See Table 8-1)

TYPE IV-A

CULVERT FLOWING FULL WITH SUBMERGED OUTLET

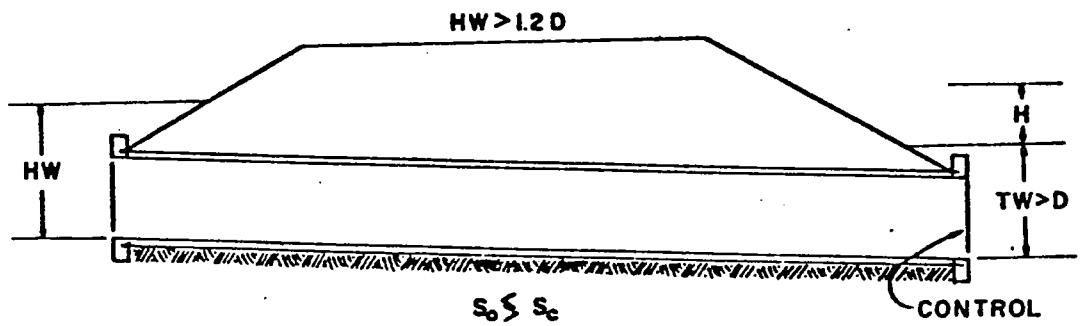


Figure 8-4

Conditions

The entrance is submerged (HW >1.2D). The tailwater completely submerges the outlet.

Most culverts flow with free outlet, but depending on topography, a tailwater pool of a depth sufficient to submerge the outlet may form at some installation. Generally, these will be considered at the outlet. For an outlet to be submerged, the depth at the outlet must be equal to or greater than the diameter of pipe of height of box. The capacity of a culvert flowing full with a submerged outlet shall be governed by the following equation when the approach velocity is considered zero. Outlet Velocity is based on full flow at the outlet.

$HW = H + TW - S_oL$

HW = Headwater depth above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.

H = Head for culvert flowing full.

TW = Tailwater depth in feet.

S_o = Slope of culvert in feet per foot.

L = Length of culvert in feet.

TYPE IV-B

CULVERT FLOWING FULL
WITH PARTIALLY SUBMERGED OUTLET

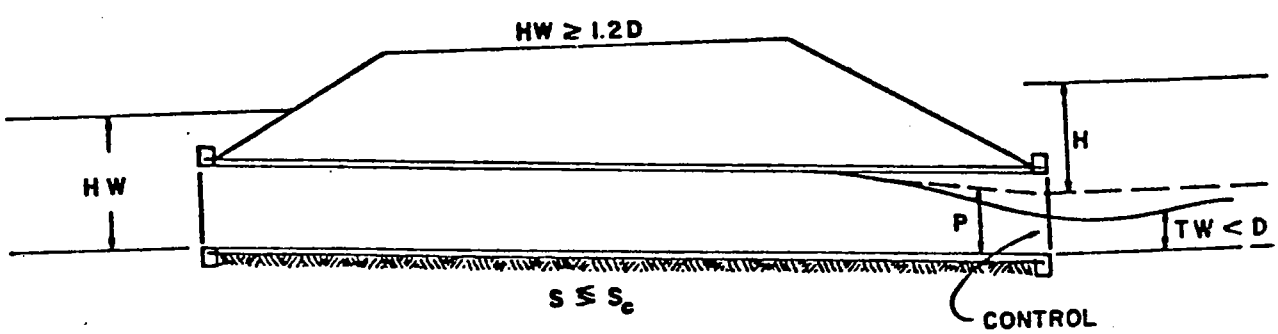


Figure 8-5

Conditions
(Partially Submerged Outlet)

The entrance is submerged (HW > 1.2D). The tailwater depth is less than D (TW < D).

The capacity of a culvert flowing full with a partially submerged outlet shall be governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on critical depth if TW depth is less than critical depth. If TW depth is greater than critical depth, outlet velocity is based on TW depth.

$$HW = H + P - S_oL$$

HW = Headwater Depth above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.

H = Head for culverts flowing full.

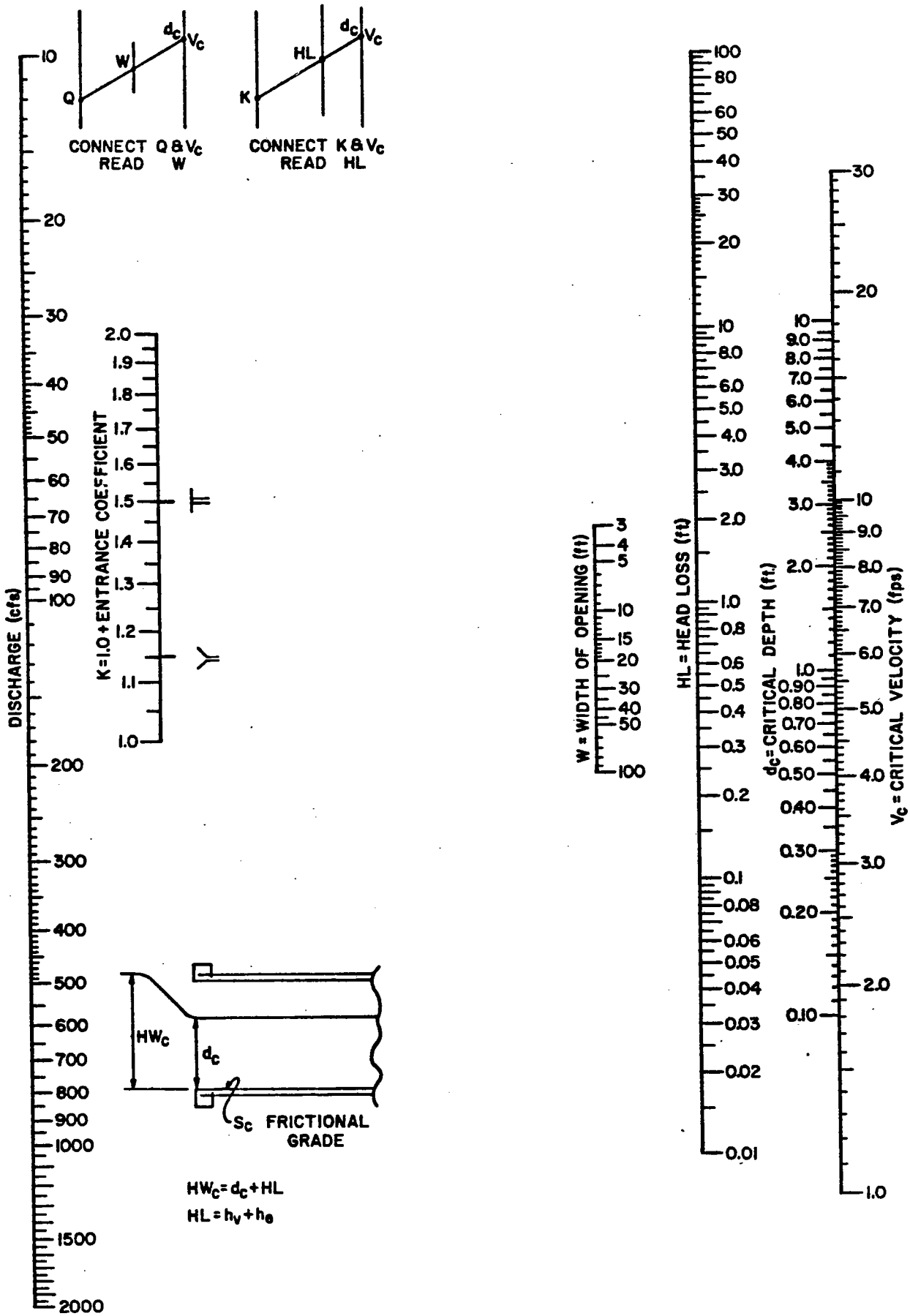
P = Pressure line height = $\frac{d_c + D}{2}$

d_c = Critical depth in feet.

D = Diameter or height of structure in feet.

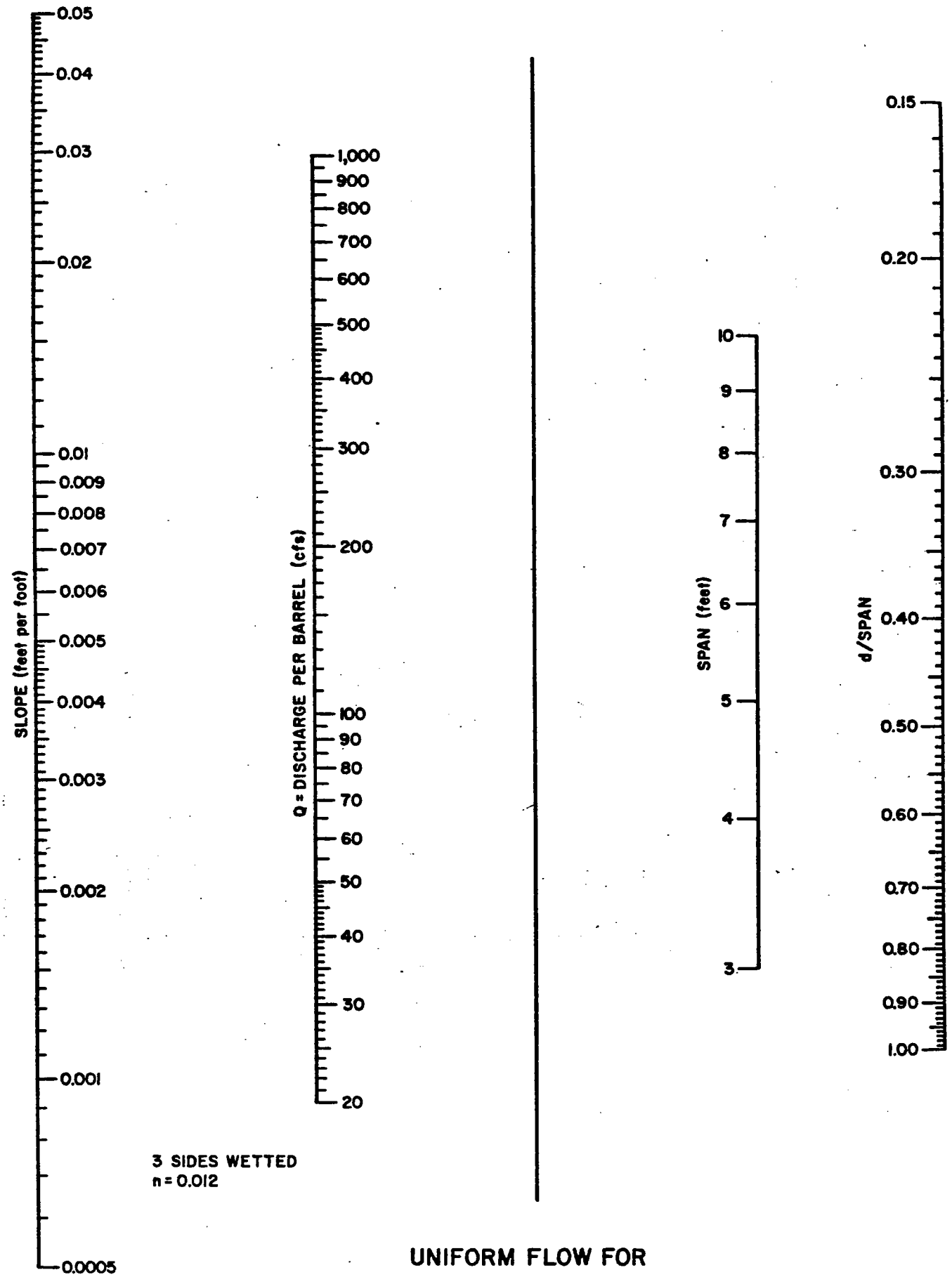
S_o = Slope of culvert in feet per foot.

L = Length of culvert in feet.



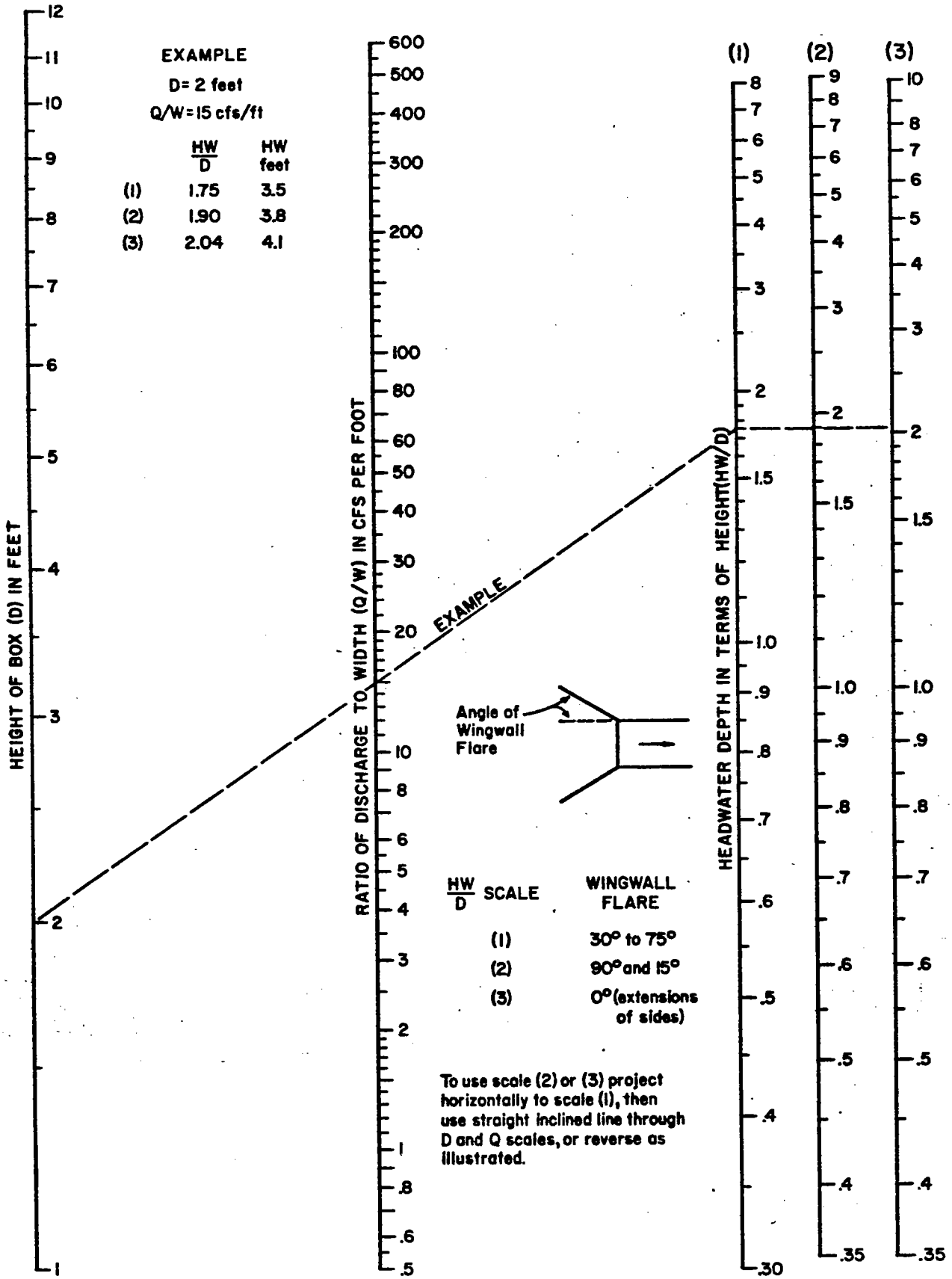
CRITICAL FLOW FOR
BOX CULVERTS

Figure 8-6



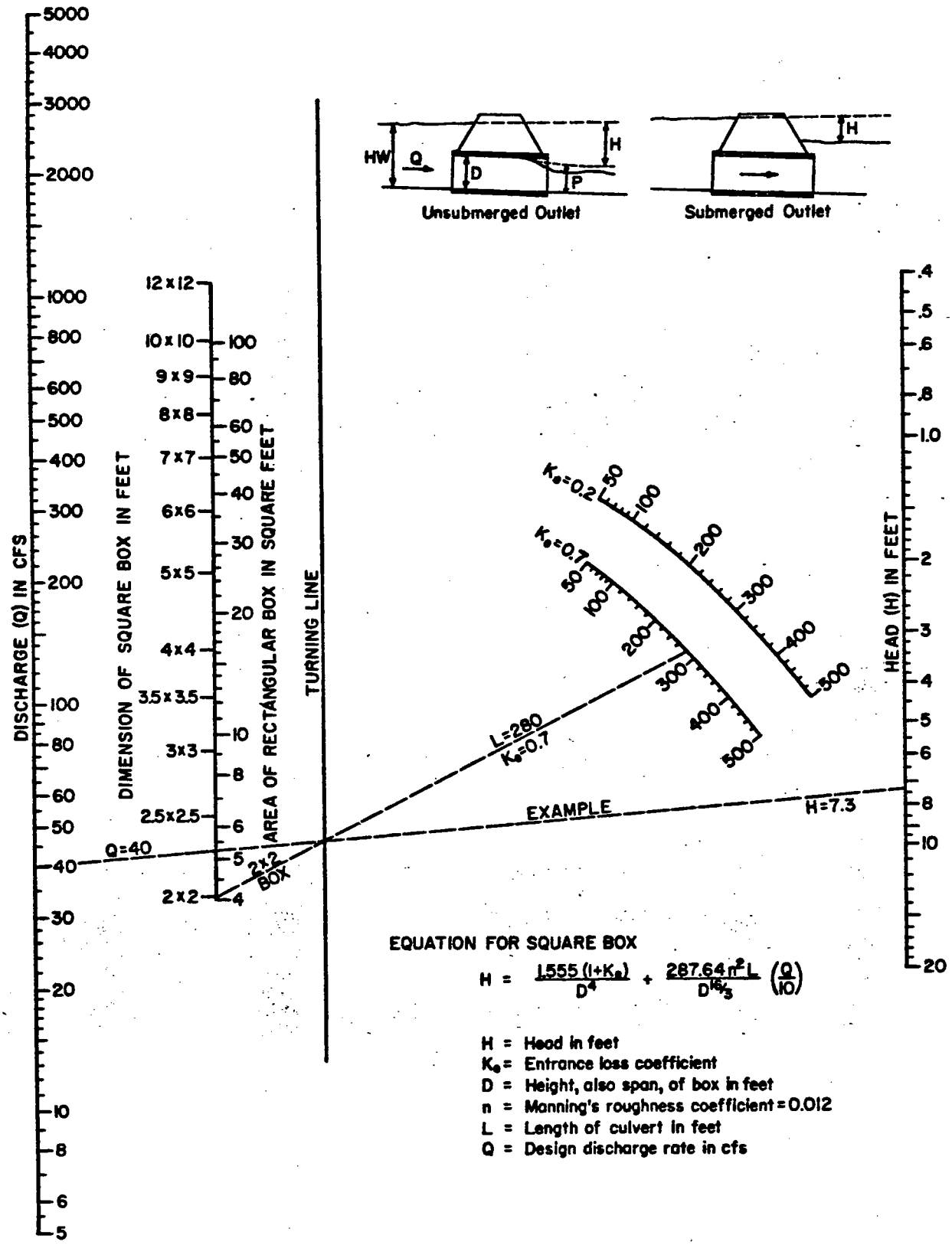
UNIFORM FLOW FOR
BOX CULVERTS

Figure 8-7



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

Figure 8-8



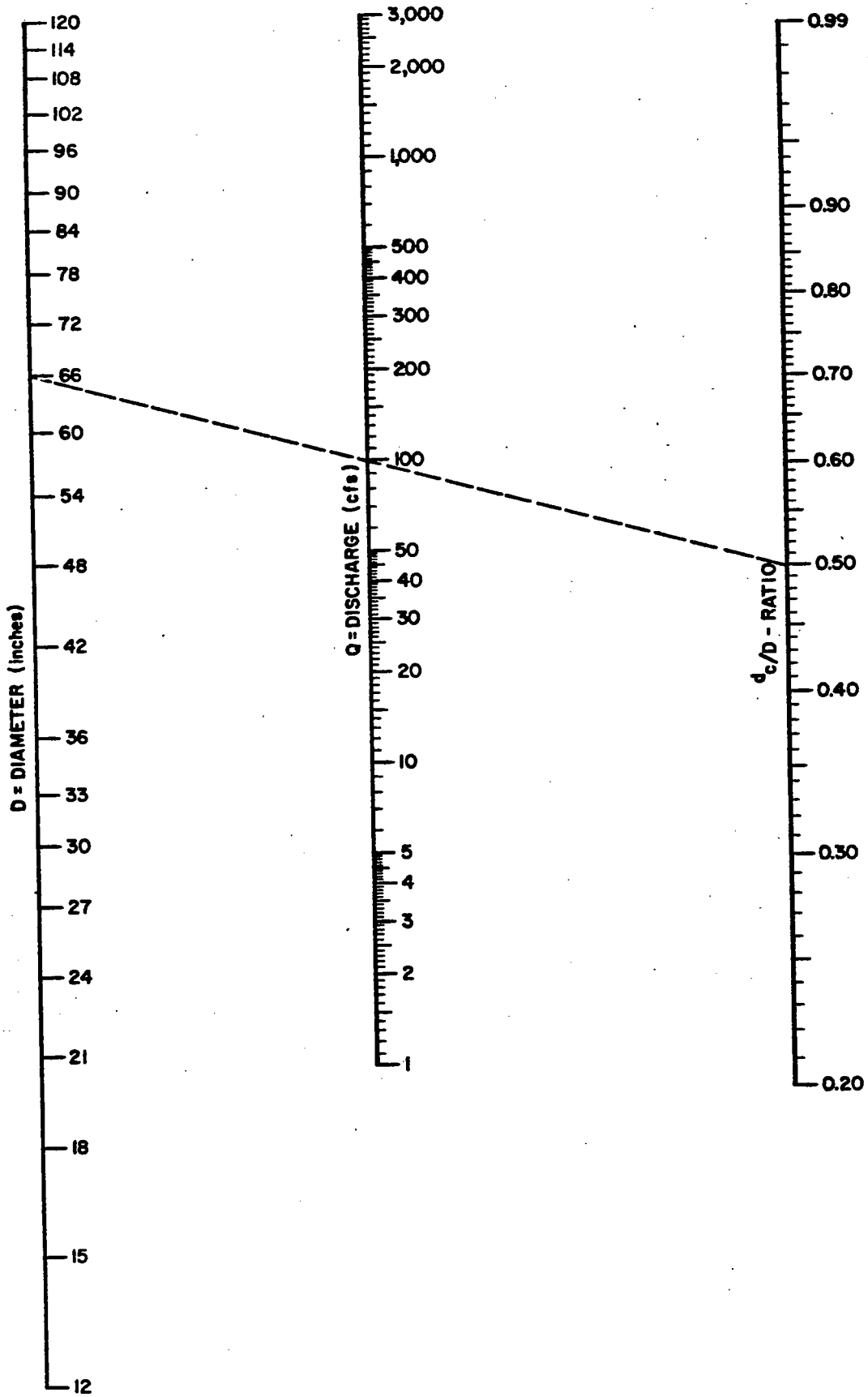
EQUATION FOR SQUARE BOX

$$H = \frac{1.555\,(1+K_e)}{D^4} + \frac{287.64\,n^2 L}{D^{16/3}}\left(\frac{Q}{10}\right)$$

H = Head in feet
K_e = Entrance loss coefficient
D = Height, also span, of box in feet
n = Manning's roughness coefficient = 0.012
L = Length of culvert in feet
Q = Design discharge rate in cfs

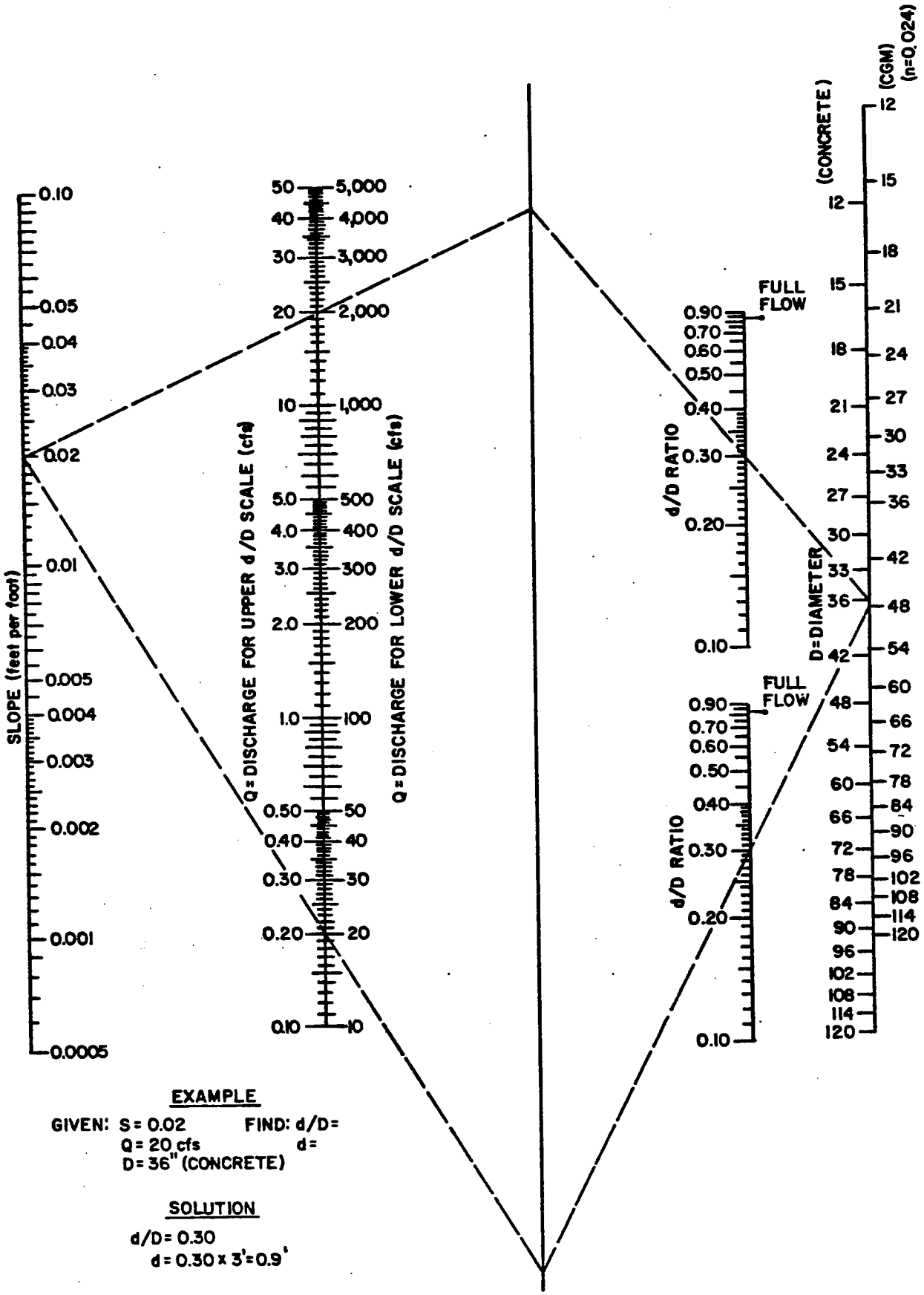
HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL

Figure 8-9



CRITICAL DEPTH OF FLOW
FOR CIRCULAR CONDUITS

Figure 8-10



EXAMPLE

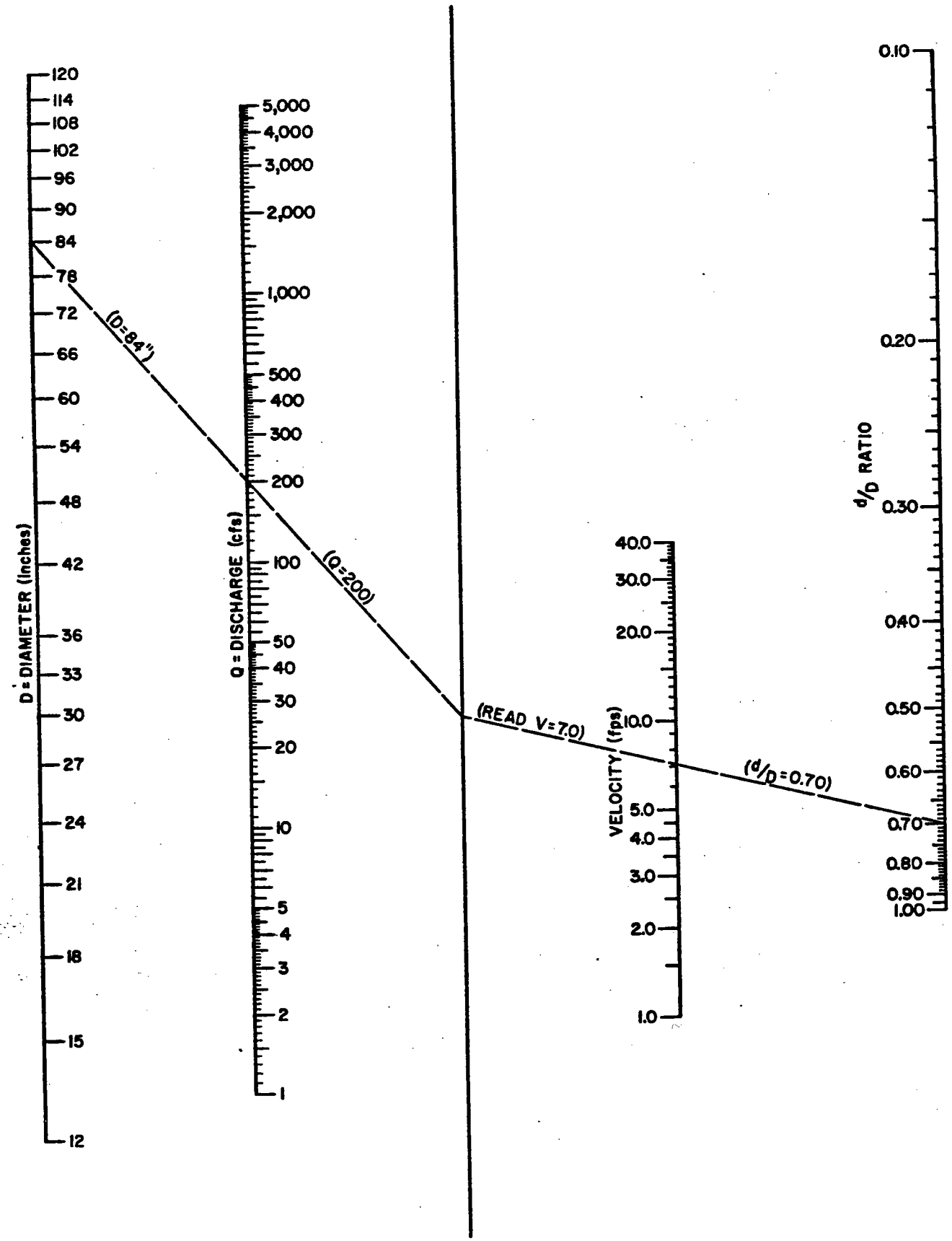
GIVEN: S = 0.02 FIND: d/D =
Q = 20 cfs d =
D = 36" (CONCRETE)

SOLUTION

d/D = 0.30
d = 0.30 x 3' = 0.9'

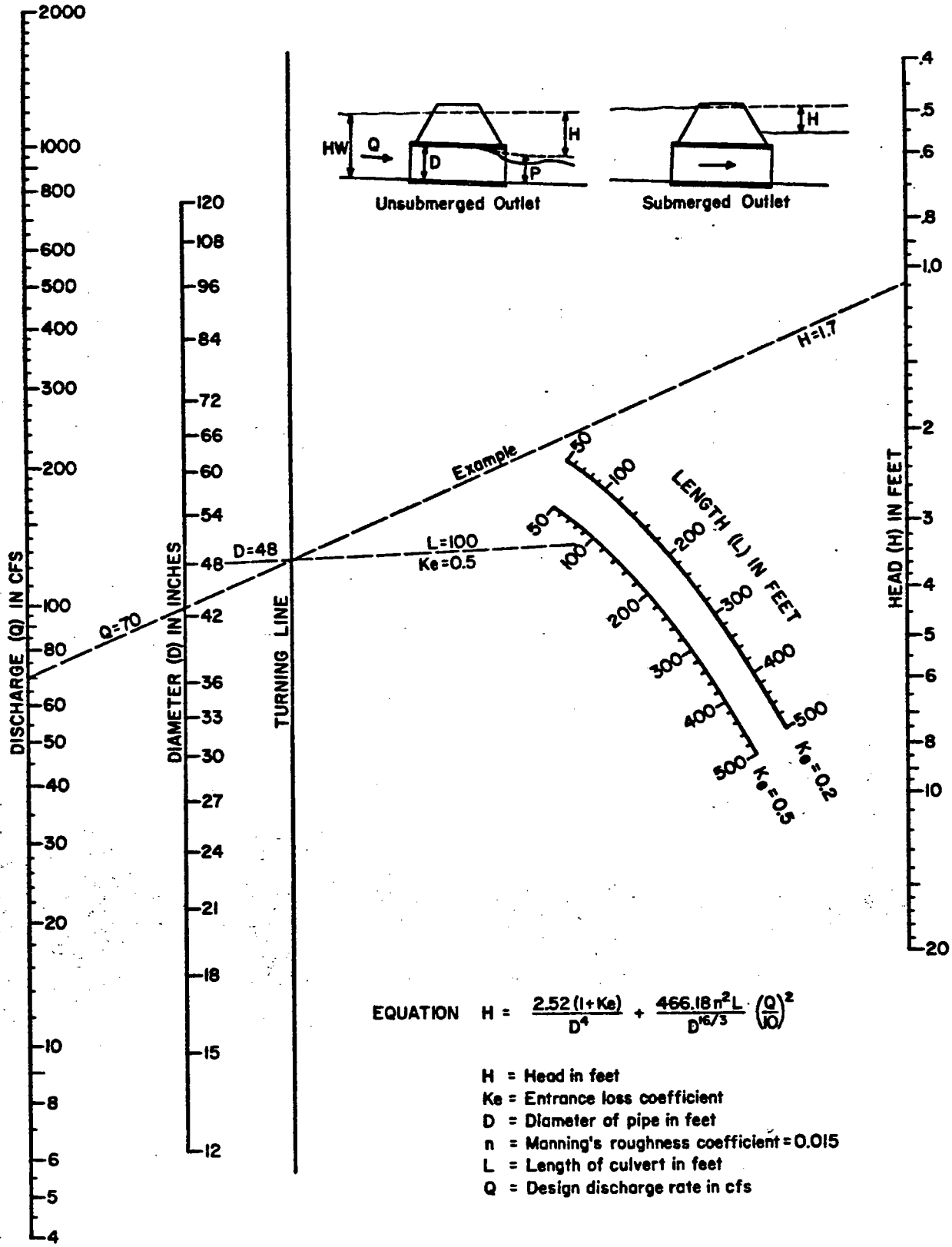
UNIFORM FLOW FOR PIPE CULVERTS

Figure 8-11



VELOCITY IN PIPE CONDUITS

Figure 8-12

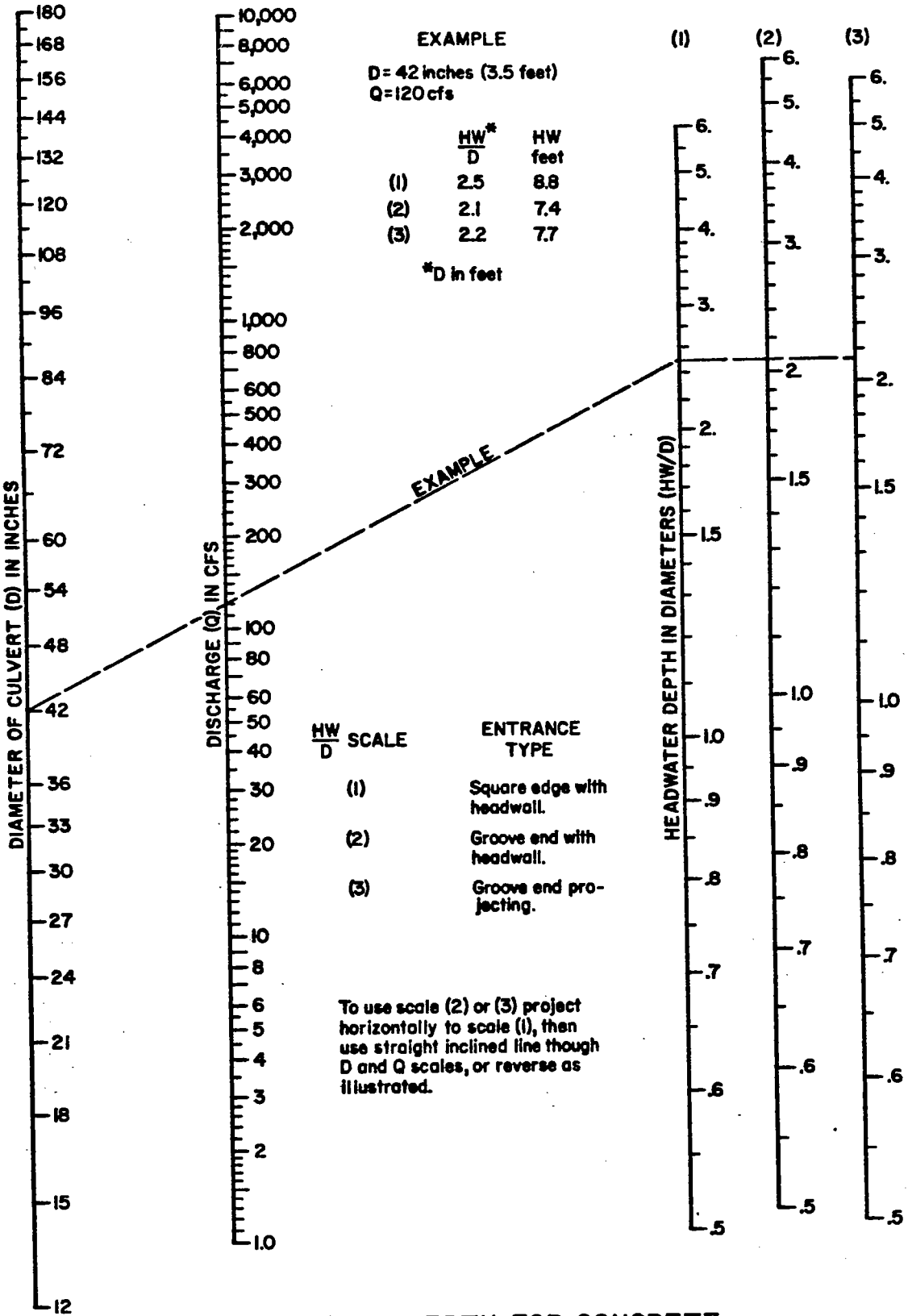


EQUATION $H = \frac{2.52(1+Ke)}{D^4} + \frac{466.18 n^2 L}{D^{16/3}} \cdot \left(\frac{Q}{10}\right)^2$

H = Head in feet
Ke = Entrance loss coefficient
D = Diameter of pipe in feet
n = Manning's roughness coefficient = 0.015
L = Length of culvert in feet
Q = Design discharge rate in cfs

HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL

Figure 8-13



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

Figure 8-14

B. Examples of Culvert Sizing Computations

Example 1:

Given:

$Q = 326$ cfs

$S_o = 0.002$ ft./ft.

Allowable headwater depth, HW = 6.0 ft.

Allowable outlet velocity, V = 8.0 fps

Length of Culvert, L = 200 ft. $\pm$

Tailwater depth, TW = 2.6 ft.

Flared Wingwalls

Required: The most economical concrete box culvert that will pass the design discharge.

Solution:

(1) Enter Figure 8-6 with $Q = 326$ and $V_c = 8.0$ and read approximate width of opening. W = 20', and $d_c = 2.0'$, then connect K value for flared wings = 1.15 with $V_c = 8.0$ and read HL = 1.2'. Then

$HW_c = d_c + HL$ or $2.0 + 1.2 = 3.2'$

From the above calculations it appears that a culvert having a width of 20' and a height of 3.2' will adequately pass the design discharge. In order to fit a standard design it is decided to try a 4 - 5' x 4' multiple box culvert.

(2) The next step is to determine the type of culvert operation. This is accomplished by first de-

termining the critical slope by entering Figure 8-7 with $\frac{d_c}{W} = \frac{2}{5} = 0.4$ and W = 5 and establishing a point on the turning line. Connect the point on turning line with $Q = \frac{326}{4} = 81.5$ and read

$S_c = 0.0037$

We have now assembled the following data:

Existing Channel	Culvert
$S_o = 0.002$ ft./ft.	$S_c = 0.0037$
TW = 2.6'	$d_c = 2.0'$
	D = 4.0'

Also we know the following:

$S_o < S_c$

$TW > d_c$

$TW < D$

This culvert will function as Type II operation with the control at the outlet providing $HW < 1.2D$.

(3) The next step is to determine the actual head-water depth and to confirm the Type II operation.

$$HW = TW + \left(\frac{V_{TW}}{2g}\right)^2 + h_e + h_f - S_oL$$

TW = 2.6'

$$\left(\frac{V_{TW}}{2g}\right)^2 = \frac{\left(\frac{Q}{A}\right)^2}{64.4} = \frac{\left(\frac{326}{20 \times 2.6}\right)^2}{64.4} = \frac{39.31}{64.4} = 0.61'$$

$h_e = K_e \left(\frac{V_{TM}}{2g}\right)^2 = 0.15 \times 0.6 = 0.09$

(For K_e see Table 8-1)

$h_f = S_f L$ Enter Figure 8-7 with

$\frac{d_{TW}}{W} = \frac{2.6}{5} = 0.52$, W = 5 and

$Q = \frac{326}{4} = 81.5$ and read $S_f = 0.0019$ ft./ft.

$h_f = 0.0019 \times 200 = 0.38'$

$S_o L = 0.002 \times 200 = 0.40'$

$HW = 2.60 + 0.61 + 0.09 + 0.38 - 0.40 = 3.28'$

The computation of the headwater depth confirms the Type II operation since $HW \leq 1.2D$.

(4) The outlet velocity $= \frac{Q}{A} = \frac{326}{20 \times 2.6} = 6.3$ fps

Since the calculated HW = 3.27' which is substantially less than the allowable HW = 6.0' and the calculated V = 6.3 fps which is less than the allowable V = 8.0 fps, the above structure is considered uneconomical.

<u>Example 2:</u>

Given: Same data as in Example 1.

Try 2 - 6.5' x 4' multiple box culvert.

Solution:

(1) From Figure 8-6 $d_c = 2.65$, $V_c = 9.30$

(2) From Figure 8-7 $S_c = 0.0035$ ft./ft.

since $S_o < S_c$

and $TW < d_c$

We have a Type I operation with control at the outlet providing $HW \leq 1.2D$.

(3) Check HW for Type I operations:

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_oL$$

$$d_c = 2.65'$$

$$\frac{V_c^2}{2g} = \frac{(9.30)^2}{64.4} = 1.34'$$

$$h_e = K_e \left(\frac{V^2}{2g}\right) = 0.25 \times 1.34' = 0.20'$$

$h_f = S_fL$ Enter Figure 8-7 with

$$\frac{1.1d_c}{W} = \frac{1.1 \times 2.65}{6.5} = 0.45, \; W = 6.5'$$

$$Q = \frac{326}{2} = 163 \text{ and read } S_f = 0.00275 \text{ ft./ft}$$

$$h_f = S_fL = 0.00275 \times 200 = 0.55'$$

$$S_oL = 0.02 \times 200 = 0.40'$$

$$HW = 2.65 + 1.34 + 0.20 + 0.55 - 0.40 = 4.34'$$

Since $HW < 1.2D$ the installation will function as a Type I operation.

(4) Outlet Velocity = V_c = 9.30 fps.

HW is still lower than the allowable HW = 6.0'; however, the outlet velocity is greater than the

allowable which was assumed to be 8 fps. The designer has the choice to provide riprap in the downstream channel, select a multiple box culvert of greater width or consider Type IV operation.

Example 3:

Given: Same data as in Example 1.

Required: Multiple Box Culvert for Type IV operation.

Solution:

For the given data let us select a 2 - 5' x 4' multiple box culvert. HW must be equal to or greater than 1.2D, or HW = 1.2 x 4.0 = 4.8' minimum. A partially submerged outlet (Type IV-B) will be considered. Under these conditions:

$HW = H + P - S_oL$

(1) Area of one barrel = 5 x 4 = 20 sq. ft.
Length of Culvert = 200 ft. K_e (Flared Wing-walls) = 0.4 (Table 8-1)
Q per barrel = $\frac{326}{2}$ = 163 cfs

(2) Use Figure 8-9. Connect area of one barrel -20 sq. ft. with 200 ft. length on K_e = 0.4 scale. The position of K_e = 0.4 must be interpolated between the limits K_e = 0.2 and K_e = 0.7. Mark point on turning line. Connect this point with Q = 163 and read H - 2.3'.

(3) According to the definition,

$$P = \frac{d_c + D}{2}$$

Enter Figure 8-6 with Q = 326, W = 10 and read

$d_c = 3.1'$

Then $P = \frac{3.1 + 4.0}{2} = 3.55'$

and HW = 2.3 + 3.55 - (0.002 x 200)

HW = 5.45'

(4) V (outlet) $= \frac{Q}{A} = \frac{326}{10 \times 3.1} = 10.5$ fps (concrete apron req'd.)

Note: Had TW been higher than D we would have had a submerged outlet and Type IV - A Flow would have controlled

$HW = H + TW - S_oL$ and V (outlet) $= \frac{Q}{A}$

Example 4:

Given: To illustrate Type III operation assume the same data as in Example 1 except that $S_o = 0.005$ and the allowable outlet velocity = 10.0 fps.

Required: To determine the size of concrete box culvert.

Solution:

(1) Enter Figure 8-6 with Q = 326 cfs and $V_c = 10.0$ fps and read W = 10', $d_c = 3.1'$ and HL = 1.3'.

Then $HW_c = d_c + HL = 3.1 + 1.3 = 4.4'$

(2) 10' x 5' single box culvert.

To determine the type of operation first find S_c by entering Figure 8-7 with $\frac{d_c}{W} = \frac{3.1}{10}$

$= 0.31$, $W = 10'$

and establish a point on the turning line. Connect this point with $Q = 326$ cfs and read $S_c = 0.00295$ ft./ft.

We now have assembled the following data:

Existing Channel	Culvert
$S_o = 0.005$ ft./ft.	$S_c = 0.00295$ ft./ft.
$TW = 2.6'$	$d_c = 3.1'$

Since $S_o > S_c$

and $TW < D$

indications are the structure will function as Type III operation providing the $HW < 1.2D$.

(3) For Type III operation the control is critical depth at the entrance and

$HW = \frac{HW}{D}$ (from Nomograph) x D

check HW:

Enter Figure 8-8 with $\frac{Q}{W} = \frac{326}{10} = 32.6$ and $D = 5'$

and determine $\frac{HW}{D} = 1.0$

Then $HW = 1.0 \times D = 5 = 5'$

(4) The velocity for Type III culverts varies from critical velocity at the entrance to uniform velocity at the outlet provided the culverts is sufficiently long. We assume in this example that

the outlet velocity is equal to the uniform velocity which is computed as follows:

Enter Figure 8-7 with $S_o = 0.005$, $Q = 326$ and $W = 10$ and determine $\frac{d}{W} = 0.26$

$d = 0.26\ W = 0.26 \times 10 = 2.6$

$A = 10 \times 2.6 = 26.0$ sq. ft.

$V = \text{(uniform)} = \frac{Q}{A} = \frac{326}{26.0} = 12.5$ fps (Outlet requires riprap)

Example 5:

Given:

$Q = 326$ cfs

$S_o = 0.002$ ft./ft.

Allowable headwater depth, HW = 6.5 ft.

Allowable outlet velocity, V = 8.0 fps

Length of Culvert, L = 200 ft. ±

Tailwater depth, TW = 2.6 ft.

Square edge with headwall

Required: Determine size of concrete pipe culvert to pass the design discharge.

Solution:

(1) Use Figure 8-14, connect $\frac{HW}{D} = 1.2$ with $Q = 326$ and read approximate opening required = 80 inches. Since the allowable HW is restricted to 6.5' and HW for 80" pipe = 1.2 x 6.7 = 8.0' the designer trys 2 - 60" pipes, and HW = 1.2 x 5.0 = 6.0'.

(2) Use Figure 8-10; connect $Q = \frac{326}{2} = 163$ with $D = 60"$

and read $\frac{d_c}{D} = 0.73$.

$d_c = 0.73D = 0.73 \times 5.0 = 3.65'$

(3) Use Figure 8-11; connect 60" with $\frac{d_c}{D} = 0.73$

and intersect turning line. Connect turning line with $Q = 163$ and determine $S_c = 0.0046$ for concrete pipe.

We have now assembled the following data:

Existing Channel	Culvert
$S_o = 0.002$ ft./ft.	$S_c = 0.0046$ ft./ft. (Conc.)
$TW = 2.6'$	$d_c = 3.65'$
	$D = 5.0'$

Since $s_o < S_c$ and $TW < d_c$, we have a Type I operation with control at the outlet, providing $HW \leq 1.2D$.

(4) The next step in this design is to determine the actual headwater depth and to confirm the Type I operation.

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_oL$$

$d_c = 3.65'$

For $\frac{d_c}{D} = 0.73$; V_c (Figure 8-12) = 10.7 fps

$\frac{V_c^2}{2g} = \frac{(10.7)^2}{64.4} = 1.77'$

$h_e = 0.5 \times 1.77 = 0.89'$

h_f is calculated as follows:

$1.1\ d_c = 1.1 \times 3.65 = 4.01'$

$\frac{1.1\ d_c}{D} = \frac{4.01}{5.0} = 0.8$

To determine the friction slope S_f,

enter Figure 8-11 with D = 60", $\frac{d_c}{D} = 0.8$

$Q = 163$ and determine $S_f = 0.0038$

$h_f = S_f L = 0.0038 \times 200 = 0.76'$

$S_o L = 0.002 \times 200 = 0.40'$

$HW = 3.65 + 1.77 + 0.89 + 0.76 - 0.40 = 6.67'$

(5) Since HW > 1.2D for the concrete pipe, the concrete pipe will not function as Type I operation. Also the HW exceeds the allowable.

(6) The designer must now try another pipe size to carry the design flow. Try 2-66" pipes.

(7) Use Figure 8-10; connect Q = 163 cfs with D = 66" and read $\frac{d_c}{D} = 0.65$.

$d_c = 0.65D = 0.65 \times 5.5 = 3.58'$

(8) Use Figure 8-11; connect 66" with $\frac{d_c}{D} = 0.65$

and intersect turning line. Connect turning line with Q = 163 and determine $S_c = 0.004$.

We have now assembled the following data:

Existing Channel	Culvert
S_o = 0.002 ft./ft.	S_c = 0.004 ft./ft.
TW = 2.6'	d_c = 3.58'
	D = 5.5'

Since $S_o < S_c$ and $TW < d_c$, we have a Type I operation, providing $HW < 1.2D$.

(9) Check to determine the actual headwater depth and to confirm the Type I operation.

$$HW = d_c + \frac{v_c^2}{2g} + h_e + h_f - S_oL$$

$$d_c = 3.58'$$

For $\frac{d_c}{D} = 0.65$; from Figure 8-12, $V_c = 10.0$ fps

$$\frac{V_c^2}{2g} = \frac{(10)^2}{64.4} = 1.55'$$

$$h_e = 0.5 \times 1.55 = 0.78'$$

$$\frac{1.1d_c}{D} = \frac{1.1\ (3.58)}{5.5} = 0.72$$

From Figure 8-11 with D = 66", $\frac{d}{c} = 0.72$,

$$Q = 163$$

determine $S_f = 0.0032$

$$h_f = S_fL = 0.0032 \times 200 = 0.64'$$

$$S_oL = 0.002 \times 200 = 0.40'$$

$$HW = 3.58 + 1.55 + 0.78 + 0.64 - 0.40 = HW = 6.1'$$

(10) Since $HW < 1.2D$, the pipe will function as a Type I operation. Also the headwater is calculated to be less than the allowable.

(11) Check outlet velocity to determine if within allowable.

Outlet velocity = V_c = 10 fps

This velocity is greater than allowable. The designer must consider providing riprap in the downstream channel or some type of energy disipation method or try another size pipe culvert.

Example 6:

Given: To illustrate Type III operation assume the same data as in Example 5 except that S_o = 0.02 and the allowable outlet velocity is 15 fps due to a solid rock channel.

Solution:

Follow the same procedure as in Example 5 for determining the initial size, critical depth and critical slope which is summarized below:

Existing Channel	Culvert
S_o = 0.02 ft./ft.	S_c = 0.0046 ft./ft. (Conc.)
TW = 2.6'	d_c = 3.65'
	D = 5.0'

Since $S_o > S_c$ and TW $<$D, the installation will function as Type III operation providing the entrance is unsubmerged, i.e. HW $<$1.2D

(1) The next step in this design is to determine the actual headwater depth and to confirm the Type III

operation.

$HW = \frac{HW}{D} \times D$

$\frac{HW}{D}$ (Figure 8-14 = 1.13 for concrete pipe.)

HW (conc. - grooved end with headwall) =

1.13D = 1.13 x 5.0 = 5.65'.

Since HW $<$1.2D the concrete pipe will function as Type III operation.

(2) The velocity for Type III operation varies from critical velocity at the entrance to uniform velocity at the outlet providing the installation is sufficiently long and the TW depth = uniform depth.

Enter Figure 8-11 with S_o = 0.02, Q = 163,

D = 60" and determine

$\frac{d}{D}$ = 0.45, d = 0.45D = 0.45 x 5.0 = 2.25

Since TW $\geq$2.25 the outlet velocity is based on TW depth as follows:

$\frac{d_{TW}}{D} = \frac{2.25}{5.0} = 0.45$

Enter Figure 8-12 with D = 60", Q = 163 and the controlling $\frac{d}{D}$ ratios and determine

V (outlet - Conc.) = 19.0 fps

Some provision must be made to reduce the outlet velocity to the allowable velocity.

STORAGE
SECTION 9

9.01 General

9.02 Storage Classification

A. Rainfall Storage

B. Runoff Storage

9.03 Rainfall Storage

A. Roof Tops

B. Parking Lots

C. Recreation Areas

D. Property Line Swales

E. Porous Ground

F. Combinations

9.04 Runoff Storage

A. Offstream Storage

B. Channel Storage

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9.05 Multipurpose Use

Lower Cost Recreation

Multi-Use Criteria

9.06 Hydraulic Design Criteria

A. General

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9.07 Hydraulic Design Methods

A. General

B. Rational Method Storage Analysis

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STORAGE
SECTION 9

9.01 GENERAL

Storm water management programs aimed at controlling increased stream water runoff generated by development are a top priority in urban watershed planning. More frequent flooding, increased rates and volumes of storm water runoff, increased stream channel erosion and degradation, increased sedimentation, and increased water pollution are all problems intensified by increased storm water runoff resulting from development. Storage of excess urban storm runoff is one of the most promising methods available in preventing urban flood damage.

Storm runoff storage with reduced release rates can hold downstream flood flows to within the safe conveyance capacity of the storm sewer and stream system. In most cases, it can be shown that storage is more economical than increasing downstream conveyance capacity. Storage facilities should be planned and designed to assure an effective and efficient operation and maintenance program.

Retention and detention are two generalized types of storm runoff storage used to control flooding. Retention storage refers to storm runoff collected and stored for a significant period and released or used after the storm runoff has ended. Retention storage usually consists of "wet reservoirs" which often have agricultural,

recreational, and/or aesthetic value. Detention storage consists of reducing the rate of runoff for a short period of time to reduce peak flows by controlling the discharge through an outlet structure and by extending the period of runoff.

Skillful use of the procedures outlined in this section will provide reasonable solutions to Drainage Policy requirements. Individual and community experience in the use of these techniques will evolve into a methodology which will allow urban growth without the attendant drainage problems.

9.02 STORAGE CLASSIFICATION

Storage can be classified by location as follows:

A. Rainfall Storage

The storage of water near the point of rainfall occurrence or before storm runoff accumulates significantly. Rainfall storage is usually accomplished by roof top ponding, parking lot ponding, property line swale ponding, and small on-site ponds.

B. Runoff Storage

Water stored after runoff has accumulated significantly and begun to flow in a drainage system. Runoff storage is usually accomplished by offstream storage, channel storage, or on-stream storage.

9.03 RAINFALL STORAGE

A. Roof Tops

Some city codes require roof load designs for rain and snow. The design load may be converted to equivalent water depth in inches which can be safely contained on flat roofs. The maximum storage allowed for design purposes should not exceed this depth unless a building is designed to withstand a greater roof load. The depth of water can be controlled by proper sizing of downspouts and by constructing scuppers through the parapet walls.

Another method of achieving roof top ponding involves placing loose gravel windrows or dikes a few inches in heights so that storm runoff is trapped as it moves toward the drainage outlet. Individual wedge-shaped ponds form behind the small dikes and slowly drain as the storm water filters through the gravel. This solution is of limited value, however, as there is no way to assure it's permanence.

The possibility of overflows exist and will occur quite often during major storms. Overflow drains should be used to protect against roof overloading. Periodic inspection and proper maintenance will reduce the possibility and the hazards of overflows.

Special attention must be given to the water tightness of the roof to assure that no leakage occurs due to the accumulation of water.

B. Parking Lots

Considerable area in urban development is occupied by parking lots. If planned correctly, these paved areas can provide adequate detention with minimum inconvenience to the public and without functional interference.

There are two general methods of storm water detention that can be utilized on parking areas. One form involves the storage of runoff in depressions constructed near drains or outfall structures. The second method of storm water detention on parking lots consists of using the paved parking areas to channel storm water to grassed or gravel filled areas for maximum infiltration and a decrease in overland flow.

C. Recreation Areas

Recreation areas, such as sports fields, generally have a substantial area of grass cover which often has a high infiltration rate. Storm runoff from such fields is minimal. A secondary use of such recreation fields can be made by providing for limited detention storage of runoff from adjacent areas.

Since these areas are not used during periods of precipitation, detention ponding should not seriously impede their primary use. To minimize after effects, the recreation area should be designed so that it will thoroughly drain. In addition, the vegetation used on the area should be tolerant of periodic inundation and wetness.

D. Property Line Swales

Subdivision planning and layout requires adequate surface drainage away from buildings. This is obtained by sloping the finished grade in all directions away from the building. The layout often calls for a swale to be located along the back property line, which then drains longitudinally through the block.

Temporary ponding facilities along the rear lot line may include small controlled discharges along the back property line or, if the subsoil conditions are favorable, several inches of rainfall could be expected to percolate into the ground during and after a storm.

Prior to planning for property line swale ponding, the engineer should determine that saturation of the subsoil will not have an adverse effect on building foundations. In cases where significant subsoil problems exist, water should not be ponded or percolated into the ground.

E. Porous Ground

Porous pavement used in parking and other areas can reduce flood peaks by allowing water to penetrate and flow through it. Porous pavement can be of two types: (1) porous asphalt with a crushed rock base, or (2) lattice-work concrete blocks.

Since the use of porous pavements as a method of storing or attenuating storm runoff in the development state, there may be additional design modifications required. Provisions should

be made to insure that the surface of the paving does not become clogged as this will decrease its permeability. When porous asphalt is utilized, storms greater than the design storm may generate excess runoff and provisions should be made for this overflow. Certain soils when saturated, suffer diminished strength, pumping occurs and a general deterioration of the pavement results.

F. Combinations

In many instances, one on-site detention method cannot conveniently or economically satisfy the required or needed amount of storm water storage. Limitations in storage capacities, site development conditions, soils limitations and other related constraints may require that more than one method be utilized. For example, rooftop, parking lot, and surface pond storage might all be required to compensate for increase in runoff due to development of a particular site. Whatever combinations are suitable may be incorporated into a site development plan.

9.04 RUNOFF STORAGE

A. Offstream Storage

The storage of water in depressed open areas, in reservoirs, and on low lying recreation fields to which storm peak flows are routed is usually termed offstream storage. It is usually characterized by a side channel spillway or overflow

from the main channel.

B. Channel Storage

Although all channels inherently store water, channels can be made to attenuate runoff by altering their hydraulic characteristics in a way that will reduce peak flows. Side channels that run essentially parallel to the main stream channel are also a means of temporarily storing water during excessive rainfall events.

C. Onstream Storage

The construction of an embankment across a channel so that a storage pond is formed represents onstream storage. Spillway considerations are important to pass large floods exceeding the storage capacity. Properly protected roadway embankments are well suited for their purpose.

9.05 MULTIPURPOSE USE

Rainfall storage and runoff systems are most practical when designed for multi-purpose use. For maximum land use efficiency the design engineer must consult early with the Planning Department and the Parks and Recreation Department of the City.

Lower Cost Recreation. Making use of detention storage areas as parks and greenbelts, ballfields and playing fields, satisfies two needs and reduces the cost of each. The minipark concept, where small recreational areas are

provided in neighborhoods within safe walking distances for children, can be a typical joint effort of the drainage design engineer and the planning and recreational staffs of the City. Provisions must be made for maintenance of such areas.

Multi-Use Criteria. The multiple use of storage areas is a field of endeavor where the different disciplines should develop desirable and acceptable criteria fitting the needs of the local community. It is with storage, including both detention and retention storage, that an important potential exists for the reduction of flooding, drainage costs, and to some degree recreation costs.

9.06 HYDRAULIC DESIGN CRITERIA

A. General

Hydraulic and hydrologic design criteria provide the guidelines for design and construction of storm water storage facilities. These criteria are a necessary part of a storm water management program needed to protect low lying areas by preventing unacceptable increases in runoff rates as urban development progresses.

Existing land contours of the property should be a consideration in developing the drainage plan. In many instances, storage can be achieved economically by blocking the overland flow of storm water runoff with various land forms, curbs, walls, terraces, and other means. This amounts to designing a drainage system that will minimize the reduction in the critical time of concentration as the property is developed.

A drainage project located in a watershed that has a master plan may involve additional considerations which require review. Consultation with the Department of Public Utilities is essential to insure compliance with the overall watershed master plan.

B. Design Factors and Procedures

Before proceeding with engineering design of an on site storage facility, the physical and technical factors should be identified and the basic design procedures established. A discussion of design factors and procedures follow.

1. The drainage plans for a particular storage facility should include all property lines, topography, area, location, and all other items necessary to meet policy requirements outlined in the "Drainage Policy for the Parish of Jefferson" in Section 1 of this manual.

2. Care must be taken to locate, layout, and design the

storage facility in a manner to insure safety and ease of maintenance.

3. The rate of inflow to the storage facility (inflow hydrographs) and all hydrologic considerations must assume ultimate development of the site's contributing area. Several inflow hydrographs should be prepared to examine their effect on downstream flooding for the various design storm frequencies. The number of inflow hydrographs to be provided may vary but should always adequately describe the range of flows expected for the design storm frequencies. These various design storm inflow hydrographs and an acceptable flood routing procedure will be necessary in sizing the outlet works for the storage facility. An emergency spillway should be provided to pass runoff that exceeds the design capacity of the detention facility.

4. The maximum allowable release rate from a storage facility is selected after careful review of runoff rates for all pertinent storms. As a minimum, the pertinent design storm frequencies should include the 10, 25 and 100-year events. However, specific site conditions may require that additional consideration be given a particular area. The Department of Public Utilities should be contacted to identify special design conditions. The maximum allowable peak flow released from a particular site is selected to comply

with the policy section of this manual. In all cases, the maximum release rate should be calculated using the hydraulic gradient created when the storage area is being utilized at full capacity.

Key items to consider in determining the maximum allowable release rate are: a) the peak flow rates for the various design storm frequencies prior to development, b) the capacity of the downstream storm water channel or storm sewer, c) compliance with a master plan for the watershed (if one exists), d) problems, if any, that are created on site by the storage of storm water runoff, e) potential problems created downstream when the on-site storage capacity is fully utilized and the excess runoff flows overland into downstream areas.

5. The storm water storage volume required is given by the maximum difference, at any time, between cumulative total inflow volume and cumulative outflow volume measured from the beginning of inflow for a 25-year storm. The maximum allowable release rate is not to be exceeded by the outflow peak discharge.

6. The outfall or outlet structures should be designed on the basis of inlet or outlet control, whichever is applicable. The structures should be capable of safely and properly passing the flow range of design storm frequencies including the 100-year storm without causing

downstream flooding or exceeding the maximum allowable release rate. Outlet works must also pass all of the runoff from the 100-year design storm within a reasonable length of time or a permit may be required from the Louisiana Water Rights Commission. Wet ponds will in many cases also require a permit from the Louisiana Water Rights Commission.

7. Special attention should be given to the provision of an emergency or overflow spillway which would pass excess flows greater than those of the 25-year design storm and overflows caused by clogging of the principal outlets. Downstream watershed considerations such as the potential loss of life and damage to property due to overtopping or failure of the structure and the storage facilities, area and capacity dictate the emergency spillway design storm. It is advisable to contact state agencies such as the Louisiana Water Development Board and the Louisiana Water Rights Commission when planning a storage facility in order to meet their criteria.

9.07 HYDRAULIC DESIGN METHODS

A. General

The two basic methods suggested for predicting the volume of runoff with time and the peak flow rate are the Rational Method and the SCS Hydrograph Method. The SCS Hydrograph

Method is best suited for storage analysis of urban runoff but for relatively small drainage areas some form of the Rational Method is generally acceptable.

B. Rational Method Storage Analysis

When designing storage facilities for an area smaller than 400 acres, it is often justifiable to use the Rational Method to compute the design inflow hydrographs. This method relies heavily upon personal judgment but it is relatively simple and can give acceptable results if proper procedures are followed. Three different techniques of employing the Rational Method are discussed and two of the techniques have examples to show their use. These two examples, Example 1 and Example 2, outline the procedure to compute an inflow hydrograph for only one return period flood.

One of the three Rational Method techniques has been utilized by the Federal Aviation agency in designing airport drainage facilities. The procedure is presented in "Airport Drainage", prepared by the Federal Aviation Agency (1966). The FAA technique is basically a graphical procedure which represents the cumulative storm runoff volume and the cumulative volume released from the storage facility through an outlet structure as functions of time. The maximum difference between the two volume curves represents the required storage volume of the storage basin. The procedure will consistently result in underdesigned storage facilities if a constant release rate is assumed from time zero. This is caused by sizing

the outlet pipe using the Manning Equation and not recognizing the stage-discharge relationship. A reservoir routing technique utilizing the stage-discharge relationship of the storage facility should be used.

Examples using the FAA technique are not given here as they may be found in the FAA document mentioned above. It is recommended that in most instances the contributing drainage area not be more than 25 acres when using this tecnhique.

The second technique that employs the Rational Method is referred to as the Modified Rational Method Analysis. This technique manipulates the Rational Method to reflect the fact that storms with durations greater than the normal time of concentration for a basin will result in a larger volume of runoff even though the peak discharge is reduced. Even though rainfall intensities and resulting peak discharges associated with longer duration storms are less than those for short duration storms, the inflow may still be considerably greater than the outflow thus requiring more storage than in the case of shorter, higher intensity storms.

This approach becomes more valid on progressively smaller basins. The technique should, therefore, be limited to relatively small areas such as parking lots, rooftops, or other upstream facilities with contributing areas less than 25 acres.

Example 1 shows the application of the Modified Rational Method 11.6 acre tract of land using 10-year frequency intensities.

<u>Example 1</u>. Computation of Modified Rational Method Hydrographs

Given: A drainage area, when fully developed, will have the following characteristics:

Drainage area = 11.6 acres

Type of ultimate development: Office District

C = 0.90

Design rainfall frequency: 10-year

Rainfall intensity-duration-frequency curves: see SECTION 2, Figure 2-3 Rainfall Curves for Metropolitan New Orleans.

Time of consentration (t_c) = 15 minutes (SECTION 2)

Required: Develop a family of curves representing Modified Rational Method hydrographs for the 15, 30, 60 and 90 minute rainfall durations.

Solution: Solution is shown in Table 9-1 and Figure 9-1.

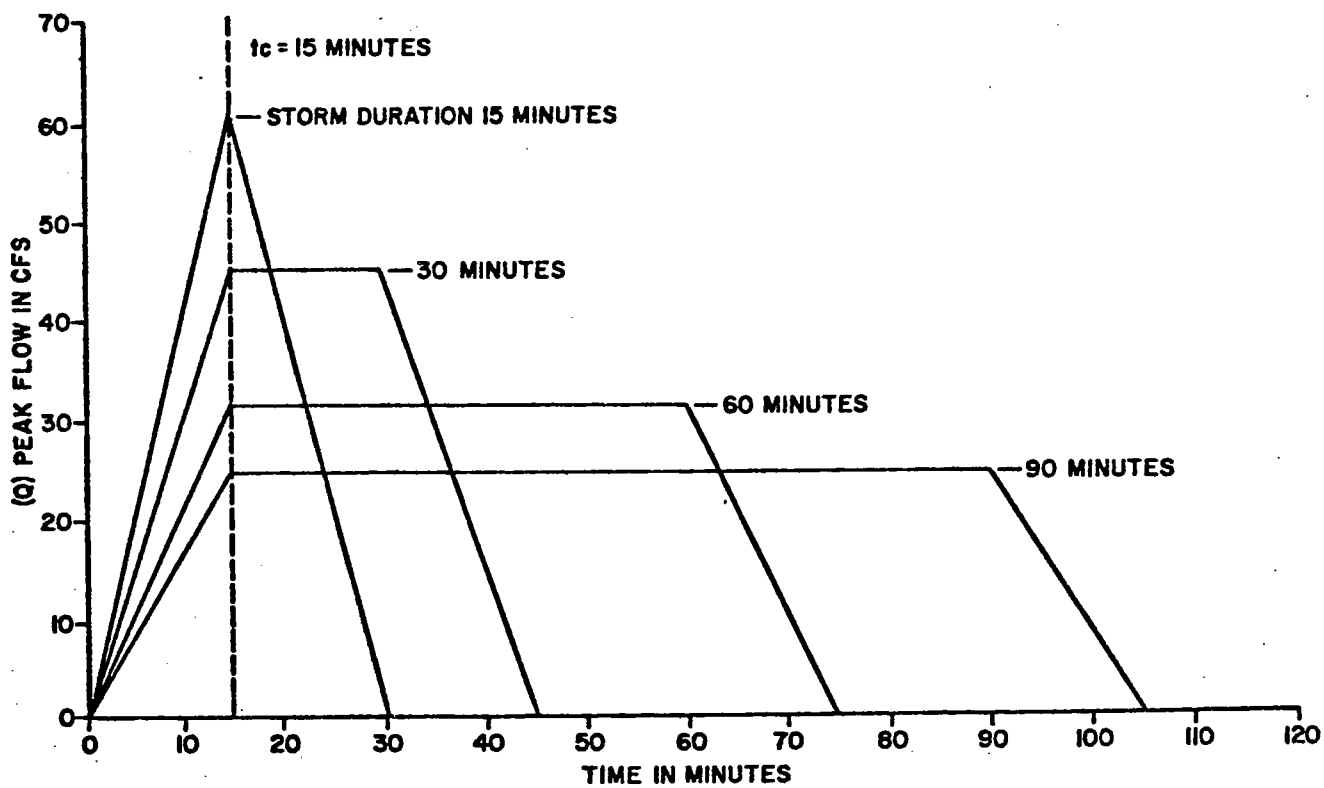
TABLE 9-1

HYDROGRAPH DEVELOPMENT BY
MODIFIED RATIONAL METHOD PROCEDURE
FOR 10-YEAR RAINFALL INTENSITIES

Rainfall Duration (Min.)	Rainfall Intensity (In/Hr)	Peak Runoff Rate (cfs)
15	6.0	62.6
30	4.4	45.9
60	3.1	32.4
90	2.4	25.0

Figure 9-1 shows the resulting hydrographs. These hydrographs are developed using the basic Rational Method assumptions of constant rainfall intensity (i), time of concentration (t), and the coefficient of runoff (C). The triangular shaped hydrograph with the peak discharge coinciding with the time of concentration for the area is computed using the Rational Method formula, Q = CiA. Following this, a family of trapezoidal shaped hydrographs representing storms of greater duration are developed. The peak discharge for each hydrograph is equal to CiA where (i) is the rainfall intensity for each respective storm duration. The rising and falling limbs of the hydrograph are, in each case, equal to (t) for the drainage area. The basic assumption of this method is that the volume of the developed trapezoidal hydrographs equals the volume of runoff from the theoretical rainfall. The volume under the hydrograph is also equal to the peak discharge rate for that particular rainfall times the rainfall duration.

The Modified Rational Method technique should be used to develop hydrographs over the range of required storm frequencies to evaluate the effect of resultant flooding that might occur. SECTION 2 of this manual should be consulted to select the appropriate rainfall intensity (i) and coefficient of runoff (C). After developing the hydrographs for various durations and return frequencies, a routing routine should be employed in designing the optimum outlet facilities and required storage capacity.



MODIFIED RATIONAL METHOD HYDROGRAPHS FOR 100-YEAR RAINFALL INTENSITIES

Fig. 9-1

The third technique utilizing the Rational Method considers a design storm with varying rainfall intensities. The technique begins with the intensity-duration-frequency curve shown in Figure 2-3 located in SECTION 2 of this manual. A

design frequency is chosen and a design storm is symmetrically arranged such that the storm rainfall pattern will have average intensities of the same recurrence interval for all durations. The total storm length should be at least one or two hours or until storm intensities are low enough that no appreciable runoff will occur. It is suggested the design storm be built in small time increments of two to five minutes and that the maximum intensity be located near the storm's center.

The basic assumption made in developing the runoff hydrograph is that the runoff rate at any point of time is equal to $Q = CiA$. The rainfall intensity (i) is taken as the average for the time of concentration immediately preceding the time at which the runoff rate is being computed. This procedure is followed only when computing the ordinates for the rising limb of the hydrograph. The falling limb of the hydrograph is made to be twice as long as the rising limb which basically characterizes the shape of most hydrographs. The increment of time between any one particular characterizes the shape of most hydrographs. The increment of time between any one particular flow (Q) value and the peak time will be twice as long for the falling limb compared to the rising limb. Example 2 and Tables 9-2 and 9-3 outline the procedure.

This technique also provides for analysis based on different times of concentrations for any particular drainage area that is being developed. The different (t) values can greatly effect the shape of the flood hydrograph and the resulting

amount of storm water to be retained.

Example 2. Rational Method Using Variable Rainfall Intensities

Given: A drainage area, when fully developed, will have the following characteristics:

Drainage area = 300 acres

Type of development: Residential C = 0.45

Design rainfall frequency: 10-year

Rainfall intensity-duration-frequency curves: see SECTION 2, Figure 2-3. Rainfall Curves for Metropolitan New Orleans.

Time of concentration - 40 minutes.

Required: Develop a 10-year design storm and resulting flood hydrograph as part of an analysis in planning and designing a storage facility.

Solution: The solution is outlined in Table 9-2 which shows the development of the design 10-year frequency storm and Table 9-3 which shows the computation of the design 10-year flood hydrograph.

Table 9-3 summarizes the procedure to compute the 10-year flood hydrograph for the drainage area described in Example 2. Referring to Table 9-3, the columns are identified and computed as follows:

(1) = Time from beginning of storm in minutes.

(2) = i_{10} is the rainfall intensity in inches per hour for various times during the 10-year design

storm.

(3) = Column (3) accumulates the values from column (2).

(4) = Values from column (3) are displaced a time equal to the time of concentration which is 40 minutes in this case.

(5) = Column (4) subtracted from column (3).

(5) = (3)-(4).

(6) = Values for this column are compiled from the equation

$$\frac{(5)}{t_c/AT}$$

(5) = values taken from Column (5)

t_c = time of concentration = 40 min. in this case

AT = time increment from column (1) = 5 minutes

Thus column (6) = $\frac{(5)}{40/5}$ = $\frac{(5)}{8}$ in this case which is simply an average intensity over the previous t_c increment

(7) = The rising limb of a runoff hydrograph from a 300 acre area with a C value of 0.45 is computed with the expression CAq_{40}. This becomes (0.45) x (300) x (q_{40}) in this case.

(8) = The increment of time for the falling limb of the hydrograph is doubled from the time of the peak and thereafter folded back so that the computed runoff figures can be used.

The computations were stopped in column (7) when the rising

limb of the hydrograph reached its peak value. At this point the time scale can be folded as shown in column (8). Doubling the time increments for the falling limb serves to double the volume that would have been under that portion of the runoff hydrograph. The volume under the entire discharge hydrograph will be three times that under the rising limb. With this assumption, the volume of runoff expressed as a percentage from an area with a runoff coefficient of 0.45, becomes approximately 67.5% rather than 45% of the rainfall. In this procedure the C value from the Rational Method formula represents the ratio of the peak runoff to the average rainfall intensity rate for a period equal to the time of concentration and not a simple runoff to rainfall ratio.

Having examined the design 10-year storm, other return frequency storms should be converted to flood hydrographs using the same procedure. As in the other Rational Method techniques, a procedure should be used to aid in sizing the outfall structures by routing all pertinent return frequency hydrographs through the planned facility.

TABLE 9-2

DEVELOPMENT OF A 10-YEAR FREQUENCY STORM

Duration (Min)	Accumulated Depth (In)	Incremental Depth (In)	Incremental Intensity (In/Hr)	Incremental Order
5	0.67	0.67	8.04	19
10		0.41	4.92	20
15	1.50	0.41	4.92	18
20		0.22	2.64	21
25	1.95	0.22	2.64	17
30		0.17	2.04	22
35	2.40	0.17	2.04	16
40		0.16	1.92	23
45	2.70	0.16	1.92	15
50		0.14	1.68	24
55	2.98	0.14	1.68	14
60		0.11	1.32	25
65	3.19	0.11	1.32	13
70		0.09	1.08	26
75	3.37	0.09	1.08	12
80		0.09	1.08	27
85	3.54	0.08	0.96	11
90		0.08	0.96	28
95	3.70	0.08	0.96	10
100		0.07	0.84	29
105	3.84	0.07	0.84	9
110		0.06	0.72	30
115	3.96	0.06	0.72	8

120		0.06	0.72	31
125	4.08	0.06	0.72	7
130		0.06	0.72	32
135	4.20	0.06	0.72	6
140		0.05	0.60	33
145	4.40	0.05	0.60	5
150		0.05	0.60	34
155	4.50	0.05	0.60	4
160		0.05	0.60	35
165	4.60	0.05	0.60	3
170		0.04	0.48	36
175	4.68	0.04	0.48	2
180		0.04	0.48	37
185	4.76	0.04	0.48	1

TABLE 9-3

RUNOFF COMPUTATIONS FROM A 300 ACRE AREA WITH A TIME OF CONCENTRATION OF 40 MINUTES AND C = 0.45

Time (Min)	i_{10} (In/Hr)	Σi_{10}	Σi_{10} (Lagged 40 Min)	(3)-(4)	q_{40} (In/Hr)	Q(cfs)	Time Folded
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0						0	330
5	0.48	0.48		0.48	0.06	8.1	320
10	0.48	0.96		0.96	0.12	16.2	310
15	0.60	1.56		1.56	0.20	27.0	300
20	0.60	2.16		2.16	0.27	36.5	290
25	0.60	2.76		2.76	0.35	47.3	280
30	0.72	3.48		3.48	0.44	59.4	270
35	0.72	4.20		4.20	0.53	71.6	260
40	0.72	4.92		4.92	0.62	83.7	250
45	0.84	5.76	0.48	5.28	0.66	89.10	240
50	0.96	6.72	0.96	5.76	0.72	97.2	230
55	0.96	7.68	1.56	6.12	0.76	102.6	220
60	1.08	8.76	2.16	6.60	0.82	110.7	210
65	1.32	10.08	2.76	7.32	0.91	122.8	200
70	1.68	11.76	3.48	8.28	1.03	139.0	190
75	1.92	13.68	4.20	9.48	1.18	159.3	180
80	2.04	15.72	4.92	10.80	1.26	170.1	170
85	2.64	18.36	5.76	12.60	1.58	213.3	160
90	4.92	23.28	6.72	16.56	2.07	279.4	150
95	8.04	31.32	7.68	23.64	2.96	399.6	140

100	4.92	36.24	8.76	27.48	3.43	463.0	130
105	2.64	38.88	10.08	28.80	3.60	486.0	120
110	2.04	40.92	11.76	29.16	3.64	491.4	(peak)
115	1.92	42.84	13.68	29.16	3.64	491.4	
120	1.68	44.32	15.72	28.60	3.57	481.95	

C. S.C.S. Hydrograph Method Storage Analysis

The S.C.S. Hydrograph Method procedure enables the designer to construct the design 10, 25 and 100-year flood hydrograph for areas greater than 400 acres. Other hydrograph methods can also be used with the approval of the Department of Public Utilities.

Basic procedures for design of a storage facility using S.C.S. Hydrograph Method is summarized as follows:

Step 1. Establish the maximum allowable release rates for the design. 10-year, 25-year and 100-year flood hydrographs following the procedures in Section 2 of this manual for an undeveloped watershed for which the storage facility is designed.

Step 2. Establish the design 10-year, 25-year and 100-year flood (inflow) hydrograph for the same watershed when fully developed.

Step 3. The storage facility and outlet work will be designed with a rate of release not to exceed

the capacity of the downstream drainage facilities.

Step 4. The facility is to be located on the downstream property line so that the runoff from the entire area can be controlled. The layout and location are such that the facility will be safe and easily maintained.

Step 5. Establish storage-outflow relationship for the detention facility based on maximum allowable release rate for the range of design return period, floods examined, site topographic conditions, allowable ponding depth, possible outlet works, and economic factors.

Step 6. The design 10-year, 25-year and 100-year flood hydrographs will be routed through the planned facility. The respective outflow rates will be examined to verify that the undeveloped peak discharge was not exceeded in each case.

Step 7. The volume of storage required is the area of the inflow hydrograph above the outflow hydrograph, of course, the area under the outflow hydrograph must be equal to the area under the inflow hydrograph.

APPENDIX-A

CHECK LIST FOR STORM DRAINAGE PLANS

A. DRAINAGE AREA MAP

1. Use 1"=100' scale for addition and 1"=100' for creeks off site and show match lines between any two or more maps. All plans are to be submitted on a 2'x3' sheet.

2. Show existing and proposed storm sewers and inlets.

3. Indicate sub areas for each alley, street and off site.

4. Use design criteria as shown in design manual.

5. Indicate zoning on drainage area.

6. Indicate runoff at all inlets, dead end streets and alleys or to adjacent additions or acreage.

7. Provide runoff calculations for all areas showing acreage, runoff coefficient, inlet time.

8. For cumulative runoff show calculations.

9. Indicate all crests, sags and street and alley intersections with flow arrows.

B. DRAINAGE FACILITIES

1. Show plan and profile of all storm sewers. The plans should show all distances, size of lines, slope of lines, invert elevation at catch basins, manholes, discharge and the natural ground elevations. In the case of existing ditches or canals, a cross-section is necessary at all discharge points.

2. 12 inch minimum cover will be required over drain lines in those area where no traffic is passing over the lines.

3. 12 inch minimum cover will be required in those areas where light traffic is passing over drainage lines; these lines are to be reinforced concrete pipe.

4. A minimum of 30 inches of cover over reinforced concrete pipe will be required in those areas where moderate to heavy traffic will pass over the drainage lines.

5. Provide inlets where street capacity is exceeded. Provide inlets where alley runoff exceeds intersecting street capacity. Use a maximum of 200 feet spacing between catch basins and high points.

6. Indicate property lines along storm sewer and show easements with dimensions.

7. Show all existing utilities in plan and profile of storm sewers.

8. Indicate existing and proposed ground line and improvements on all street, alley, and storm sewer profiles.

9. Show all hydraulics, velocity head changes gradients, computations and profile outfall with typical section and computations.

10. Show laterals on trunk profile with stations.

11. Indicate size of inlet on plan view, lateral size and flow line, paving station and top of curb elevation.

12. Indicate runoff concentrating at all inlets and direction of flow. Show runoff for all stub out, pipes and intakes.

13. Show future streets and grades where applicable.

14. Do not use bends on storm sewers or outfall. Provide good alignment.

15. Discharge storm sewers at the flow line of canals and channels with the last 10 feet at a grade not to exceed one percent.

16. Show water surface at outfall of storm sewer.

17. Where fill is proposed or trench cut in creeks or outfall ditches, specify compacted fill.

18. Where connections are made to existing storm sewer, show computations of existing system.

19. Use heavier pipes where crossing railroads, deep fill and heavy loads.

20. Show details of all connection boxes, headwalls on storm sewer, flumes or any other item not a standard detail.

21. Use a minimum size of 15"Ø on drainage lines and 12"Ø on street laterals.

22. Provide lateral profiles where utilities are crossed and show all utilities in profile.

23. Provide headwalls or grouted rip rap for all storm sewers at outfall.

24. Provide 2 feet beween top of inlet and beginning lateral hydraulic gradient. Provide 1.5 $V^2/2G$, using trunk line velocity, between top of inlet and hydraulic gradient of trunk line at lateral connection, on normal length laterals such as street and alley.

25. Indicate flow line elevations of storm sewers on profile, show percent grade. Match top inside of pipe where adjacent to other size pipe.

26. Tie storm sewer stationing with paving stations.

27. Do not flow storm water from streets into alleys.

28. On all dead end streets and alleys, show grade out for drainage on the profiles.

29. Specify concrete strength for all structures.

30. Where quantities of runoff are shown on plan or profile, indicate storm frequency design.

31. Provide sections for road, railroad and other ditches with profiles and hydraulic computations. Show design water surface on profile.

32. One hundred year flood elevations, as set by the Flood Insurance Administration, are to be shown on the Master Drainage Plan. Street cuts will be limited to a maximum of eighteen (18) inches below this elevation. However, street cuts are not to exceed twenty-four (24) inches from existing grade under any circumstances.

33. All streets must be provided with drains running along their length to eliminate bird bath.

C. PLANS AND SPECIFICATIONS

1. One set of plans and specifications must be submitted for review on any project containing sewerage and drainage facilities which will be operated and/or maintained by the Parish.

   Review on submittals will not commence until both plans and specifications have been received.

2. All plans are to be submitted on a 2'x3' sheet, with a

layout plan drawn to a one inch to a one hundred foot scale.

3. A permanent bench mark shall be located on the plans within the area of construction, to be used for both construction layout and as-built checkout. This bench mark shall be tied into a coast and geodetic bench mark.

4. Manholes shall have a minimum inside dimension of 4.0' with a one inch thick mortar coat both inside and out on brick manholes. Concrete manholes shall consist of reinforced concrete with a minimum wall thickness of five (5) inches in accordance with ASTM C76 for concrete pipe, Wall B.

5. A note shall appear on the plans referring to the use of the Department of Public Utilities specifications (both general and construction specifications) for the construction of storm sewers. Any deviation from these specifications, either general or construction, shall be in the form of "special conditions" submitted with the plans for approval.

6. When submitting the plans for approval, a letter of transmittal must accompany them, in which the owner's name and address appears, and the intended use of the property noted.

D. BRIDGES

1. Clear the lowest member of the bridge by 4 feet above design water surface.

2. Indicate borings on plans.

3. Provide soils report.

4. Show bridge sections upstream and downstream.

5. Provide hydraulic calculations on all sections.

6. Provide structural details and calculations with dead load deflection diagram.

7. Provide vertical and horizontal alignment.

E. CANALS AND DITCHES

1. Show stationing in plan and profile.

2. Indicate flow line, banks, design water surface. Show hydraulic computations.

3. Label all canals (name, size, servitude, etc.).

4. Provide drainage area map and show all computations for runoff quantities.

5. Provide cross-sections with ties to property lines and easements.

6. Specify spot sodding where fill is provided or cut made on sections.

7. Provide a minimum of 3:1 side slope from street, alley property lines or rear lot lines to flow line of creek.

8. Specify compacted fill where fill is proposed.

9. Do not use velocities greater than original stream velocities.

10. Where natural creek is to be altered or filled, a standard master plan policy for filling in flood plain will be followed.

11. Indicate adjacent alley or street elevations on creek profiles.

12. Provide minimum 10 feet servitude on the center line of closed ditches.

13. Provide minimum 10 feet servitudes on each side of open ditches.

14. Provide minimum 35 feet servitudes on each side of open canals.

15. If bulkheads along canals are necessary, details should be supplied. All bulkheads should have tiebacks.

16. Use one length of corrugated metal pipe at all system outfalls where outfall pipe protrudes from ditch bank. Outfall to canals shall be in accordance with Drawing No. furnished by the Department of Public Utilities.