STATE DEPARTMENT OF HIGHWAYS
AND PUBLIC TRANSPORTATION

ADMINISTRATIVE CIRCULAR NO. 89-83

To:	ALL DISTRICT ENGINEERS, ENGINEER-MANAGER, RESIDENT ENGINEERS & DIVISION HEADS	Date:	October 27, 1983
Subject:	Revised Chapter IX, Hydraulic Manual	Expires:	Upon Receipt of Manual Revision
Reference:		File:	D-5

BRIDGE MANUAL CHANGE LETTER

Manual Change Transmittal No. 2

Attached is a completely revised Chapter IX, PIPE STRENGTH, for the Department's Hydraulic Manual. The old Chapter IX should be removed from the manual and replaced with the attached.

The revised Chapter IX gives D-load, bedding, and trench shape requirements for concrete pipe and wall thickness requirements for steel and aluminum pipe under a large range of cover heights. Also included is a discussion on pipe that is to be jacked.

This Administrative Circular will expire upon receipt. If additional copies of Chapter IX are needed they can be obtained thru D-4.

Sincerely yours,

M. G. Goode

M. G. Goode
Engineer-Director

Attachment

DISTRIBUTION:
DISTRICT ENGINEERS
ENGINEER-MANAGER
RESIDENT ENGINEERS
DIVISION HEADS

Texas Highway Department

ADMINISTRATIVE CIRCULAR NO. 48-75

To: ALL DISTRICT ENGINEERS, ENGINEER-MANAGER AND DIVISION HEADS

Date: June 5, 1975

Subject: Hydraulics: Rainfall Intensity Coefficients

Expires: Upon Receipt of Manual Revision

Reference: Bridge Division Hydraulic Manual Revision

File: D-5

HYDRAULIC MANUAL CHANGE LETTER

Manual Change Transmittal No. 1

The attached table contains newly derived rainfall intensity coefficients to be used in the rainfall intensity formula found in Chapter II of the Bridge Division Hydraulic Manual, Second Edition. The new coefficients should replace those in Table VI, Chapter II of the Hydraulic Manual.

While there may be occasional instances of moderate variation of calculated intensities between the old and new sets of coefficients, usually the variation will be small or negligible. The attached coefficients are based upon more comprehensive observations and approximately 25 more years of record than was available when the currently used coefficients were derived.

Due to the derivation techniques used, the 'd' factor is constant for all frequencies while the 'e' factor changes for each frequency. This is the reverse of the situation in the currently used tabulation of factors.

Sincerely yours

B. L. DeBerry

B. L. DeBerry
State Highway Engineer

Attachments

DISTRIBUTION:
District Engineers
Engineer-Manager
Division Heads
Resident Engineers

HYDRAULIC MANUAL

Prepared and Compiled

by

THE TEXAS HIGHWAY DEPARTMENT

BRIDGE DIVISION

SECOND EDITION

SEPTEMBER 1970

ACKNOWLEDGMENT

This manual was prepared by the Bridge Division of the Texas Highway Department. In compiling all data presented herein various publications and individuals were consulted. Due to the number of sources, it is impossible to acknowledge them by name; however, the editors wish to convey their appreciation to all contributors, both within the Department and otherwise, who assisted in the preparation of the material. The publications of the United States Department of Transportation, Federal Highway Administration, the United States Department of Agriculture, Soil Conservation Service, the United States Department of the Interior, Geological Survey and the United States Corps of Engineers were freely consulted and their contribution is gratefully acknowledged.

INTRODUCTION

INTRODUCTION

Regardless of the type of hydraulic structure to be investigated, the design procedure always is essentially the same.

The time and involvement in any of the design procedures discussed throughout the manual should be commensurate with the relative importance of the structure.

The order in which the chapters are arranged follows the usual order of design (Figure 1) and is briefly outlined as follows:

FIELD SURVEYS: Field surveys, as discussed in Chapter 1, represent the beginning point for a proper design procedure. Items are gathered during the field survey that cannot be obtained elsewhere; therefore, particular attention is given numerous individual items necessary for the complete design of a hydraulic structure.

RUNOFF: Discharge, or rate of runoff, at a given site depends upon the hydrologic characteristics of the area. Methods for determination of discharge or runoff are discussed in detail in Chapter II. Discharge frequency requirements also are covered in this chapter.

OPEN CHANNELS: When designing a hydraulic structure that will restrict a channel or stream, existing or proposed conditions must be analysed prior to restricting the channel so that the extent of the restriction can be evaluated. When a new channel is being designed, the proposed cross-section can be analysed to determine the depth of flow that will occur at design discharge. It is essential that design highwater be established in the natural channel, an improved channel, or a proposed new channel prior to restricting the flow with a hydraulic structure. Also, the design highwater should be established at the outlet end of proposed storm sewers prior to design. Chapter III discusses methods for designing or analysing channels and ditches.

CULVERTS: After all conditions affecting the design have been gathered, a tentative decision must be made regarding type of structure required. The choice of structures may lie between a culvert or a bridge. Generally, culverts would be used for lower discharges and bridges for higher discharges. For some intermediate discharge rates, an economic comparison between a bridge and a culvert must be made. Hydraulic calculations and calculation procedures for all types of culverts may be found in Chapter IV. Also included are design procedures for special hydraulic tools such as improved entrances, outlet velocity devices, etc. which may be used to improve the operating characteristics of the culvert.

BRIDGES: Hydraulic calculation procedures for bridge waterways may be found in Chapter V. Included in this chapter are discussions of spur dikes, skewness, sizing, and multiple structure proportioning.

STORM-SEWERS: If an underground drainage system is required, information pertaining to storm-sewer design may be found in Chapter VI.

PUMP STATIONS: It is economically desirable from time to time to mechanically lift water in the outfall by means of pumps. Chapter VII discusses the procedure for sizing and analysing pump systems.

RESERVOIRS: A special treatment has been made in Chapter VIII that covers factors affecting highways either crossing or bordering reservoirs. Also, the governing design criteria and guidelines, are presented.

PIPE STRENGTH: Chapter IX is devoted to structural design directly concerned with pipe structures. Included is a discussion of conduit bedding and embankment loads.

DOCUMENTATION: A general discussion is presented in Chapter X covering certain points of preliminary submissions, design files and plan preparation with regard to drainage structures.

DESIGN AIDS: Nomographs, tables and other graphs are presented throughout the manual as valuable aids in hydraulic design. These tools allow for more rapid calculation of hydraulic problems. In addition to these aids, several computer programs which solve hydraulic problems are available and in production in the Highway Department. These computer programs and their uses are discussed more fully in another manual.

TERMINOLOGY

Throughout the manual the user will encounter certain terminology that appears to have the same meaning but actually does not. This is the case with the terms highwater, headwater, backwater, and tailwater and abbreviations of same. The decision was made not to depart from the use of these terms because they are traditionally entrenched in highway hydraulic terminology. These terms are briefly defined below although the definitions may also appear elsewhere in the manual.

a. Highwater Elevation - The calculated water surface elevation that will result from the passage of the design discharge in the natural stream at the highway site, without the restriction created by the highway, or the observed water surface elevation of an actual flood. The terms should be accompanied by either "observed" or "calculated" as the case may be. This term is usually used in connection with bridge type structures and denotes an elevation.

b. Backwater - The calculated depth of ponded water above the unrestricted highwater surface elevation, as a result of the passage of the design discharge in the natural stream with highway restrictions. This term is used in connection with bridge type structures and denotes a depth.

c. Tailwater - The calculated depth of the water at design discharge flowing in the natural stream without highway restriction, and is measured from the downstream flowline of a culvert to the water surface. This term is usually used in connection with culverts and denotes a depth, not an elevation.

d. Headwater - The calculated depth of water upstream of a culvert as a result of a design discharge flowing in the natural stream with highway restrictions, and is measured from the upstream culvert flowline to the water surface. This term usually is used in connection with culverts and denotes a depth, not an elevation.

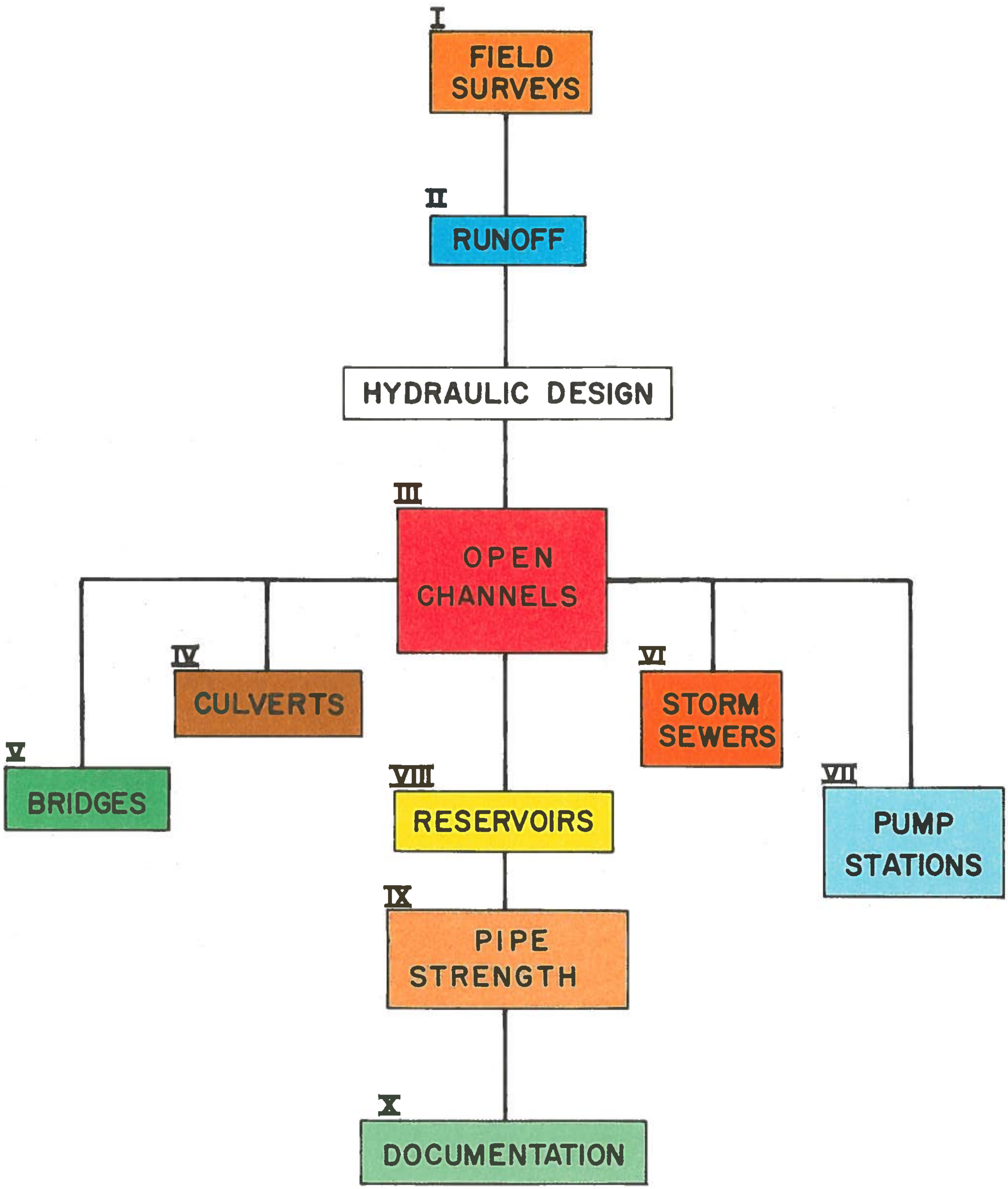
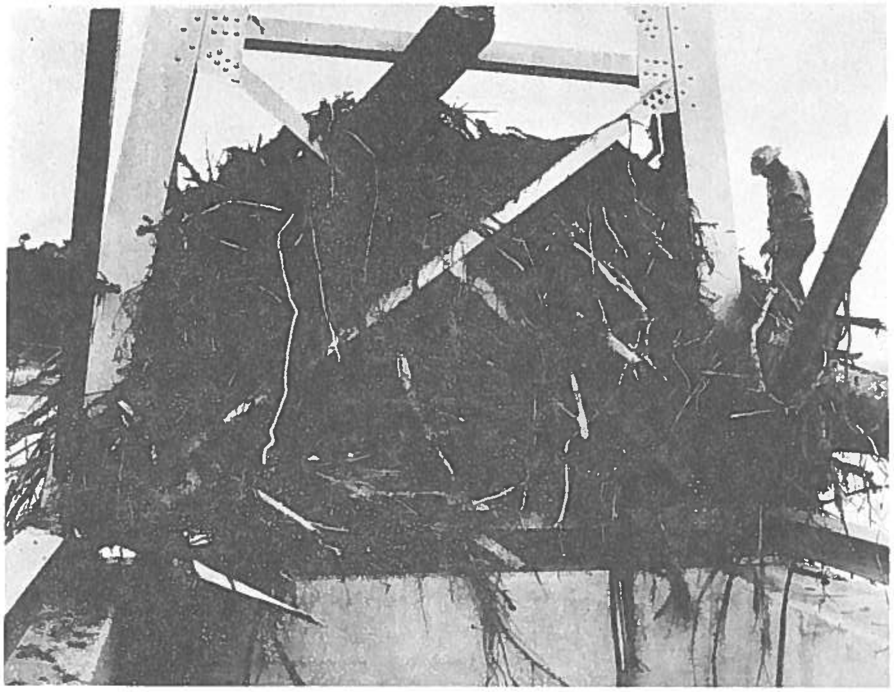
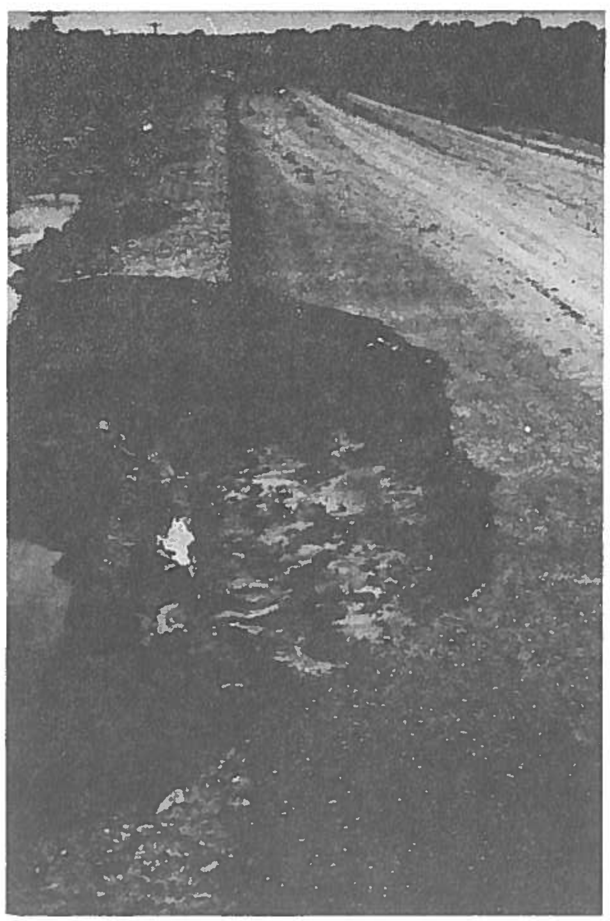

I
FIELD SURVEYS
II
RUNOFF
HYDRAULIC DESIGN
III
OPEN CHANNELS
IV
CULVERTS
V
BRIDGES
VI
STORM SEWERS
VII
PUMP STATIONS
VIII
RESERVOIRS
IX
PIPE STRENGTH
X
DOCUMENTATION

Figure-1

CHAPTER I

FIELD SURVEYS





FIELD SURVEYS

1-100. FIELD SURVEYS - GENERAL

In order to properly approach a correct hydraulic design there are several items of data which must be collected by means of field surveys. Generally these items are as follows:

1. Drainage Area Characteristics
2. Stream Course Data
3. Data on Existing Structures
4. Allowable Headwater Elevation
5. Drift
6. Allowable Outlet Velocities

Many of the items described may be obtained by a simple on-site inspection or by referring to old highway plans. Other items may require a more detailed survey but all items mentioned are essential to a proper hydraulic design. Only after a thorough study of the area and a complete collection of all information mentioned above should the designer proceed with sizing and shaping the hydraulic facility. All pertinent data and facts gathered through the field survey should be shown on the plans as explained in Chapter 10.

1-200. DRAINAGE AREA CHARACTERISTICS

1-201. DRAINAGE AREA SIZE

The size of drainage area may be determined by one of the following methods:

a. Direct field survey with conventional surveying instruments

b. Use of topographic maps together with field checks as to artificial barriers such as terraces, ponds, etc. (USGS topographic maps are available for many areas of the State in the Austin office, File D-10.)

c. Use of State Highway Planning Survey Maps

d. Aerial maps or aerial photographs (many areas have been photographed and mapped by the Soil Conservation Service)

1-202. NATURE OF TERRAIN

The nature of the terrain of the watershed should be determined by thorough field inspection of the area.

The average slope of the watershed may be classified as follows:

Classification	Slope (%)
1. flat	0 to 1
2. rolling	1 to 3 1/2
3. hilly	3 1/2 to 5 1/2
4. mountains	5 1/2 +

The above table may be helpful but on site field inspection is necessary.

1-203. SOIL CLASSIFICATION

The soil included within the limits of the watershed should be classified as follows:

1. very slowly permeable - includes clay with high swelling properties and some shallow soils with practically impermeable subhorizons near the surface.

2. slowly permeable - comprises shallow soils and soils containing considerable clay and colloid but to a lesser degree than soils under (1). These soils have below average infiltration rate after presaturation.

3. permeable - includes deep sandy soils and loess with very little silt and clay with above average infiltration rate after thorough wetting.

1-204. SOIL COVER

The soil cover should be classified as follows:

1. cultivated
2. pasture
3. timber
4. terraced for cultivation

Again, on site inspection is necessary for a proper determination of soil cover classification.

1-205. LAND USE

Possible future development of the drainage area or watershed must be taken into account. Prediction as to development of an area for 20 years or so hence is often difficult. However, an estimation of future development should be made by interviewing local owners, developers, officials, etc.

1-300. STREAM COURSE DATA

Of prime importance to a good hydraulic design is the acquirement of a complete understanding of the nature of the natural stream course.

1-301. STREAM MEANDERS

The meander of the main stream and the nature of flow over the entire flood plain should be determined. Data concerning the meanders and flood plain is necessary to determine the feasibility of possible channel changes or improvements in the vicinity of the crossing, or to establish the general skewed condition with respect to the centerline of the highway. The meanders are preferably shown on the plans by contours. They should be reasonably accurate for some distance on each side of the crossing and within the limits of possible channel changes.

1-302. ROUGHNESS COEFFICIENTS

Roughness coefficients, ordinarily in the form of Manning's "n" values, should be estimated for the entire flood limits of the stream. A tabulation of Manning's "n" values with descriptions of their applications can be found in Chapter III. These values depend upon the nature of the channel lining with respect to vegetation, soil roughness and size, and general roughness of the waterway.

1-303. STREAM PROFILE

The profile of the stream should be extended sufficiently upstream and downstream to determine the average slope and to encompass any channel changes. A minimum distance of 500 feet both upstream and downstream (for a total of 1,000 feet) or a distance equal to twice the length of the structure, whichever is greater, should be used in determining the stream profile.

1-304. STREAM CROSS-SECTIONS

Usually, downstream conditions such as roughness, slope, shape and size of cross-section control the highwater and channel velocity. However, any section in the vicinity of the highway crossing which may be considered as typical will serve as well. This typical cross-section will be subsequently used in calculations of a stage-discharge curve; however, more than one cross-section may be required depending on choice of methods outlined in Chapter III. All channel sections should be taken at right angles to the anticipated direction of general stream flow at flood stages.

1-305. FLOOD STAGES

Past flood stages are necessary and very helpful. Much information can be obtained from THD personnel, city and county personnel, and local residents. In addition, much can be learned about the behavior patterns of a stream by watching it flow at flood stage. Such things as direction of current with relation to low flow channel, estimated velocity, drift (amount and size), erosion, the drop from upstream side to downstream side of structure, the highest stage and date of occurrence can be pertinent information to later channel analysis or design. All observations will be more useful if the discharge is determined for the observed stage. Generally, the discharge can be determined only by recognized methods such as current meter measurement, slope-area determinations, etc. To take only a single stream cross-section and a flood stage leaves much to be desired from the standpoint of accuracy. Photographs of the flood can provide useful information for future studies at a given site.

Observed flood flow data across roadway sections can be used to approximate the discharge over the roadway by using the following weir formula: (Figure 1)

$$Q = CLH^{3/2}$$

where

Q = Discharge over roadway

C = Coefficient of discharge and approximates 3.02 (use $C = 3.02$)

L = Length of overflow associated with average depth along roadway center line

H = Depth of flow above roadway estimated just upstream

Downstream conditions should be noted, because, for the roadway to be controlling the flood there should be a drop in the water surface downstream of the roadway.

When a structure exists under the roadway at the site where the overflow occurred, additional calculations are necessary to determine the flow through the structure. This is done by determining the headwater and the tailwater for the structure, and calculating the flow through the culvert from procedures in Chapter IV.

1-306. DOWNSTREAM CONTROL

Any ponds or reservoirs, along with their spillway elevations and design levels of operation, should be noted as their backwater effect may bear directly on the proposed structure. Also, any downstream confluence of two or more streams should be studied to determine the backwater effects on the proposed site.

1-307. UPSTREAM CONTROLS

Upstream control of runoff in the watershed should be noted. Often, conservation and/or flood control dams in the watershed may be used to distinct advantage by, in effect, eliminating some of the drainage area under consideration. Capacities and operation designs for these fixtures should be obtained. The Soil Conservation Service, the Corps of Engineers, the Bureau of Reclamation, consulting engineers, reservoir sponsors, etc. often have complete reports concerning the operation and design of any proposed or existing conservation and/or flood control dams.

1-308. BORROW PITS IN CHANNELS AND FLOOD PLAINS

Existing borrow pits and proposed borrow pits in channels and flood plains can adversely affect the roadway and bridges that cross the same stream in the near vicinity to the said borrow pits. Any pits should be noted, but the borrow pits within 500 feet upstream or downstream of the roadway should be carefully analyzed and evaluated as to their affect on the flood distribution and as a potential source of stream scour problems.

1-400. DATA ON EXISTING STRUCTURES

The location, size, description, observed flood stages, and channel section of existing structure on the water course should be secured in order to determine their capacity and their effect on the stream flow. Any structures, downstream or upstream, which may cause backwater or retard normal stream flow should be noted. Also, the manner in which existing structures are and have been functioning with regard to scour, overtopping, etc. should be noted. This data should include span lengths and type of piers which generally may be secured from existing structure plans.

Sometimes the Engineer has the opportunity to observe the structure under extreme flood conditions. When possible, photographs of the flood action in the vicinity of the structure should be taken for use in future studies. If the difference in water surface elevations between the upstream and downstream end of the structure is measured, a fairly accurate estimate of the discharge can be made.

1-500. ALLOWABLE HEADWATER ELEVATION

Improvements, property use, etc. adjacent to the proposed site may directly effect the allowable headwater. Elevations of these improvements or fixtures involved in determining allowable headwater should be noted. The allowable headwater may be based on freeboard requirements to the highway itself, (but not necessarily so.)

1-600. DRIFT

The probable nature, size and volume of drift should be noted in order to determine the amount of freeboard that will be required for the proposed structure.

1-700. OUTLET VELOCITIES

Scour or erosion characteristics inherent to the soil in the vicinity of the proposed structure should be noted. These characteristics will be used in determining allowable outlet velocities and structure type and geometry.

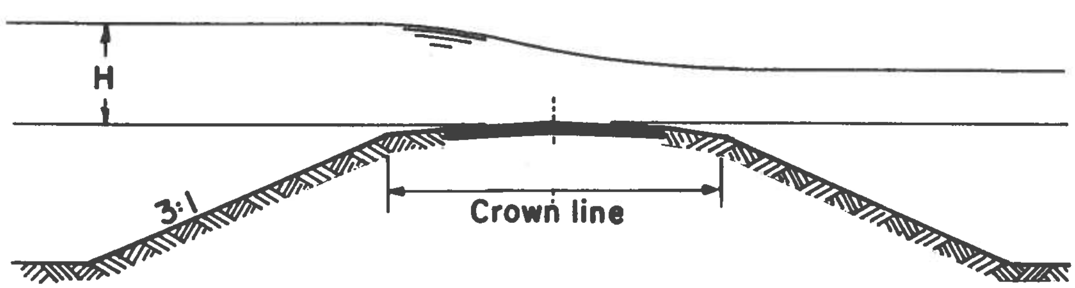
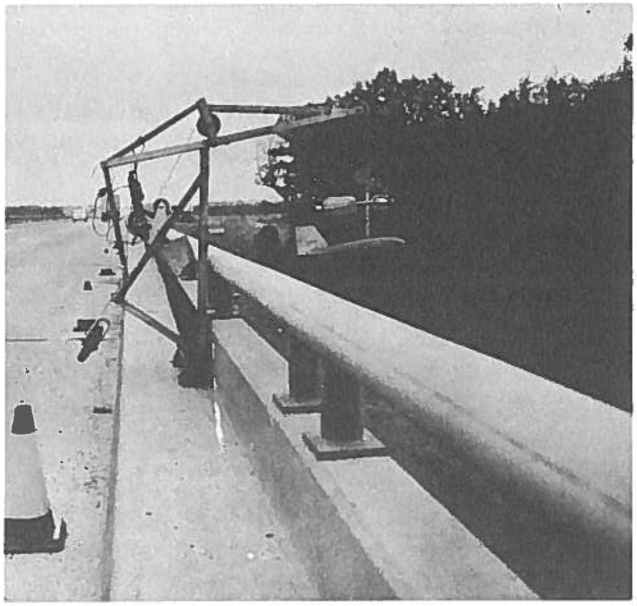
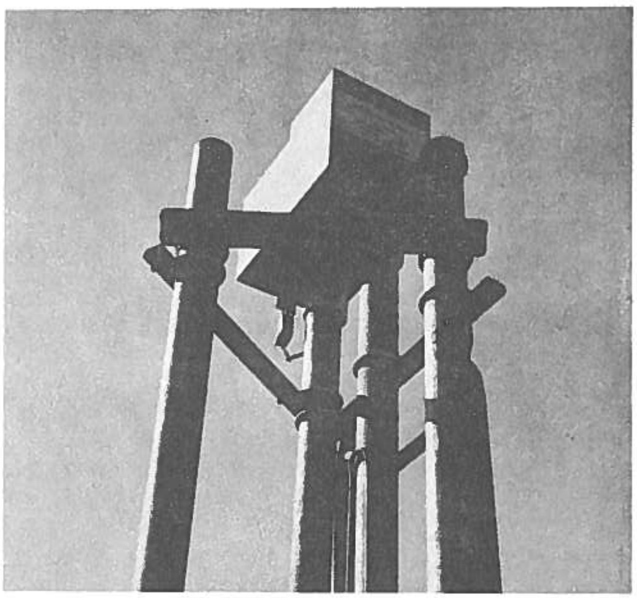


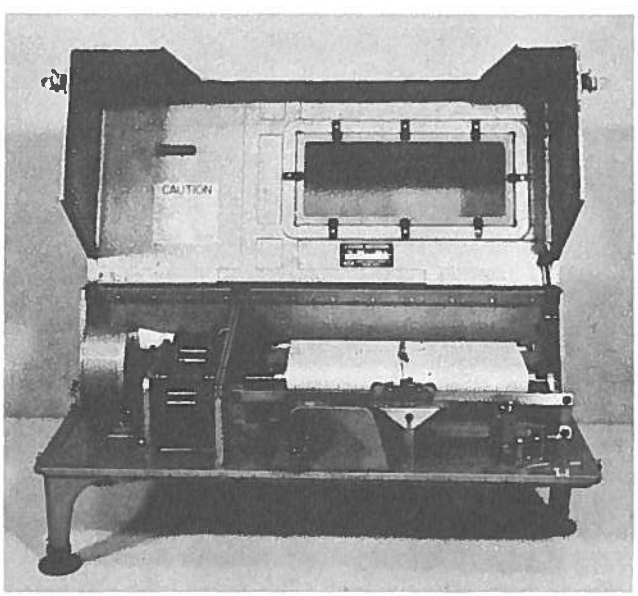
Figure I.

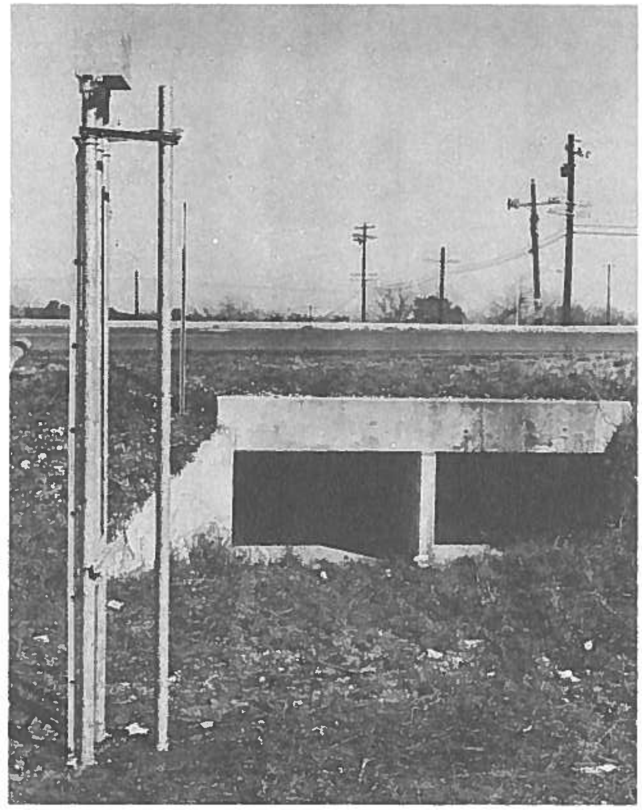
CHAPTER II

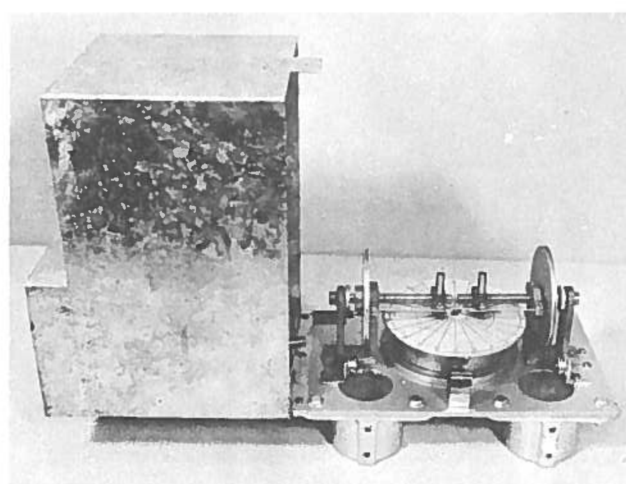
RUNOFF






CAUTION





RUNOFF

2-100. GENERAL

Prior to making any hydraulic design, the amount of runoff, or discharge, that a given area is capable of yielding at a given site must first be determined. The discharge quantity is the "hydraulic load" that the hydraulic structure is to accommodate; therefore, this discharge quantity governs the size of the hydraulic structure.

The cost of drainage structures, whether they are bridges, culverts or storm sewers, constitutes a large portion of the overall cost in highway construction. It is imperative that these structures perform efficiently and yet be designed as economically as possible.

2-101. SOURCES OF INFORMATION

There are various sources of reliable hydrological information in addition to the charts and publications contained in this chapter. Some additional sources are listed below:

STREAMFLOW DATA:

a. The U. S. Geological Survey has complete streamflow data available for each gaging station site, as well as data at miscellaneous sites where flood measurements have also been made.

b. U. S. Corps of Engineers

c. Texas Water Development Board

d. Texas Highway Department data on small watersheds (less than 20 sq. mi.)

MAPS:

a. Texas Highway Department Planning Survey Maps

b. U. S. Geological Quadrangles

c. U. S. Soil Conservation Service Aerial Photography

2-200. DESIGN FREQUENCY

Since it is not economically feasible to design a structure for the maximum runoff a drainage area is capable of yielding, a design frequency must be established. The storm frequency (recurrence interval or return period) may be defined as the average interval of time within which the given flood will be equaled or exceeded once. (American Society of Civil Engineers, 1953, page 1221.) Therefore, a storm with 50 year frequency or recurrence interval would be expected to recur within an approximate 50 year period following its last occurrence. Note that it does not have to recur at the end of the 50 year period but within the period, i.e. the storm has a 2% chance of occurring in any one year, but if it does, it does not mean that it will not occur again for the remainder of the 50 year period. The chance for such a flood remains at 2% in any one year. Consequently, the recurrence chance for a 5 year storm is 20%, a 10 year storm is 10%, a 25 year storm is 4%.

Table I has been prepared to show generally the design frequencies used for the various hydraulic structures on certain highways. The frequency or chance of occurrence on which the design is to be based should be made only after due consideration of several factors including importance of the highway, land use, etc. Possible land use should be considered for 20 years into the future.

TABLE I

Type of Structure	Design Frequency in Years: Interstate & Controlled Access Hwys. Main Lanes	Design Frequency in Years: Other Highways & Frontage Roads
Inlets and Sewers	10	2 to 5
Inlets for Depressed Roadways	50	2 to 10
Culverts	50	2 to 10
Small Bridges	50	10 to 50
River Crossings	50	10 to 50

2-300. METHODS OF DISCHARGE DETERMINATION

Drainage areas smaller than 20 square miles differ materially in a hydrological manner from areas of 100 square miles or more. Factors such as degree of soil moisture content, soil type and depth, land use, shape and size of watershed have marked effects on peak rates of run-off quite distinct from those of larger areas.

It is granted at this point that the discharge determination is one of the most nebulous to make and much work is being done to increase the accuracy of the discharge determination. Regardless, there are, at present, four methods recommended below that each designer shall use. If there is reason to vary from the procedures discussed below, a complete documentation of those reasons shall accompany the hydraulic design to which the variance pertains.

In order of preference, the four recommended procedures presently employed are outlined below, and the selection of the proper method is accomplished by the process of elimination with the decisive parameter being the drainage area size.

2-301. TEXAS WATER COMMISSION BULLETIN 6311

The primary tool for discharge determination for most areas exceeding 20 square miles is the information contained in Texas Water Commission Bulletin 6311, "Floods in Texas, Magnitude and Frequency of Peak

Flows". This publication has priority over all methods for determining discharge in large (over 20 square miles) areas in Texas. As may be seen in the publication, there are still some sections of Texas where a correct discharge determination is difficult, if not impossible. For sites that fall on an area or region boundary line, the selection of the proper area or region should be based upon the area and region in which the drainage area is located. Without doubt there will be a few sites for which certain deviation from the published methods must be made. When a deviation is deemed proper, complete documentation must be made. This documentation will consist of compiling a statement of all factual information which contributed to the decision to deviate from the established method as published.

The Texas Water Commission Bulletin 6311 "Floods in Texas, Magnitude and Frequency of Peak Flows" contains a method on pages 1 through 30 for estimating runoff based on regional analysis plus curves, limitation, etc. The Bulletin 6311 method divides the State into 8 hydrologic areas, each area having a minimum size that varies from 20 square miles to 100 square miles but never less than 20 square miles. If the designer has a drainage area less than the minimum size for a given hydrologic area, then the Bulletin 6311 method is eliminated and stream gaging data should be investigated.

2-302. STREAM GAGE DATA

As a general rule, stream gaging stations are installed to measure discharge, etc. on large drainage areas of over 100 square miles. However, many stations exist at locations where drainage areas are somewhat less than 100 square miles. There are instances when a gaging station may yield data that is instrumental in a discharge determination, or the data may be used to supplement Bulletin 6311. However, it should be noted that the Bulletin 6311 method usually takes precedence over a single stream gaging station because the single station has only recorded what has happened at a single site to date. The Bulletin 6311 method by the nature of the regional evaluation considers all stream gage data for similar hydrologic areas and then determines the discharge a certain drainage area is capable of yielding based on what the other similar hydrologic areas have yielded. This statistical method yields more probable results.

If the Bulletin 6311 method of determining discharge does not apply, then stream gage data should be sought. If the drainage area being studied has no stream gage, and there is no gaged area that will favorably compare hydrologically, then stream gage data is eliminated and the rational method should be investigated.

The two situations where stream gage data will be useful are as follows:

a. One or more stream gaging stations located within the area.

b. The area itself is not gaged but stream gaging stations are located in nearby comparable areas with similar hydrologic characteristics so that the records can be transposed and flow data obtained in this manner.

The determination of the discharge for Cases (a) and (b) requires the use of Flood Frequency Curves which may be derived by either an annual or partial duration series. The annual flood frequency series is the simplest method to apply and is used in this manual. The required discharge data for preparing these curves can be found in Texas Water Commission Bulletin 6311, from U. S. Geological Survey yearly water supply papers, or from computer output for each gage. The computer output is prepared by D-5 personnel from U. S. Geological Survey data. These printouts are updated each year and each District Engineer is furnished an output print on all gages within each district.

Step 1 - The proper gaging station or stations in the vicinity of a proposed structure should be selected. A record of 10 years or more is considered desirable, but records as short as 5 years may be used if no other data is available. The influence of usual storage or artificial controls should be properly evaluated by adjusting the recorded discharge to conditions of uncontrolled flow. Only the latest revised records should be considered.

Step 2 - As shown on Tables II, III and IV the peak stages and discharges for each water year are listed in chronological order. A water year extends from October 1 through September 30. Partial yearly records are not to be included.

Step 3 - After the discharges have been listed they are then numbered in order of their magnitude; that is, the highest discharge for a particular gaging station is assigned the number 1, the next highest 2, and etc. The numbering system will indicate the relative distribution of floods during a given period of years.

Step 4 - The recurrence intervals are next computed from the formula

$$\text{Recurrence Interval} = \frac{N+1}{M}$$

where

N = Number of years of record

M = Relative magnitude of floods beginning with the highest which is 1

It is apparent from the above formula that the greatest known flood will plot at a recurrence interval equal to one plus the number of years of record; however, in computing recurrence intervals by any formula there are times when the computations must be modified. For example, the highest flood in a 40-year record should not always have a recurrence interval of 41 years. There is frequently additional historical knowledge which might indicate, for example, that the highest flood in a 40-year record is the highest in 300 years. Its plotting position should then be computed as though it were the highest in a series of 300 items or as 301 instead of 41 years. The second flood (in 40 years) would then be computed as usual.

Step 5 - The recurrence interval versus discharge for each water year is next plotted as shown in Figures 1, 2, and 3. These points are then connected by a line using as many of these points as possible. More weight should be given to the large grouping of points rather than single plots of extremely high discharges.

If the bridge site is located on a stream having an established gaging station with adequate record (Case a), the determination of the design discharge is a simple matter. First construct a Frequency versus Discharge Curve for the station as previously discussed and determine the design discharge for the desired frequency at the gaging station. This discharge must now be projected up or down stream of a selected bridge site.

This is accomplished by multiplying the gage discharge by the direct ratio of the respective drainage area raised to the 2/3 power. Thus for a bridge site on Richland Creek having 500 square miles of contributing area, a 25-year discharge would be

$$62{,}000\left(\frac{500}{737}\right)^{2/3} = 47{,}926 \text{ cfs}$$

(Projected Upstream)

where

62,000 = Discharge at the gaging station for a 25-year frequency flood. (See Figure 2.)

500 = Contributing area at the bridge site.

737 = Contributing area at the gaging station.

Note: Tables for 2/3 powers are in Chapter III.

On certain streams where no gaging stations are in existence, records of other nearby gaging stations may be utilized provided all areas have similar hydrologic characteristics. The discharge for such an ungaged stream may be determined by the transposal of records (Case b.). Reference is made to Figures 1, 2 and 3 showing Discharge versus Frequency for three gaging stations. For example, consider a location with a drainage area of 450 square miles having similar run-off characteristics of the above three mentioned stations. To determine the 25-year discharge, first project the 25-year discharge of each station outlined in Case (a) and average the results. The computations are as follows:

Cedar Creek $38{,}000\left(\frac{450}{734}\right)^{2/3} = 27{,}322$ cfs

Chambers Creek $45{,}000\left(\frac{450}{971}\right)^{2/3} = 26{,}820$ cfs

Richland Creek - $62{,}000\left(\frac{450}{731}\right)^{2/3} = 45{,}074$ cfs

Total = 99,216 cfs

$$\text{25-year Design Discharge} = \frac{99{,}216}{3} = 33{,}072 \text{ cfs}$$

2-303. RATIONAL METHOD

The rational method is accepted as a means of determining the discharge from drainage areas up to approximately 5 square miles. If the drainage area in question is greater than 5 square miles, then the rational method is usually eliminated, and the Regional Curves should be investigated as discussed in Section 2-304.

The rational method is based on the principle that the maximum rate of run-off from a given drainage area for an assumed rainfall intensity occurs when all parts of the area are contributing to the flow at the point of discharge.

The rational method is especially applicable to urban areas or developed sections of right of way where the influence of subsections of varying hydrologic characteristics may be weighted in the final analysis.

The method is expressed by the formula Q = CIA.

where

Q = Discharge in cfs

C = Run-off coefficient which varies with the topography, land use and moisture content of the soil at the time the run-off producing rainfall occurs and the existence or non-existence of well defined drainage channels. In selecting the run-off coefficient from Table V, consideration should be given to possible future land developments that might take place in the next 20 years.

I = Rainfall intensity in inches per hour =

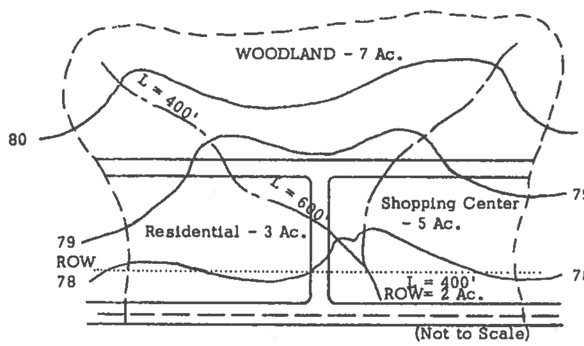
$$\frac{b}{(t+d)^e}$$

where b, d and e are constants listed in Table VI. The time of concentration or "t" is the time in minutes required for the run-off to flow from the most remote point in the drainage area to the structure site, and is equal to the average run-off velocity in feet per minute of flow divided into the distance in feet along the course. In certain cases it is advisable to divide the calculations in segments of overland flow and channel flow to arrive at a more realistic time of concentration. Average velocities may be obtained by actual measurements, by the use of Manning's formula or by using the average values for the velocity of various conditions of topography and land use listed in Table VII.

Rainfall intensities may also be determined from the nomograph at Figure 4, which is also based on the above mentioned b, d, and e constants listed in Table VI.

Example:

Given:



Location - District 7
Tom Green County
Design Frequency - 10 years
Drainage Area - 17 Acres

Required: Discharge for a 10 year frequency.

Solution:

Step 1 - Divide the drainage area into sections having similar run-off characteristics and determine the coefficient of run-off from Table V for each section. Multiply each sectional area by the corresponding coefficient c and add the products to arrive at the total CA.

C(Woodland)	= 0.18 x 7 acres = 1.26
C(Shopping Center)	= 0.90 x 5 acres = 4.50
C(Residential)	= 0.50 x 3 acres = 1.50
C(ROW)	= 0.70 x 2 acres = 1.40
	Total CA = 8.66

Step 2 - Calculate the time of concentration by determining the longest route the run-off will follow and divide this length by the average run-off velocity as determined from Table VII. The time of concentration will be affected by the slope and type of watershed. Therefore, a time of concentration must be calculated for each section that the natural water course passes through.

Time of Concentration (Woodland) $= \frac{400}{2x60} = 3.33$

Time of Concentration (Residential and Shopping Center) $= \frac{600}{5x60} = 2.0$

Time of Concentration (ROW) $= \frac{400}{2x60} = 3.33$

Total = 8.66

use t = 10 minutes

(generally, the time of concentration used should be no less than 10 minutes)

Step 3 - Determine I_{10} from the formula

$$I_{10} = \frac{b}{(t+d)^{e}}$$

$$= \frac{130}{(10+22)^{0.875}} = 6.3 \text{ inches per hour}$$

or use nomograph, Figure 4.

Step 4 - Calculate the discharge from the above information.

$$Q_{10} = CA \times I$$

$$= 8.66 \times 6.3 = 54.5 \text{ cfs}$$

2-304. REGIONAL CURVES

If the drainage area being studied is too small for the Bulletin 6311 method to apply, and there is no stream gage data, and the size is greater than five square miles, then use the applicable curve from Figures 6, 7, or 8. These curves, and the classification of regions, Figure 5, were prepared with the assistance of Mr. G. G. Commons, Texas Board of Water Engineers, in about 1946. They represent an attempt at a regional analysis and although the curves have served well where there was nothing else, they are now generally accepted as yielding higher than expected discharges for given frequencies. Interpolation of curves is usually allowable if the area in question does not fall within one of the outlined area limits.

To determine the 25-year discharge for a drainage area of 10 square miles in Wichita County, select the "North Texas Discharge Frequency Curve" from Figure 5. Enter Figure 8 with the drainage area of 10 square miles on the abscissa and determine Q_{25} = 3,500 cfs on the ordinate.

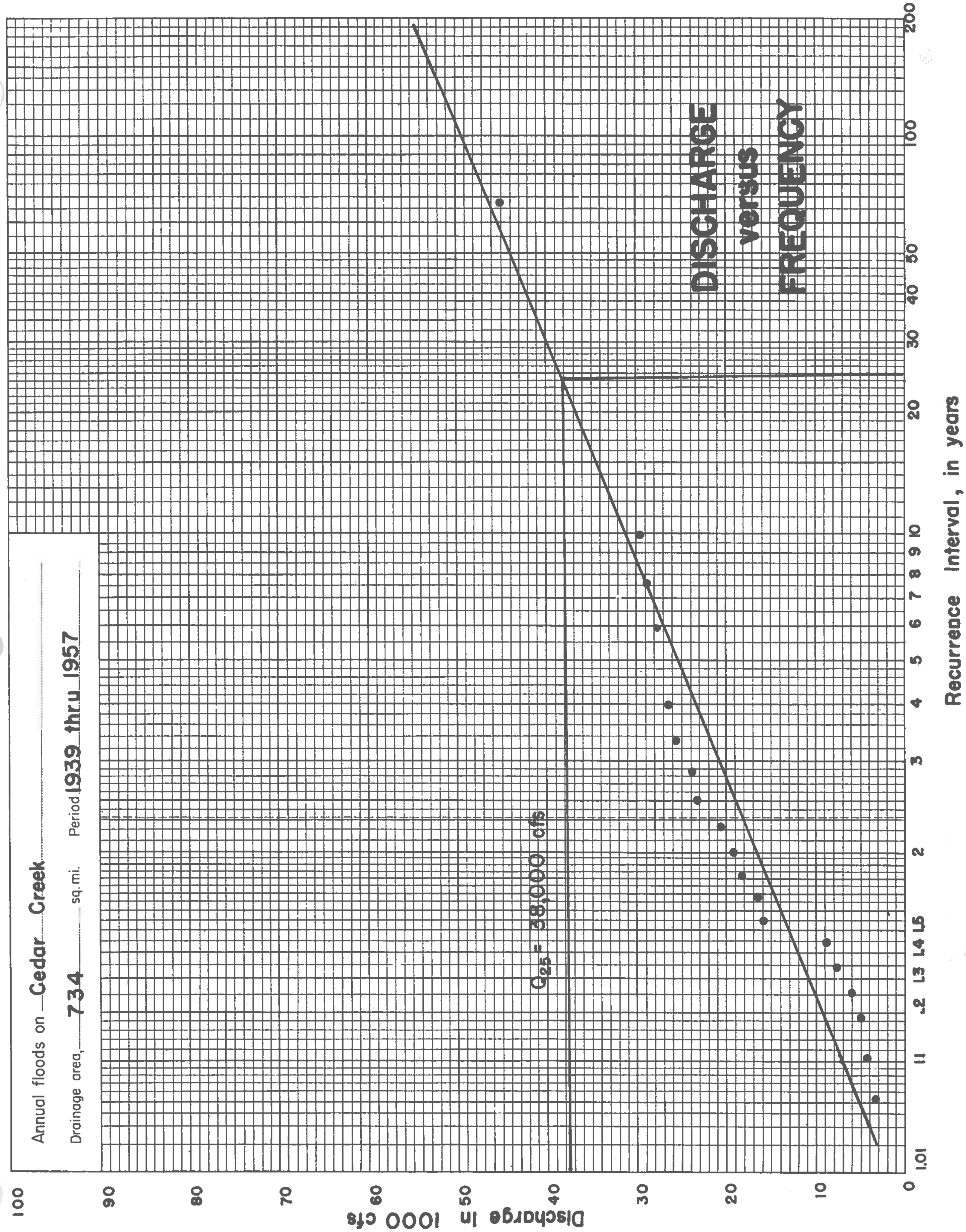

Annual floods on Cedar Creek
Drainage area, 734 sq.mi.
Period 1939 thru 1957
Q_{25} = 38,000 cfs
DISCHARGE versus FREQUENCY
Recurrence Interval, in years
1.01 1.1 1.2 1.3 1.4 1.5 2 3 4 5 6 7 8 9 10 20 30 40 50 100 200
Discharge in 1000 cfs
0 10 20 30 40 50 60 70 80 90 100

Figure 1

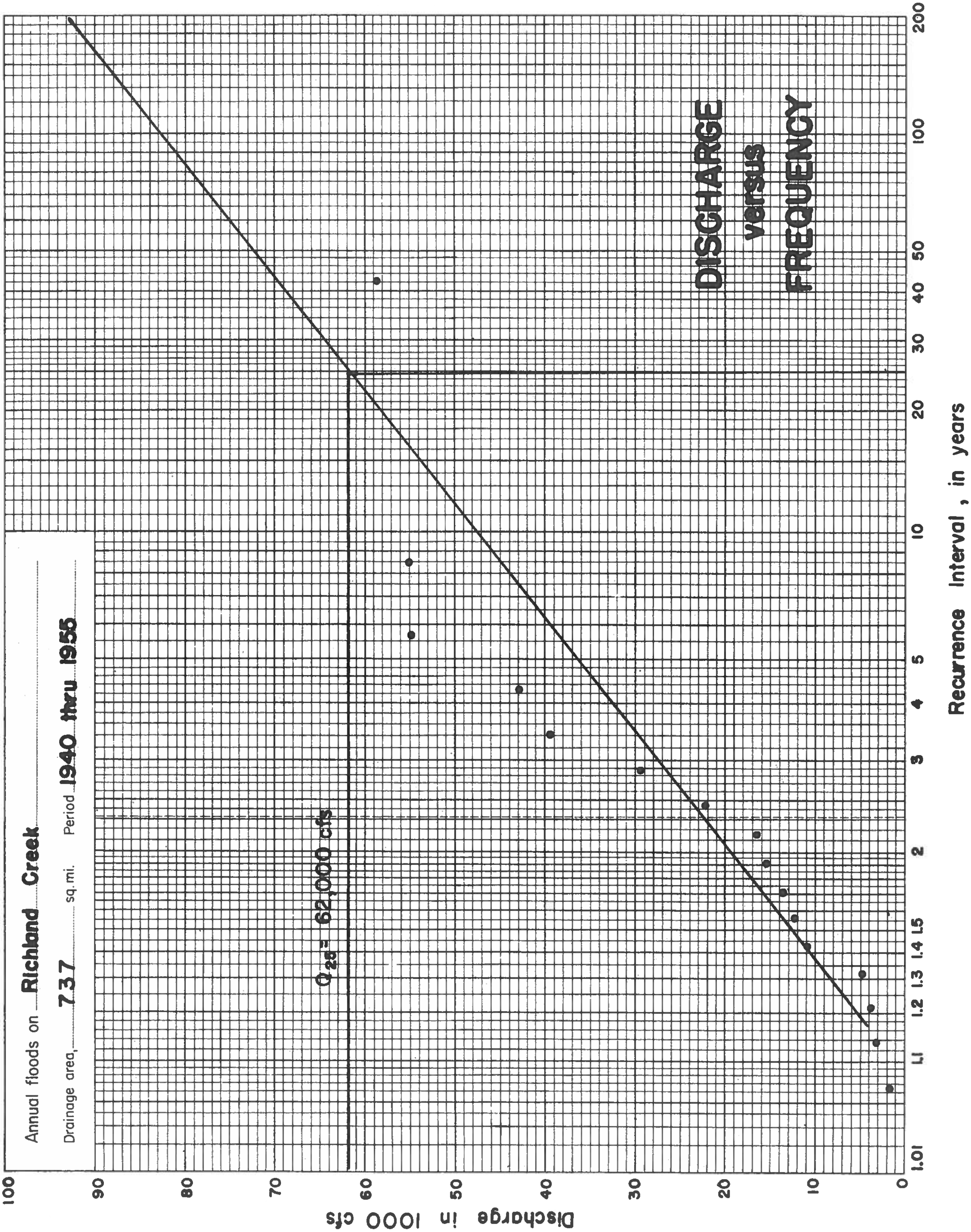
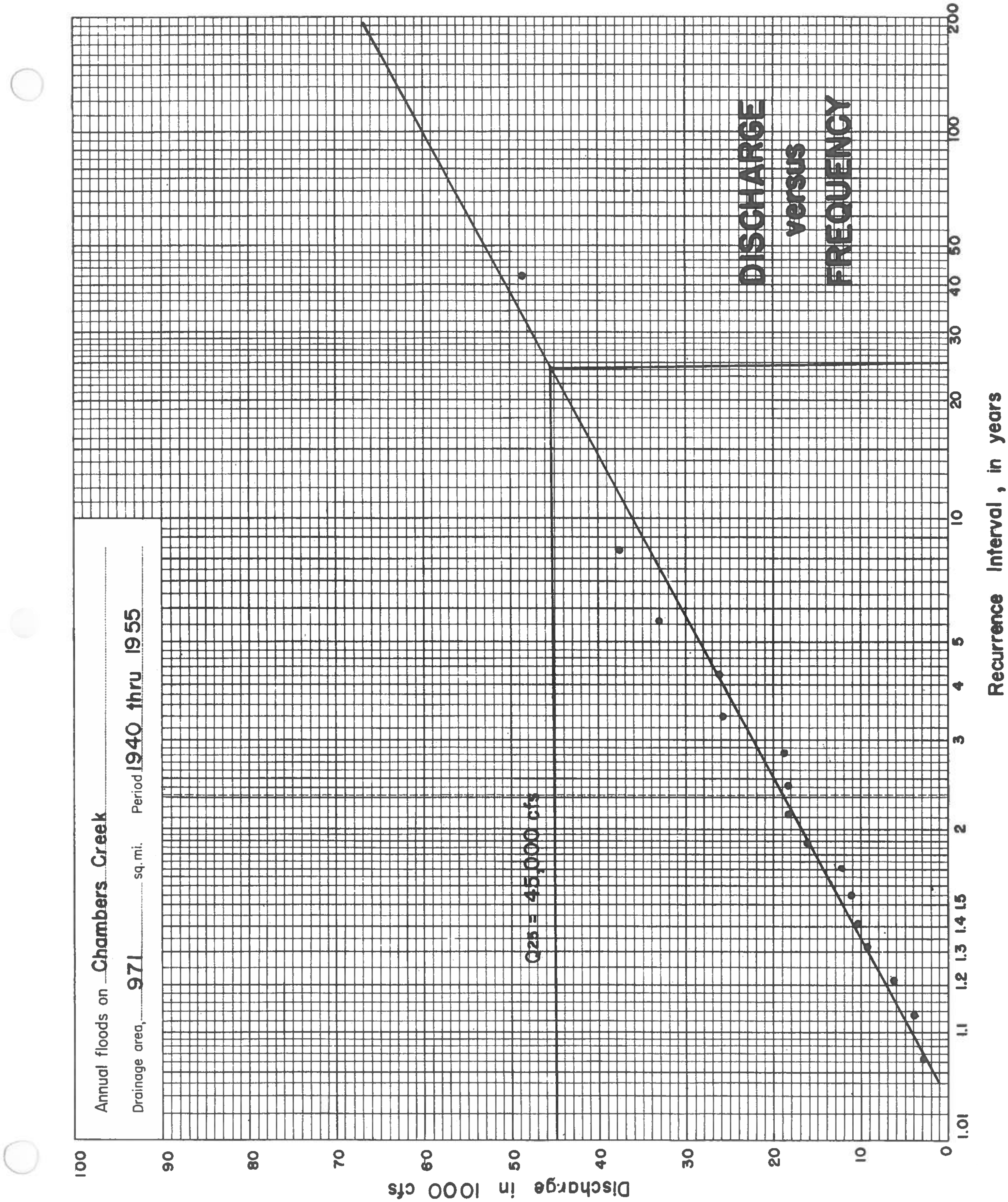

DISCHARGE
versus
FREQUENCY
Annual floods on Richland Creek
Drainage area, 737 sq. mi.
Period 1940 thru 1955
Q25 = 62,000 cfs
Recurrence Interval, in years
Discharge in 1000 cfs
1.01
1.1
1.2
1.3
1.4
1.5
2
3
4
5
10
20
30
40
50
100
200
0
10
20
30
40
50
60
70
80
90
100

Figure 2



DISCHARGE versus FREQUENCY
Annual floods on Chambers Creek
Drainage area, 971 sq.mi.
Period 1940 thru 1955
Q25 = 45,000 cfs
Discharge in 1000 cfs
0
10
20
30
40
50
60
70
80
90
100
Recurrence Interval , in years
1.01
1.1
1.2
1.3
1.4
1.5
2
3
4
5
10
20
30
40
50
100
200

Figure 3

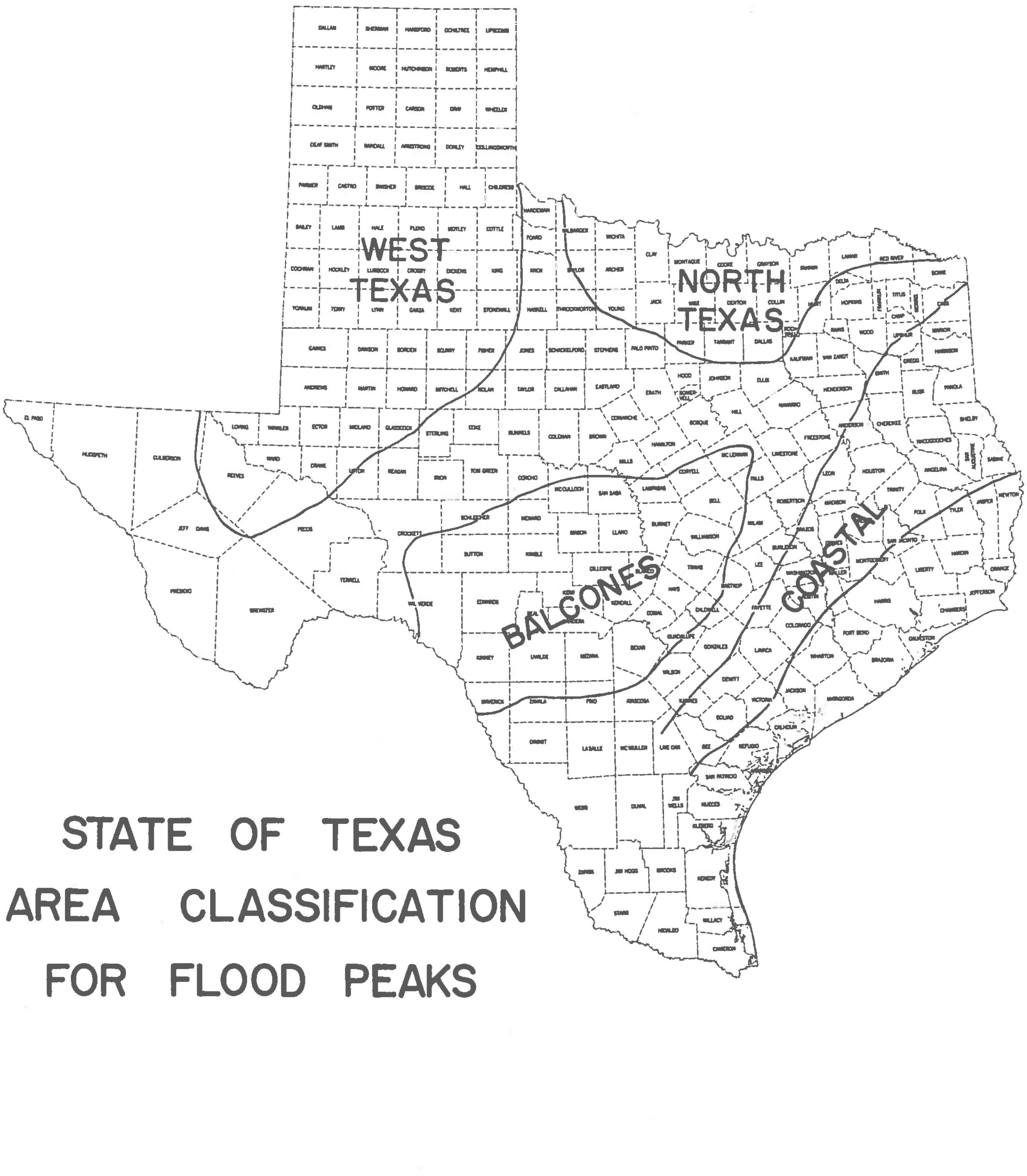

WEST TEXAS
NORTH TEXAS
BALCONES
COASTAL
STATE OF TEXAS
AREA CLASSIFICATION
FOR FLOOD PEAKS

Figure 5

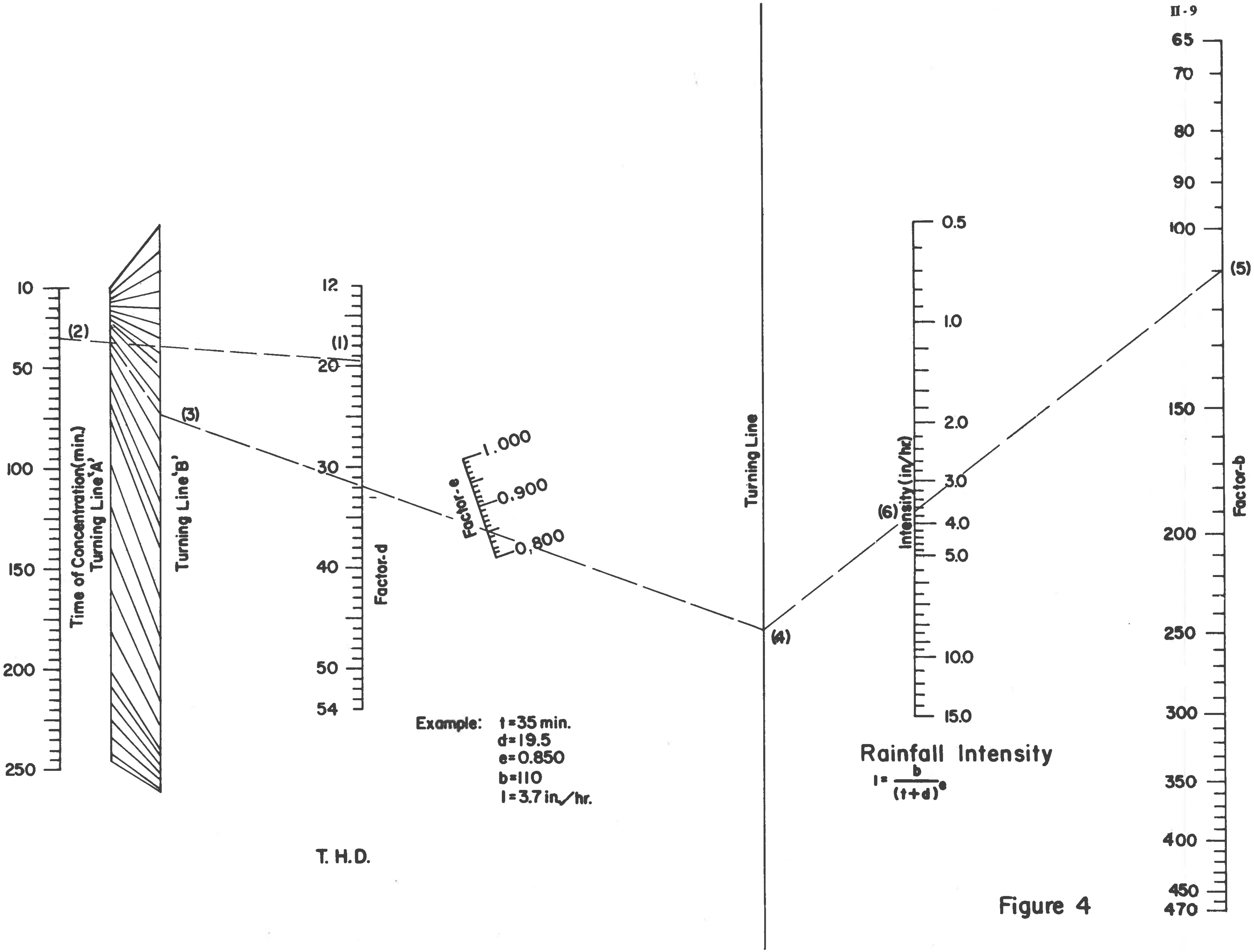

Time of Concentration (min.)
10
50
100
150
200
250
(2)
Turning Line 'A'
Turning Line 'B'
(3)
12
(1)
20
30
40
50
54
Factor-d
Factor-e
1.000
0.900
0.800
Turning Line
(4)
Intensity (in/hr)
0.5
1.0
2.0
3.0
4.0
5.0
10.0
15.0
(6)
Factor-b
65
70
80
90
100
(5)
150
200
250
300
350
400
450
470
Example: t=35 min.
d=19.5
e=0.850
b=110
I=3.7 in./hr.
Rainfall Intensity
$I = \frac{b}{(t+d)^e}$
T. H. D.

Figure 4

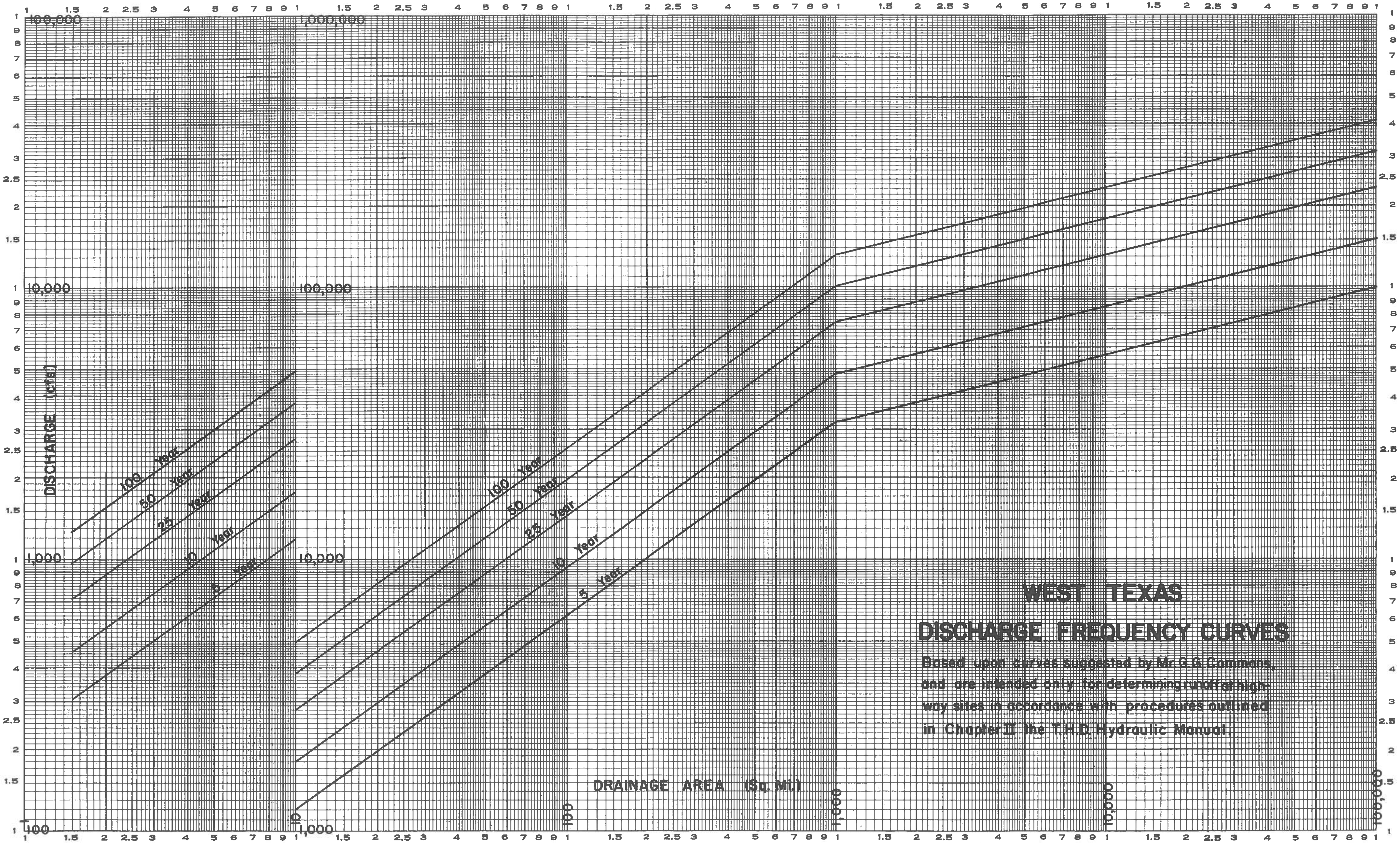

WEST TEXAS
DISCHARGE FREQUENCY CURVES
Based upon curves suggested by Mr. G. G. Commons, and are intended only for determining runoff at highway sites in accordance with procedures outlined in Chapter II the T.H.D. Hydraulic Manual.
DISCHARGE (cfs)
DRAINAGE AREA (Sq. Mi.)
100 Year
50 Year
25 Year
10 Year
5 Year
100,000
1,000,000
10,000
100,000
1,000
10,000
100
1,000
10
100
1,000
10,000
100,000

Figure 6

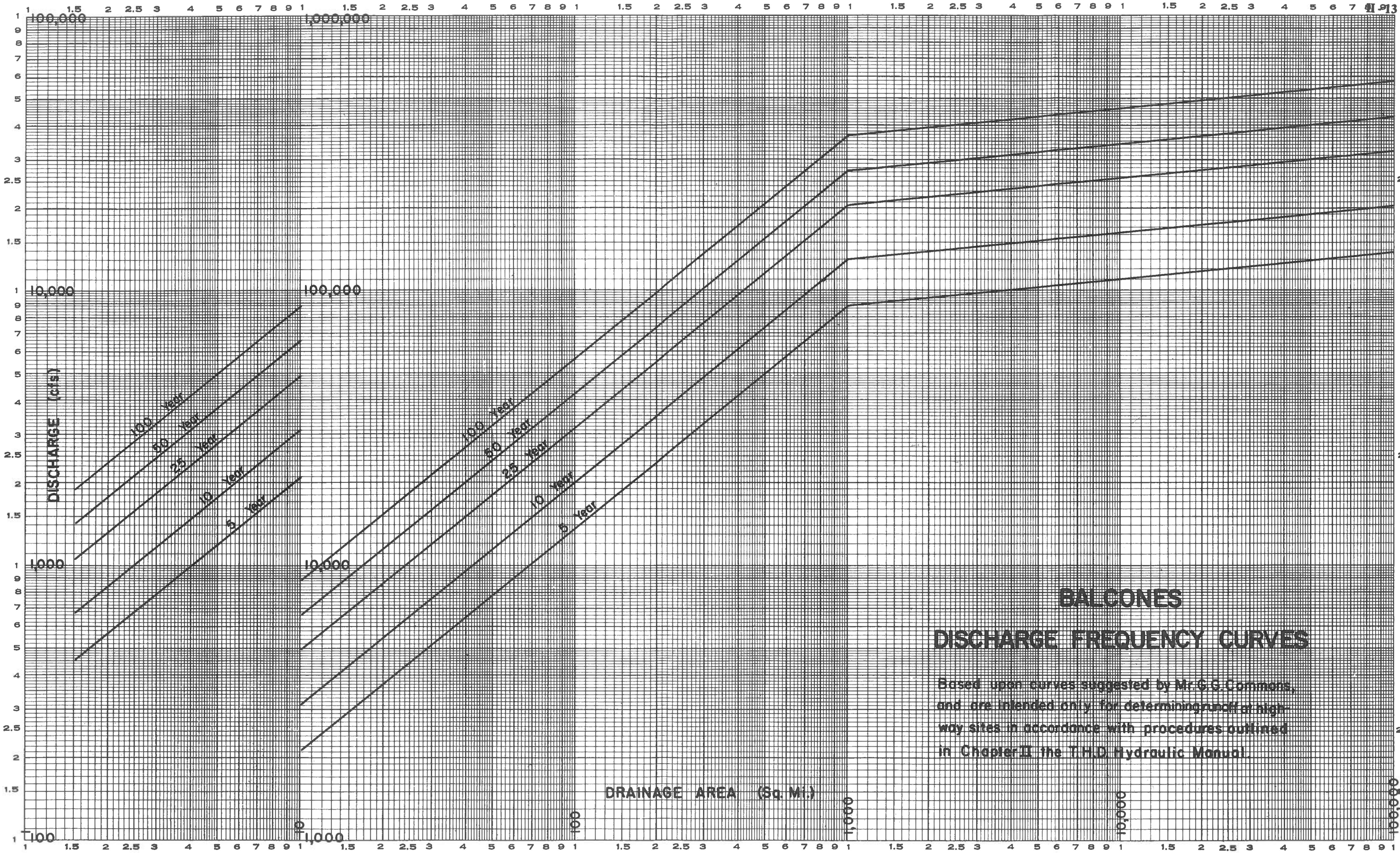

BALCONES
DISCHARGE FREQUENCY CURVES
Based upon curves suggested by Mr. G.G. Commons, and are intended only for determining runoff at highway sites in accordance with procedures outlined in Chapter II the T.H.D. Hydraulic Manual.
DISCHARGE (cfs)
DRAINAGE AREA (Sq. Mi.)
100 Year
50 Year
25 Year
10 Year
5 Year
100
1,000
10,000
100,000
1,000,000
10
100
1,000
10,000
100,000

Figure 7

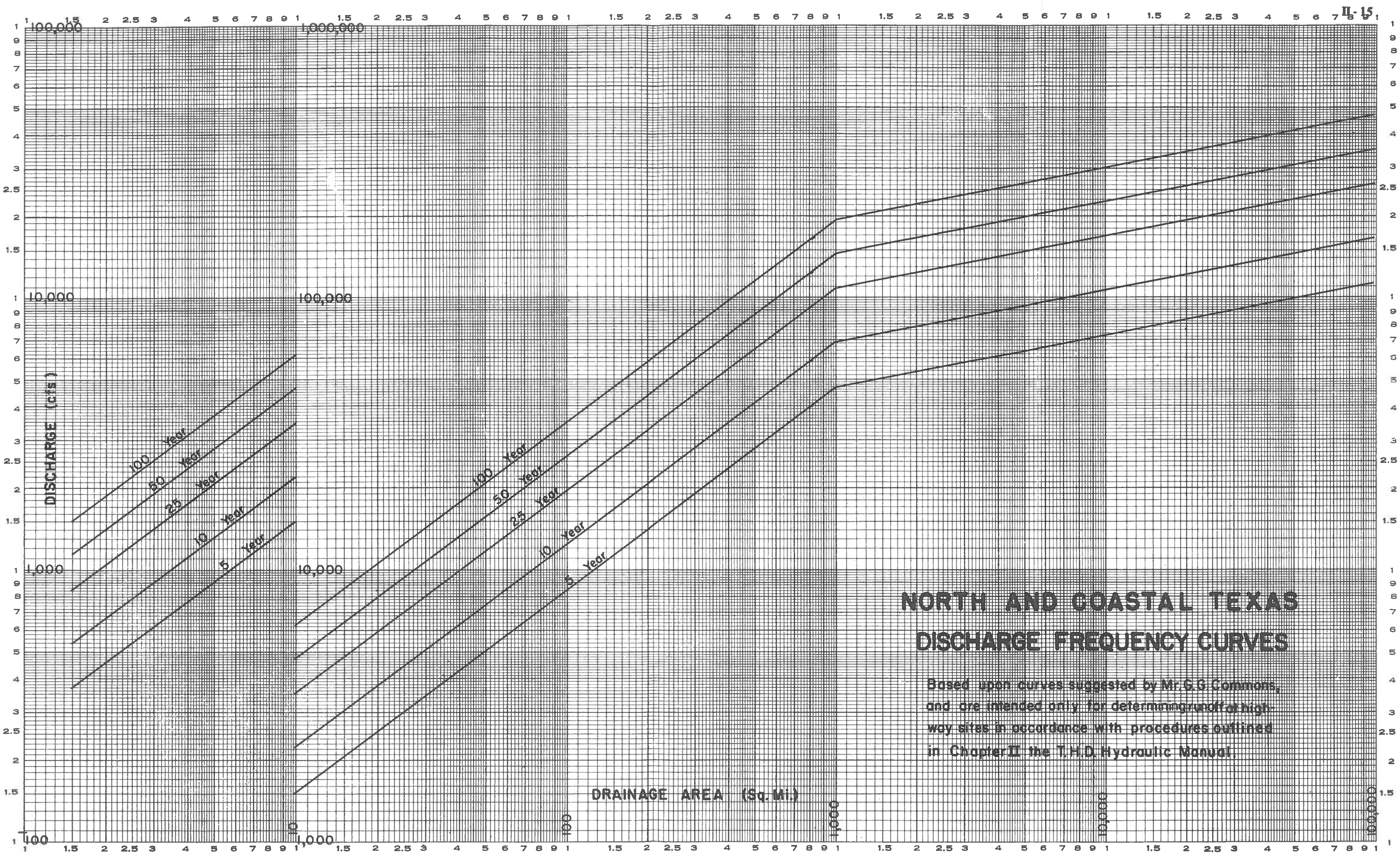

NORTH AND COASTAL TEXAS
DISCHARGE FREQUENCY CURVES
Based upon curves suggested by Mr. G.G. Commons, and are intended only for determining runoff at highway sites in accordance with procedures outlined in Chapter II the T.H.D. Hydraulic Manual.
DISCHARGE (c.f.s.)
DRAINAGE AREA (Sq. Mi.)
100 Year
50 Year
25 Year
10 Year
5 Year
100
1,000
10,000
100,000
1,000,000
10
100
1,000
10,000
100,000

Figure 8

TABLE II

TEXAS HIGHWAY DEPARTMENT
BRIDGE DIVISION

Flood data for Cedar Creek near Mabank
Drainage area 734 sq. miles. Period of record 1939 thru 1957
Location of Gage at bridge on Farm Road 85

YEAR	DATE	ELEVATION (feet)	DISCHARGE (second-feet)	ANNUAL FLOODS ORDER (M)	ANNUAL FLOODS RECURRENCE INTERVAL (years)	REMARKS
1939	2-27-39		3,720	19	1.05	
1940	4- 8-40		9,000	14	1.43	
1941	6-17-41		16,800	12	1.67	
1942	4- 9-42		29,400	2	10.0	
1943	6- 7-43		27,600	4	5.0	
1944	5- 3-44		28,800	3	6.67	
1945	3-30-45	310.93	44,800	1	69.0	* (Stage = 25.43 Ft.)
1946	5-31-46		23,500	8	2.5	
1947	4-10-47		20,900	9	2.22	
1948	12- 9-47		16,100	13	1.54	
1949	2-25-49		19,300	10	2.0	
1950	2-13-50		25,800	6	3.33	
1951	6- 5-51		5,120	17	1.18	
1952	4-23-52		18,600	11	1.82	
1953	5-16-53		24,000	7	2.86	
1954	5-13-54		8,000	15	1.33	
1955	3-22-55		4,720	18	1.11	
1956	5- 4-56		6,170	16	1.25	
1957	4-28-57		26,300	5	4.0	

*Historical Data:

Maximum stage since at least 1889, that of March 30, 1945.

$$RI = \frac{(1957-1889)+1}{1} = 69.0$$

TABLE III

TEXAS HIGHWAY DEPARTMENT
BRIDGE DIVISION

Flood data for Richland Creek near Richland
Drainage area 737 sq. miles. Period of record 1940 thru 1955
Location of Gage at bridge on US Hwy. 75

YEAR	DATE	ELEVATION (feet)	DISCHARGE (second-feet)	ANNUAL FLOODS ORDER (M)	ANNUAL FLOODS RECURRENCE INTERVAL (years)	REMARKS
1940	4- 7-40		10,600	12	1.42	
1941	11-24-40		43,000	4	4.25	
1942	4-26-42		39,600	5	3.40	
1943	5- 8-43		16,600	8	2.12	
1944	5- 2-44		55,000	3	5.67	
1945	3-31-45		55,000	2	8.50	
1946	5-13-46		15,200	9	1.89	
1947	1-19-47		22,400	7	2.43	
1948	5-12-48	323.73	58,900	1	43.0	* (Stage = 24.61 Ft.)
1949	5-27-49		1,740	16	1.06	
1950	2-13-50		12,100	11	1.55	
1951	9-13-51		3,000	15	1.13	
1952	4-23-52		13,500	10	1.70	
1953	5-13-53		29,500	6	2.83	
1954	5-13-54		4,600	13	1.31	
1955	3-22-55		3,740	14	1.21	

*Historical Data:

Maximum stage since 1913, that of May 12, 1948.

$$RI = \frac{(1955-1913)+1}{1} = 43.0$$

TABLE IV

TEXAS HIGHWAY DEPARTMENT

BRIDGE DIVISION

Flood data for Chambers Creek near Corsicana
Drainage area 971 sq. miles. Period of record 1940 thru 1955
Location of Gage at bridge on St. Hwy. 31

YEAR	DATE	ELEVATION (feet)	DISCHARGE (second-feet)	ANNUAL FLOODS		REMARKS
				ORDER (M)	RECURRENCE INTERVAL (years)	
1940	4- 7-40		18,400	6	2.83	
1941	11-24-40		25,400	5	3.40	
1942	4-25-42		37,400	2	8.5	
1943	7- 6-43		16,000	9	1.89	
1944	5- 3-44	321.45	48,000	1	43.0	* (Stage = 27.19 Ft.)
1945	3-31-45		32,900	3	5.67	
1946	2-19-46		12,000	10	1.70	
1947	6-22-47		18,200	7	2.43	
1948	5-12-48		18,200	8	2.12	
1949	2-25-49		9,140	13	1.31	
1950	2-13-50		11,000	11	1.55	
1951	6- 5-51		6,060	14	1.21	
1952	4-23-52		26,000	4	4.25	
1953	5-16-53		10,200	12	1.42	
1954	5-13-54		3,700	15	1.13	
1955	5-21-55		2,350	16	1.06	

*Historical Data:

Maximum stage since 1913, that of May 3, 1944.

$$RI = \frac{(1955-1913) + 1}{1} = 43.0$$

TABLE V

Values of C (Run-off Coefficient) in Formula Q = CIA

SLOPE	LAND USE (estimate up to 20 years in future)	SOIL CLASSIFICATION: Rolling Plains Min.	Rolling Plains Max.	Sand or Sandy Loam Soils (Pervious) Min.	Sand or Sandy Loam Soils (Pervious) Max.	Black or Loessial Soils (Impervious) Min.	Black or Loessial Soils (Impervious) Max.
Flat (0%-1%)	Cultivated			0.25	0.35	0.30	0.40
	Woodlands			0.15	0.20		
	Pasture			0.20	0.25		
	Paved	0.90		0.90		0.90	
	Residential	0.50	0.60	0.50	0.60	0.50	0.60
	Commercial	0.60	0.90	0.60	0.90	0.60	0.90
Rolling (1%-3.5%)	Cultivated	0.40	0.45	0.45	0.65	0.50	0.70
	Woodlands			0.15	0.20	0.18	0.25
	Pasture	0.25	0.30	0.30	0.40	0.35	0.45
	Paved	0.90		0.90		0.90	
	Residential	0.55	0.60	0.50	0.60	0.35	0.60
	Commercial	0.60	0.90	0.60	0.90	0.60	0.90
Hilly (3.5%-5.5%)	Cultivated			0.60	0.75	0.70	0.85
	Woodlands			0.20	0.25	0.25	0.30
	Pasture			0.35	0.45	0.45	0.55
	Paved	0.90		0.90		0.90	
	Residential	0.60		0.50	0.60	0.50	0.60
	Commercial	0.60	0.90	0.60	0.90	0.60	0.90
Mountainous (5.5% +)	Woodlands					0.70	0.80
	Bare					0.80	0.90
Steep Grassed Slopes		0.70		0.70		0.70	

TABLE VI

CONSTANTS FOR USE IN FORMULA $I = \frac{b}{(t+d)^e}$

Based on Weather Bureau (NWS) Technical Paper No. 40
"Rainfall Frequency Atlas of the United States"

COUNTY	5 year		10 year		25 year		50 year		All Freqs.
	e	b	e	b	e	b	e	b	d
Anderson	0.792	77	0.763	78	0.772	92	0.744	93	8.8
Andrews	0.827	57	0.809	63	0.793	69	0.807	84	10.2
Angelina	0.762	69	0.746	73	0.726	77	0.727	86	7.6
Aransas	0.787	77	0.753	79	0.745	88	0.739	95	8.5
Archer	0.783	61	0.794	74	0.789	86	0.792	100	8.5
Armstrong	0.819	63	0.820	75	0.835	95	0.831	105	10.4
Atascosa	0.791	74	0.780	80	0.770	90	0.757	95	9.0
Austin	0.781	75	0.757	79	0.739	85	0.733	92	8.1
Bailey	0.848	62	0.777	55	0.806	72	0.819	85	9.0
Bandera	0.770	62	0.774	72	0.764	82	0.765	94	8.2
Bastrop	0.781	72	0.765	77	0.762	88	0.748	93	8.4
Baylor	0.780	59	0.797	75	0.791	87	0.801	103	8.8
Bee	0.796	78	0.762	79	0.760	92	0.748	97	8.7
Bell	0.780	69	0.773	77	0.771	90	0.754	93	8.5
Bexar	0.784	70	0.779	79	0.769	88	0.756	94	8.7
Blanco	0.777	65	0.776	75	0.766	85	0.758	93	8.4
Borden	0.808	58	0.802	66	0.805	80	0.795	87	10.2
Bosque	0.779	66	0.772	74	0.776	91	0.765	97	8.6
Bowie	0.769	64	0.753	68	0.743	76	0.742	83	8.0
Brazoria	0.751	70	0.749	80	0.730	85	0.718	90	8.0
Brazos	0.785	76	0.763	80	0.754	89	0.745	98	8.5
Brewster	0.819	60	0.833	70	0.818	78	0.833	91	9.6
Briscoe	0.824	63	0.815	73	0.823	89	0.818	101	10.3
Brooks	0.797	80	0.769	82	0.771	97	0.746	98	9.1
Brown	0.770	57	0.763	66	0.768	80	0.770	91	7.6
Burleson	0.784	74	0.763	79	0.760	90	0.743	97	8.4
Burnet	0.773	64	0.777	75	0.769	86	0.758	94	8.5
Caldwell	0.785	72	0.769	77	0.764	88	0.748	92	8.5
Calhoun	0.772	73	0.760	81	0.745	88	0.737	95	8.4
Callahan	0.767	55	0.776	68	0.770	80	0.783	95	8.1
Cameron	0.786	78	0.781	90	0.760	95	0.727	93	8.8
Camp	0.782	70	0.761	74	0.758	84	0.743	87	8.8
Carson	0.828	65	0.829	77	0.842	97	0.849	113	10.6
Cass	0.768	65	0.750	69	0.746	78	0.738	83	8.3
Castro	0.819	56	0.802	65	0.827	84	0.829	95	9.5
Chambers	0.735	66	0.742	76	0.727	83	0.711	87	7.4
Cherokee	0.783	74	0.754	75	0.751	85	0.742	91	8.5
Childress	0.807	65	0.814	78	0.821	97	0.822	108	10.2
Clay	0.787	64	0.790	74	0.787	88	0.790	99	8.4
Cochran	0.840	60	0.782	57	0.802	70	0.813	83	9.5

TABLE VI (continued)

COUNTY	5 year e	5 year b	10 year e	10 year b	25 year e	25 year b	50 year e	50 year b	All Freqs. d
Coke	0.766	51	0.777	65	0.778	77	0.787	91	9.3
Coleman	0.763	54	0.770	66	0.763	78	0.781	92	8.0
Collin	0.781	67	0.778	79	0.779	92	0.776	102	8.8
Collings-worth	0.821	68	0.822	81	0.837	102	0.825	110	10.5
Colorado	0.778	74	0.756	79	0.744	87	0.736	94	8.3
Comal	0.781	69	0.775	78	0.766	87	0.753	91	8.6
Comanche	0.770	59	0.765	68	0.773	83	0.769	92	7.8
Concho	0.752	50	0.779	67	0.755	73	0.785	91	8.0
Cooke	0.779	65	0.780	77	0.791	93	0.787	104	8.7
Coryell	0.774	66	0.777	76	0.774	89	0.761	95	8.5
Cottle	0.799	61	0.805	75	0.811	91	0.817	105	9.8
Crane	0.819	55	0.818	66	0.785	69	0.801	82	10.2
Crockett	0.791	56	0.781	63	0.782	74	0.777	82	9.3
Crosby	0.815	61	0.809	69	0.811	83	0.807	92	10.1
Culberson	0.851	53	0.831	58	0.814	62	0.818	72	10.8
Dallam	0.866	68	0.824	68	0.846	90	0.872	111	10.6
Dallas	0.782	68	0.777	78	0.774	90	0.771	101	8.7
Dawson	0.818	59	0.814	68	0.807	77	0.801	86	10.4
Deaf Smith	0.844	60	0.794	61	0.831	82	0.837	94	9.3
Delta	0.783	68	0.775	77	0.770	89	0.759	94	9.1
Denton	0.777	65	0.779	77	0.781	90	0.780	102	8.5
DeWitt	0.785	75	0.758	78	0.758	90	0.747	96	8.7
Dickens	0.807	61	0.803	71	0.808	85	0.808	96	10.0
Dimmit	0.806	74	0.795	82	0.783	94	0.781	105	9.4
Donley	0.832	68	0.825	79	0.836	96	0.823	103	10.6
Duval	0.802	79	0.781	84	0.772	94	0.755	98	9.2
Eastland	0.772	58	0.771	69	0.772	81	0.775	92	7.8
Ector	0.821	56	0.816	65	0.789	68	0.802	82	10.5
Edwards	0.759	54	0.759	63	0.769	76	0.776	88	7.5
Ellis	0.788	71	0.777	79	0.771	91	0.766	98	8.8
El Paso	0.802	34	0.795	42	0.843	60	0.900	90	12.0
Erath	0.772	61	0.770	72	0.785	89	0.772	96	8.1
Falls	0.786	72	0.771	79	0.772	93	0.750	94	8.5
Fannin	0.778	66	0.782	79	0.782	93	0.770	99	9.1
Fayette	0.782	73	0.758	76	0.758	88	0.743	94	8.2
Fisher	0.779	55	0.793	70	0.790	83	0.798	96	9.5
Floyd	0.821	63	0.809	71	0.818	85	0.813	97	10.0
Foard	0.787	60	0.806	77	0.807	91	0.817	109	9.5
Fort Bend	0.760	71	0.751	80	0.729	84	0.726	91	8.1
Franklin	0.782	69	0.765	74	0.759	84	0.751	89	8.8

TABLE VI (continued)

COUNTY	5 year		10 year		25 year		50 year		All Freqs.
	e	b	e	b	e	b	e	b	d
Freestone	0.795	77	0.769	80	0.775	95	0.749	94	9.0
Frio	0.789	71	0.789	82	0.772	89	0.765	96	9.1
Gaines	0.832	58	0.805	63	0.797	70	0.807	84	10.0
Galveston	0.739	66	0.742	78	0.727	85	0.704	88	7.6
Garza	0.811	60	0.800	67	0.810	83	0.799	88	10.2
Gillespie	0.766	60	0.767	71	0.765	82	0.764	94	8.1
Glasscock	0.796	55	0.803	66	0.789	74	0.789	83	10.0
Goliad	0.789	75	0.758	77	0.755	89	0.746	97	8.7
Gonzales	0.788	74	0.763	77	0.760	89	0.747	95	8.6
Gray	0.837	70	0.836	83	0.842	99	0.841	114	10.8
Grayson	0.778	65	0.779	78	0.790	95	0.781	104	8.9
Gregg	0.783	71	0.750	72	0.753	84	0.740	87	8.6
Grimes	0.784	75	0.760	81	0.744	87	0.742	95	8.3
Guadalupe	0.787	72	0.772	78	0.765	89	0.750	93	8.7
Hale	0.827	61	0.815	69	0.823	84	0.812	92	9.9
Hall	0.821	66	0.815	75	0.822	92	0.819	103	10.3
Hamilton	0.770	62	0.761	72	0.778	89	0.766	95	8.3
Hansford	0.846	73	0.842	84	0.862	104	0.867	124	11.3
Hardeman	0.794	61	0.810	78	0.816	95	0.817	110	9.5
Hardin	0.738	65	0.740	74	0.720	80	0.718	87	7.5
Harris	0.749	70	0.753	81	0.724	81	0.728	91	7.7
Harrison	0.773	69	0.750	70	0.747	80	0.730	82	8.4
Hartley	0.855	67	0.814	67	0.840	85	0.863	106	10.2
Haskell	0.779	57	0.799	74	0.787	85	0.805	103	9.2
Hays	0.783	69	0.776	78	0.765	87	0.747	90	8.6
Hemphill	0.851	76	0.842	87	0.843	103	0.840	115	10.7
Henderson	0.796	77	0.770	79	0.773	93	0.752	93	9.0
Hidalgo	0.795	80	0.778	87	0.771	98	0.749	99	9.2
Hill	0.790	72	0.777	78	0.773	91	0.764	98	8.8
Hockley	0.832	60	0.807	64	0.812	78	0.810	87	10.0
Hood	0.773	63	0.773	75	0.782	90	0.773	98	8.3
Hopkins	0.783	70	0.775	77	0.767	88	0.754	93	9.1
Houston	0.780	73	0.757	78	0.748	86	0.740	93	8.3
Howard	0.800	56	0.802	65	0.796	76	0.791	86	10.1
Hudspeth	0.840	44	0.827	50	0.819	60	0.856	78	11.4
Hunt	0.785	69	0.783	80	0.776	93	0.764	99	9.2
Hutchinson	0.844	70	0.837	80	0.851	100	0.863	121	11.0
Irion	0.789	55	0.775	61	0.783	77	0.759	81	9.6
Jack	0.779	63	0.786	75	0.782	88	0.782	98	8.5
Jackson	0.769	73	0.757	80	0.745	89	0.737	95	8.5

TABLE VI (continued)

COUNTY	5 year		10 year		25 year		50 year		All Freqs.
	e	b	e	b	e	b	e	b	d
Jasper	0.743	65	0.736	73	0.719	78	0.715	84	7.4
Jeff Davis	0.865	63	0.853	69	0.802	65	0.818	77	10.8
Jefferson	0.733	65	0.727	74	0.730	86	0.710	87	7.5
Jim Hogg	0.813	82	0.786	87	0.780	98	0.756	99	9.4
Jim Wells	0.797	79	0.765	80	0.768	94	0.748	98	8.9
Johnson	0.779	67	0.773	77	0.776	90	0.771	99	8.6
Jones	0.771	55	0.794	73	0.780	82	0.803	102	9.0
Karnes	0.791	75	0.766	77	0.767	91	0.751	95	8.9
Kaufman	0.790	72	0.776	80	0.777	93	0.763	98	9.0
Kendall	0.771	64	0.773	74	0.764	84	0.762	94	8.3
Kenedy	0.793	79	0.771	84	0.763	94	0.735	96	8.7
Kent	0.801	60	0.796	70	0.806	85	0.799	93	10.0
Kerr	0.765	58	0.764	69	0.763	80	0.766	91	8.0
Kimble	0.758	53	0.763	66	0.757	75	0.772	88	7.6
King	0.794	59	0.804	74	0.802	87	0.811	103	9.6
Kinney	0.780	62	0.773	70	0.777	82	0.790	97	8.0
Kleberg	0.794	79	0.764	81	0.761	93	0.739	96	8.7
Knox	0.788	58	0.801	76	0.798	88	0.809	104	9.3
Lamar	0.777	66	0.770	74	0.769	87	0.759	93	8.9
Lamb	0.828	60	0.800	63	0.821	80	0.818	88	9.6
Lampasas	0.770	63	0.770	73	0.771	86	0.762	94	8.4
LaSalle	0.801	77	0.795	86	0.776	91	0.767	99	9.4
Lavaca	0.782	74	0.757	78	0.750	89	0.741	95	8.5
Lee	0.782	73	0.764	77	0.761	89	0.745	94	8.3
Leon	0.788	76	0.766	80	0.762	92	0.746	95	8.8
Liberty	0.739	65	0.747	78	0.716	78	0.726	90	7.4
Limestone	0.792	74	0.772	79	0.772	92	0.753	93	8.8
Lipscomb	0.858	78	0.846	89	0.849	106	0.851	121	10.7
Live Oak	0.795	76	0.768	79	0.765	92	0.752	96	8.9
Llano	0.768	60	0.768	72	0.769	84	0.763	93	8.2
Loving	0.825	50	0.822	61	0.804	63	0.813	77	10.2
Lubbock	0.821	60	0.813	69	0.816	82	0.808	88	10.1
Lynn	0.821	59	0.813	69	0.815	81	0.802	87	10.3
Madison	0.785	76	0.765	81	0.751	90	0.755	100	8.7
Marion	0.770	67	0.750	71	0.746	79	0.736	83	8.4
Martin	0.813	57	0.820	70	0.796	74	0.797	85	10.4
Mason	0.766	56	0.760	68	0.770	81	0.769	91	7.9
Matagorda	0.757	71	0.759	83	0.737	87	0.727	94	8.5
Maverick	0.801	70	0.788	76	0.782	88	0.796	106	8.7
McCulloch	0.761	53	0.767	67	0.766	79	0.772	91	7.8

TABLE VI (continued)

COUNTY	5 year		10 year		25 year		50 year		All Freqs.
	e	b	e	b	e	b	e	b	d
McLennan	0.787	71	0.777	78	0.774	91	0.757	94	8.7
McMullen	0.797	77	0.782	83	0.770	91	0.758	96	9.2
Medina	0.779	67	0.784	79	0.771	86	0.765	95	8.7
Menard	0.751	51	0.781	69	0.750	72	0.779	91	7.6
Midland	0.812	57	0.823	71	0.788	71	0.795	83	10.5
Milam	0.784	72	0.769	78	0.768	91	0.748	93	8.4
Mills	0.769	59	0.763	69	0.772	84	0.766	93	7.9
Mitchell	0.778	54	0.782	64	0.792	78	0.792	88	9.7
Montague	0.785	65	0.783	75	0.785	89	0.788	100	8.6
Montgomery	0.771	73	0.757	81	0.728	82	0.736	92	7.7
Moore	0.847	65	0.825	72	0.851	96	0.864	118	10.7
Morris	0.777	67	0.760	71	0.757	81	0.741	85	8.5
Motley	0.817	63	0.806	72	0.807	85	0.814	101	10.0
Nacogdoches	0.773	71	0.748	73	0.740	81	0.733	86	8.0
Navarro	0.795	75	0.776	80	0.777	94	0.757	95	8.9
Newton	0.741	65	0.734	71	0.718	76	0.709	82	7.4
Nolan	0.768	52	0.780	68	0.781	79	0.793	96	9.3
Nueces	0.794	79	0.762	79	0.759	91	0.741	96	8.7
Ochiltree	0.847	74	0.848	88	0.858	106	0.866	127	11.2
Oldham	0.844	64	0.804	62	0.833	84	0.850	103	9.6
Orange	0.733	65	0.728	73	0.730	85	0.709	87	7.5
Palo Pinto	0.775	61	0.781	74	0.781	87	0.778	96	8.4
Panola	0.773	70	0.743	70	0.741	79	0.724	81	8.3
Parker	0.777	63	0.781	76	0.781	88	0.777	99	8.4
Parmer	0.845	62	0.783	56	0.821	76	0.827	88	9.1
Pecos	0.819	56	0.811	64	0.786	69	0.806	84	9.7
Polk	0.761	69	0.748	75	0.718	76	0.730	88	7.5
Potter	0.817	58	0.818	70	0.841	93	0.855	112	10.2
Presidio	0.860	62	0.854	72	0.837	75	0.848	90	10.6
Rains	0.786	73	0.781	81	0.771	91	0.759	94	9.2
Randall	0.793	52	0.811	68	0.837	89	0.846	106	9.5
Reagan	0.800	58	0.802	67	0.785	74	0.781	82	9.9
Real	0.769	57	0.771	68	0.770	78	0.769	91	7.6
Red River	0.776	66	0.758	70	0.758	81	0.752	86	8.4
Reeves	0.846	57	0.834	65	0.793	62	0.810	77	10.5
Refugio	0.788	77	0.753	78	0.751	90	0.743	96	8.6
Roberts	0.848	72	0.845	87	0.849	103	0.856	121	11.2
Robertson	0.784	74	0.767	80	0.765	91	0.748	96	8.6
Rockwall	0.787	70	0.780	79	0.778	92	0.768	100	8.9
Runnels	0.756	50	0.778	67	0.767	76	0.792	96	8.5

TABLE VI (continued)

COUNTY	5 year		10 year		25 year		50 year		All Freqs.
	e	b	e	b	e	b	e	b	d
Rusk	0.781	72	0.749	72	0.751	84	0.738	87	8.5
Sabine	0.758	66	0.740	71	0.721	75	0.715	80	7.6
San Augustine	0.758	67	0.741	72	0.725	76	0.719	82	7.6
San Jacinto	0.761	70	0.751	79	0.723	79	0.731	91	7.5
San Patricio	0.789	78	0.759	78	0.757	91	0.743	96	8.7
San Saba	0.767	59	0.762	69	0.771	83	0.767	92	8.0
Schleicher	0.771	53	0.772	63	0.769	74	0.771	86	8.6
Scurry	0.800	57	0.790	67	0.800	82	0.791	89	10.0
Shackelford	0.777	58	0.786	72	0.777	82	0.788	97	8.5
Shelby	0.768	68	0.742	71	0.737	78	0.717	80	8.0
Sherman	0.857	71	0.836	77	0.859	101	0.870	120	10.9
Smith	0.787	74	0.760	76	0.760	87	0.746	90	8.8
Somervell	0.771	63	0.770	73	0.782	90	0.770	97	8.4
Starr	0.805	82	0.791	89	0.780	99	0.758	100	9.4
Stephens	0.775	59	0.782	72	0.776	83	0.781	94	8.1
Sterling	0.779	53	0.780	62	0.788	76	0.783	85	9.6
Stonewall	0.790	59	0.801	74	0.797	85	0.806	100	9.5
Sutton	0.760	51	0.764	61	0.765	73	0.772	85	8.1
Swisher	0.808	58	0.807	69	0.826	87	0.822	98	10.0
Tarrant	0.778	66	0.777	77	0.774	88	0.775	101	8.5
Taylor	0.760	52	0.781	69	0.776	80	0.796	100	8.7
Terrell	0.793	54	0.800	67	0.793	76	0.806	91	9.0
Terry	0.828	59	0.806	65	0.809	77	0.806	86	10.0
Throckmorton	0.779	59	0.792	74	0.783	84	0.795	98	8.6
Titus	0.780	69	0.759	72	0.758	82	0.744	87	8.6
Tom Green	0.769	51	0.777	64	0.771	75	0.782	88	9.0
Travis	0.780	69	0.775	77	0.766	87	0.751	91	8.6
Trinity	0.764	71	0.750	76	0.729	80	0.734	90	7.7
Tyler	0.750	66	0.740	74	0.716	77	0.721	86	7.4
Upshur	0.782	71	0.760	74	0.757	84	0.742	87	8.7
Upton	0.808	56	0.824	71	0.788	72	0.793	82	10.0
Uvalde	0.779	64	0.779	73	0.772	83	0.773	95	8.2
Val Verde	0.767	52	0.767	61	0.780	76	0.794	92	8.0
Van Zandt	0.792	74	0.775	79	0.772	91	0.755	93	9.1
Victoria	0.782	75	0.755	78	0.752	90	0.745	97	8.6
Walker	0.778	73	0.759	80	0.739	84	0.740	94	8.0
Waller	0.785	77	0.757	80	0.736	84	0.729	91	8.1
Ward	0.833	56	0.810	61	0.789	65	0.808	81	10.5
Washington	0.784	75	0.759	79	0.747	87	0.740	94	8.2
Webb	0.812	81	0.801	90	0.781	95	0.774	103	9.6

TABLE VI (continued)

COUNTY	5 year		10 year		25 year		50 year		All Freqs.
	e	b	e	b	e	b	e	b	d
Wharton	0.767	72	0.758	81	0.738	86	0.733	93	8.3
Wheeler	0.832	71	0.832	85	0.836	103	0.833	114	10.6
Wichita	0.784	62	0.795	76	0.792	88	0.797	104	8.7
Wilbarger	0.782	60	0.797	77	0.799	90	0.807	107	9.0
Willacy	0.790	78	0.777	87	0.762	95	0.732	95	8.8
Williamson	0.779	68	0.777	77	0.769	88	0.751	92	8.5
Wilson	0.792	74	0.769	78	0.768	90	0.752	95	8.9
Winkler	0.829	56	0.811	61	0.790	66	0.817	83	10.4
Wise	0.778	65	0.782	77	0.782	89	0.781	100	8.6
Wood	0.787	73	0.770	78	0.761	87	0.753	92	8.9
Yoakum	0.841	59	0.798	59	0.797	69	0.812	82	9.6
Young	0.779	61	0.786	74	0.781	85	0.787	97	8.4
Zapata	0.815	84	0.795	91	0.785	99	0.763	100	9.6
Zavala	0.794	69	0.785	77	0.779	89	0.778	98	8.9

TABLE VII

Approximate Average Velocities of Run-off Flow for Calculating Time of Concentration

Description of Water Course	Slope in Percent			
	0-3	4-7	8-11	12-
	(Ft/Sec.)	(Ft/Sec.)	(Ft/Sec.)	(Ft/Sec.)
Unconcentrated*				
Woodlands	0-1.5	1.5 - 2.5	2.5 - 3.25	3.25 -
Pastures	0-2.5	2.5 - 3.5	3.5 - 4.25	4.25 -
Cultivated	0-3.0	3.0 - 4.5	4.5 - 5.5	5.5 -
Pavements	0-8.5	8.5 -13.5	13.5 -17	17 -
Concentrated**				
Outlet Channel - Determine velocity by Manning's Formula				
Natural Channel Not Well Defined	0-2	2-4	4-7	7 -

* This condition usually occurs in upper extremity of watershed prior to the overland flow accumulating in a watercourse.

** These values vary with the channel size and other conditions. Where possible, more accurate determinations should be made for particular conditions by the Manning Channel Formula for velocity.

CHAPTER III

OPEN CHANNELS

OPEN CHANNELS

3-100. GENERAL

Channel design involves the determination of the channel cross-section required to accommodate a given design discharge. Channel design is most frequently used for sizing outfall channels and various roadway ditches.

Channel analysis is made to determine the depth and velocity at which a discharge would flow in a channel where the cross section is already established. Channel analysis is most frequently used for establishing a water surface elevation prior to the design or analysis of a hydraulic structure and to establishing a roadway grade line.

The three methods outlined in this chapter for calculating depth of flow may be used either for analysing an existing channel or for the design of a proposed new or improved channel.

3-200. METHODS FOR COMPUTING DEPTHS OF FLOW

3-201. GENERAL

A proper determination of a highwater elevation or a tailwater (TW) depth for a structure or channel design is most important and should never be neglected. Methods presented herein for analysing or designing a channel are:

a. Single-section stage-discharge curve
b. Two-section slope-determination method
c. Multi-section backwater curve

Computer routines are available for each of the three methods. Where the expense is justified, it is recommended that the multi-section method take precedence over the other two methods. In turn, that the two section method take precedence over the single-section method.

3-202. SINGLE-SECTION STAGE-DISCHARGE CURVE

The single-section stage-discharge curve (or rating curve) has been the most widely used for TW determination. This method employs the use of Manning's formula:

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

where

Q = Discharge, cfs

A = Cross-sectional area of flow, sq. ft.

R = Hydraulic Radius

S = Slope of water surface, which "S" is usually assumed to be parallel to streambed slope.

n = Manning's Roughness Coefficient

Advantages are that it is a relatively simple procedure, usually inexpensive, and expedient. However, due to the assumptions necessary for its use, its reliability is often reduced to cause such a disadvantage as to outweigh the advantages.

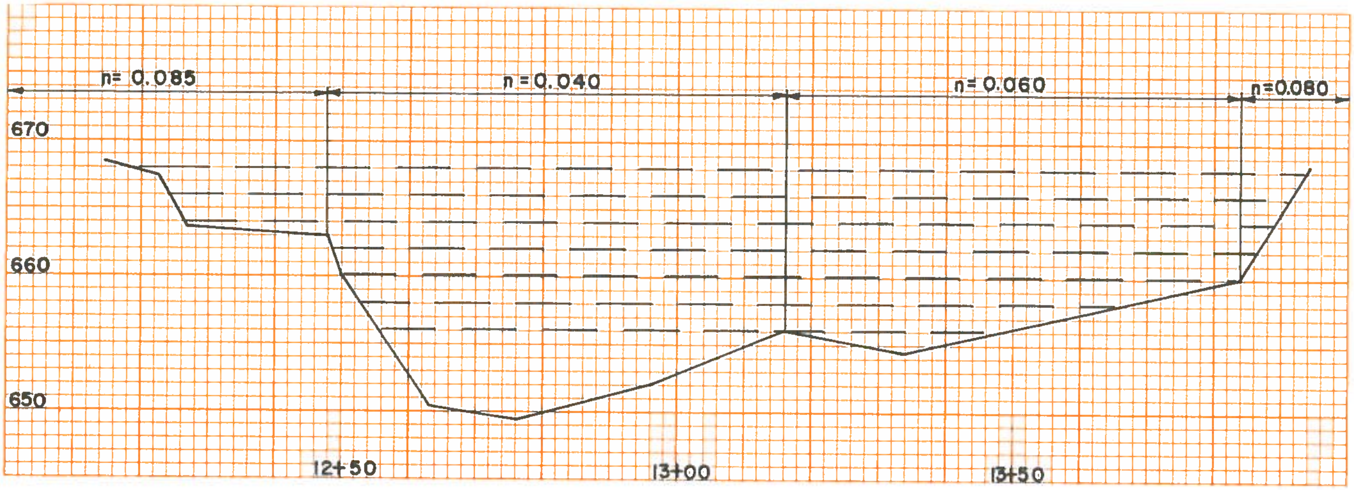
For the usual culvert design, this single section method will suffice for determining tailwater (TW) depth at the culvert outlet.

The data required for this method includes: a channel cross section which might safely be considered as typical of the stream reach under consideration. Most engineers prefer to locate this section downstream from the proposed structure site. If a "typical"section cannot be found, then a "controlling" section (also downstream) should be used. A "controlling" section would be where the depth of flow is controlled by a constriction of channel, a damming effect across the channel, or possibly a heavily wooded area with high roughness coefficients. Any section taken should be normal to the direction of stream flow under flood condition. Once the section has been obtained, Manning's roughness coefficients ("n" values) should be applied. Table II gives Manning's roughness coefficients for various conditions. The cross section should be subdivided with vertical boundaries at severe changes in cross section shape or at changes in vegetation cover. Additional consideration should be given to such things as depth of flow, size of boulders, etc.

It should be borne in mind that, however inexact the "n" value determination may be, there are definite and unchangeable "n" values in the cross section. Therefore, once the engineer has chosen "n" values to the best of his ability, the "n" values chosen should not be juggled just to provide another answer. If there is uncertainty about particular "n" value choices, a more experienced engineer should be consulted.

The third important factor necessary to calculate a single-section rating curve is the slope. Manning's formula is based on the slope of the water surface. Often this slope will correspond to the average slope of the channel bed. However, some reaches of stream may have a water surface slope quite different from the bed slope at flood flow. The least expensive and most expedient method of slope-determination is the surveying and analysis of the bed profile for some distance in a stream reach. Due to the importance of the slope in Manning's formula, it is recommended that, wherever possible, an accurate water slope at flood stage be determined for use in Manning's formula.

Obviously, the closer to the proposed site a typical section is taken, the less error in final water surface is



TYPICAL SECTION PLOT

Figure — I.

possible. This is because once the highwater elevation or tailwater depth is determined, it must be projected to the proposed site along some uncertain slope (as shown above) which is assumedly constant.

After the section shape, "n" values, and slope are obtained or determined, the calculation of the stage-discharge curve should proceed as follows.

Step 1. Assumed incremental water surface elevations should be applied to a plot of the cross section. See Figure 1. A total discharge under each assumed elevation is then calculated by use of Manning's formula. Care should be taken to apply the individual "n" values to their proper subsection. In addition, the wetted perimeter is considered only along the solid boundaries of the cross-section and not along the water interface between subsections.

Step 2. After several discharges have been calculated so that there are some discharges which are less than design discharge and some which are greater, a plot should be made of elevation versus discharge. (Figure 2) A water surface elevation corresponding to the design discharge may then be scaled from the elevation discharge curve.

For trapezoidal channels a nomograph is provided to aid the solution of Manning's formula (Figure 3).

3-203. TWO-SECTION METHOD

Of the three indicated methods for highwater elevation or tailwater depth, the two-section method is probably the best compromise for engineering expenses and design reliability. The choice of sections and "n" values should proceed as explained above in the single-section method. The chief difference being that two sections must be selected instead of one. These sections should straddle the proposed structure site, i.e., one upstream and one downstream. Distance between the two sections should also be determined.

A slope for use in Manning's formula is not necessary for this method. This is explained by the following.

If it is assumed that discharge and slope are constant in the reach of stream under consideration, then conveyance at the upstream section necessarily equals conveyance at the downstream section. Therefore, for a particular design discharge, there is at least one conveyance value which defines a slope mathematically and also subtends the same slope physically along the longitudinal surface of the water.

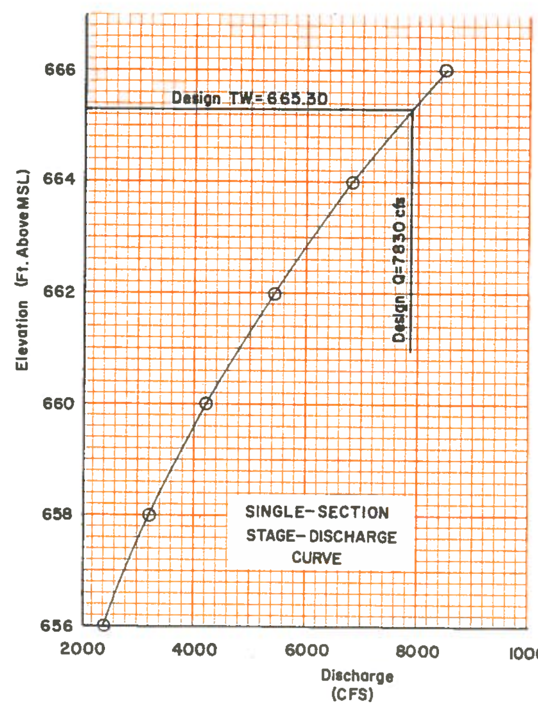
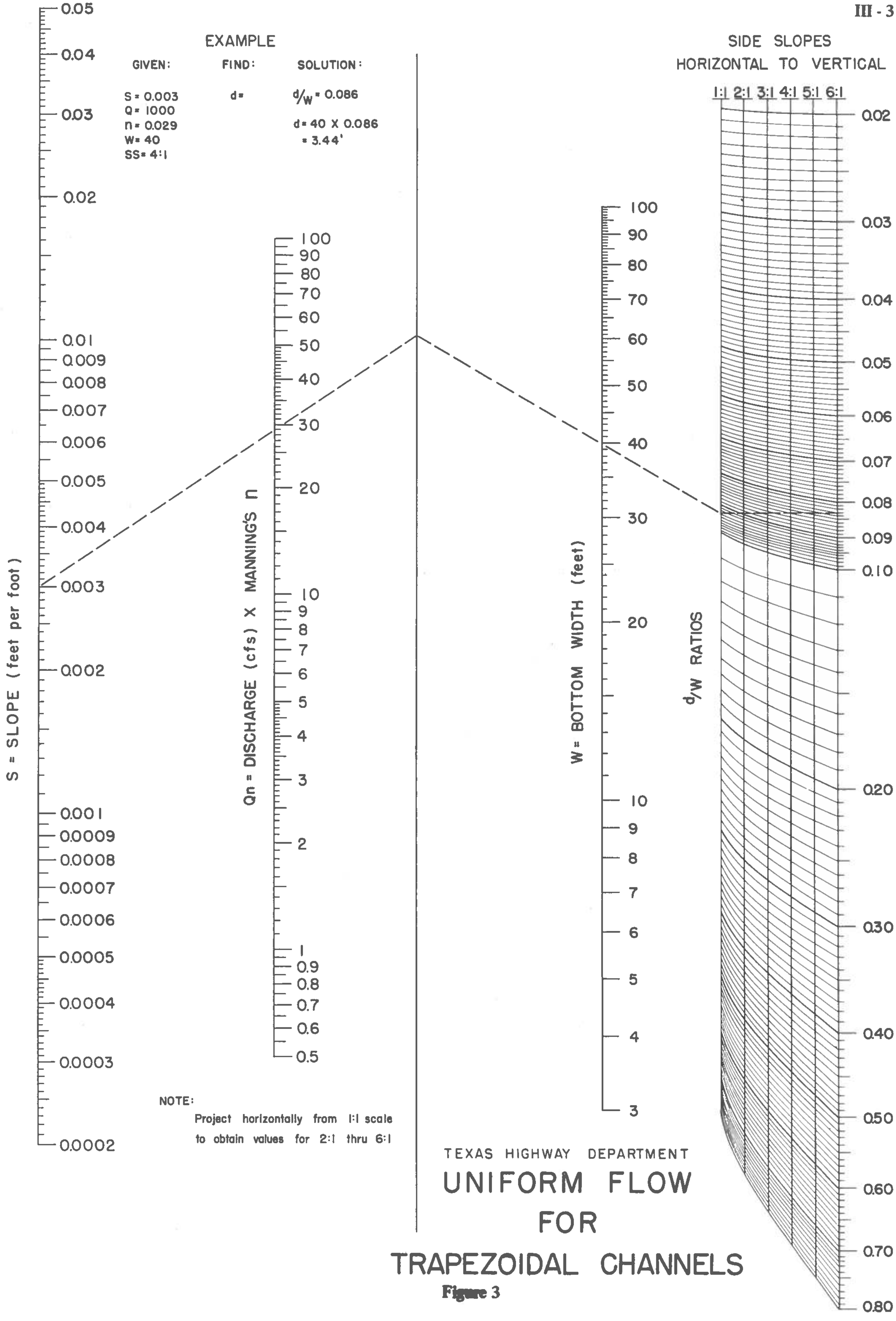


Figure 2.



TEXAS HIGHWAY DEPARTMENT

UNIFORM FLOW FOR TRAPEZOIDAL CHANNELS

Figure 3

The calculation procedure is as follows:

Step 1. Since

$$K = Q/S^{1/2}$$

or

$$K = \frac{1.486}{n} AR^{2/3}$$

the conveyance at each of the two sections may be calculated for a range of water surface elevations. The result would be plots of the upstream and downstream stage-conveyance curves (plotted on the same graph, Figure 4).

Step 2. Find conveyance based on design discharge and assumed slope according to the relation $K_1 = Q/S_1^{1/2}$. Since any slope (S_1) may be assumed at the start, the channel bed slope may be used.

Step 3. Draw a vertical line representing K_1 intercepting both conveyance curves.

Step 4. Scale the vertical elevation difference (ΔE) between the two curve intercepts.

Step 5. Find $S' = \frac{\Delta E}{L}$ (L = distance between the two sections)

At this point there are two slope values. One slope (S_1) is assumed and used to calculate K_1. K_1 is then used to establish the corresponding slope (S'_1) from the actual conveyance curves of the two sections. The proper slope is arrived at when $S = S'$. This usually requires trial and error and a plot (Figure 5) is used to aid this solution.

Step 6. If S' does not equal S, plot S' versus the assumed slope (Figure 5) and increase or decrease the assumed slope as necessary so that by repeating the procedure outlined above, a plot of several S' versus S intercepts the line representing S' = S. The value at this intercept is the derived slope.

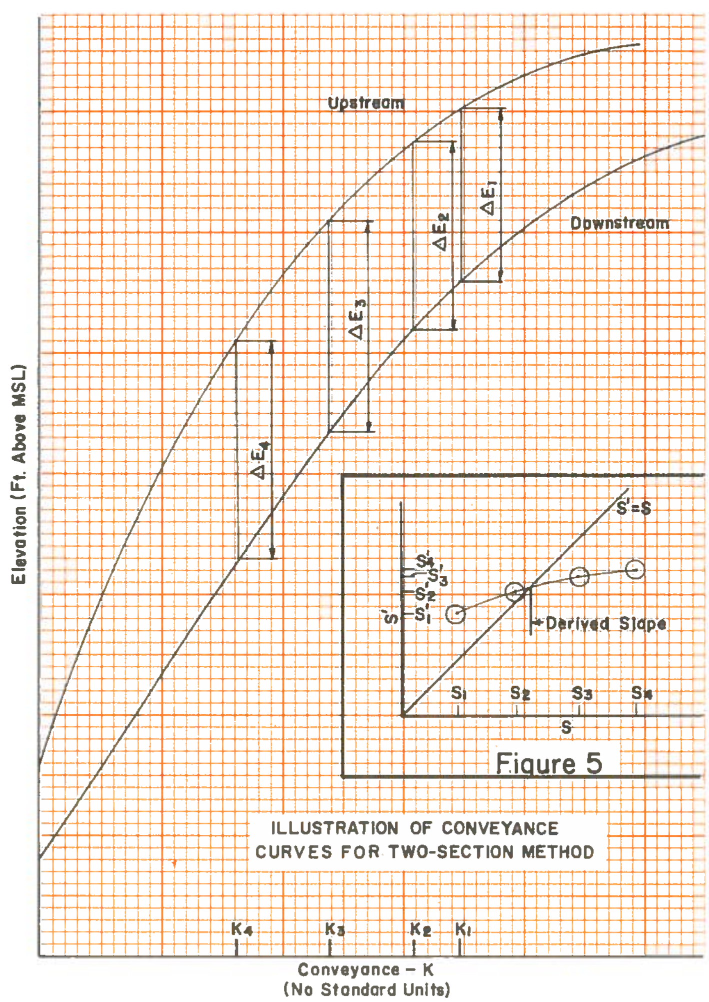


Figure 4

Step 7. Divide design discharge by the square root of the derived slope and draw a vertical line representing the resulting conveyance on Figure 4. The elevations on this line represent the predicted water surface elevations between the upstream section and the downstream section at design discharge. The highwater elevation at the proposed structure site need be only picked off this vertical line (Figure 4) according to its location between the two sections.

3-204. MULTI-SECTION BACKWATER CURVE

A calculated water surface profile according to Bernoulli's energy equation is the most accurate of TW determination methods. Unfortunately, the method is also the most expensive to complete. This is because several cross sections must be obtained. A minimum of three cross sections (all downstream) is necessary if the starting elevation in the most downstream section is known for the design discharge. This is very rarely the case. Therefore, when this starting elevation is not known, a minimum of 20 downstream cross sections is necessary to provide a reliable water surface profile.

The procedure outlined herein is applicable to most open channel flow, including those streams having an irregular channel with the cross section consisting of a main channel and separate overbank areas with individual "n" factors. Referring to Figure 6, the objective is to find the water surface profile at Section 2 knowing or assuming its elevation over the given datum at Section 1. Figure 6 shows in graphical form Bernoulli's Energy Equation which is written:

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

where

Z_2 and Z_1 = streambed elevation with respect to a given datum at upstream and downstream sections respectively.

d_2 and d_1 = depth of flow at upstream and downstream sections respectively.

h_{v2} and h_{v1} = velocity head of upstream and downstream sections respectively.

h_f = friction head loss.

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

The basic equations involved are

$$Q = AV$$

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

and Manning's formula

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

which is defined elsewhere in this chapter.

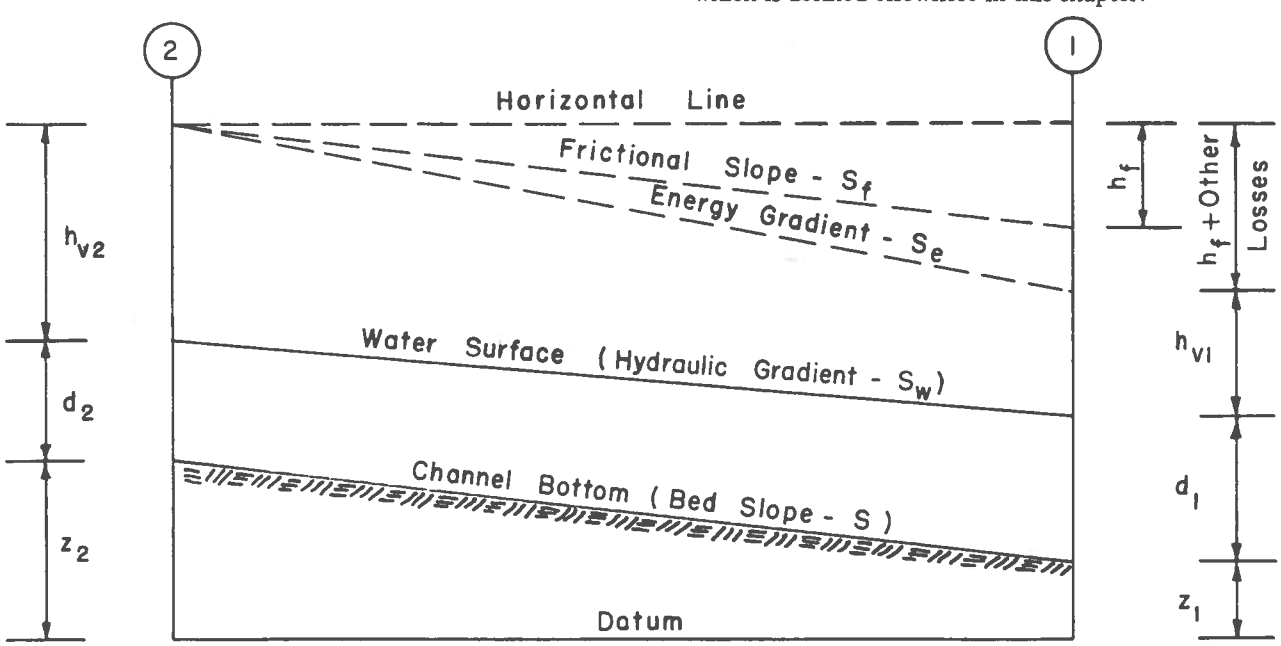


Figure 6

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

$$S_f = \left(\frac{Q\,n}{1.486\,AR^{2/3}}\right)^2$$

thus giving the total friction head

$$h_f = L\left(\frac{S_{f1} + S_{f2}}{2}\right)$$

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

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

$$h_v = \left(\frac{\sum(V_p{}^2 Q_p)}{2gQ}\right)$$

where

V_p = Velocity in subdivision of the cross section

A_p = Area of the subdivision of the cross section

Q_p = Discharge in the subdivision of the cross section

$V_p = Q_p/A_p$

Table I shows typical backwater calculations for a stream carrying 42,000 cfs. Each cross section, numbered 1 through 6 is divided into subsections covering areas with different roughness coefficients "n".

1. Calculations are based on a known or assumed water surface elevation at Section 1 and assumed elevations verified thru the calculations at subsequent sections. Col. 3 shows the length between cross sections.
2. Col. 4 shows the subsection areas calculated from the plotted cross sections.
3. Col. 5. The wetted perimeter is the wetted boundary of each subsection exclusive of the water interfaces between adjacent sections.
4. Col. 6. The roughness coefficient "n" for Manning's formula is chosen from the ranges indicated in Table II.
5. Col. 7. The values shown are obtained by dividing 1.486 x Col. 4 by Col. 6.
6. Col. 8. The hydraulic radius equals the area divided by the wetted perimeter or Col. 4 divided by Col. 5.
7. Col. 9 shows values of the hydraulic radius to the two-thirds power.
8. Col. 10 is the conveyance factor for the section which is $K = \frac{1.486}{n} AR^{2/3}$ or Col. 7 multiplied by Col. 9.
9. Col. 11 is the friction slope of the entire Section, S_f.

$$S_f = \left(\frac{\text{The total discharge}}{\text{The total conveyance}}\right)^2$$

or in this case

$$S_f = \left(\frac{42{,}000}{K}\right)^2$$

10. Col. 12 is the average friction slope between sections and the

$$S_f = \frac{S_{f_1} + S_{f_2}}{2}$$

or in the case of Section 1 and 2 the mean S_f is

$$S_f = \frac{.01297 + .00461}{2} = .00879.$$

11. Col. 13 shows the loss in friction head for the length between sections or for Sections 1 and 2 of the example 0.00879 x 500 = 4.40 ft.
12. Col. 14 shows values for the discharge of each individual subsection. The discharges vary directly with the conveyance value and for Section 1 of the example the multiplication factor is

$$\frac{\text{Total discharge}}{\text{Total Conveyance}} = \frac{42{,}000}{368{,}800}$$

For Section 1a,

$$42{,}000 \times \frac{65{,}100}{368{,}800} = 7410$$

The sum of the values must equal the total discharge.

13. Col. 15 is the value of the velocity for each subsection or

$$\frac{\text{Subsection Discharge}}{\text{Subsection Area}} = \frac{\text{Col. 14}}{\text{Col. 4}}$$

14. Col. 16 is self-explanatory.
15. Col. 17 contains values of Col. 14 multiplied by

Col. 16 to obtain the weighted velocity head h_v from the relationship

$$h_v = \frac{\Sigma(V^2Q)}{2gQ}$$

In the example the values for Section 1 are (V^2Q) = 14,102,100 and $2gQ = 2 \times 32.2 \times 42{,}000$ i.e., total discharge multiplied by twice the acceleration of gravity. Values of h_v are shown in Col. 18.

16. Col. 19 shows the algebraic differences in velocity head between sections. In the example the velocity head loss between Sections 1 and 2 in (5.21 - 2.95) = 2.26 feet and between Sections 2 and 3, (2.95 - 3.31) = -.36. The negative value indicates that kinetic head has been converted into static head.

17. Col. 20 shows eddy losses. The eddy losses are calculated as ten percent of the value $h_{v1} - h_{v2}$ when such value is positive and 50 percent thereof when the value is negative.

18. Col. 21 is H for the friction head loss + the velocity head loss + the eddy loss or Col. 13 ± Col. 19 + Col. 20. Note that values in Col. 19 may be positive or negative.

19. Col. 22 shows the water surface elevation. In the example the water surface elevation at Section 1 is 115.0. At Section 2 the water surface elevation is 115.0 + 6.89 = 121.89, etc. If the elevations calculated for subsequent sections do not agree within reasonable limits with the assumed elevations shown in Col. 2, the assumed elevations for such sections must be revised and the section properties and losses recomputed until the desired accuracy is obtained as shown for Section 4. An accuracy of ± 0.3 feet is considered adequate for this type of computation.

When the water surface profile method is used and the starting water surface elevation is definite, the computer can be of great use in calculating several backwater profiles based on several arbitrary starting elevations. If these profiles are plotted as shown in Figure 7, they will tend to converge to a common curve at some point along the way. This is because each successive calculation in proceeding upstream brings the water level nearer the correct profile. The purpose of plotting the curves and finding the convergence point is to determine whether or not the proposed structure site is at or upstream from the convergence point. If so, the calculations have started far enough downstream to define a proper TW from an unknown starting elevation. Otherwise, the calculations should begin again at a point further downstream.

3-300. EROSION CONTROL

3-301. CHANNEL LININGS

It is obvious that lining is costly. But many times its initial cost may be more than offset by the reduced channel size and right of way width caused by the improved hydraulic characteristics of the lining. One of the primary considerations when deciding to line the channel or not, is initial cost.

3-301.1 CONCRETE LINING

Concrete channel lining at present is the most popular material for channel lining. Concrete riprap has the advantages of greatly improving the hydraulic characteristics of the channel and of eliminating considerable future maintenance.

3-301.2 ROCK LINING

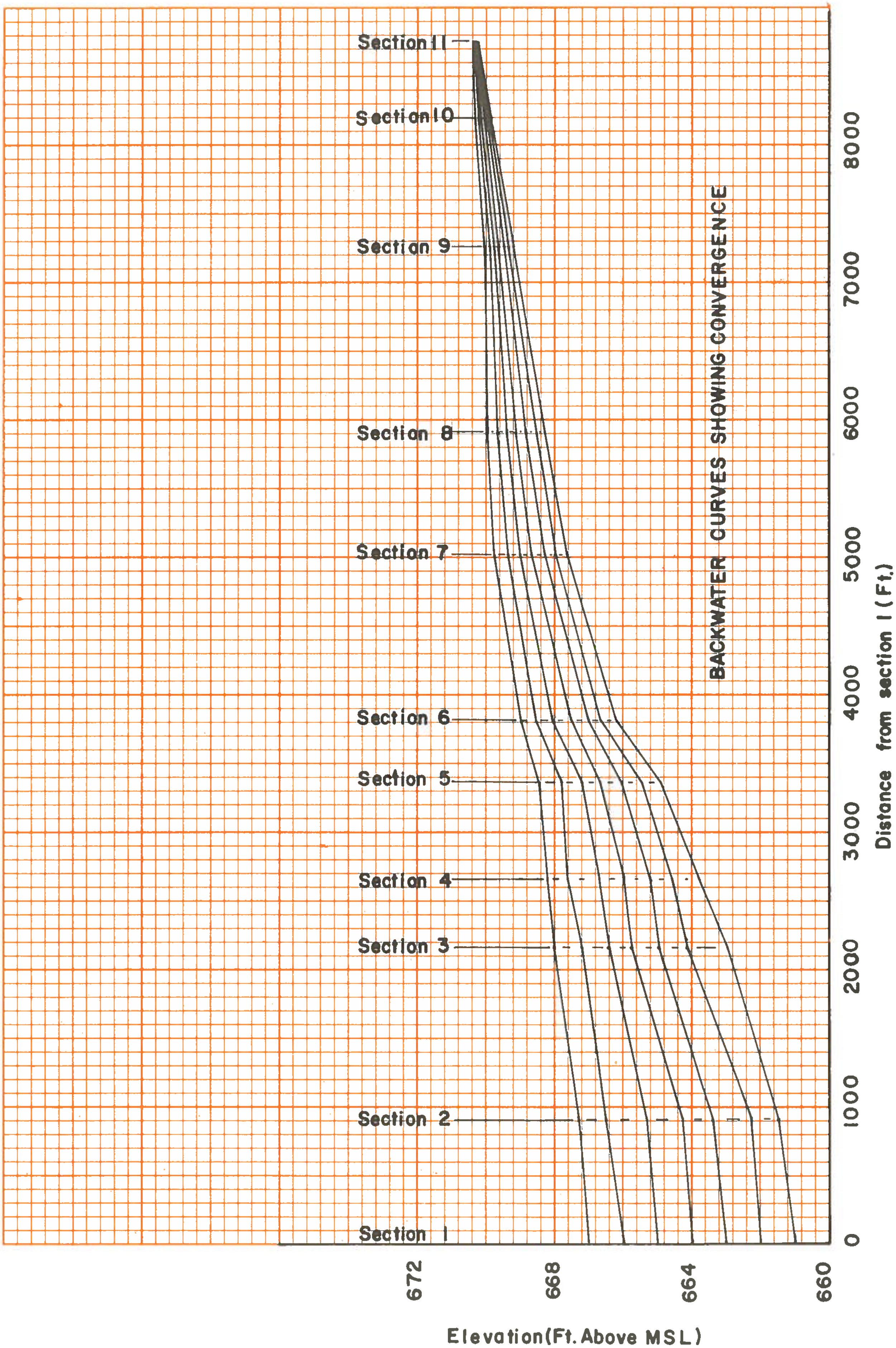
Rock lining, where rock is available, is a suitable substitute for concrete lining from the standpoint of erosion. This rock lining is composed of a graded rock (the size depending on discharge and velocity) that is dumped and bladed to line and grade and left loose. Unlike concrete lining, there are no standards, so the rock lining must be designed. At present a design developed at the University of Minnesota is available from D-5 upon request. Erosion can be effectively controlled and the rock lining should prove to be economically more favorable than concrete lining when there is a local rock source. Hydraulic efficiency is approximately the same as a new earthen channel and vegetation is controlled either by mowing or the application of a herbicide.

3-302. DITCH RETARDS

For ditches on steep grades, ditch retards properly designed provide an acceptable erosion control. The retards are a series of barriers, constructed in the ditch, normal to flow, which cause the water to pond so that the flow is "stepped" down the steep portion of the ditch. Ditches requiring retards, that are near a travel lane, are not compatible with safety requirements, but there are numerous ditches and outfalls where safety is not the problem.

A proper design would include dimensioning the retards so they are high enough to pond the water and cause a hydraulic jump upstream, and also long enough so they extend sufficiently into the side slopes to prevent erosion around the ends. Further, the ditch at the downstream foot of the retard must be properly protected to receive the "spill" over the retard.

A proper height for ditch retards may be obtained by contacting D-5 and furnishing cross section of channel, slope of channel, "n" value, and discharge.



BACKWATER CURVES SHOWING CONVERGENCE
Section 11
Section 10
Section 9
Section 8
Section 7
Section 6
Section 5
Section 4
Section 3
Section 2
Section 1
8000
7000
6000
5000
4000
3000
2000
1000
0
Distance from section 1 (Ft.)
672
668
664
660
Elevation(Ft. Above MSL)

Figure 7.

TABLE I

WATER SURFACE PROFILE COMPUTATIONS

DISCHARGE = 42,000 cfs

(1) Section		(2) Known or Assumed W.S. Elevation	(3) Length L	(4) Area A	(5) Wetted Perimeter WP	(6) Roughness Coefficient n	(7) $\frac{1.486\ A}{n}$	(8) $R = \frac{A}{WP}$	(9) $R^{2/3}$	(10) K (Col.7)(Col.9)	(11) $S_f = \left(\frac{Q}{K}\right)^2$	(12) Mean S_f	(13) Mean $S_f \times L$	(14) Q	(15) V	(16) V^2	(17) V^2Q	(18) $hv = \frac{\sum(V^2Q)}{2gQ}$	(19) $hv_1 - hv_2$	(20) Eddy Losses	(21) ΔH	(22) Water Surface Elevation
1	a	115.0		560	101	0.040	20,800	5.54	3.13	65,100				7,410	13.20	174.24	1,291,100					
	b			580	60	0.070	12,310	9.67	4.54	55,900				6,360	11.00	121.00	769,600					
	c			460	34	0.035	19,530	13.53	5.68	110,900				12,630	27.50	756.25	9,551,400					
	d			560	60	0.070	11,890	9.33	4.43	52,700				6,000	10.71	114.70	688,200					
	e			700	101	0.045	23,120	6.93	3.64	84,200				9,600	13.70	187.69	1,801,800					
										368,800	0.01297			42,000			14,102,100	5.21				115.00
			500																			
2	a	121.80		450	91	0.050	13,370	4.95	2.91	38,900				2,640	5.87	34.46	90,970					
	b			800	80	0.040	29,720	10.00	4.64	137,900				9,370	11.71	137.12	1,284,810					
	c			820	70	0.055	22,150	11.71	5.16	114,300				7,770	9.48	89.87	698,290					
	d			1,050	88	0.030	52,010	11.93	5.22	271,500				18,440	17.56	308.35	5,685,970					
	e			510	85	0.045	16,840	6.00	3.30	55,600				3,780	7.41	54.91	207,560					
										618,200	0.00461	0.00879	4.40	42,000			7,967,600	2.95	2.26	0.23	6.89	121.89
			610																			
3	a	124.80		1,760	123	0.045	58,120	14.31	5.89	342,300				24,430	13.88	192.65	4,706,440					
	b			540	60	0.060	13,370	9.00	4.33	57,900				4,130	7.65	58.52	241,690					
	c			610	40	0.035	25,900	15.25	6.15	159,300				11,370	18.64	347.45	3,950,510					
	d			420	80	0.065	9,600	5.25	3.02	29,000				2,070	4.93	24.30	50,300					
	e									588,500	0.00510	0.00486	2.96	42,000			8,948,940	3.31	-0.36	0.18	2.78	124.67
			1,100																			
4	a	133.20		700	72	0.050	20,800	9.72	4.56	94,800				4,320	6.17	38.07	164,460					
	b			1,780	160	0.070	37,790	11.12	4.98	188,200				8,580	4.82	23.23	199,310					
	c			615	80	0.040	22,850	7.69	3.90	89,100				4,060	6.60	43.56	176,850					
	d			1,435	75	0.035	60,930	19.13	7.15	435,600				19,860	13.84	191.55	3,804,180					
	e			1,040	128	0.055	28,100	8.13	4.04	113,500				5,180	4.98	24.80	128,460					
										921,200	0.00208	0.00359	3.98	42,000			4,473,260	1.65	1.66	0.16	5.80	130.47
			NOTE: As assumed water surface elevation of 133.20 for section 4 does not agree with the calculated water surface elevation of 130.47, the assumed water surface elevation must be revised and properties of section recomputed as shown below.																			
4(rev.)	a	130.20		604	65	0.050	17,950	9.29	4.42	79,300				5,350	8.86	78.50	419,980					
	b			1,300	160	0.070	27,600	8.13	4.04	111,500				7,520	5.78	33.41	251,240					
	c			375	80	0.040	13,930	4.69	2.80	39,000				2,630	7.01	49.14	129,240					
	d			1,210	75	0.035	51,370	16.13	6.38	327,700				22,120	18.28	334.16	7,391,620					
	e			700	110	0.055	18,910	6.36	3.43	64,900				4,380	6.26	39.19	171,650					
										622,400	0.00456	0.00483	5.31	42,000			8,363,730	3.09	0.22	0.02	5.55	130.22
			900																			
5	a	135.70		1,300	160	0.070	27,600	8.13	4.04	111,500				11,480	8.83	77.97	896,000					
	b			1,400	103	0.040	52,010	13.59	5.70	296,500				30,520	21.80	475.24	14,504,320					
										408,000	0.01060	0.00758	6.82	42,000			15,400,320	5.69	-2.60	1.30	5.52	135.74
			700																			
6	a	144.30		860	82	0.065	19,660	10.49	4.79	94,200				11,200	13.02	169.52	1,898,620					
	b			1,110	112	0.035	47,130	9.91	4.61	217,300				25,840	23.28	541.96	14,004,250					
	c			470	79	0.055	12,700	5.95	3.28	41,700				4,960	10.55	111.30	552,050					
										353,200	0.01414	0.01237	8.66	42,000			16,454,920	6.08	-0.39	0.20	8.47	144.21

FORM 1290

TABLE II

MANNING'S ROUGHNESS COEFFICIENTS

NATURAL STREAM CHANNELS	Min.	Max.
I. Minor Streams		
A. Fairly regular section		
1. Some grass and weeds; little or no brush	0.030	0.035
2. Dense growth of weeds, depth of flow materially greater than weed height	0.035	0.050
3. Some weeds, light brush on banks	0.035	0.050
4. Some weeds, heavy brush on banks	0.050	0.070
5. Some weeds, dense willows on banks	0.060	0.080
6. For trees within channels with branches submerged at high stage, increase all values above by	0.010	0.020
B. Irregular section with pools, slight channel meander, use 1A to 5A above, and increase all values by	0.010	0.020
C. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage		
1. Bottom; gravel, cobbles and few boulders	0.040	0.050
2. Bottom; cobbles with large boulders	0.050	0.070
II. Flood Plain (adjacent to natural streams)		
A. Pasture, no brush		
1. Short grass	0.030	0.035
2. Tall grass	0.035	0.050
B. Cultivated areas		
1. No crop	0.030	0.040
2. Mature row crops	0.035	0.045
3. Mature field crops	0.040	0.050
C. Heavy weeds, scattered brush	0.050	0.070
D. Wooded	0.075	0.150

This varies depending on undergrowth, height of foliage on trees, etc. The area of "n" = 0.10 and greater indicates an extremely heavily wooded condition. These instances of high "n" values (greater than "n" = 0.10) should be thoroughly investigated (photographs, consultation with experienced engineers, complete knowledge of area, etc.). The D-5 hydraulic section has several references available for "n" value determination.

III. Major Streams

Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of "n" for larger streams of mostly regular Sections, with no boulders or brush may be in the range of 0.028 to 0.033.

LINED CHANNELS	Min.	Max.
1. Metal corrugated	0.021	0.024
2. Neat cement lined	0.012	0.018
3. Concrete	0.012	0.018
4. Cement rubble	0.017	0.030

TABLE II (Continued)

GRASS COVERED SMALL CHANNELS, SHALLOW DEPTH

1.	No rank growth	0.035	0.045
2.	Rank growth	0.040	0.050

UNLINED CHANNELS

1.	Earth, straight and uniform	0.017	0.025
2.	Dredged	0.025	0.033
3.	Winding and sluggish	0.022	0.030
4.	Stony beds, weeds on bank	0.025	0.040
5.	Earth bottom, rubble sides	0.028	0.035
6.	Rock cuts, smooth and uniform	0.025	0.035
7.	Rock cuts, rugged and irregular	0.035	0.045

PIPE

1.	Cast iron, coated	0.010	0.014
2.	Cast iron, uncoated	0.011	0.015
3.	Wrought iron, galvanized	0.013	0.017
4.	Wrought iron, black	0.012	0.015
5.	Steel, riveted and spiral - smooth	0.013	0.017
6.	Steel, corrugated (1/2")	0.021	0.024
7.	Steel, corrugated (2" Structural Plate)	0.034	0.038
8.	Concrete	0.010	0.017
9.	Vitrified sewer pipe	0.010	0.017
10.	Clay, common drainage tile	0.011	0.017

TABLE III

Square Roots of Decimal Numbers

Number	.---0	.---1	.---2	.---3	.---4	.---5	.---6	.---7	.---8	.---9
.00001	.003162	.003317	.003464	.003606	.003742	.003873	.004000	.004123	.004243	.004359
.00002	.004472	.004583	.004690	.004796	.004899	.005000	.005099	.005196	.005292	.005385
.00003	.005477	.005568	.005657	.005745	.005831	.005916	.006000	.006083	.006164	.006245
.00004	.006325	.006403	.006481	.006557	.006633	.006708	.006782	.006856	.006928	.007000
.00005	.007071	.007141	.007211	.007280	.007348	.007416	.007483	.007550	.007616	.007681
.00006	.007746	.007810	.007874	.007937	.008000	.008062	.008124	.008185	.008246	.008307
.00007	.008367	.008426	.008485	.008544	.008602	.008660	.008718	.008775	.008832	.008888
.00008	.008944	.009000	.009055	.009110	.009165	.009220	.009274	.009327	.009381	.009434
.00009	.009487	.009539	.009592	.009644	.009695	.009747	.009798	.009849	.009899	.009950
.00010	.010000	.010050	.010100	.010149	.010198	.010247	.010296	.010344	.010392	.010440
.0001	.01000	.01049	.01095	.01140	.01183	.01225	.01265	.01304	.01342	.01378
.0002	.01414	.01449	.01483	.01517	.01549	.01581	.01612	.01643	.01673	.01703
.0003	.01732	.01761	.01789	.01817	.01844	.01871	.01897	.01924	.01949	.01975
.0004	.02000	.02025	.02049	.02074	.02098	.02121	.02145	.02168	.02191	.02214
.0005	.02236	.02258	.02280	.02302	.02324	.02345	.02366	.02387	.02408	.02429
.0006	.02449	.02470	.02490	.02510	.02530	.02550	.02569	.02588	.02608	.02627
.0007	.02646	.02665	.02683	.02702	.02720	.02739	.02757	.02775	.02793	.02811
.0008	.02828	.02846	.02864	.02881	.02898	.02915	.02933	.02950	.02966	.02983
.0009	.03000	.03017	.03033	.03050	.03066	.03082	.03098	.03114	.03130	.03146
.0010	.03162	.03178	.03194	.03209	.03225	.03240	.03256	.03271	.03286	.03302
.001	.03162	.03317	.03464	.03606	.03742	.03873	.04000	.04123	.04243	.04359
.002	.04472	.04583	.04690	.04796	.04899	.05000	.05099	.05196	.05292	.05385
.003	.05477	.05568	.05657	.05745	.05831	.05916	.06000	.06083	.06164	.06245
.004	.06325	.06403	.06481	.06557	.06633	.06708	.06782	.06856	.06928	.07000
.005	.07071	.07141	.07211	.07280	.07348	.07416	.07483	.07550	.07616	.07681
.006	.07746	.07810	.07874	.07937	.08000	.08062	.08124	.08185	.08246	.08307
.007	.08367	.08426	.08485	.08544	.08602	.08660	.08718	.08775	.08832	.08888
.008	.08944	.09000	.09055	.09110	.09165	.09220	.09274	.09327	.09381	.09434
.009	.09487	.09539	.09592	.09644	.09695	.09747	.09798	.09849	.09899	.09950
.010	.10000	.10050	.10100	.10149	.10198	.10247	.10296	.10344	.10392	.10440
.01	.1000	.1049	.1095	.1140	.1183	.1225	.1265	.1304	.1342	.1378
.02	.1414	.1449	.1483	.1517	.1549	.1581	.1612	.1643	.1673	.1703
.03	.1732	.1761	.1789	.1817	.1844	.1871	.1897	.1924	.1949	.1975
.04	.2000	.2025	.2049	.2074	.2098	.2121	.2145	.2168	.2191	.2214
.05	.2236	.2258	.2280	.2302	.2324	.2345	.2366	.2387	.2408	.2429
.06	.2449	.2470	.2490	.2510	.2530	.2550	.2569	.2588	.2608	.2627
.07	.2646	.2665	.2683	.2702	.2720	.2739	.2757	.2775	.2793	.2811
.08	.2828	.2846	.2864	.2881	.2898	.2915	.2933	.2950	.2966	.2983
.09	.3000	.3017	.3033	.3050	.3066	.3082	.3098	.3114	.3130	.3146
.10	.3162	.3178	.3194	.3209	.3225	.3240	.3256	.3271	.3286	.3302

TABLE IV

Two-thirds Powers of Numbers

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.046	.074	.097	.117	.136	.153	.170	.186	.201
.1	.215	.229	.243	.256	.269	.282	.295	.307	.319	.331
.2	.342	.353	.364	.375	.386	.397	.407	.418	.428	.438
.3	.448	.458	.468	.477	.487	.497	.506	.515	.525	.534
.4	.543	.552	.561	.570	.578	.587	.596	.604	.613	.622
.5	.630	.638	.647	.655	.663	.671	.679	.687	.695	.703
.6	.711	.719	.727	.735	.743	.750	.758	.765	.773	.781
.7	.788	.796	.803	.811	.818	.825	.832	.840	.847	.855
.8	.862	.869	.876	.883	.890	.897	.904	.911	.918	.925
.9	.932	.939	.946	.953	.960	.966	.973	.980	.987	.993
1.0	1.000	1.007	1.013	1.020	1.027	1.033	1.040	1.046	1.053	1.059
1.1	1.065	1.072	1.078	1.085	1.091	1.097	1.104	1.110	1.117	1.123
1.2	1.129	1.136	1.142	1.148	1.154	1.160	1.167	1.173	1.179	1.185
1.3	1.191	1.197	1.203	1.209	1.215	1.221	1.227	1.233	1.239	1.245
1.4	1.251	1.257	1.263	1.269	1.275	1.281	1.287	1.293	1.299	1.305
1.5	1.310	1.316	1.322	1.328	1.334	1.339	1.345	1.351	1.357	1.362
1.6	1.368	1.374	1.379	1.385	1.391	1.396	1.402	1.408	1.413	1.419
1.7	1.424	1.430	1.436	1.441	1.447	1.452	1.458	1.463	1.469	1.474
1.8	1.480	1.485	1.491	1.496	1.502	1.507	1.513	1.518	1.523	1.529
1.9	1.534	1.539	1.545	1.550	1.556	1.561	1.566	1.571	1.577	1.582
2.0	1.587	1.593	1.598	1.603	1.608	1.613	1.619	1.624	1.629	1.634
2.1	1.639	1.645	1.650	1.655	1.660	1.665	1.671	1.676	1.681	1.686
2.2	1.691	1.697	1.702	1.707	1.712	1.717	1.722	1.727	1.732	1.737
2.3	1.742	1.747	1.752	1.757	1.762	1.767	1.772	1.777	1.782	1.787
2.4	1.792	1.797	1.802	1.807	1.812	1.817	1.822	1.827	1.832	1.837
2.5	1.842	1.847	1.852	1.857	1.862	1.867	1.871	1.876	1.881	1.886
2.6	1.891	1.896	1.900	1.905	1.910	1.915	1.920	1.925	1.929	1.934
2.7	1.939	1.944	1.949	1.953	1.958	1.963	1.968	1.972	1.977	1.982
2.8	1.987	1.992	1.996	2.001	2.006	2.010	2.015	2.020	2.024	2.029
2.9	2.034	2.038	2.043	2.048	2.052	2.057	2.062	2.066	2.071	2.075
3.0	2.080	2.085	2.089	2.094	2.099	2.103	2.108	2.112	2.117	2.122
3.1	2.126	2.131	2.135	2.140	2.144	2.149	2.153	2.158	2.163	2.167
3.2	2.172	2.176	2.180	2.185	2.190	2.194	2.199	2.203	2.208	2.212
3.3	2.217	2.221	2.226	2.230	2.234	2.239	2.243	2.248	2.252	2.257
3.4	2.261	2.265	2.270	2.274	2.279	2.283	2.288	2.292	2.296	2.301
3.5	2.305	2.310	2.314	2.318	2.323	2.327	2.331	2.336	2.340	2.345
3.6	2.349	2.353	2.358	2.362	2.366	2.371	2.375	2.379	2.384	2.388
3.7	2.392	2.397	2.401	2.405	2.409	2.414	2.418	2.422	2.427	2.431
3.8	2.435	2.439	2.444	2.448	2.452	2.457	2.461	2.465	2.469	2.474
3.9	2.478	2.482	2.486	2.490	2.495	2.499	2.503	2.507	2.511	2.516
4.0	2.520	2.524	2.528	2.532	2.537	2.541	2.545	2.549	2.553	2.558
4.1	2.562	2.566	2.570	2.574	2.579	2.583	2.587	2.591	2.595	2.599
4.2	2.603	2.607	2.611	2.616	2.620	2.624	2.628	2.632	2.636	2.640
4.3	2.644	2.648	2.653	2.657	2.661	2.665	2.669	2.673	2.677	2.681
4.4	2.685	2.689	2.693	2.698	2.702	2.706	2.710	2.714	2.718	2.722
4.5	2.726	2.730	2.734	2.738	2.742	2.746	2.750	2.754	2.758	2.762
4.6	2.766	2.770	2.774	2.778	2.782	2.786	2.790	2.794	2.798	2.802
4.7	2.806	2.810	2.814	2.818	2.822	2.826	2.830	2.834	2.838	2.842
4.8	2.846	2.850	2.854	2.858	2.862	2.865	2.869	2.873	2.877	2.881
4.9	2.885	2.889	2.893	2.897	2.901	2.904	2.908	2.912	2.916	2.920

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
5.0	2.924	2.928	2.932	2.936	2.940	2.944	2.947	2.951	2.955	2.959
5.1	2.963	2.967	2.971	2.975	2.979	2.982	2.986	2.990	2.994	2.998
5.2	3.001	3.005	3.009	3.013	3.017	3.021	3.024	3.028	3.032	3.036
5.3	3.040	3.014	3.047	3.051	3.055	3.059	3.063	3.067	3.070	3.074
5.4	3.078	3.082	3.086	3.089	3.093	3.097	3.101	3.105	3.108	3.112
5.5	3.116	3.120	3.123	3.127	3.131	3.135	3.138	3.142	3.146	3.150
5.6	3.154	3.157	3.161	3.165	3.169	3.172	3.176	3.180	3.184	3.187
5.7	3.191	3.195	3.198	3.202	3.206	3.210	3.213	3.217	3.221	3.224
5.8	3.228	3.232	3.236	3.239	3.243	3.247	3.250	3.254	3.258	3.261
5.9	3.265	3.269	3.273	3.276	3.280	3.284	3.287	3.291	3.295	3.298
6.0	3.302	3.306	3.309	3.313	3.317	3.320	3.324	3.328	3.331	3.335
6.1	3.339	3.342	3.346	3.350	3.353	3.357	3.360	3.364	3.368	3.371
6.2	3.375	3.379	3.382	3.386	3.389	3.393	3.397	3.400	3.404	3.408
6.3	3.411	3.415	3.418	3.422	3.426	3.429	3.433	3.436	3.440	3.444
6.4	3.447	3.451	3.454	3.458	3.461	3.465	3.469	3.472	3.476	3.479
6.5	3.483	3.486	3.490	3.494	3.497	3.501	3.504	3.508	3.511	3.515
6.6	3.519	3.522	3.526	3.529	3.533	3.536	3.540	3.543	3.547	3.550
6.7	3.554	3.558	3.561	3.565	3.568	3.572	3.575	3.579	3.582	3.586
6.8	3.580	3.593	3.596	3.600	3.603	3.607	3.610	3.614	3.617	3.621
6.9	3.624	3.628	3.631	3.635	3.638	3.642	3.645	3.649	3.652	3.656
7.0	3.659	3.663	3.666	3.670	3.673	3.677	3.680	3.684	3.687	3.691
7.1	3.694	3.698	3.701	3.705	3.708	3.712	3.715	3.718	3.722	3.725
7.2	3.729	3.732	3.736	3.739	3.742	3.746	3.749	3.753	3.756	3.760
7.3	3.763	3.767	3.770	3.773	3.777	3.780	3.784	3.787	3.791	3.794
7.4	3.797	3.801	3.804	3.808	3.811	3.814	3.818	3.821	3.825	3.828
7.5	3.832	3.835	3.838	3.842	3.845	3.849	3.852	3.855	3.859	3.862
7.6	3.866	3.869	3.872	3.876	3.879	3.883	3.886	3.889	3.893	3.896
7.7	3.899	3.903	3.906	3.910	3.913	3.916	3.920	3.923	3.926	3.930
7.8	3.933	3.937	3.940	3.943	3.947	3.950	3.953	3.957	3.960	3.963
7.9	3.967	3.970	3.973	3.977	3.980	3.983	3.987	3.990	3.993	3.997
8.0	4.000	4.003	4.007	4.010	4.013	4.017	4.020	4.023	4.027	4.030
8.1	4.033	4.037	4.040	4.043	4.047	4.050	4.053	4.057	4.060	4.063
8.2	4.066	4.070	4.073	4.076	4.080	4.083	4.086	4.090	4.093	4.096
8.3	4.099	4.103	4.106	4.109	4.113	4.116	4.119	4.122	4.126	4.129
8.4	4.132	4.136	4.139	4.142	4.145	4.149	4.152	4.155	4.159	4.162
8.5	4.165	4.168	4.172	4.175	4.178	4.181	4.185	4.188	4.191	4.194
8.6	4.198	4.201	4.204	4.207	4.211	4.214	4.217	4.220	4.224	4.227
8.7	4.230	4.233	4.237	4.240	4.243	4.246	4.249	4.253	4.256	4.259
8.8	4.262	4.266	4.269	4.272	4.275	4.279	4.282	4.285	4.288	4.291
8.9	4.295	4.298	4.301	4.304	4.307	4.311	4.314	4.317	4.320	4.324
9.0	4.327	4.330	4.333	4.336	4.340	4.343	4.346	4.349	4.352	4.356
9.1	4.359	4.362	4.365	4.368	4.372	4.375	4.378	4.381	4.384	4.387
9.2	4.391	4.394	4.397	4.400	4.403	4.407	4.410	4.413	4.416	4.419
9.3	4.422	4.426	4.429	4.432	4.435	4.438	4.441	4.445	4.448	4.451
9.4	4.454	4.457	4.460	4.464	4.467	4.470	4.473	4.476	4.479	4.482
9.5	4.486	4.489	4.492	4.495	4.498	4.501	4.504	4.508	4.511	4.514
9.6	4.517	4.520	4.523	4.526	4.530	4.533	4.536	4.539	4.542	4.545
9.7	4.548	4.551	4.555	4.558	4.561	4.564	4.567	4.570	4.573	4.576
9.8	4.580	4.583	4.586	4.589	4.592	4.595	4.598	4.601	4.604	4.608
9.9	4.611	4.614	4.617	4.620	4.623	4.626	4.629	4.632	4.635	4.639
10.0	4.642									

TABLE IV

Two-thirds Powers of Numbers (Cont.)

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
10.0	4.642	4.645	4.648	4.651	4.654	4.657	4.660	4.663	4.666	4.669
10.1	4.672	4.676	4.679	4.682	4.685	4.688	4.691	4.694	4.697	4.700
10.2	4.703	4.706	4.709	4.712	4.716	4.719	4.722	4.725	4.728	4.731
10.3	4.734	4.737	4.740	4.743	4.746	4.749	4.752	4.755	4.758	4.762
10.4	4.765	4.768	4.771	4.774	4.777	4.780	4.783	4.786	4.789	4.792
10.5	4.795	4.798	4.801	4.804	4.807	4.810	4.813	4.816	4.819	4.822
10.6	4.825	4.828	4.832	4.835	4.838	4.841	4.844	4.847	4.850	4.853
10.7	4.856	4.859	4.862	4.865	4.868	4.871	4.874	4.877	4.880	4.883
10.8	4.886	4.889	4.892	4.895	4.898	4.901	4.904	4.907	4.910	4.913
10.9	4.916	4.919	4.922	4.925	4.928	4.931	4.934	4.937	4.940	4.943
11.0	4.946	4.949	4.952	4.955	4.958	4.961	4.964	4.967	4.970	4.973
11.1	4.976	4.979	4.982	4.985	4.988	4.901	4.994	4.997	5.000	5.003
11.2	5.006	5.009	5.012	5.015	5.018	5.021	5.024	5.027	5.030	5.033
11.3	5.036	5.039	5.042	5.044	5.047	5.050	5.053	5.056	5.059	5.062
11.4	5.065	5.068	5.071	5.074	5.077	5.080	5.083	5.086	5.089	5.092
11.5	5.095	5.098	5.101	5.104	5.107	5.110	5.113	5.115	5.118	5.121
11.6	5.124	5.127	5.130	5.133	5.136	5.139	5.142	5.145	5.148	5.151
11.7	5.154	5.157	5.160	5.163	5.166	5.168	5.171	5.174	5.177	5.180
11.8	5.183	5.186	5.189	5.192	5.195	5.198	5.201	5.204	5.207	5.209
11.9	5.212	5.215	5.218	5.221	5.224	5.227	5.230	5.233	5.236	5.238
12.0	5.241	5.244	5.247	5.250	5.253	5.256	5.259	5.262	5.265	5.268
12.1	5.271	5.273	5.276	5.279	5.282	5.285	5.288	5.291	5.294	5.297
12.2	5.300	5.302	5.305	5.308	5.311	5.314	5.317	5.320	5.323	5.326
12.3	5.329	5.331	5.334	5.337	5.340	5.343	5.346	5.349	5.352	5.354
12.4	5.357	5.360	5.363	5.366	5.369	5.372	5.375	5.377	5.380	5.383
12.5	5.386	5.389	5.392	5.395	5.398	5.400	5.403	5.406	5.409	5.412
12.6	5.415	5.418	5.421	5.423	5.426	5.429	5.432	5.435	5.438	5.440
12.7	5.443	5.446	5.449	5.452	5.455	5.458	5.461	5.463	5.466	5.469
12.8	5.472	5.475	5.478	5.480	5.483	5.486	5.489	5.492	5.495	5.498
12.9	5.500	5.503	5.506	5.509	5.512	5.515	5.517	5.520	5.523	5.526
13.0	5.529	5.532	5.534	5.537	5.540	5.543	5.546	5.549	5.551	5.554
13.1	5.557	5.560	5.563	5.566	5.568	5.571	5.574	5.577	5.580	5.582
13.2	5.585	5.588	5.591	5.594	5.597	5.600	5.602	5.605	5.608	5.611
13.3	5.614	5.616	5.619	5.622	5.625	5.628	5.630	5.633	5.636	5.639
13.4	5.642	5.644	5.647	5.650	5.653	5.656	5.658	5.661	5.664	5.667
13.5	5.670	5.672	5.675	5.678	5.681	5.684	5.686	5.689	5.692	5.695
13.6	5.698	5.700	5.703	5.706	5.709	5.712	5.714	5.717	5.720	5.723
13.7	5.725	5.728	5.731	5.734	5.737	5.739	5.742	5.745	5.748	5.751
13.8	5.753	5.756	5.759	5.762	5.765	5.767	5.770	5.773	5.776	5.778
13.9	5.781	5.784	5.787	5.789	5.792	5.795	5.798	5.801	5.803	5.806
14.0	5.809	5.812	5.814	5.817	5.820	5.823	5.825	5.828	5.831	5.834
14.1	5.836	5.839	5.842	5.845	5.847	5.850	5.853	5.856	5.859	5.861
14.2	5.864	5.867	5.870	5.872	5.875	5.878	5.880	5.883	5.886	5.889
14.3	5.892	5.894	5.897	5.900	5.902	5.905	5.908	5.911	5.913	5.916
14.4	5.919	5.922	5.924	5.927	5.930	5.933	5.935	5.938	5.941	5.944
14.5	5.946	5.949	5.952	5.955	5.957	5.960	5.963	5.965	5.968	5.971
14.6	5.974	5.976	5.979	5.982	5.985	5.987	5.990	5.993	5.995	5.998
14.7	6.001	6.004	6.006	6.009	6.012	6.014	6.017	6.020	6.023	6.025
14.8	6.028	6.031	6.034	6.036	6.039	6.042	6.044	6.047	6.050	6.052
14.9	6.055	6.058	6.061	6.063	6.066	6.069	6.071	6.074	6.077	6.080

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
15.0	6.082	6.085	6.088	6.090	6.093	6.096	6.098	6.101	6.104	6.106
15.1	6.109	6.112	6.115	6.117	6.120	6.123	6.125	6.128	6.131	6.133
15.2	6.136	6.139	6.141	6.144	6.147	6.150	6.152	6.155	6.158	6.160
15.3	6.163	6.166	6.168	6.171	6.174	6.176	6.179	6.182	6.184	6.187
15.4	6.190	6.193	6.195	6.198	6.200	6.203	6.206	6.209	6.211	6.214
15.5	6.217	6.219	6.222	6.225	6.227	6.230	6.233	6.235	6.238	6.241
15.6	6.243	6.246	6.249	6.251	6.254	6.257	6.259	6.262	6.265	6.267
15.7	6.270	6.273	6.275	6.278	6.281	6.283	6.286	6.289	6.291	6.294
15.8	6.297	6.299	6.302	6.305	6.307	6.310	6.312	6.315	6.318	6.320
15.9	6.323	6.326	6.328	6.331	6.334	6.336	6.339	6.342	6.344	6.347
16.0	6.350	6.352	6.355	6.358	6.360	6.363	6.365	6.368	6.371	6.373
16.1	6.376	6.379	6.381	6.384	6.387	6.389	6.392	6.394	6.397	6.400
16.2	6.402	6.405	6.408	6.410	6.413	6.416	6.418	6.421	6.423	6.426
16.3	6.429	6.431	6.434	6.437	6.439	6.442	6.444	6.447	6.450	6.452
16.4	6.455	6.458	6.460	6.463	6.465	6.468	6.471	6.473	6.476	6.479
16.5	6.481	6.484	6.486	6.489	6.492	6.494	6.497	6.500	6.502	6.505
16.6	6.507	6.510	6.513	6.515	6.518	6.520	6.523	6.526	6.528	6.531
16.7	6.533	6.536	6.539	6.541	6.544	6.546	6.549	6.552	6.554	6.557
16.8	6.560	6.562	6.565	6.567	6.570	6.573	6.575	6.578	6.580	6.583
16.9	6.586	6.588	6.591	6.593	6.596	6.599	6.601	6.604	6.606	6.609
17.0	6.611	6.614	6.617	6.619	6.622	6.624	6.627	6.620	6.632	6.635
17.1	6.637	6.640	6.643	6.645	6.648	6.650	6.653	6.655	6.658	6.661
17.2	6.663	6.666	6.668	6.671	6.674	6.676	6.679	6.681	6.684	6.686
17.3	6.689	6.692	6.694	6.697	6.699	6.702	6.705	6.707	6.710	6.712
17.4	6.715	6.717	6.720	6.723	6.725	6.728	6.730	6.733	6.735	6.738
17.5	6.740	6.743	6.746	6.748	6.751	6.753	6.756	6.758	6.761	6.764
17.6	6.766	6.769	6.771	6.774	6.776	6.779	6.782	6.784	6.787	6.789
17.7	6.792	6.794	6.797	6.799	6.802	6.805	6.807	6.810	6.812	6.815
17.8	6.817	6.820	6.822	6.825	6.828	6.830	6.833	6.835	6.838	6.840
17.9	6.843	6.845	6.848	6.850	6.853	6.856	6.858	6.861	6.863	6.866
18.0	6.868	6.871	6.873	6.876	6.878	6.881	6.884	6.886	6.889	6.891
18.1	6.894	6.896	6.899	6.901	6.904	6.906	6.909	6.911	6.914	6.917
18.2	6.919	6.922	6.924	6.927	6.920	6.932	6.934	6.937	6.939	6.942
18.3	6.944	6.947	6.949	6.952	6.954	6.957	6.960	6.962	6.965	6.967
18.4	6.970	6.972	6.975	6.977	6.980	6.982	6.985	6.987	6.990	6.992
18.5	6.995	6.997	7.000	7.002	7.005	7.007	7.010	7.013	7.015	7.018
18.6	7.020	7.023	7.025	7.028	7.030	7.032	7.035	7.038	7.040	7.043
18.7	7.045	7.048	7.050	7.053	7.055	7.058	7.060	7.063	7.065	7.068
18.8	7.070	7.073	7.075	7.078	7.080	7.083	7.085	7.088	7.090	7.093
18.9	7.095	7.098	7.100	7.103	7.105	7.108	7.110	7.113	7.115	7.118
19.0	7.120	7.123	7.125	7.128	7.130	7.133	7.135	7.138	7.140	7.143
19.1	7.145	7.148	7.150	7.153	7.155	7.158	7.160	7.163	7.165	7.168
19.2	7.170	7.173	7.175	7.178	7.180	7.183	7.185	7.188	7.190	7.193
19.3	7.195	7.198	7.200	7.203	7.205	7.208	7.210	7.212	7.215	7.217
19.4	7.220	7.222	7.225	7.227	7.230	7.232	7.235	7.237	7.240	7.242
19.5	7.245	7.247	7.250	7.252	7.255	7.257	7.260	7.262	7.265	7.267
19.6	7.270	7.272	7.274	7.277	7.279	7.282	7.284	7.287	7.289	7.292
19.7	7.294	7.297	7.299	7.302	7.304	7.307	7.309	7.311	7.314	7.316
19.8	7.319	7.321	7.324	7.326	7.329	7.331	7.334	7.336	7.339	7.341
19.9	7.343	7.346	7.348	7.351	7.353	7.356	7.358	7.361	7.363	7.366

TABLE IV

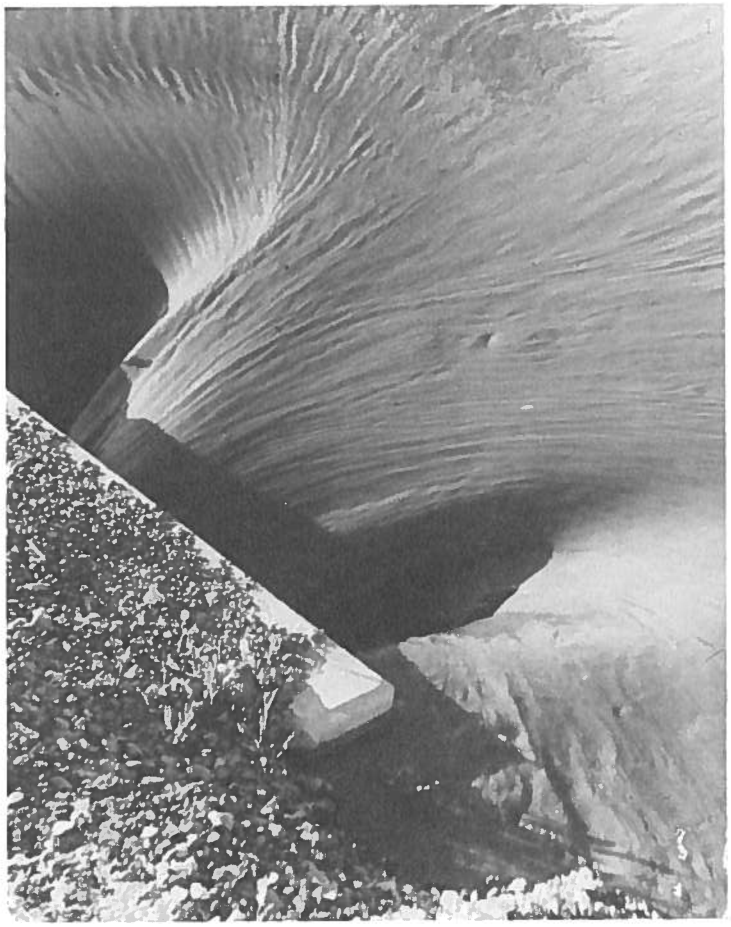
Two-thirds Powers of Numbers (Cont.)

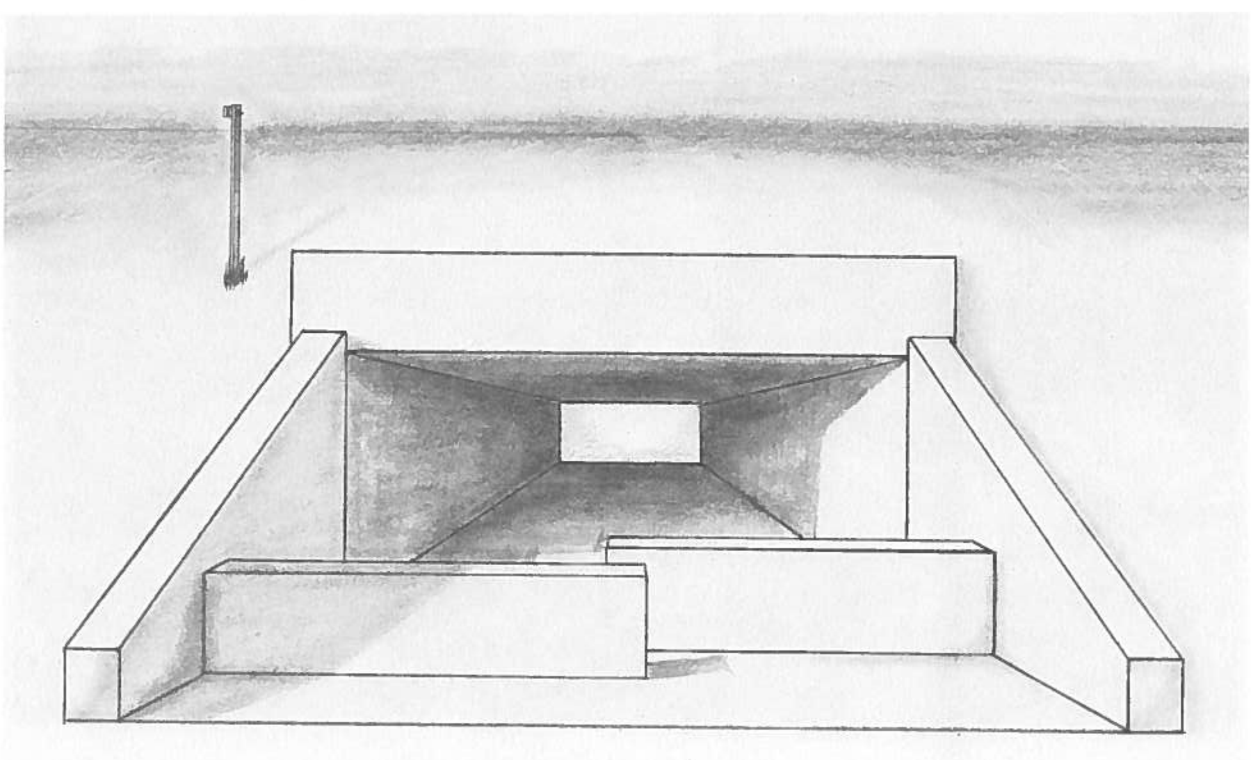
Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
20.0	7.368	7.371	7.373	7.375	7.378	7.380	7.383	7.385	7.388	7.390
20.1	7.393	7.395	7.397	7.400	7.402	7.405	7.407	7.410	7.412	7.415
20.2	7.417	7.420	7.422	7.424	7.427	7.429	7.432	7.434	7.437	7.439
20.3	7.442	7.444	7.446	7.449	7.451	7.454	7.456	7.459	7.461	7.464
20.4	7.466	7.468	7.471	7.473	7.476	7.478	7.481	7.483	7.485	7.488
20.5	7.490	7.493	7.495	7.498	7.500	7.503	7.505	7.507	7.510	7.512
20.6	7.515	7.517	7.520	7.522	7.524	7.527	7.529	7.532	7.534	7.537
20.7	7.539	7.541	7.544	7.546	7.549	7.551	7.554	7.556	7.558	7.561
20.8	7.563	7.566	7.568	7.571	7.573	7.575	7.578	7.580	7.583	7.585
20.9	7.587	7.590	7.592	7.595	7.597	7.600	7.602	7.604	7.607	7.609
21.0	7.612	7.614	7.617	7.619	7.621	7.624	7.626	7.629	7.631	7.633
21.1	7.636	7.638	7.641	7.643	7.645	7.648	7.650	7.653	7.655	7.658
21.2	7.660	7.662	7.665	7.667	7.670	7.672	7.674	7.677	7.679	7.682
21.3	7.684	7.686	7.689	7.691	7.694	7.696	7.698	7.701	7.703	7.706
21.4	7.708	7.710	7.713	7.715	7.718	7.720	7.722	7.725	7.727	7.730
21.5	7.732	7.734	7.737	7.739	7.742	7.744	7.746	7.749	7.751	7.754
21.6	7.756	7.758	7.761	7.763	7.766	7.768	7.770	7.773	7.775	7.778
21.7	7.780	7.782	7.785	7.787	7.789	7.792	7.794	7.797	7.799	7.801
21.8	7.804	7.806	7.809	7.811	7.813	7.816	7.818	7.820	7.823	7.825
21.9	7.828	7.830	7.832	7.835	7.837	7.840	7.842	7.844	7.847	7.849
22.0	7.851	7.854	7.856	7.859	7.861	7.863	7.866	7.868	7.870	7.873
22.1	7.875	7.878	7.880	7.882	7.885	7.887	7.889	7.892	7.894	7.897
22.2	7.809	7.901	7.904	7.906	7.908	7.911	7.913	7.916	7.918	7.920
22.3	7.923	7.925	7.927	7.930	7.932	7.934	7.937	7.939	7.942	7.944
22.4	7.946	7.949	7.951	7.953	7.956	7.958	7.960	7.963	7.965	7.968
22.5	7.970	7.972	7.975	7.977	7.979	7.982	7.984	7.986	7.989	7.991
22.6	7.994	7.996	7.998	8.001	8.003	8.005	8.008	8.010	8.012	8.015
22.7	8.017	8.019	8.022	8.024	8.027	8.029	8.031	8.034	8.036	8.038
22.8	8.041	8.043	8.045	8.048	8.050	8.052	8.055	8.057	8.059	8.062
22.9	8.064	8.066	8.069	8.071	8.074	8.076	8.078	8.081	8.083	8.085
23.0	8.088	8.090	8.092	8.095	8.097	8.099	8.102	8.104	8.106	8.109
23.1	8.111	8.113	8.116	8.118	8.120	8.123	8.125	8.127	8.130	8.132
23.2	8.134	8.137	8.139	8.141	8.144	8.146	8.148	8.151	8.153	8.155
23.3	8.158	8.160	8.162	8.165	8.167	8.169	8.172	8.174	8.176	8.179
23.4	8.181	8.183	8.186	8.188	8.190	8.193	8.195	8.197	8.200	8.202
23.5	8.204	8.207	8.209	8.211	8.214	8.216	8.218	8.221	8.223	8.225
23.6	8.228	8.230	8.232	8.235	8.237	8.239	8.242	8.244	8.246	8.249
23.7	8.251	8.253	8.255	8.258	8.260	8.262	8.265	8.267	8.269	8.272
23.8	8.274	8.276	8.279	8.281	8.283	8.286	8.288	8.290	8.293	8.295
23.9	8.297	8.300	8.302	8.304	8.306	8.309	8.311	8.313	8.316	8.318
24.0	8.320	8.323	8.325	8.327	8.330	8.332	8.334	8.337	8.339	8.341
24.1	8.343	8.346	8.348	8.350	8.353	8.355	8.357	8.360	8.362	8.364
24.2	8.366	8.369	8.371	8.374	8.376	8.378	8.380	8.383	8.385	8.387
24.3	8.390	8.392	8.394	8.396	8.399	8.401	8.403	8.406	8.408	8.410
24.4	8.413	8.415	8.417	8.419	8.422	8.424	8.426	8.429	8.431	8.433
24.5	8.436	8.438	8.440	8.442	8.445	8.447	8.449	8.452	8.454	8.456
24.6	8.458	8.461	8.463	8.465	8.468	8.470	8.472	8.474	8.477	8.479
24.7	8.481	8.484	8.486	8.488	8.490	8.493	8.495	8.497	8.500	8.502
24.8	8.504	8.507	8.509	8.511	8.513	8.516	8.518	8.520	8.522	8.525
24.9	8.527	8.529	8.532	8.534	8.536	8.538	8.541	8.543	8.545	8.548

Number	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
25.0	8.550	8.552	8.554	8.557	8.559	8.561	8.564	8.566	8.568	8.570
25.1	8.573	8.575	8.577	8.580	8.582	8.584	8.586	8.589	8.591	8.593
25.2	8.595	8.598	8.600	8.602	8.605	8.607	8.609	8.611	8.614	8.616
25.3	8.618	8.620	8.623	8.625	8.627	8.629	8.632	8.634	8.636	8.639
25.4	8.641	8.643	8.645	8.648	8.650	8.652	8.654	8.657	8.659	8.661
25.5	8.664	8.666	8.668	8.670	8.673	8.675	8.677	8.679	8.682	8.684
25.6	8.686	8.688	8.691	8.693	8.695	8.697	8.700	8.702	8.704	8.706
25.7	8.709	8.711	8.713	8.716	8.718	8.720	8.722	8.725	8.727	8.729
25.8	8.731	8.734	8.736	8.738	8.740	8.743	8.745	8.747	8.749	8.752
25.9	8.754	8.756	8.758	8.761	8.763	8.765	8.767	8.770	8.772	8.774
26.0	8.776	8.779	8.781	8.783	8.785	8.788	8.790	8.792	8.794	8.797
26.1	8.799	8.801	8.803	8.806	8.808	8.810	8.812	8.815	8.817	8.819
26.2	8.821	8.824	8.826	8.828	8.830	8.833	8.835	8.837	8.839	8.842
26.3	8.844	8.846	8.848	8.850	8.853	8.855	8.857	8.859	8.862	8.864
26.4	8.866	8.868	8.871	8.873	8.875	8.877	8.880	8.882	8.884	8.886
26.5	8.889	8.891	8.893	8.895	8.897	8.900	8.902	8.904	8.906	8.909
26.6	8.911	8.913	8.915	8.918	8.920	8.922	8.924	8.927	8.929	8.931
26.7	8.933	8.935	8.938	8.940	8.942	8.944	8.947	8.949	8.951	8.953
26.8	8.956	8.958	8.960	8.962	8.964	8.967	8.969	8.971	8.973	8.976
26.9	8.978	8.980	8.982	8.984	8.987	8.989	8.991	8.993	8.996	8.998
27.0	9.000	9.002	9.004	9.007	9.009	9.011	9.013	9.016	9.018	9.020
27.1	9.022	9.024	9.027	9.029	9.031	9.033	9.036	9.038	9.040	9.042
27.2	9.044	9.047	9.049	9.051	9.053	9.055	9.058	9.060	9.062	9.064
27.3	9.067	9.069	9.071	9.073	9.075	9.078	9.080	9.082	9.084	9.086
27.4	9.089	9.091	9.093	9.095	9.098	9.100	9.102	9.104	9.106	9.109
27.5	9.111	9.113	9.115	9.117	9.120	9.122	9.124	9.126	9.128	9.131
27.6	9.133	9.135	9.137	9.139	9.142	9.144	9.146	9.148	9.150	9.153
27.7	9.155	9.157	9.159	9.162	9.164	9.166	9.168	9.170	9.173	9.175
27.8	9.177	9.179	9.181	9.184	9.186	9.188	9.190	9.192	9.195	9.197
27.9	9.199	9.201	9.203	9.206	9.208	9.210	9.212	9.214	9.216	9.219
28.0	9.221	9.223	9.225	9.227	9.230	9.232	9.234	9.236	9.238	9.241
28.1	9.243	9.245	9.247	9.249	9.252	9.254	9.256	9.258	9.260	9.263
28.2	9.265	9.267	9.269	9.271	9.274	9.276	9.278	9.280	9.282	9.284
28.3	9.287	9.289	9.291	9.293	9.295	9.298	9.300	9.302	9.304	9.306
28.4	9.308	9.311	9.313	9.315	9.317	9.319	9.322	9.324	9.326	9.328
28.5	9.330	9.333	9.335	9.337	9.339	9.341	9.343	9.346	9.348	9.350
28.6	9.352	9.354	9.356	9.359	9.361	9.363	9.365	9.367	9.370	9.372
28.7	9.374	9.376	9.378	9.380	9.383	9.385	9.387	9.389	9.391	9.393
28.8	9.396	9.398	9.400	9.402	9.404	9.407	9.409	9.411	9.413	9.415
28.9	9.417	9.420	9.422	9.424	9.426	9.428	9.430	9.433	9.435	9.437
29.0	9.439	9.441	9.443	9.446	9.448	9.450	9.452	9.454	9.456	9.459
29.1	9.461	9.463	9.465	9.467	9.469	9.472	9.474	9.476	9.478	9.480
29.2	9.482	9.485	9.487	9.489	9.491	9.493	9.495	9.498	9.500	9.502
29.3	9.504	9.506	9.508	9.511	9.513	9.515	9.517	9.519	9.521	9.524
29.4	9.526	9.528	9.530	9.532	9.534	9.537	9.539	9.541	9.543	9.545
29.5	9.547	9.549	9.552	9.554	9.556	9.558	9.560	9.562	9.565	9.567
29.6	9.569	9.571	9.573	9.575	9.577	9.580	9.582	9.584	9.586	9.588
29.7	9.590	9.593	9.595	9.597	9.599	9.601	9.603	9.605	9.608	9.610
29.8	9.612	9.614	9.616	9.618	9.621	9.623	9.625	9.627	9.629	9.631
29.9	9.633	9.636	9.638	9.640	9.642	9.644	9.646	9.648	9.651	9.653
30.0	9.655									

CHAPTER IV

CULVERTS





CULVERTS

4-100. GENERAL

After a complete field survey of the general area and culvert location vicinity, and after design discharge and design tailwater have been determined, the next step in a complete hydraulic design is the selection of a structure that will carry the design discharge beneath the highway satisfactorily.

It should be borne in mind that the selection of any structure should be based on hydraulic principles, on the most economical size and shape, and with a resulting headwater depth which will not cause damage to adjacent property. The resultant outlet velocity should be taken into consideration for any possible damaging effects. The allowable headwater elevation is that elevation above which damage may be caused to adjacent property and/or the highway. It is this allowable headwater depth that is primarily the basis for sizing a culvert.

The cost of maintaining highways in good condition is directly related to the adequacy of the means provided for drainage. Storm water for which adequate provisions are not made may cause severe erosion of embankment slopes and may undermine culvert outlets. Good drainage design depends on determining the proper frequency and amount of run-off and on the provisions of adequate facilities to remove the run-off at such a rate as to avoid undue interference with traffic and also keep maintenance costs at a minimum. In addition, these facilities should be provided at a minimum cost of initial investment. Structures may consist of various installations, such as:

a. Box Culverts
   1. Single barrel
   2. Multiple barrel

b. Concrete and Metal Pipes
   1. Circular
   2. Oval
   3. Pipe-arch

c. Structural Plate Pipes
   1. Circular
   2. Pipe-arch
   3. Arch

4-200. TERMINOLOGY AND DEFINITIONS

4-201. TERMINOLOGY

The hydraulic performance of a culvert is determined by the following factors. These factors must be known or estimated prior to designing a structure.

A = Area occupied by water flow in square feet

C_e = Entrance coefficient (Table I)

D = Diameter of pipe culvert in feet or height of box culvert in feet

d = Depth of flow in feet

d_c = Critical depth of flow in feet

$\frac{d}{D}$ = Ratio of depth of flow to diameter of pipe

$\frac{d}{W}$ = Ratio of depth of flow to width of culvert

g = Acceleration of gravity = 32.2 feet per second per second

H_L = Head losses

HW = Headwater depth in feet

HW_c = Headwater depth in feet for critical flows at culvert inlet

h_e = Entrance head losses in feet = $C_e \frac{V^2}{2g}$

h_f = Friction head losses in feet = $S_f L$

h_v = Velocity head in feet = $\frac{V^2}{2g}$

L = Length of barrel in feet

n = Coefficient of roughness. (Table II, Chapter III)

P = Pressure line height $= \frac{d_c + D}{2}$ (See Figure 7)

Q = Discharge in cubic feet per second, based on a predetermined design frequency

q = Discharge per foot of width for rectangular channels in cubic feet per second $= \frac{Q}{W}$

R = Hydraulic radius in feet $= \frac{A}{WP}$

S_o = Slope of Culvert in feet per foot

S_c = Critical slope in feet per foot

S_f = Friction slope = Slope that will produce uniform flow (see Figures 10 and 14) at depth under consideration. For Type I operation the friction slope is based upon $1.1d_c$. Type II operation requires that the friction slope be based upon the TW depth.

TW = Tailwater depth in feet

V = Velocity in feet per second (Figures 16,24&29)

V_c = Critical velocity in feet per second occuring at critical depth

W = Width of rectangular culvert or bottom width of channel in feet

WP = Wetted perimeter in feet

4-202. DEFINITIONS

(a) Critical Depth can best be illustrated as the depth at which water flows over a weir, this depth being attained automatically because it is the depth at which the energy content of flow is a minimum. For a given discharge and channel shape there is only one critical depth. The formula applicable for calculating critical depth in rectangular channels =

$$d_c = \sqrt[3]{\frac{q^2}{32.2}}$$

For rectangular culverts the critical depth may either be computed from the above formula or read from Figure 9.

The formulas applicable for calculating critical depth in circular, arch, and oval sections are rather tedious and for this reason are not included herein. For circular sections the critical depth may be obtained from Figure 15, Pipe-Arch, Figure 27, Oval, Figure 22.

(b) Uniform flow is possible only in a channel of constant cross section having the same discharge, velocity and depth of flow throughout the reach. This type of flow will exist in a Type III A culvert operation provided the culvert is sufficiently long to reach a uniform depth of flow.

(c) Free outlets are those outlets whose TW is equal to or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the discharge. (See Type I operation)

(d) Partially submerged outlets are those outlets whose TW is higher than critical depth and lower than D, the height of culvert. (See Type II operation)

(e) Submerged outlets are those outlets having a TW elevation higher than the soffit of the culvert. (See Type IV A operation)

(f) Critical slope is that slope at which a given discharge will pass through a structure at critical depth and critical velocity. Increasing the slope above critical slope does not increase the discharge because culvert capacity is determined by the inlet geometry. It merely makes the water flow at a depth less than critical depth and at a greater velocity. Decreasing the slope to less than critical slope will have a retarding effect upon the discharge, causing the depth of flow to be higher than critical depth and the velocity to be less than critical.

(g) Flared, Improved, or Tapered Inlet indicates a special entrance condition as illustrated in Sections 4-603 and 4-604.

(h) Soffit refers to the inside top of pipe or box.

(i) Invert refers to the flowline of pipe or box (inside bottom).

(j) Tandem Culverts refers to culverts aligned across a roadway in such a manner that it may be possible for the headwater of the downstream culvert to influence the tailwater of the culvert immediately upstream. (Figure 1)

4-300. CULVERT FLOW CONTROLS

4-301. GENERAL

Generally, the hydraulic control in a culvert will be at the culvert outlet if the culvert slope is less than the critical slope. Entrance control usually governs if the culvert slope is greater than the critical slope.

For outlet control, the head losses due to TW and barrel friction are predominate in controlling the headwater of the culvert. The entrance will allow the water to enter the culvert faster than the TW and barrel friction will allow it to flow through the culvert.

For entrance control, the entrance characteristics of the culvert are such that the entrance head losses are predominate in determining the headwater of the culvert. The barrel will carry water through the culvert faster than the water can enter the culvert. Each culvert flow, however classified, is dependent upon one or both of these controls; therefore, because of the importance of these controls, further discussion follows.

4-302. ENTRANCE CONTROL

a. If the slope of the culvert is greater than critical slope (referred to as 'steep slope') and the tailwater depth is less than S_oL or tailwater elevation is lower than the upstream flowline of the culvert, (Figure 4) headwater HW is based on entrance control for all ranges of discharge.

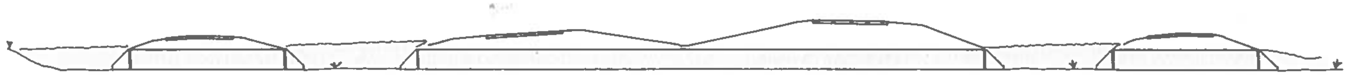


Figure 1

b. If the slope of the culvert is steep and tailwater depth TW is greater than S_oL but not greater than (S_oL+D) (see Figure 5) that is, the upstream flowline elevation is submerged by TW elevation but the upstream soffit is unsubmerged, the control may be entrance or outlet. In this case, it is recommended that both controls be checked and the one causing the higher HW be used.

See Types III A and B (Figures 4 and 5) and IV A and B (Figures 6 and 7).

4-303. OUTLET CONTROL

4-303.1 OUTLET CONTROL - MILD SLOPE

If the slope of the culvert is less than critical slope (referred to as 'mild slope'), the outlet characteristics control the flow.

a. If the culvert is on a 'mild' slope and TW depth is less than critical depth then the outlet will control and the operation is probably Type I (Figure 2). If HW is calculated by Type I procedure to be greater than 1.2D (an empirical value determined by research), the type is probably IVB and HW should be calculated according to the procedure for Type IVB (Figure 7).

b. If the culvert is on a 'mild' slope and TW depth is greater than critical depth but less than D (barrel depth), the operation is probably Type II (Figure 3). If HW is calculated by Type II procedure to be greater than 1.2D, the type is probably IVB and HW should be calculated according to the procedure for Type IVB (Figure 7).

c. If the culvert is on a 'mild' slope and TW is greater than D, the operation is Type IVA (Figure 6). Calculated HW may or may not be greater than 1.2D in this case.

4-303.2 OUTLET CONTROL - STEEP SLOPE

If the culvert is on a steep slope and TW is greater than S_oL+D (or tailwater elevation is higher than the upstream soffit elevation on the culvert) the operation is Type IVA (Figure 6). Calculated HW may or may not be greater than 1.2D in this case.

For more information concerning the various types of operation, see the schematics of each in Figures 2 through 7. A general solution for hydraulic design of culverts appears elsewhere in this chapter.

4-304. CULVERT FLOW CONDITIONS WITH APPLICABLE HW AND V_{out} EQUATIONS

4-304.1 GENERAL

The hydraulic operation of culverts may be broken down into several types depending on operating conditions. The attempt has been made in this manual to illustrate and discuss each of the major types of operation. Some variations to the types listed herein may be found but these variations should not appreciably alter the end results.

It should be further noted that actual culvert flow conforms to the laws of open-channel flow and closed-conduit flow. Since the proper calculations involved in open-channel flow are rather tedious, certain estimates and compromises are made in the calculation procedures for Types I, II and IVB indicated in this manual. These estimates and compromises cause very little, if any, differences from detailed backwater calculations and the result is a simple pencil and slide rule solution.

The most common types of culvert operations are classified as follows and apply to culverts of any type barrel cross section.

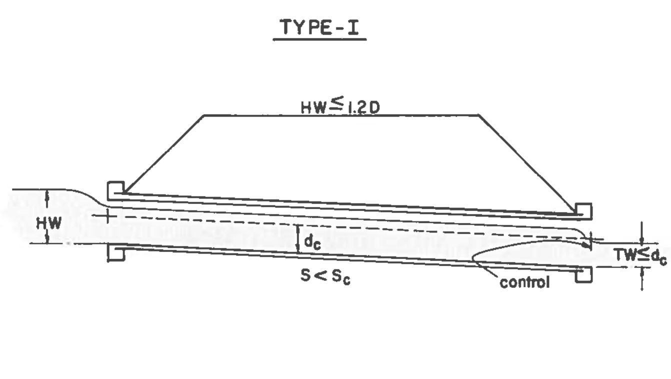


Figure 2

4-304.2 TYPE I CONDITIONS (FIGURE 2)

The entrance is unsubmerged (HW $\leq$ 1.2D), the slope is less than critical slope at design discharge ($S_o < S_c$), and the tailwater is less than or equal to 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.

$$HW = d_c + V_c^2/2g + h_e + h_f - S_oL$$

where:

d_c = critical depth

V_c = critical velocity (based on d_c)

g = 32.2 ft/sec^2

h_e = entrance head = $C_e \frac{V_c^2}{2g}$

where: C_e = entrance coefficient found in Table I

h_f = friction head = S_fL

where: L = length of culvert

S_f = that slope at which 1.1 d_c would be uniform depth

S_oL = vertical drop in culvert from upstream flowline to downstream flowline

The outlet velocity is equal to V_c.

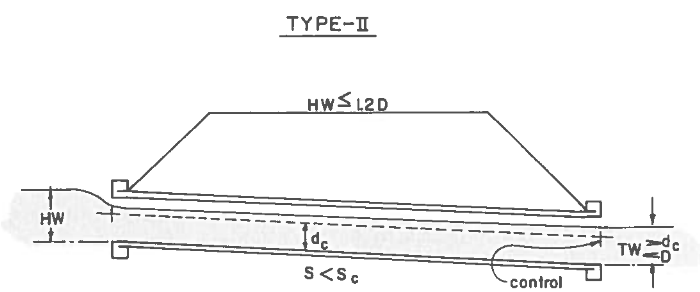


Figure 3

4-304.3 TYPE II CONDITIONS (FIGURE 3)

The entrance is unsubmerged (HW $\leq$ 1.2D), the slope is less than critical slope at design discharge ($S_o < S_c$), the TW depth is greater than critical depth (TW $>$ d_c) and TW is less than D (TW $<$ D).

The above condition is a common occurence where the channel is deep, narrow, and well defined. The control is tailwater at the culvert outlet.

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

where:

TW = tailwater depth at outlet

V_{TW} = velocity based on TW depth

g = acceleration of gravity - 32.2 ft/sec^2

h_e = entrance head = $C_e \frac{V_{TW}^2}{2g}$

where: C_e = entrance coefficient found in Table I.

h_f = friction head = S_fL

where: S_f = slope at which TW depth would be uniform depth

L = length of culvert

S_oL = drop in culvert from upstream flowline to downstream flowline

Outlet velocity = V_{TW} which is the discharge divided by the area of flow in the culvert at tailwater depth.

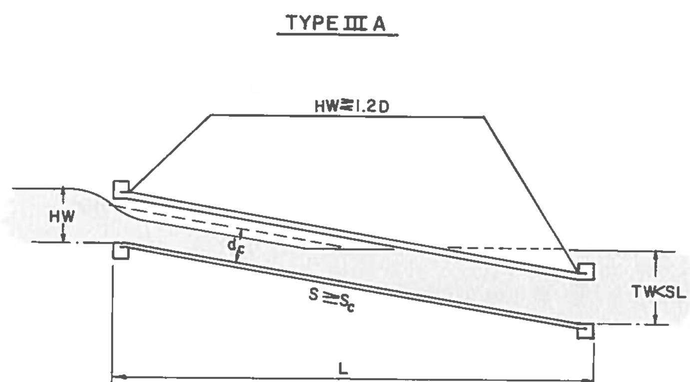


Figure 4

4-304.4 TYPE III A CONDITIONS (FIGURE 4)

The entrance may be submerged or unsubmerged (HW $\gtrless$ 1.2D), the slope of the culvert is greater than or equal to critical slope at design discharge ($S_o \geq S_c$), TW depth is less than S_oL (TW elevation is lower than the upstream flowline). TW depth with respect to D is inconsequential as long as the above conditions are met. This condition is a common occurrence for culverts in rolling or mountainous country. The control is critical depth at the entrance for HW values up to about 1.2D. Control is the entrance geometry for HW values over about 1.2D.

HW is determined from empirical curves in the form of nomographs (see Figures 11, 17, 18, 25 and 30).

If TW is greater than D, outlet velocity is based on full flow at the outlet. If TW is less than D, outlet velocity is based on uniform depth for the culvert. Uniform depth is simply that depth of water for a given discharge, culvert slope, and geometry at steady flow.

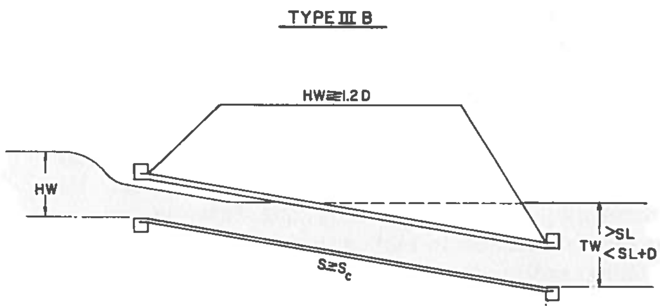


Figure 5

4-304.5 TYPE III B CONDITIONS (FIGURE 5)

The entrance may be submerged or unsubmerged. (HW $\lesseqgtr$ 1.2D), the slope of the culvert is greater than or equal to the critical slope at design discharge ($S_o \geq S_c$), TW depth is greater than S_oL (TW elevation is above the upstream flowline), and TW depth is less than S_oL+D (TW elevation is below the upstream soffit). TW depth with respect to D is inconsequential as long as the above conditions are met. This condition is a common occurrence for culverts in rolling or mountainous country. The control for this type of operation may be at the entrance or the outlet or control may transfer itself back and forth between the two. (Commonly called "slug" flow.) For this reason, it is recommended that HW be determined for both entrance control and outlet control (full flow as in Type IV A and IV B) and the higher of the two determinations be used. Entrance control HW is determined from empirical curves in the form of nomographs (see Figures 11, 17, 18, 25 and 30). Outlet control HW is determined by procedures indicated for Type IV A or IV B (depending on TW depth with respect to D).

If TW depth is less than D, outlet velocity should be based on TW depth. If TW depth is greater than D, outlet velocity should be based on full flow at the outlet.

4-304.6 TYPE IV A CONDITIONS (FIGURE 6)

This condition will exist if the culvert slope is less than critical slope at design discharge ($S_o < S_c$) and TW depth is greater than D (TW $>$ D), or; the culvert slope is greater than or equal to critical slope at design discharge ($S_o \geq S_c$) and TW is greater than S_oL+D (TW $>$ (S_oL+D)) . The HW may or may not be greater than 1.2D, though it usually is greater.

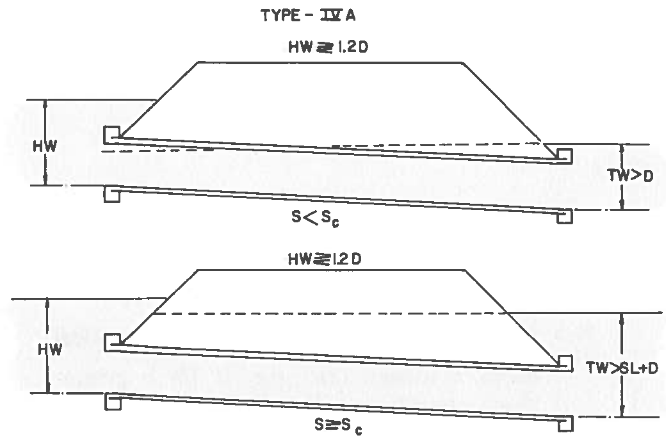


Figure 6

$$HW = H + TW - S_oL$$

where:

H = total head loss of discharge through culvert

$$H = h_v + h_e + h_f$$

where: h_v = velocity head $V^2/2g$ (where V is based on full flow in culvert)

h_e = entrance head $C_e\ h_v$

h_f = friction head = S_fL (where S_f is based on full flow in culvert)

H may be determined directly from nomographs on Figure 12, 19, 26 and 31

TW = tailwater depth

S_oL = drop in culvert elevation from upstream to downstream

Outlet velocity is based on full flow at the outlet.

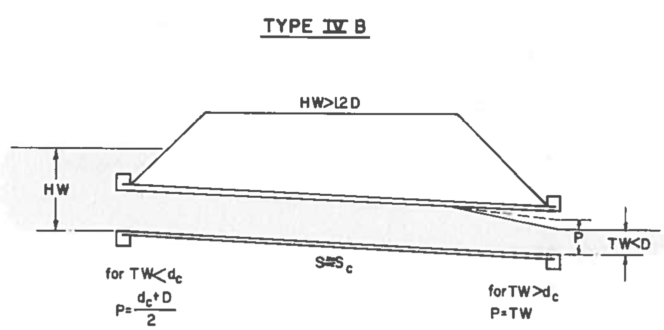


Figure 7

4-304.7 TYPE IV B CONDITIONS (FIGURE 7)

The entrance is submerged (HW ≥ 1.2D) and the tailwater depth is less than D (TW < D). Normally, the designer should arrive at this type of operation only after previous consideration of Types I, II, or IIIB.

On occasion, it may be found that (HW $>$ 1.2D) for Type I, II, or IIIB but (HW $<$ 1.2D) for Type IVB. If so, the higher HW should be used.

$$HW = H + P - S_oL$$

where

H = same definition as for Type IVA

P = empirical approximation of equivalent hydraulic grade line

P = $(d_c + D)/2$ if TW depth is less than critical depth at design discharge. If TW is greater than critical depth, then P = TW

S_oL = drop in culvert flow line from upstream to downstream

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.

4-400. OUTLET VELOCITY

Outlet velocity in any properly designed culvert is normally greater than the velocity in the natural channel and for this reason it has been the usual practice to provide riprap and/or velocity control devices of various kinds in erosive outlet channels. The engineer must bear in mind, while attempting to control flow at a structure outlet, that the main objective is to return the flow to the normal flow in the natural stream, in an economical and efficient manner.

Across Texas, velocities at which various soils become erosive may vary widely. The engineer should attempt to make an estimate of just what the threshold of erosive velocity is for each culvert location. This can be done by observing storm flows on various soil types and estimating those velocities at which erosion is occurring. A widely used threshold of erosive velocity in Texas is 8 feet per second. However, much higher velocities may be tolerated in the cases of channels with rock or shale bottoms, and lower velocities may have erosive effects in the cases of channels with silt or sand bottoms. In any case, if the outlet velocity of a culvert exceeds the maximum deemed allowable by the engineer, riprap protection and/or velocity control devices at the outlet should be provided. See Section 4-700.

It should be noted at this point that if the culvert has been properly sized according to allowable headwater elevation, it is almost always more economical to protect against excessive outlet velocity with riprap and/or velocity control devices than to try to adjust the culvert size to reduce the excessive outlet velocity.

4-500. CULVERT IN TANDEM

When culverts are arranged as depicted in Figure 1 such that water leaving one culvert must pass through another culvert immediately downstream, consideration must be given to the backwater effects of the downstream culvert upon the upstream culvert. Briefly, the headwater elevation determined for a downstream structure serves, in turn, as the tailwater elevation for the upstream structure. This is approximately true only if that elevation submerges a theoretical unrestricted channel tailwater depth. The actual water-surface profile is more complex than the above description but the difference in final tailwater elevations is negligible.

It can be seen that the entrance losses involved in such a culvert arrangement can be eliminated if the culverts are joined. Economy is often better when the culverts are joined because of more efficient operation and the elimination of two inside headwalls. However, an economic comparison in each instance is necessary before joining tandem culverts.

4-600. CULVERT DESIGN

4-601. CULVERT DESIGN DATA

The proper design of a culvert requires the definite knowledge of some items and the assumption of other items. The items which should be known either by observation or calculation include:

1. design discharge - Q_D
2. design tailwater - TW
3. culvert slope - S_o
4. allowable headwater - HWA

The items which must be assumed or estimated include:

5. allowable outlet velocity - VA
6. culvert length - L
7. entrance conditions
8. culvert material and shape (box, pipe, metal, concrete, etc.)
9. maximum allowable depth of barrel - DMAX (should usually not be greater than HWA)

Only after all the basic culvert design data is assimilated should the culvert sizing be attempted.

4-602. DESIGN PROCEDURE

The following is a step by step culvert design procedure. Preceding the design discussion is a schematic flow chart (Figure 8). This chart should be helpful in routing the designer through the design procedure. It should be noted that the procedure incorporates the items as shown on the culvert calculation sheet (Chapter 10) from left to right. Variations from normal culvert design are covered elsewhere in this chapter.

Initial Trial Size:

1. Divide the allowable headwater HWA by the maximum allowable depth DMAX.
2. (a) Box Culvert

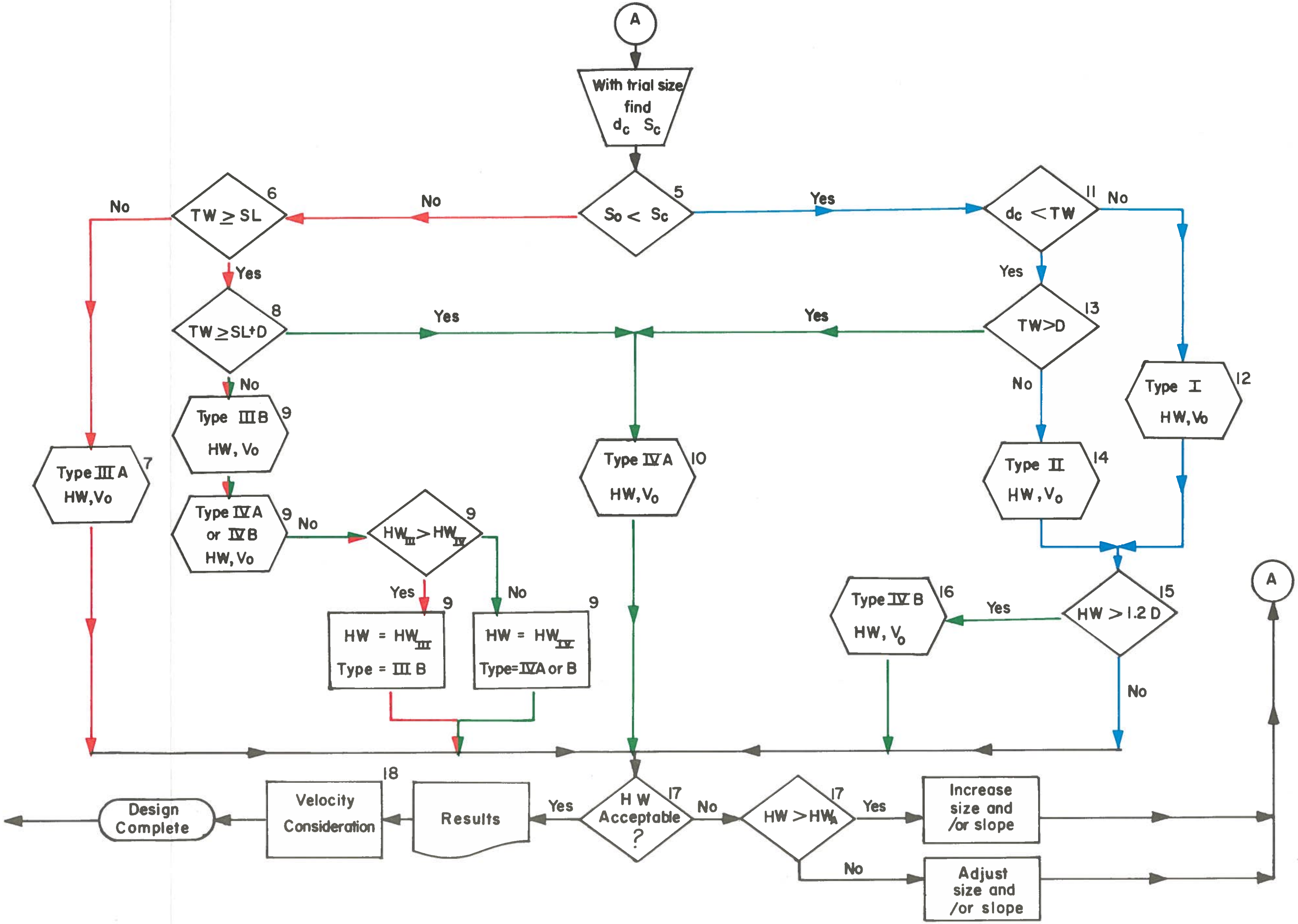

A
With trial size find d_c S_c
$S_o < S_c$
5
No
Yes
$TW \geq SL$
6
$d_c < TW$
11
No
Yes
Yes
$TW \geq SL+D$
8
Yes
Yes
$TW > D$
13
No
No
Type III B HW, Vo
9
Type I HW, Vo
12
Type III A HW, Vo
7
Type IV A HW, V_o
10
Type II HW, V_o
14
Type IV A or IV B HW, Vo
9
No
$HW_{III} > HW_{IV}$
9
Yes
9
No
9
HW = HW_{III} Type = III B
HW = HW_{IV} Type=IVA or B
Type IV B HW, V_o
16
Yes
$HW > 1.2D$
15
No
A
Design Complete
Velocity Consideration
18
Results
Yes
H W Acceptable ?
17
No
$HW > HW_A$
17
Yes
No
Increase size and /or slope
Adjust size and /or slope

Figure 8

Enter Figure 11 with D = DMAX and HW/D = HWA/DMAX, determine q. ($q = Q_D/W$) where W = total width in feet of box required. Round W up to the nearest value which yields a whole multiple of standard box widths. Divide W by the largest standard span S for which W is a multiple. This determines the number of barrels - N. At this point, the determination has been made that the initial trial box size will be:

N - S x DMAX x L

(b) Pipe Culvert

Enter Figure 17 (for circular), Figure 25 (for elliptical), Figure 30 (for pipe-arch) with D = DMAX and HW/D = HWA/DMAX. Determine Q/BBL. Divide Q_D by Q/BBL and round up to the nearest whole number. This number represents the number (N) of barrels of diameter D pipe (or of rise D for oval or arch). At this point the determination has been made that the initial trial size culvert will be N - D x L pipes (or N-RISE x SPAN x L pipes for oval or pipe-arch).

Critical Depth:

3. (a) Box Culvert

Enter Figure 9 with span width S and discharge per barrel, find d_c - or calculate d_c from formula in Section 4-202.

(b) Pipe Culvert

Enter Figure 15 (Figure 22 for elliptical, Figure 27 for pipe-arch) with diameter (or rise and span) and discharge per barrel. Find d_c/D (d_c/RISE for oval and pipe-arch).

Critical Slope:

4. (a) Box Culvert

Enter Figure 10 with d_c/S (critical depth divided by span width), span S, and discharge per barrel and find critical slope S_c.

(b) Pipe Culvert

Enter Figure 14 (Figure 23 for elliptical, Figure 28 for pipe-arch) with discharge per pipe, pipe size, and d_c/D (d_c/RISE for oval or arch) and find critical slope S_c.

At this point, critical depth and critical slope have been determined for the design discharge and the trial size. Tailwater depth TW, barrel depth D, and culvert slope S_o are known. The next step in the culvert design procedure is to determine the probable type of culvert operation and calculate HW and outlet velocity. This is accomplished by making a series of comparisons between known quantities to determine the conditions of culvert flow.

Confirmation of Trial Size:

5. If culvert slope is less than critical slope ($S_o < S_c$), skip to step 11, otherwise;

6. If tailwater depth TW is greater than or equal to culvert flow line drop S_oL, ($TW \geq S_oL$), skip to step 8, otherwise;

7. Determine HW according to the procedure for Type IIIA. Use Figure 11 for box culverts, Figures 17 or 18 for circular pipes, Figure 25 for elliptical pipes, and Figure 30 for pipe-arch. Calculate outlet velocity according to instructions under Type IIIA and skip to step 17.

8. If TW is greater than or equal to (S_oL+D) skip to step 10, otherwise;

9. Calculate HW for both entrance control and outlet control. HW (entrance control) should be determined from nomograph. Use Figure 11 for box culverts, Figures 17 or 18 for circular pipe, Figure 25 for elliptical pipes, and Figure 30 for pipe-arch. HW (outlet control) is calculated by procedures outlined for Types IVA or IVB.

   If the TW is greater than D,

   $$HW = H + TW - S_oL$$

   If the TW is less than D,

   $$HW = H + P - S_oL$$

   where

   $P = (d_c + D)/2$ if TW is less than d_c, and P = TW if TW is greater than d_c.

   H is obtained from Figure 12 for box culverts, Figures 19 or 20 for circular pipes, Figure 26 for elliptical pipes, and Figure 31 for pipe-arch. HW (entrance control) is then compared to HW (outlet control) and the higher of the two is used. If HW (entrance control) is higher, operation is Type IIIB. If HW (outlet control) is higher, operation is Type IVA or IVB. Calculate outlet velocity by dividing design discharge by the area of flow at the outlet and skip to step 17.

10. HW is based on outlet control, the culvert is on a mild slope and the outlet is submerged by TW or, the culvert is on a steep slope and the entire

culvert is submerged by TW.

$$HW = H + TW - S_oL$$

H is determined from nomographs at Figure 12 for box culvert, Figures 19 or 20 for circular pipe, Figure 26 for elliptical pipe, and Figure 31 for pipe-arch. Outlet velocity is based on design discharge and full flow at the outlet. Skip to step 17.

11. If TW is greater than critical depth d_c, skip to step 13, otherwise;

12. Calculate HW by Type I operation procedure

$$HW = d_c + V_c^2/2g + h_e + h_f - S_oL$$

Outlet velocity equals critical velocity V_c. Skip to step 15.

13. If TW is greater than D, the outlet is submerged and the culvert is on a mild slope, go to step 10, otherwise;

14. Calculate HW by Type II operation procedure.

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

Outlet velocity equals the design discharge divided by the area of flow at tailwater depth.

15. If HW is less than 1.2D, skip to step 17, otherwise;

16. The check should be made here for operation Type IV B. At this point, it has been determined that $S_o < S_c$, $TW < D$, and $HW > 1.2D$ by calculation from Type I or Type II. Calculate a new HW based on $HW = H + P - S_oL$. H is determined from a nomograph on Figure 12 for box culverts, Figures 19 or 20 for circular pipes, Figure 26 for elliptical pipes, and Figure 31 for pipe-arch. P is equal to $(d_c + D)/2$ if TW is less than d_c and P is equal to TW if TW is greater than d_c. P is an approximation of the hydraulic grade line. Calculate outlet velocity based on d_c if TW is less than d_c and based on TW if TW is greater than d_c.

17. At this point, an HW and outlet velocity have been calculated for the design discharge passage through a trial size culvert. If the calculated HW is less than or equal to the allowable headwater HWA and, in addition, the calculated HW is not appreciably lower than HWA (an indication of culvert efficiency)*, the design is complete. If the above conditions are met and the culvert is of a Type IIIA or IIIB operation, skip to step 19. If the operation is not Type IIIA or IIIB, the culvert geometry design is complete, go to step 18. If, however, the calculated HW is greater than HWA, the trial culvert size should be increased (by adding barrels, widening spans, increasing diameter, etc.). Also, the culvert slope might be adjusted slightly to cause a reduction in HW. Regardless of the changes made, the calculations must be redone; go back to step 3. If the calculated HW is considerably lower than HWA or lower than the culvert depth D, in order to find a more economical structure, the trial culvert size should be reduced, by reducing the number of barrels, reducing span widths, reducing diameter, etc. The calculations must then be redone after any changes; go back to step 3.

*A good measure of this is to compare the HW with the culvert depth D. If HW is below D, the efficiency of that culvert size is obviously not very high.

18. The culvert for which the calculated HW is satisfactory may have an excessive outlet velocity. Just what outlet velocity is 'excessive' is normally an engineering judgement based on local conditions. It is usually always most economical to provide riprap, or sills, or a stilling basin or the like at the outlet end to control any excessive velocity. (See Velocity Control Devices. Section 4-702) Generally, a properly sized culvert will have an outlet velocity greater than the natural stream velocity. Any outlet velocity control device is considered part of the hydraulic design of the culvert.

19. If the culvert is operating on entrance control as found in Type IIIA or IIIB, the possibility exists of improving the entrance conditions so as to require a less costly structure. This is done by investigating the design of a flared (or tapered) inlet and associated structure. The design procedures are discussed in Section 4-603 for pipe culverts and at Section 4-604 for box culverts. Due to the cost of the improved entrance, an economic comparison should be made between the design with a normal entrance and the design with an improved entrance. Go to step 18 after the improved entrance design is complete.

4-603. IMPROVED INLET DESIGN - PIPE

For pipe culverts on slopes GREATER THAN CRITICAL SLOPE.

Requirements:

1. Actual culvert slope MUST BE GREATER THAN CRITICAL SLOPE.

2. Culvert operates with inlet control ($HW_{oc} < HW_{ic}$).

Definitions:

HW_{tc} = HW (throat control) as calculated using flared inlet procedures

HW_{oc} = HW (outlet control) as calculated using Type IV procedures

HW_{ic} = HW (inlet control) as calculated using Type III procedures

H = Height of HW above the invert at the throat

S_o = Actual slope of culvert

L_1 = Length of flared inlet (always = 1.5D)

(See figure 39 for dimensions of improved inlet)

Procedure:

Note: For any culvert on slope greater than critical where the entrance controls the headwater, (HW inlet control) there is a possible use for a flared inlet to reduce the size of the barrel.

1. To get the trial size culvert using the flared inlet enter nomograph, Figure 21, with design discharge and select a pipe size that will yield a HW (throat control) equal to or slightly less than the design allowable HW.

2. Determine critical slope for the trial pipe size and material determined in step 1.

3. When either TW $> D$ or TW $> (d_c + S_o L)$, then determine HW (outlet control) (Type IVA). See Section 4-602, step 10. If TW is less than these requirements outlet control will not be considered.

4. Trial size is verified when:

$HW_{tc} < HW_{allow}$ (step 1)

$S_o > S_c$ (step 2)

$HW_{oc} < HW_{tc}$ (step 3)

5. If trial size is verified, compare with culvert without flared inlet for economy. Calculate outlet velocity in accordance with procedure outlined under Type IIIA conditions, Section 4-304.4.

6. If trial size is not verified then simply design culvert without flared inlet in accordance with usual procedure given in this chapter.

Note: (a) The rule that established HW = 1.2D as the dividing point between submerged and unsubmerged entrances is no criteria in this suggested design procedure for flared inlets.

(b) The entrance unit should never be cut to a skew as is done on an ordinary entrance. (See Chapter 10)

4-604. IMPROVED INLET DESIGN - BOX

If the box culvert is established as having entrance control operation under Type IIIA it follows that the barrel of the culvert is less efficient hydraulically than the entrance geometry. Steps are available to improve the efficiency of the barrel and they are as follows.

a. The barrel depth may be reduced in transition from the original depth to a minimum of one foot greater than the uniform depth flow. The transition may be made in a minimum of 20 feet. See Figure 40a. This method is arbitrary and should be used carefully only when the culvert is definitely operating as Type IIIA.

The above method is economical and simple to perform, both in design and construction. A more complex method of box culvert inlet improvement based on controlled research is the following.

b. Straight-Tapered Inlet: Box Culvert

If, in the process of ordinary box culvert design, it is found that a single barrel box-culvert on steep grade ($S_o > S_c$) under Type IIIA operation satisfies the design criteria, then a straight-tapered inlet for the box should be considered. (It is likely that if more than one barrel is involved, a tapered inlet will not be an economical alternative. This is because, each barrel must have its own taper and, therefore, the barrels must be separated.) If the culvert has been originally designed as a Type IIIA operation, then the culvert entrance is the point of hydraulic control. This means that the barrel is more efficient than the entrance. In such a case, a better balance of entrance efficiency and barrel efficiency can be attained by use of the tapered inlet. Figure 40b is an illustration of the straight -tapered inlet for a box culvert.

The procedure for such a design is as follows:

Given: 1 - Span (W) X Depth (D)

$S_c < S_o$

$HW \leq HWA$

Type IIIA operation

Q

TW

1. Try $B = W/1.6$ (round to nearest whole number) = new barrel width. If $B < D$, use $B = D$ as the first trial width.

2. If $B = W$, go to step 8. Otherwise, $q = Q/B$ = discharge/ft. of barrel width.

3. Enter Figure 13 and find H_t/D.

4. $HW = H_t \cdot (S_o L_1)$ (Where $L_1 = 1.2B$.)

5. If $HW \leq HWA$, go to step 6. Otherwise increase B by one foot and go to step 2.

6. Find d_c and S_c from Figure 9 and Figure 10 respectively. d_c and S_c are to be based upon the new culvert barrel width (B).

7. If $S_o > S_c$, the tapered inlet design will work and should be used. See Figure 13 or 40b for proper dimensions of the taper. Go to step 9.

   If $S_o < S_c$, the tapered inlet is ineffective and HW should be calculated on the basis of Types I, II, or IVB operations. It will probably be found, in this case, that $HW > HWA$. Go to step 8.

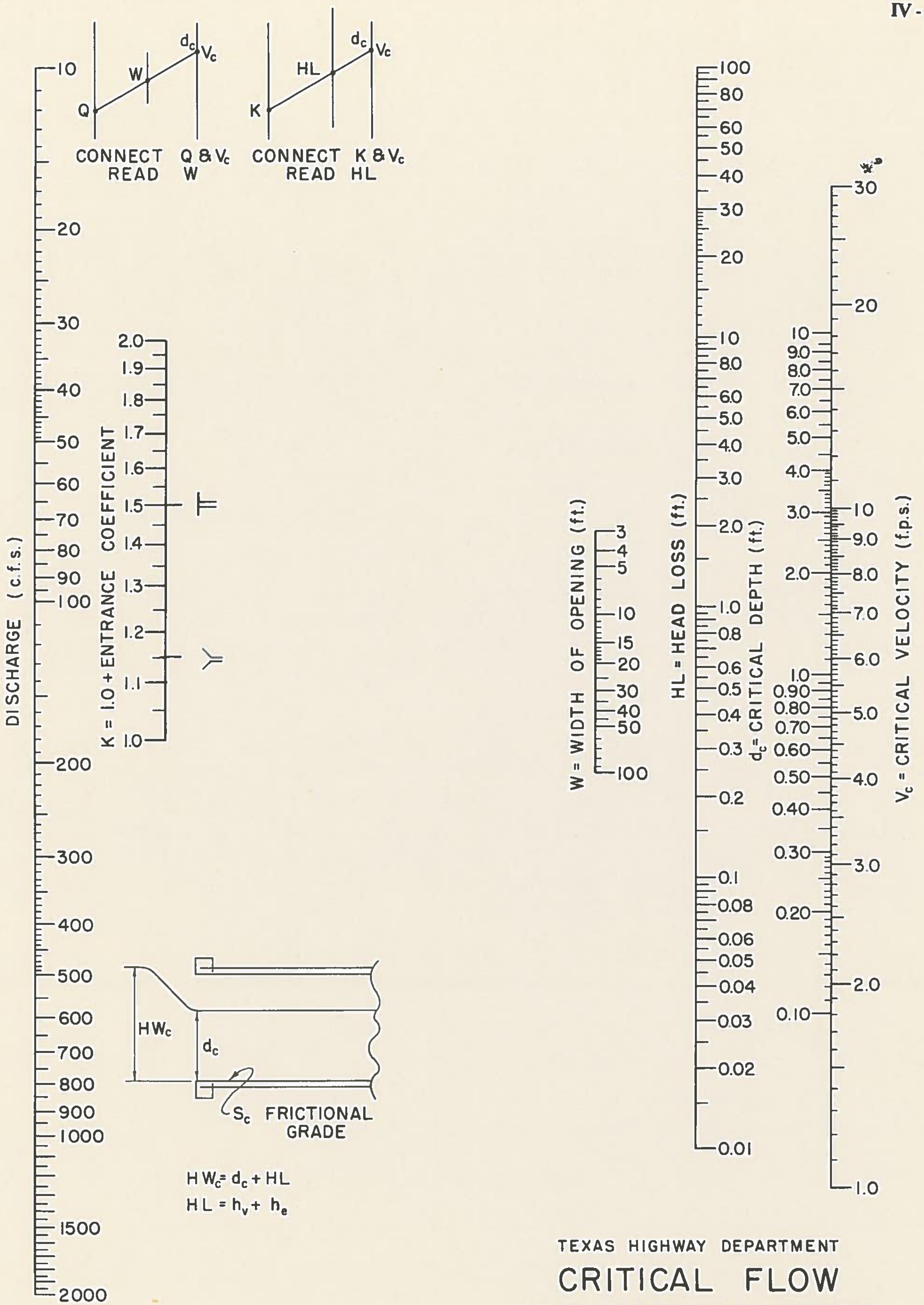
8. If this step is reached, the original ordinary culvert design likely cannot be improved upon.

9. End improved inlet design - Box. Compare with culvert without flared inlet for economics. Calculate outlet velocity in accordance with procedure outlined under Type IIIA conditions, Section 4-304.4.

TABLE I

Entrance Loss Coefficients

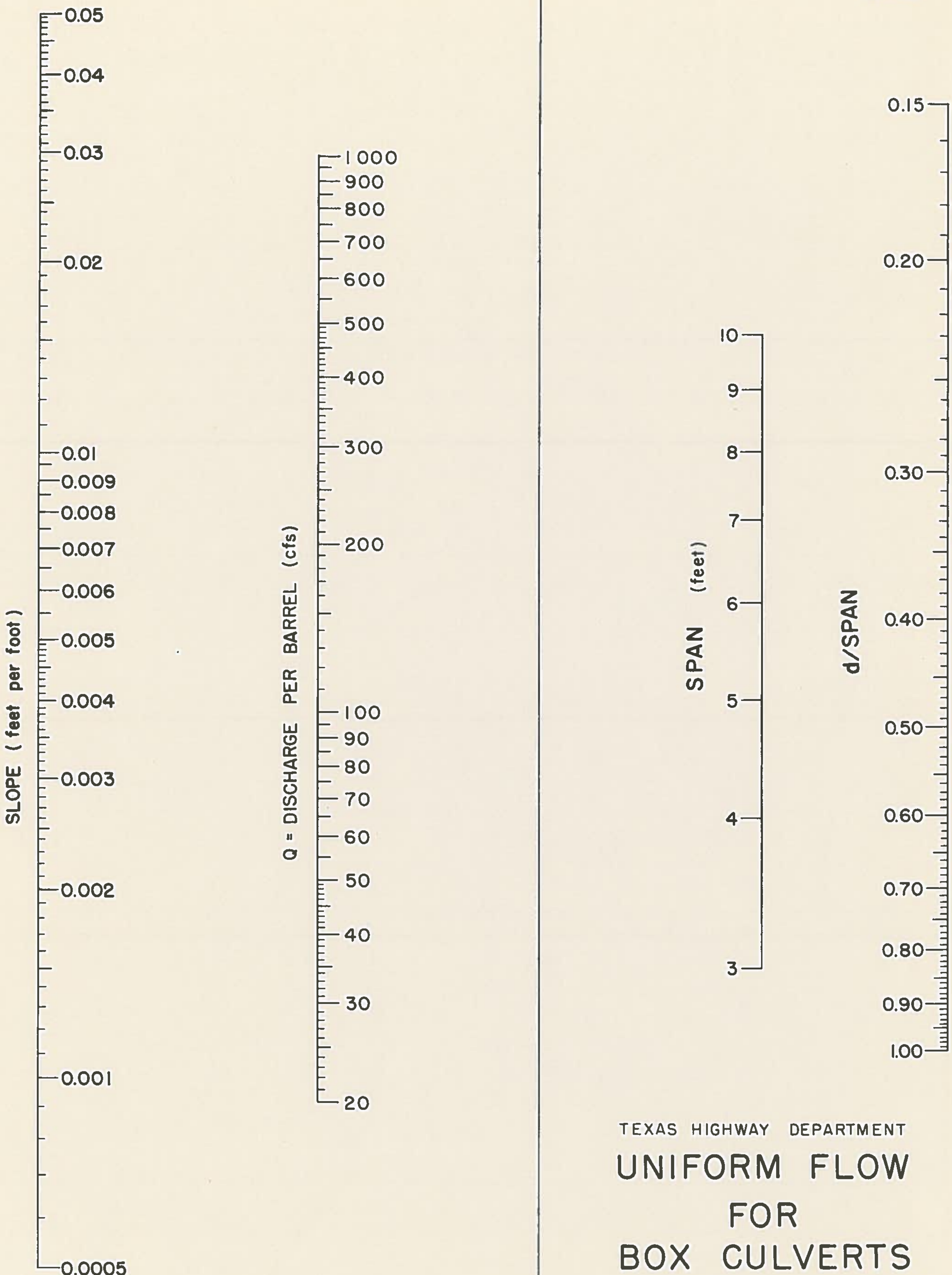
Type of Structure and Design of Entrance	Coefficient C_e
Pipe, Concrete	
Projecting from fill, socket end	0.25
Projecting from fill, sq. cut end	.0.55
Headwall or headwall and wingwalls	
Socket end of pipe	.0.2
Square-edge	.0.5
End-Section conforming to fill slope	.0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	.0.9
Headwall or headwall and wingwalls	
Square-edge	.0.5
End-Section conforming to fill slope	.0.7
Box, Reinforced Concrete	
Flared Wingwalls	.0.4
Parallel Wingwalls	.0.5
Straight Wingwalls	.0.70



TEXAS HIGHWAY DEPARTMENT

CRITICAL FLOW
FOR BOX CULVERTS
$n = 0.012$

Figure 9



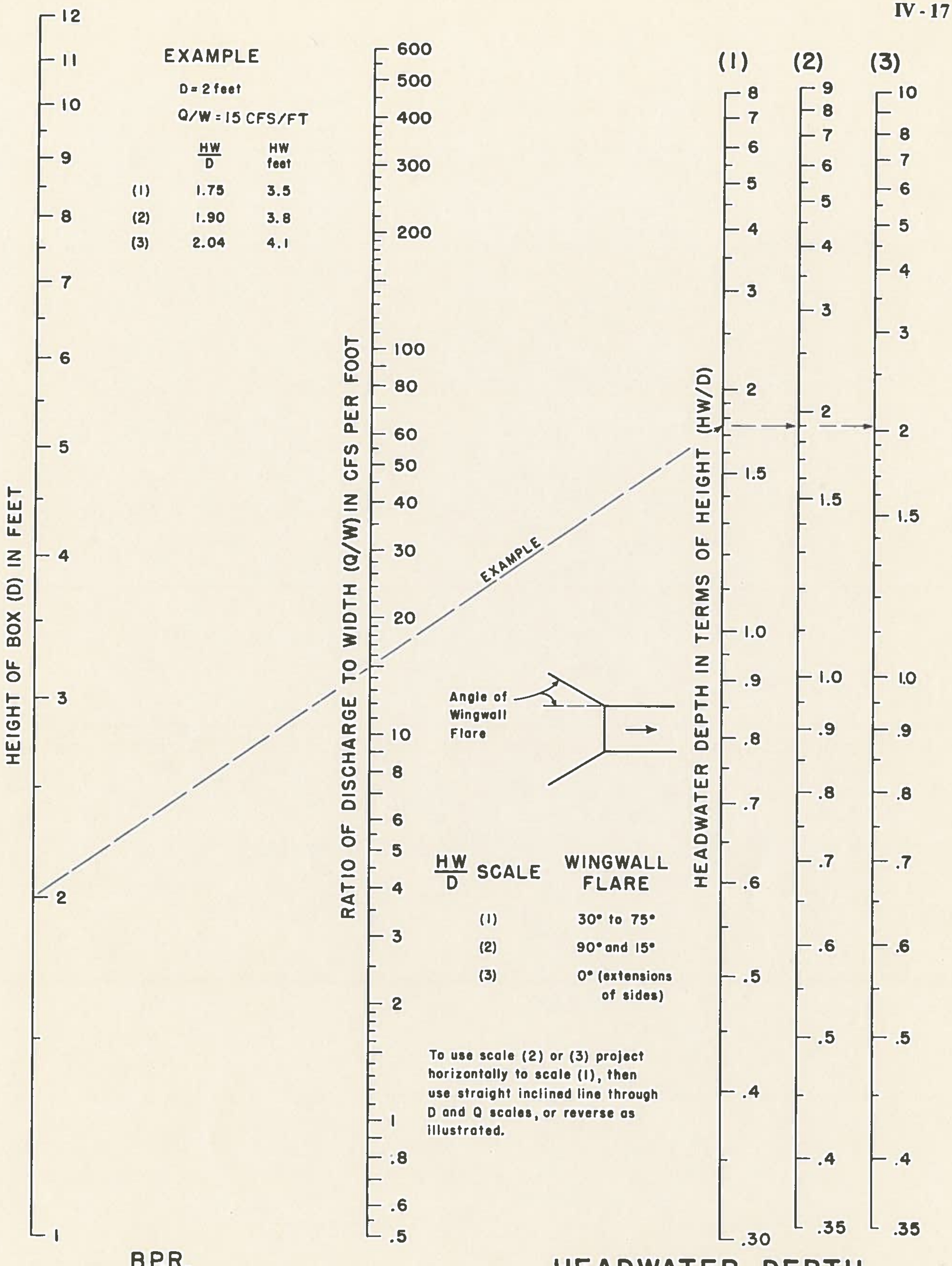
3 Sides Wetted

TEXAS HIGHWAY DEPARTMENT

UNIFORM FLOW FOR BOX CULVERTS

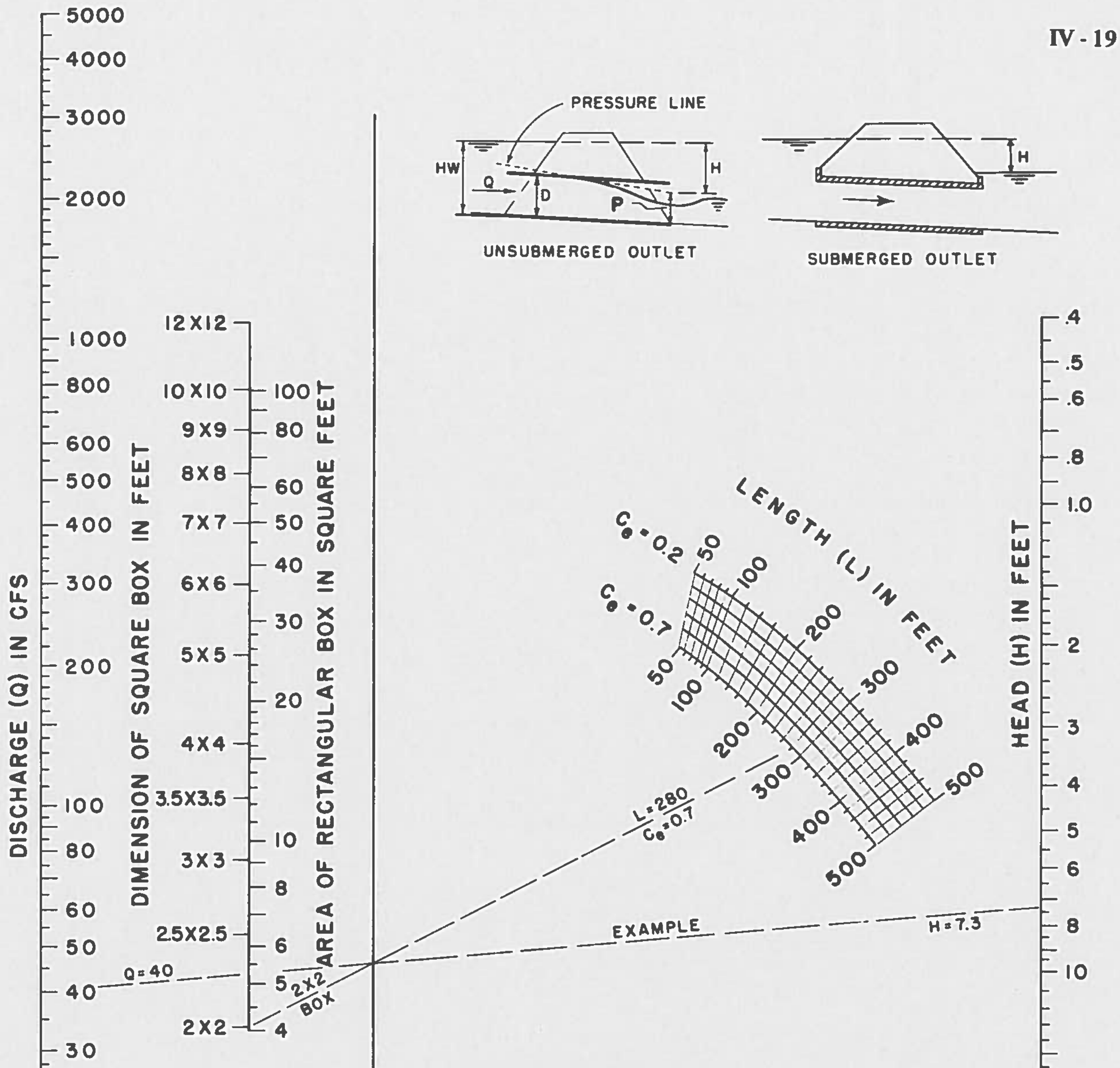
$n = 0.012$

Figure 10



HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

Figure 11



EQUATION FOR SQUARE BOX:

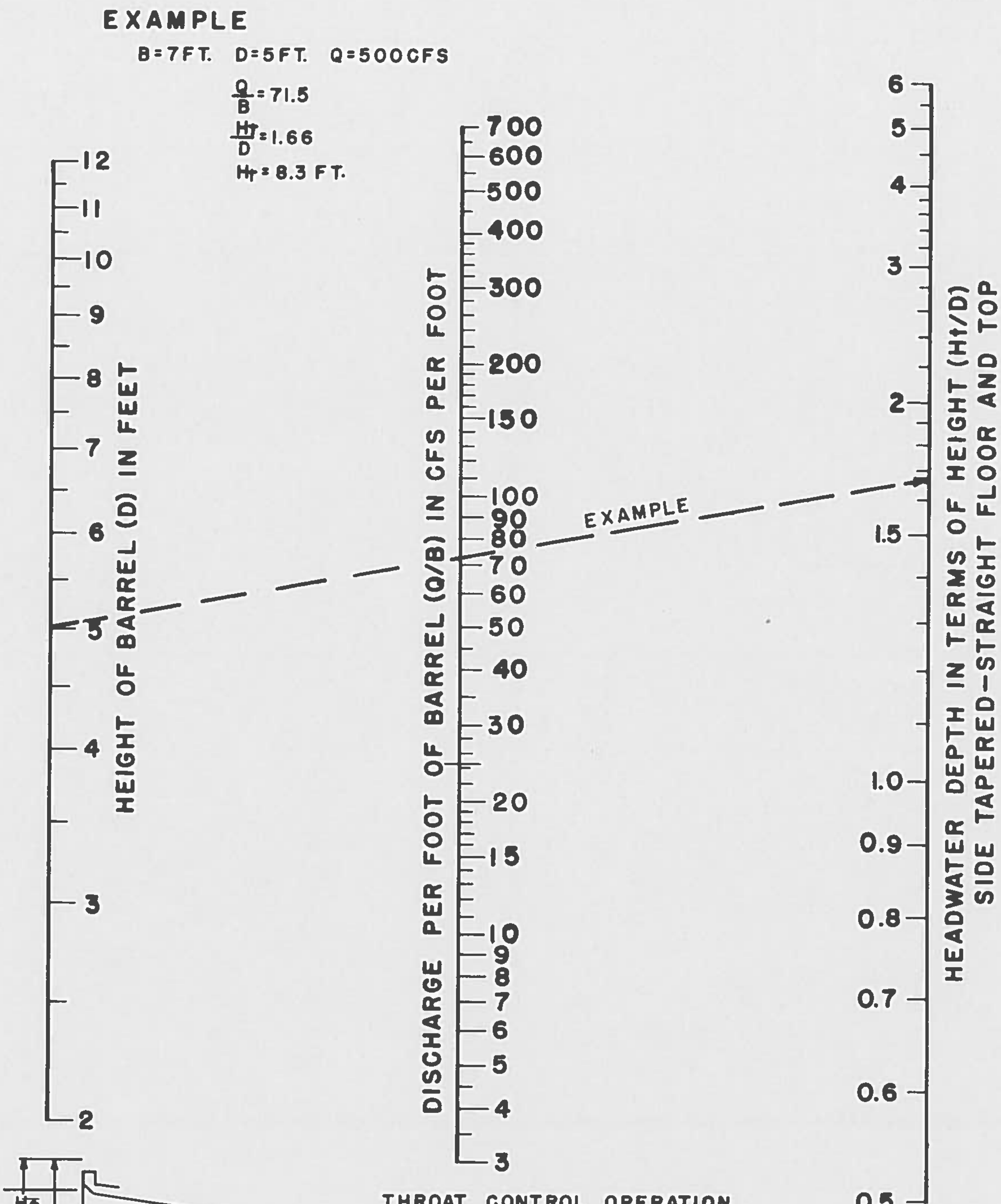
$$H = \left[\frac{1.555\ (1+C_e)}{D^4} + \frac{287.64\ n^2\ L}{D^{16/3}}\right]\left(\frac{Q}{10}\right)^2$$

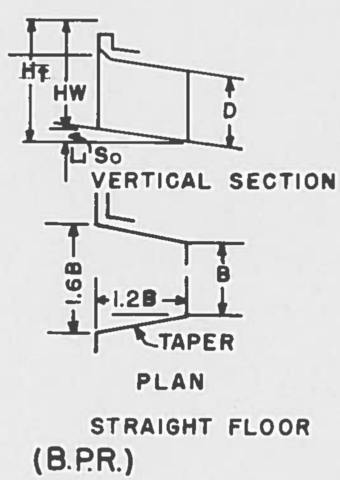
H = Head in feet
C_e = Entrance loss coefficient
D = Height, also span, of box in feet
n = Manning's roughness coefficient
L = Length of culvert in feet
Q = Design discharge rate in cfs

B.P.R.

HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL n = 0.012

Figure 12



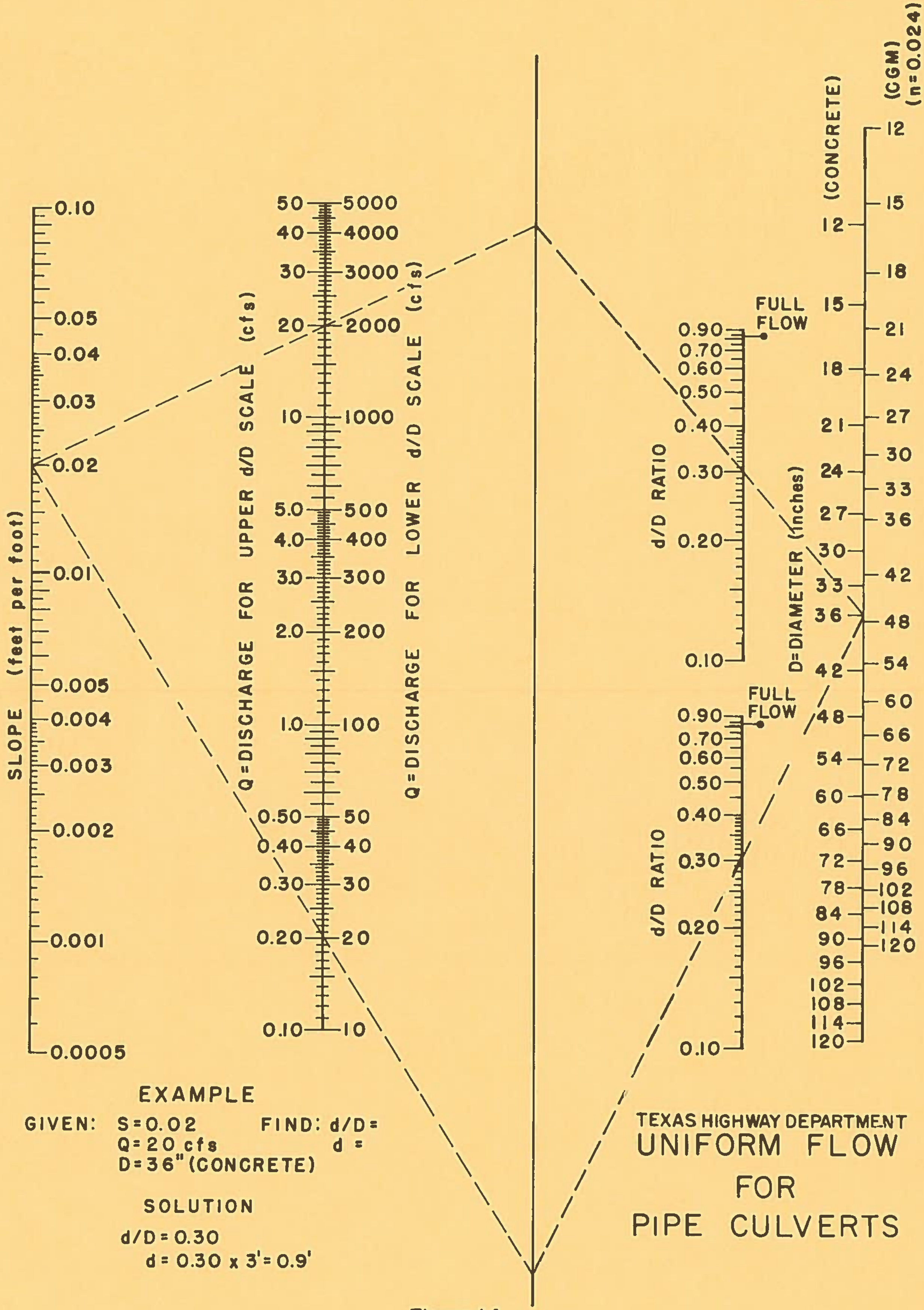


THROAT CONTROL OPERATION OF A TAPERED INLET OCCURS ONLY FOR DIMENSIONS AND DISCHARGE RATES FOR WHICH $H_t - L_1 S_o$ OR H_t-FALL EXCEEDS HW FOR THE FACE SECTION. USE THE SPECIAL INSTRUCTIONS FOR DESIGN OF TAPERED INLETS.

(B.P.R.)

TAPERED-SIDE INLETS ON BOX CULVERTS

Figure 13



EXAMPLE

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

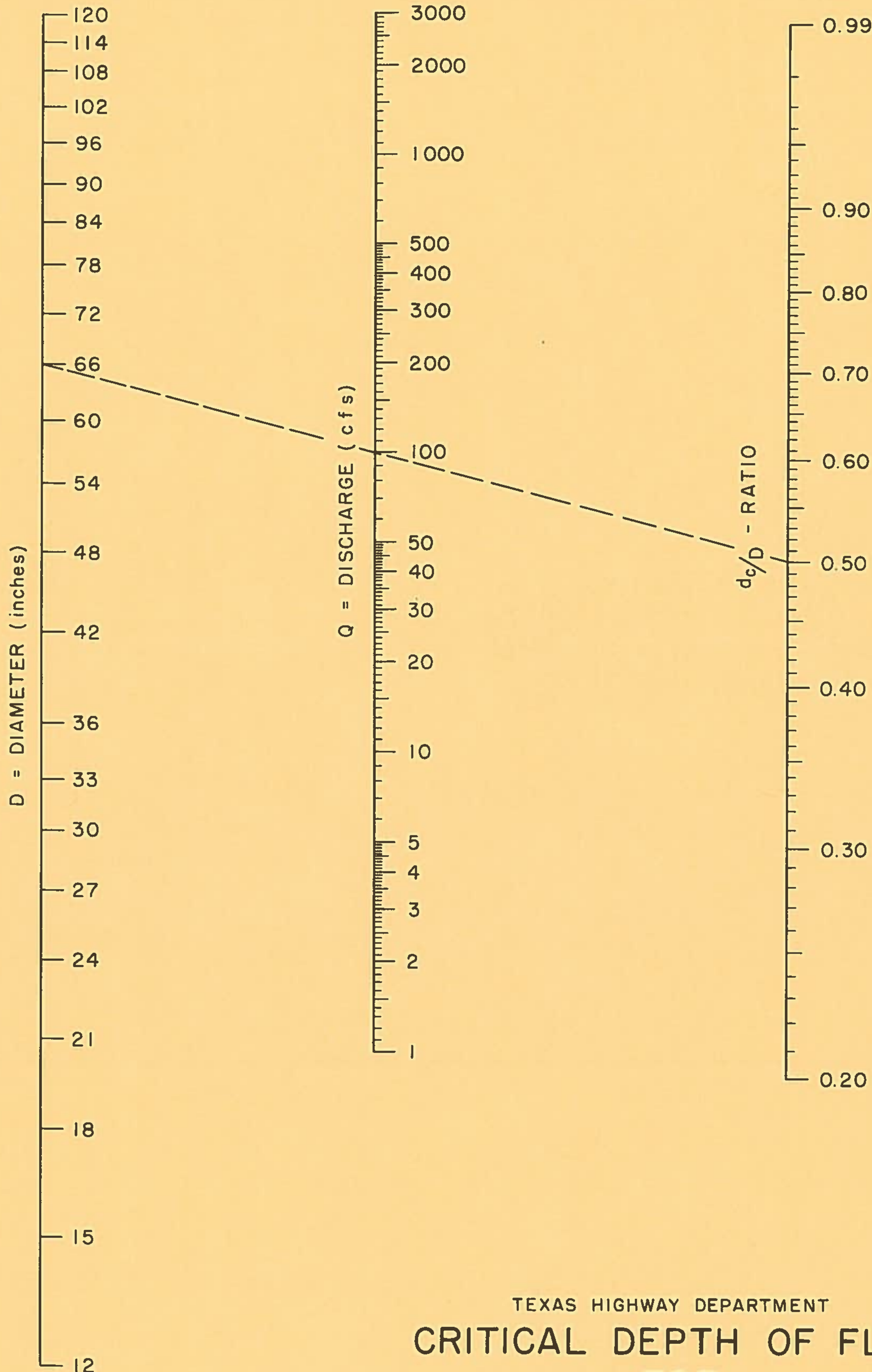
SOLUTION

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

TEXAS HIGHWAY DEPARTMENT

UNIFORM FLOW FOR PIPE CULVERTS

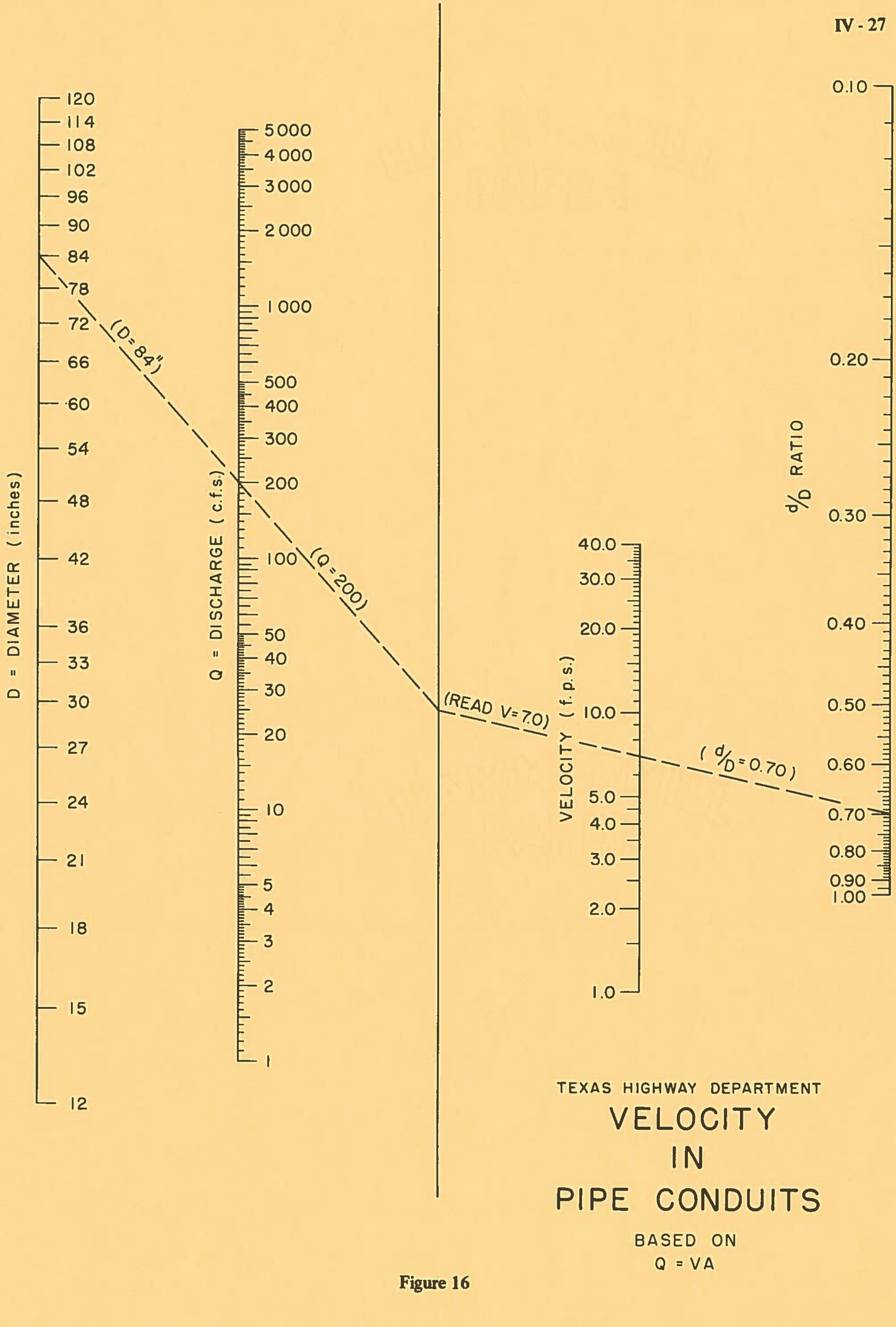
Figure 14



TEXAS HIGHWAY DEPARTMENT

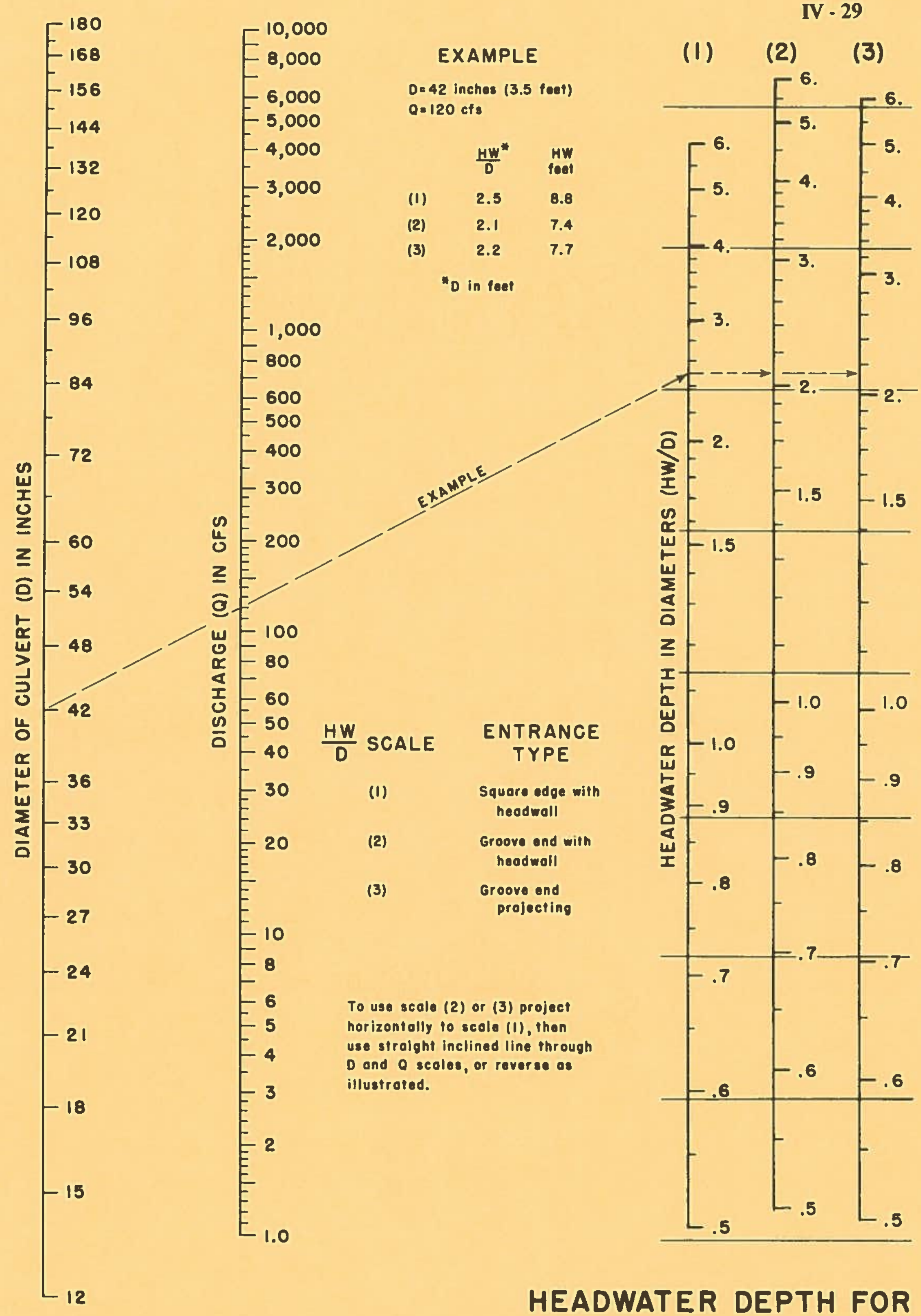
CRITICAL DEPTH OF FLOW FOR CIRCULAR CONDUITS

Figure 15



D = DIAMETER (inches)
120
114
108
102
96
90
84
78
72
66
60
54
48
42
36
33
30
27
24
21
18
15
12
Q = DISCHARGE (c.f.s.)
5000
4000
3000
2000
1000
500
400
300
200
100
50
40
30
20
10
5
4
3
2
1
(D=84")
(Q=200)
(READ V=7.0)
VELOCITY (f. p. s.)
40.0
30.0
20.0
10.0
5.0
4.0
3.0
2.0
1.0
(d/D = 0.70)
d/D RATIO
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
TEXAS HIGHWAY DEPARTMENT
VELOCITY
IN
PIPE CONDUITS
BASED ON
Q = VA

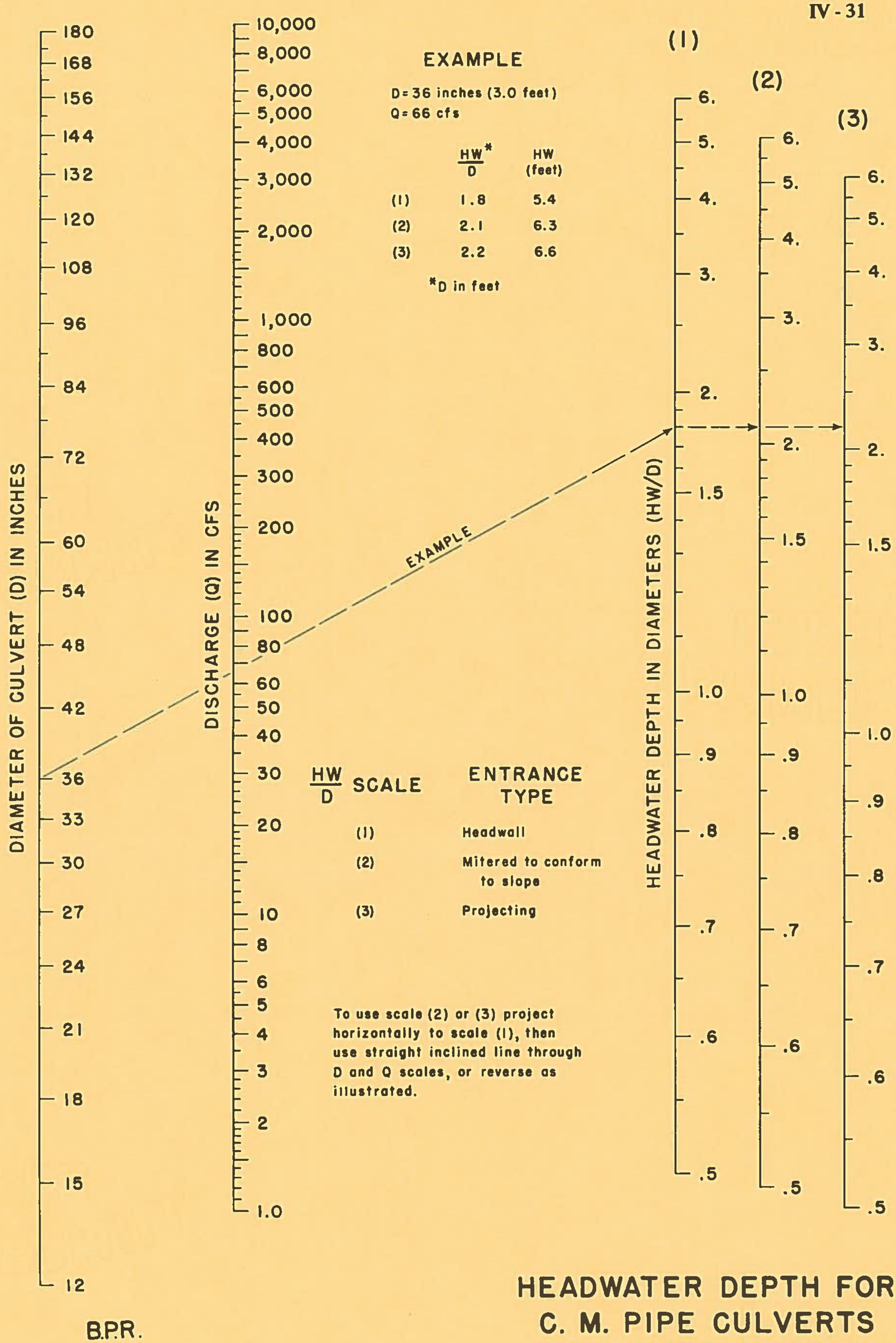
Figure 16



B.P.R.

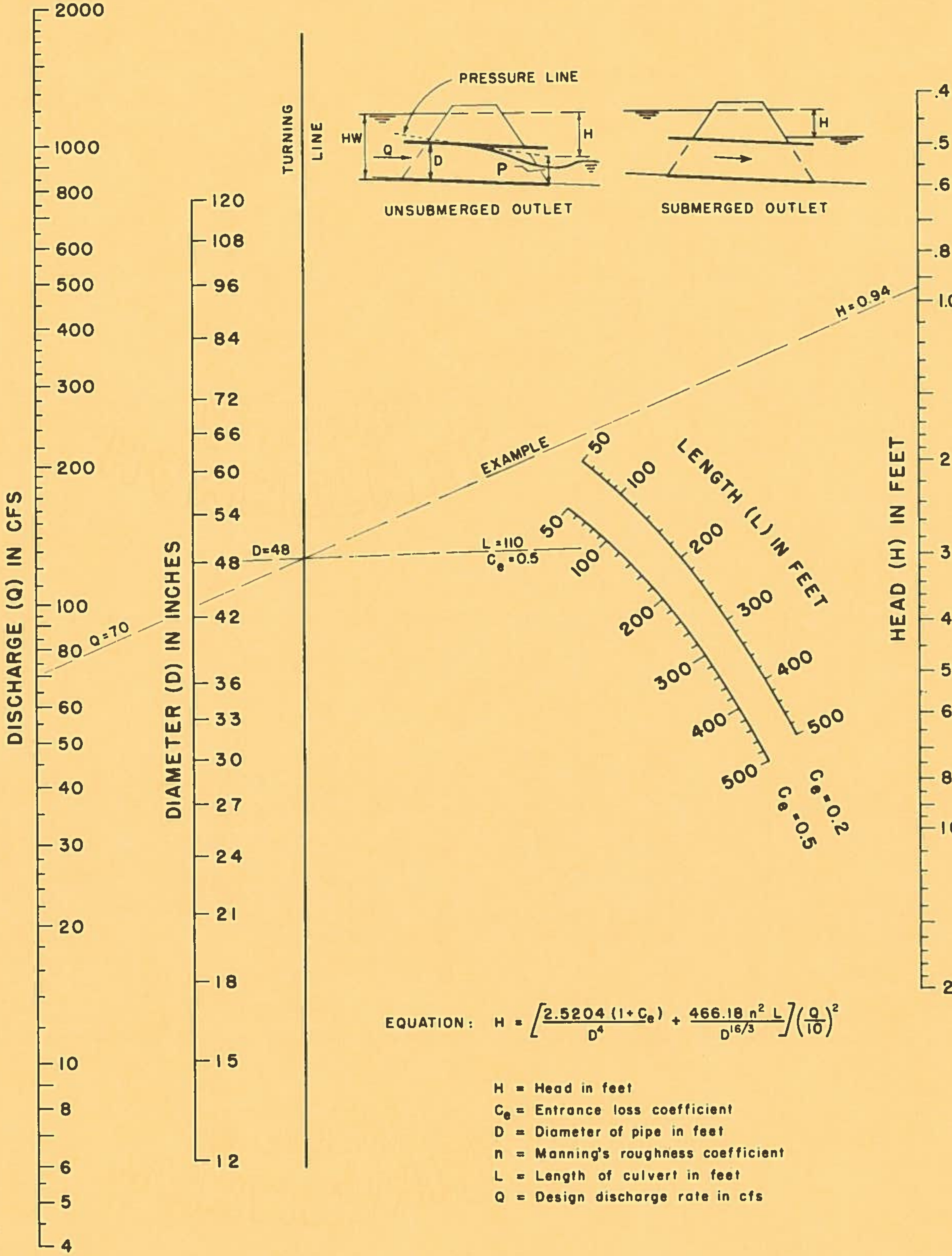
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

Figure 17



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

Figure 18



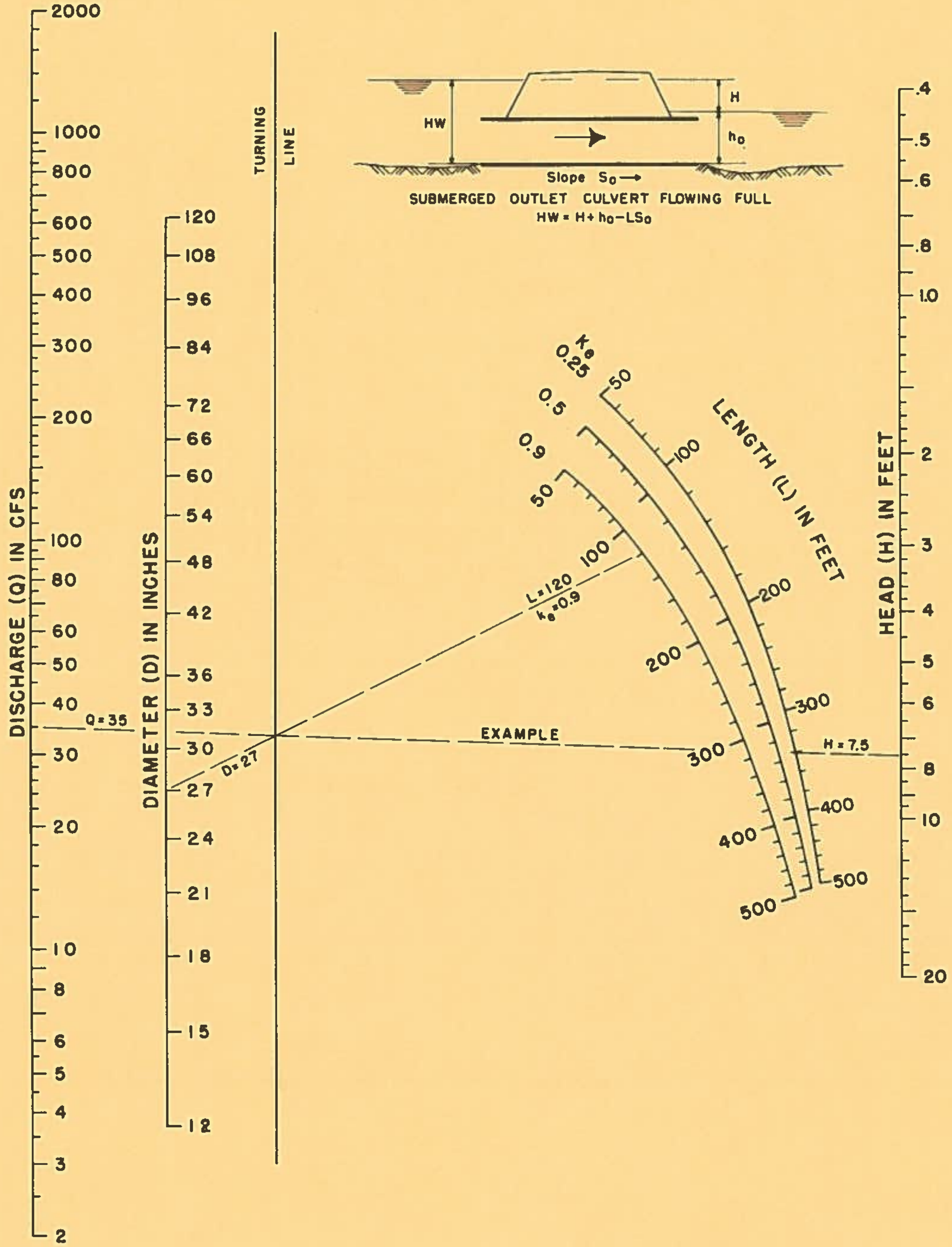
EQUATION: $$H = \left[\frac{2.5204\,(1+C_e)}{D^4} + \frac{466.18\,n^2\,L}{D^{16/3}}\right]\left(\frac{Q}{10}\right)^2$$

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

B.P.R.

HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL n = 0.012

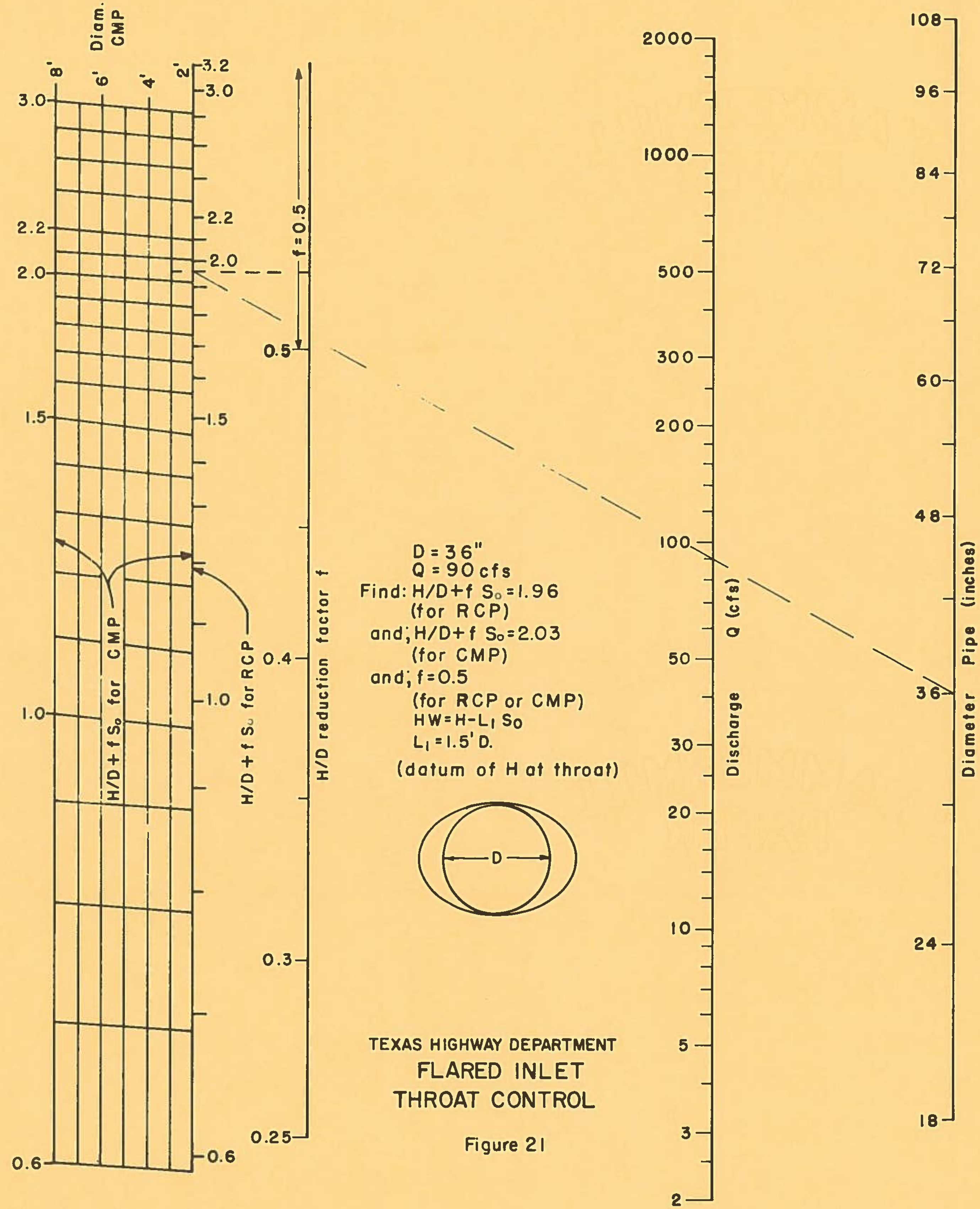
Figure 19



B.P.R.

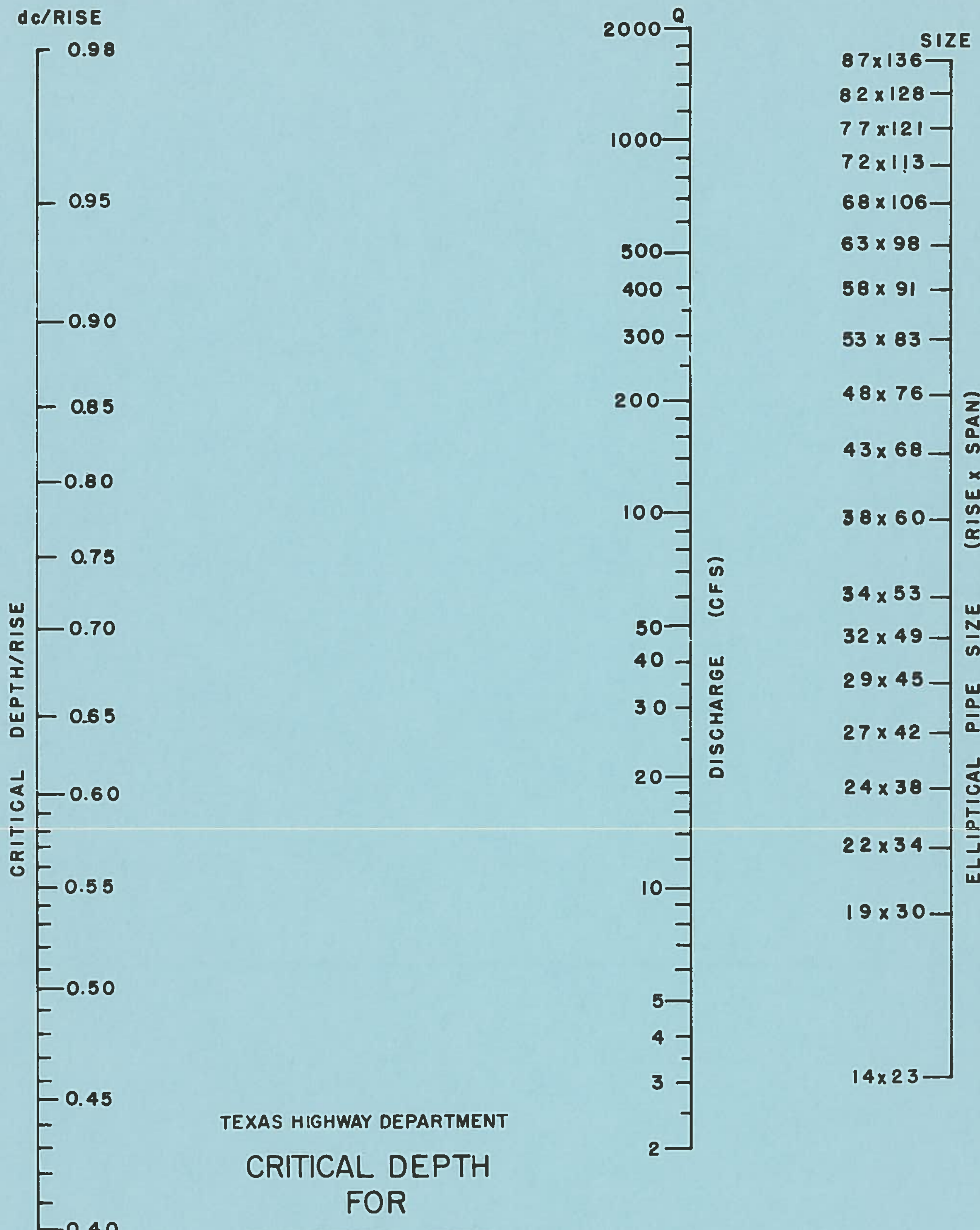
HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
n = 0.024

Figure 20



TEXAS HIGHWAY DEPARTMENT
FLARED INLET
THROAT CONTROL

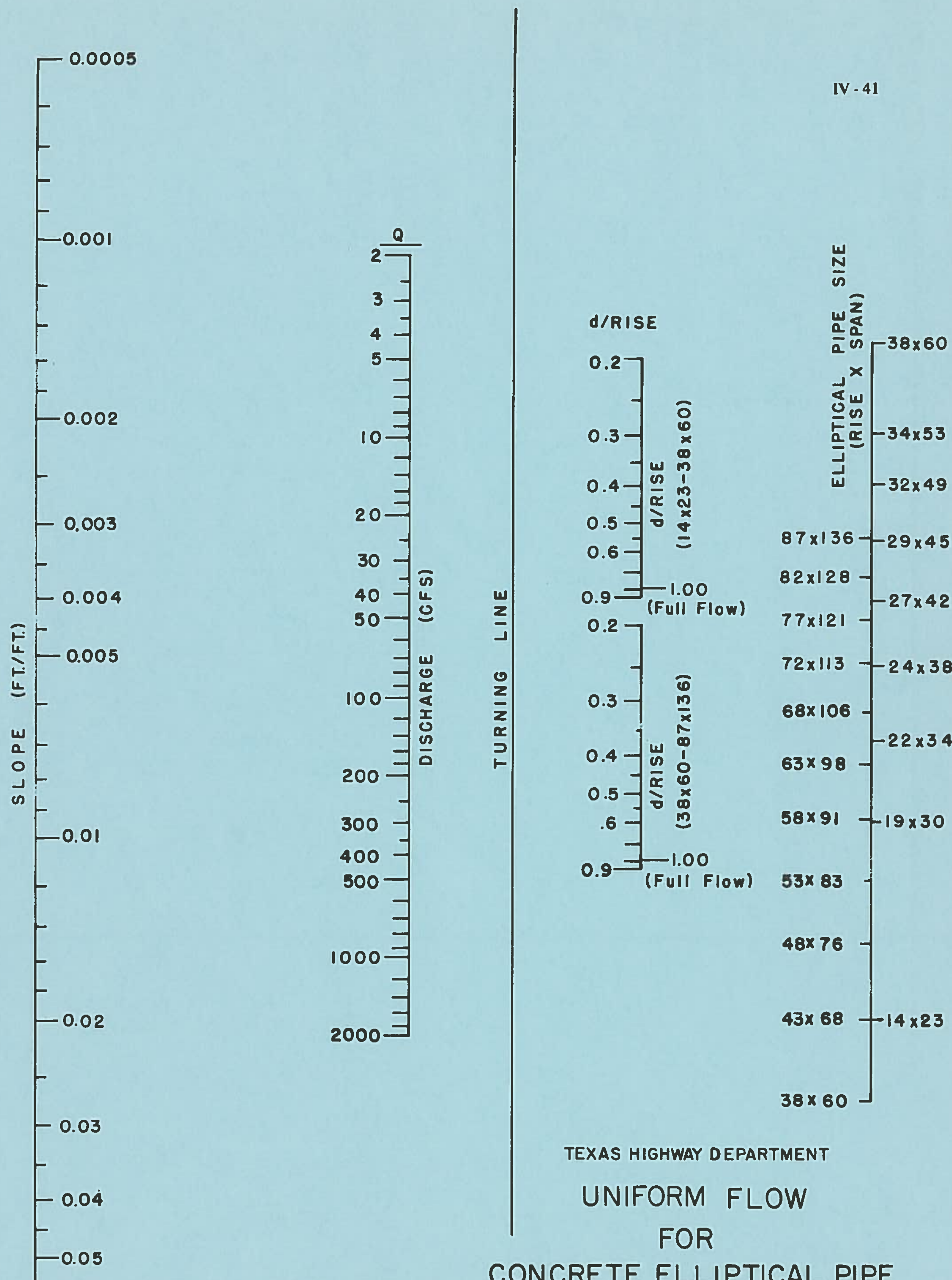
Figure 21



TEXAS HIGHWAY DEPARTMENT

CRITICAL DEPTH FOR ELLIPTICAL PIPE

Figure 22



TEXAS HIGHWAY DEPARTMENT

UNIFORM FLOW FOR CONCRETE ELLIPTICAL PIPE

Figure 23

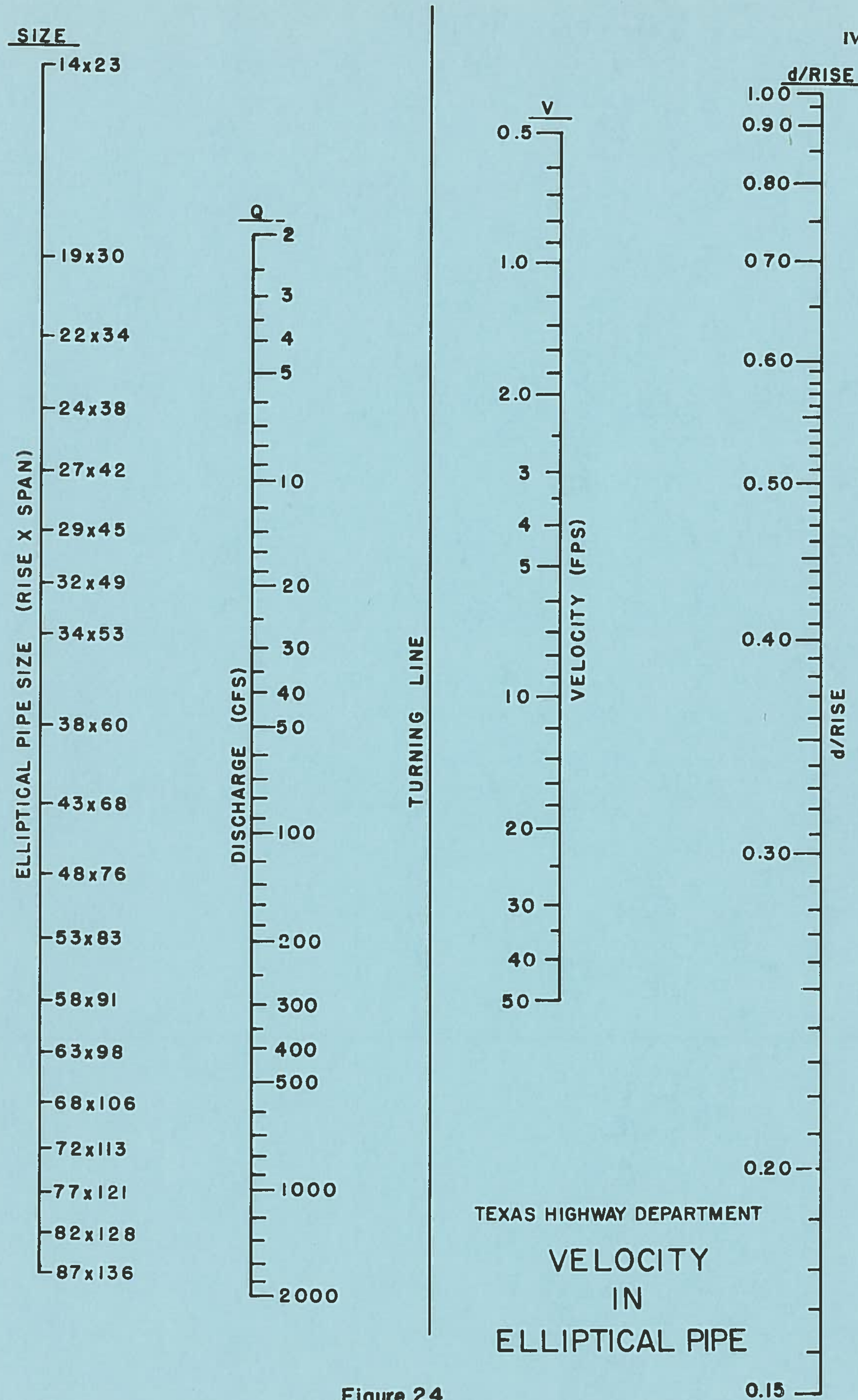
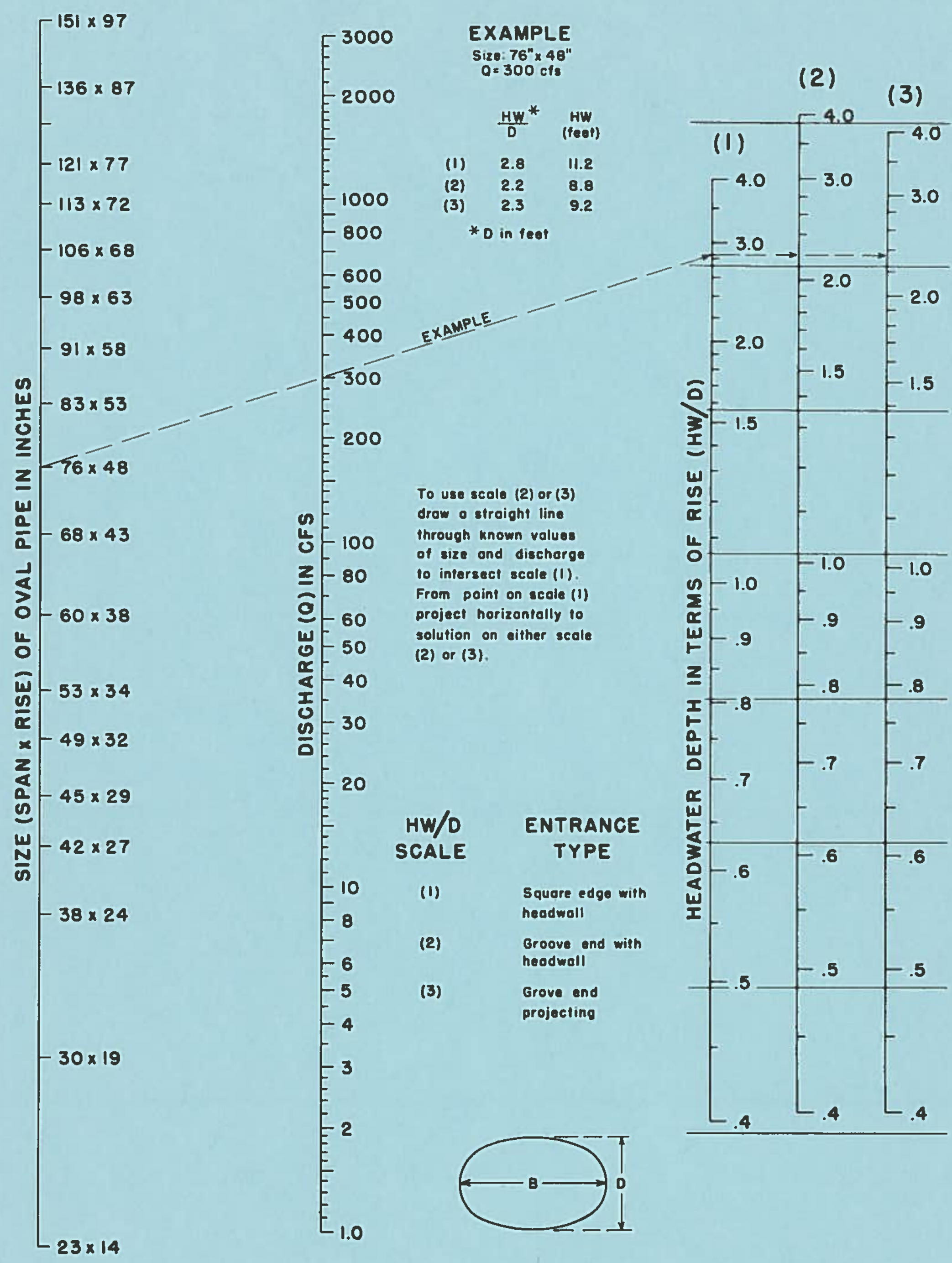

SIZE
14x23
19x30
22x34
24x38
27x42
29x45
32x49
34x53
38x60
43x68
48x76
53x83
58x91
63x98
68x106
72x113
77x121
82x128
87x136
ELLIPTICAL PIPE SIZE (RISE X SPAN)
Q
2
3
4
5
10
20
30
40
50
100
200
300
400
500
1000
2000
DISCHARGE (CFS)
TURNING LINE
V
0.5
1.0
2.0
3
4
5
10
20
30
40
50
VELOCITY (FPS)
d/RISE
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.15
d/RISE
TEXAS HIGHWAY DEPARTMENT
VELOCITY
IN
ELLIPTICAL PIPE

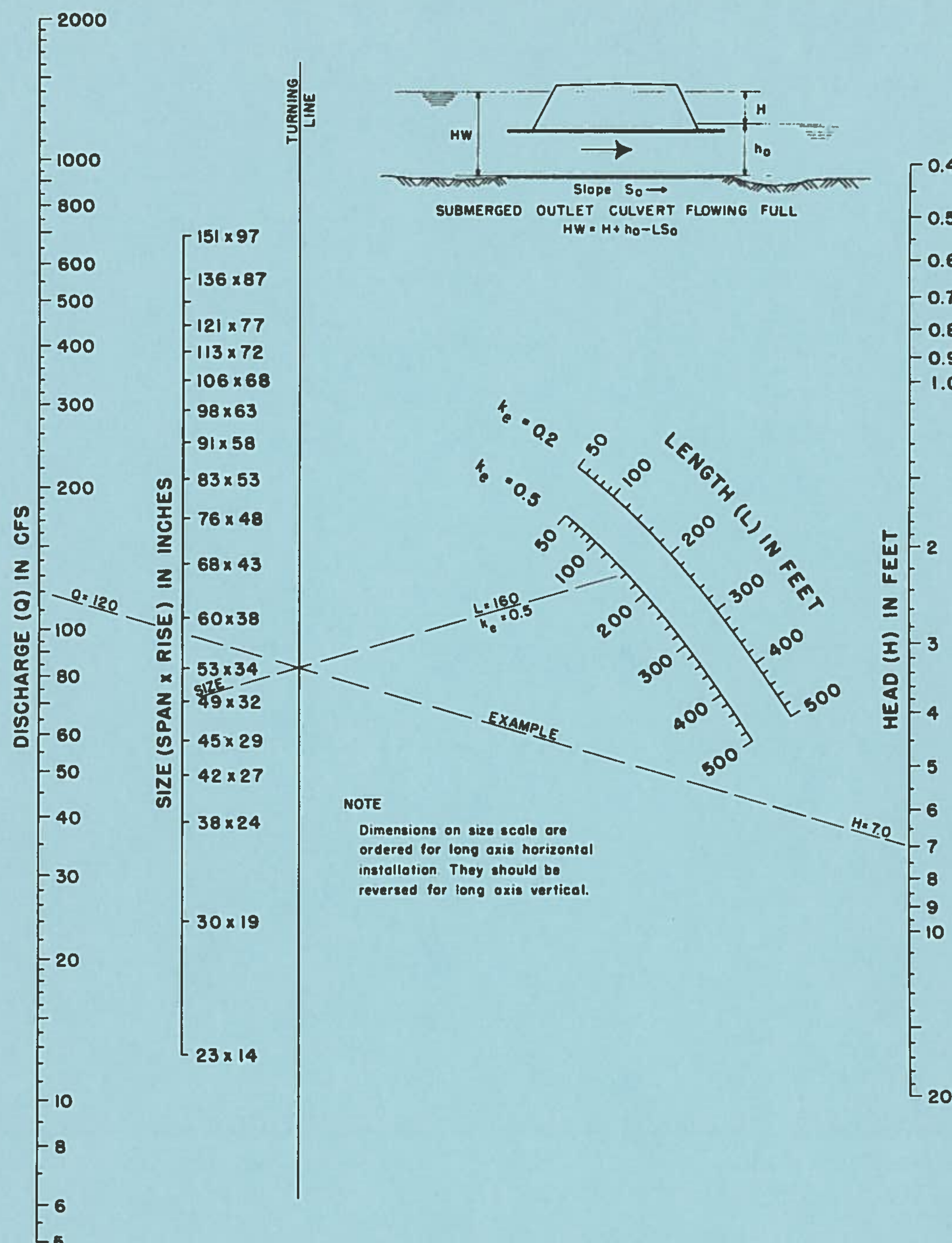
Figure 24



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

B.P.R.

Figure 25

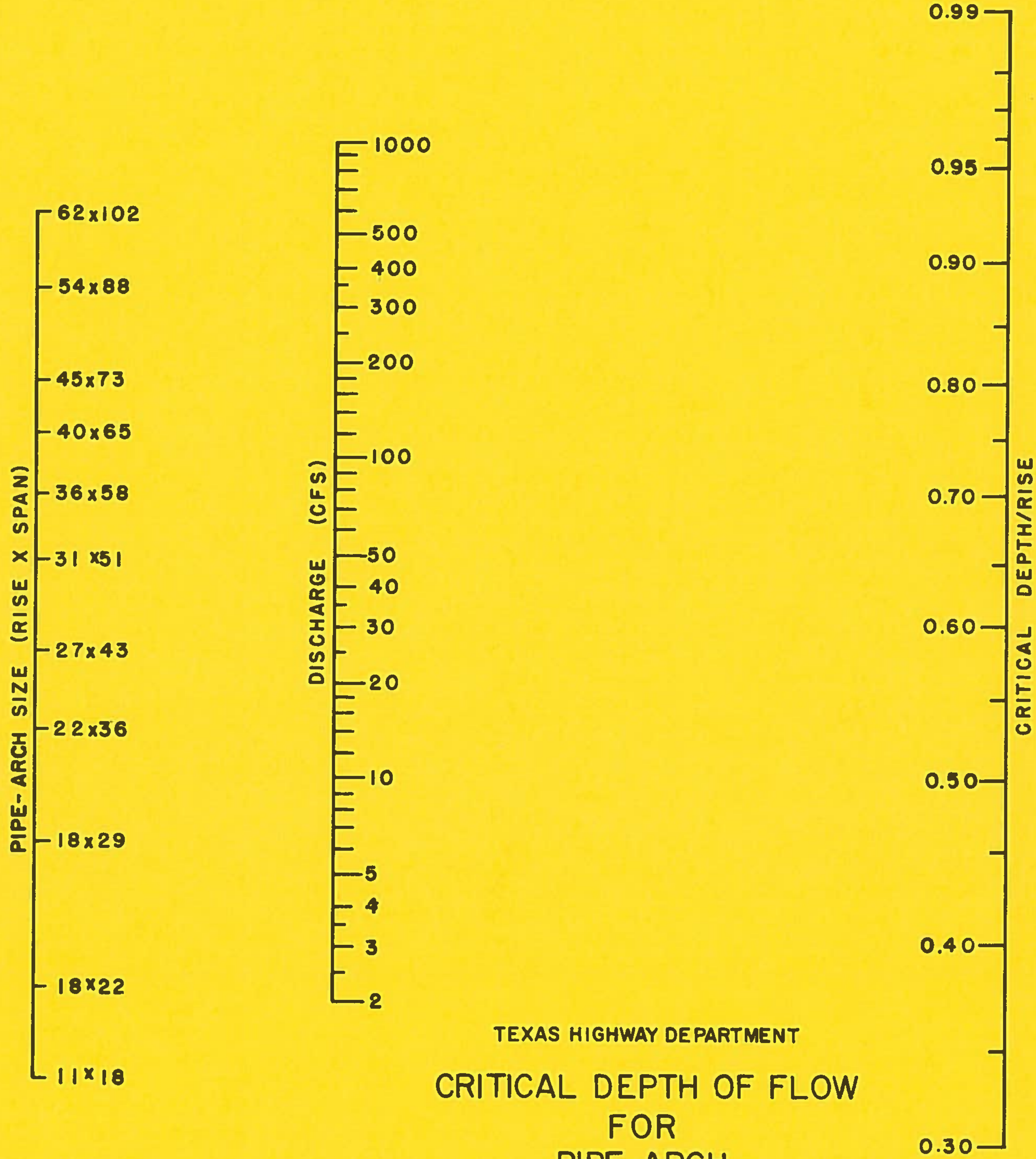


HEAD FOR ELLIPTICAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL OR VERTICAL FLOWING FULL

$n = 0.012$

B.P.R.

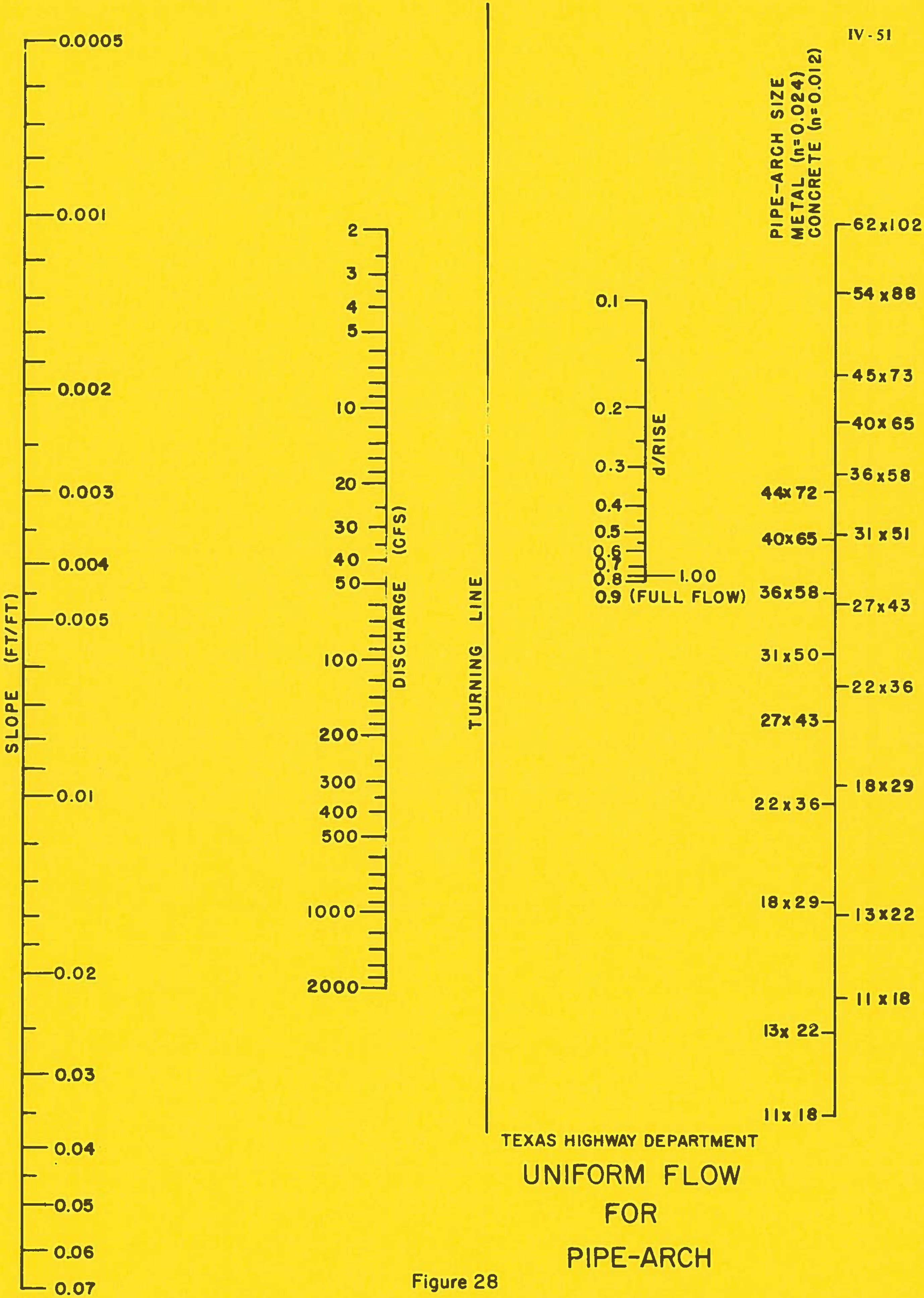
Figure 26



TEXAS HIGHWAY DEPARTMENT

CRITICAL DEPTH OF FLOW
FOR
PIPE-ARCH

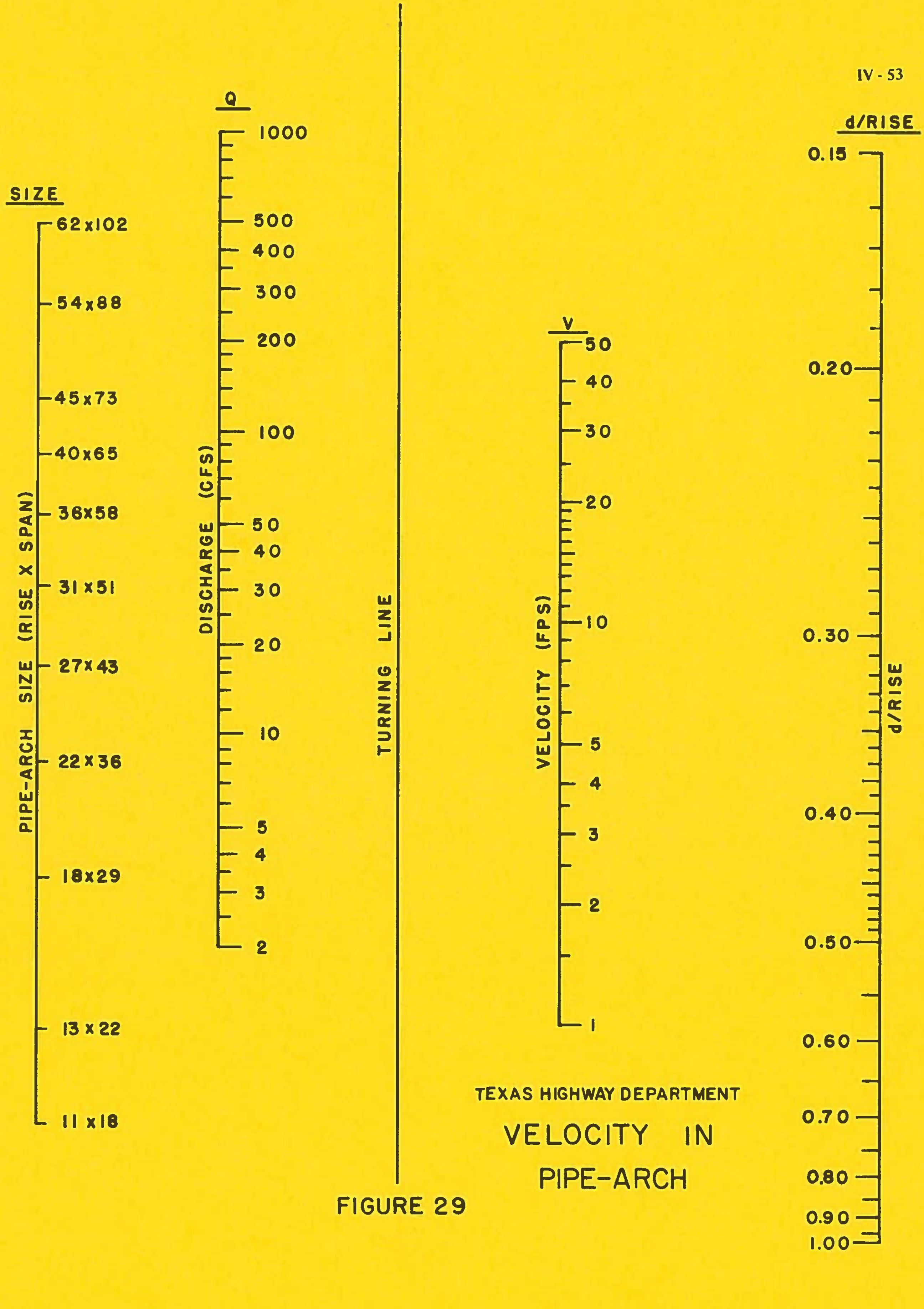
Figure 27



TEXAS HIGHWAY DEPARTMENT

UNIFORM FLOW FOR PIPE-ARCH

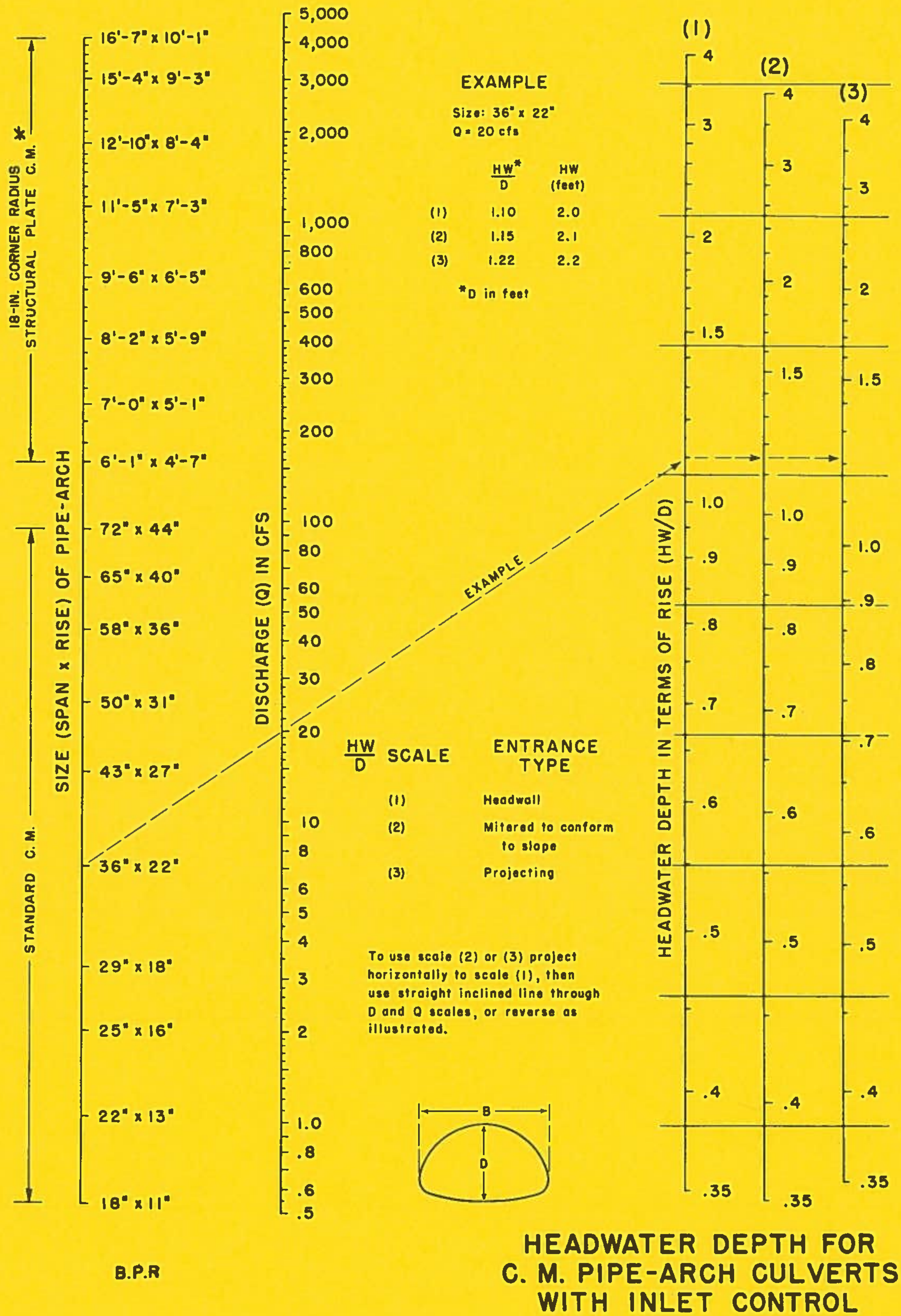
Figure 28



TEXAS HIGHWAY DEPARTMENT

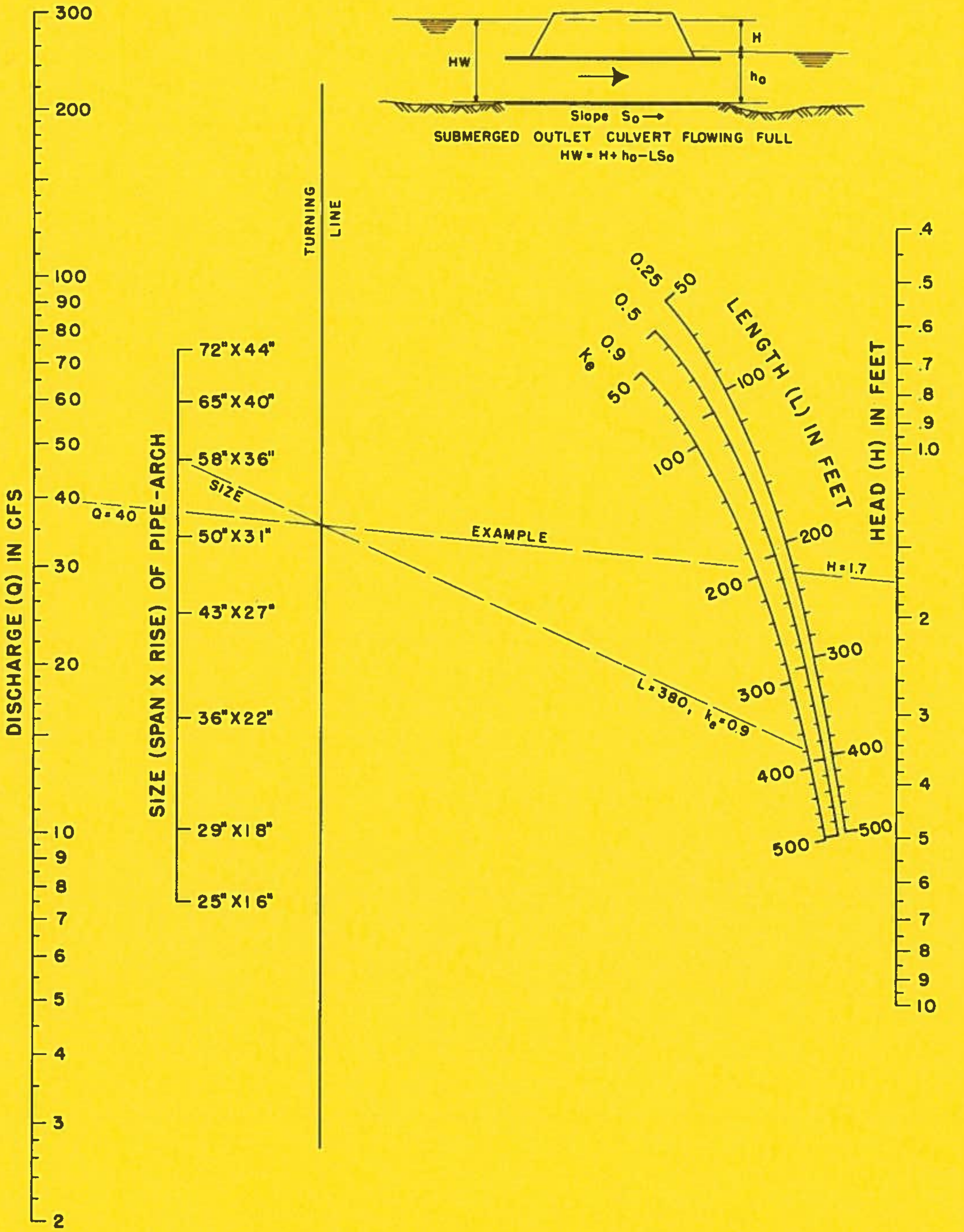
VELOCITY IN PIPE-ARCH

FIGURE 29



HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL

Figure 30



HEAD FOR STANDARD C. M. PIPE-ARCH CULVERTS FLOWING FULL

n=0.024

B.P.R.

Figure 31

4-700. OUTLET VELOCITY CONTROL

4-701. GENERAL

If the outlet velocity of a culvert is deemed excessive by the engineer, there are several possible solutions available by which velocity may be either reduced or controlled. Historically, the most widely used control has been the use of riprap which covers the channel area immediately downstream from the culvert outlet. However, there are other design possibilities available, some of which are more positive than others. Certain special culvert types have the chief function of maintaining acceptable outlet velocities which would be excessive if a straight profile culvert were used. One such special culvert type is the broken-back culvert.

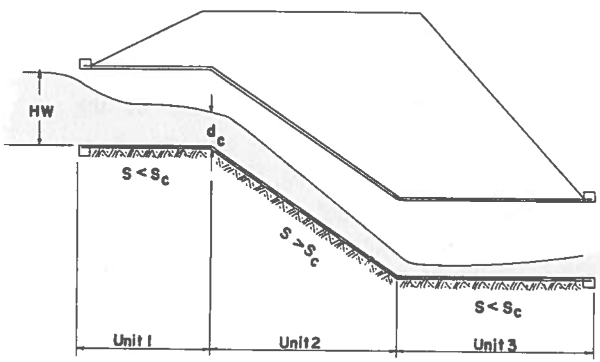


Figure 32 (a)

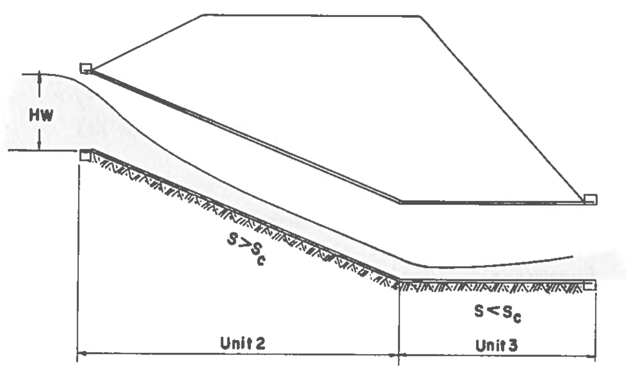


Figure 32 (b)

4-702. VELOCITY CONTROL DEVICES

4-702.1 BROKEN-BACK CULVERT DESIGN

The designer may occasionally encounter culvert locations at which the difference in elevation from one side of the highway to the other is so great as to cause outlet velocities which are unacceptably high. In this event, the engineer may choose to design a broken-back culvert as depicted in Figures 32a or 32b. The advantage of a broken-back culvert towards reducing outlet velocities lies in the theory of the hydraulic jump.

The broken-back culvert employs the hydraulic jump as an energy dissipator (or velocity reducer). But the formation of the jump requires that:

a. there be sufficient natural tailwater (not usually the case since the broken-back culvert is usually located on a steep profile such as a hillside and stream depth is rather shallow)
b. there be sufficient friction in unit 3 (see Figure 32) of the culvert. This is possible only if the culvert material is very rough and/or the unit 3 length is relatively long. Normally, in a broken-back culvert, neither of the above circumstances exist. It is therefore necessary to employ one or more of certain special devices to assist in forming the hydraulic jump and reduce velocities.

The hydraulic jump is a feature in hydraulic flow by which flow in a super-critical condition is abruptly changed to flow in a subcritical condition. In other words, a given discharge traveling at some depth less than critical depth and some velocity greater than critical velocity, upon passing through a hydraulic jump, then flows at some depth greater than critical depth and some velocity less than critical velocity.

The design of a broken-back culvert is not particularly difficult but certain provisions must be made or circumstances found so that the primary intent of reducing velocity at the outlet is realized. The design and the provisions or circumstances are discussed as follows.

1. After design discharge and tailwater are determined, the first step necessary in a broken-back culvert design is the establishment of the flow line profile. This should be done by keeping in mind the following considerations. With reference to Figures 32 a and 32b, unit 3 should be as long as possible. This means that for a given total drop, the resulting length of unit 2 is short; but this causes the slope of unit 2 to be very steep. As long as unit 1 is on a mild slope, its length has no effect on outlet velocity of any downstream hydraulic function. In fact, it is recommended that unit 1 either not be used or be very short. This allows for more adjustment in the profiles of unit 2 and unit 3.

   In carrying out water surface profile calculations, it will be seen that a longer unit 3 and a milder slope in unit 2 together enhance the possibility of realizing a hydraulic jump. However, these two conditions are contradictory and obviously not possible for a given culvert location. Therefore, some compromise must be made between length of unit 3 and slope of unit 2. Unit 3 must be on a mild slope ($S_o < S_c$) and it is recommended that this slope be only that necessary to prevent puddling.

2. The culvert should be initially sized according to procedures outlined in steps 1 and 2, section 4-602. It will be found that if a unit 1 is used, the HW will be calculated by the procedure for Type I or Type IVB. If unit 1 is not used, the HW will ordinarily be calculated by the procedure for Type IIIA.

3. At this point, the following conditions should be known: design discharge, culvert shape and size, culvert profile (unit 2 slope greater than critical slope and unit 3 slope definitely less than critical slope), and tailwater depth.

   Determine uniform depth for unit 2 with design discharge, unit 2 slope, and culvert shape and size. Use nomographs on Figures 10 for box culverts, Figure 14 for circular pipe, Figure 23 for elliptical pipe, and Figure 28 for pipe-arch. This depth is a conservative estimate of the water depth at the break in profile between units 2 and 3. Actual depth is probably slightly more than uniform depth for ordinary conditions but the computations necessary to determine this actual depth are rather tedious.

4. As the water enters unit 3 at uniform depth, a new type of water surface profile curve occurs. This is the type of curve in which the depth of the water is less than critical depth on a mild slope ($S_o < S_c$) and depth increases as the water moves downstream. The condition of flow on this type of curve (termed an M3 curve), is said to be supercritical flow. Each depth on the M3 curve has a corresponding theoretical conjugate depth. This conjugate depth is that depth at which the same discharge would be flowing in a subcritical state after the occurrence of the hydraulic jump. In simple terms, there must be a depth of water downstream from a hydraulic jump that is equal to or greater than the conjugate depth for the supercritical depth upstream from the jump before that jump can occur. This means that before a hydraulic jump is even possible there must be a tailwater downstream from the culvert of depth greater than critical depth (though this does not necessarily assure the formation of a hydraulic jump), or the length of unit 3 must be long enough so that the depth on the M3 curve can reach critical depth. The following procedure includes the calculation of supercritical depth d_1 and location L, on the M3 curve, the conjugate depth d_2, the comparison of each d_2 to TW depth*, the comparison of each d_1 to critical depth, and the comparison of each L, to unit 3 length. At any point that a hydraulic jump condition is satisfied, the outlet velocity is then computed and the process is ended. Increments to d_1 to be used may be an increment so that the change in adjacent velocities is not more than 10%. This is illustrated in the solution procedure.

   *TW depth may be a natural stream flow depth or may be artifically produced by installing a sill on the downstream apron between wingwalls. See discussion of sills under Section 4-702.3. The value of this artificial TW depth is determined by adding the sill height and the critical depth of design discharge flow over the sill. This critical depth must be based on the rectangular section formed by the top of the sill and the two vertical wingwalls.

5. The first d_1 entry for this step will be the uniform depth determined for unit 2 and its corresponding L_1 equals zero. Find velocity, velocity head, energy head and friction slope as described below. All references to 'preceding' values should be ignored for the first d_1. Skip to j. For subsequent d_1 values,

   a. find velocity for d_1 (make sure the change from the preceding velocity is no more than 10%)

   b. find velocity head ($h_v = V^2/2g$)

   c. find energy head $H_e = d_1 + h_v$

   d. find change in energy head between this d_1 and the preceding d_1 (negative value for M3 curve)

   e. find friction slope for this d_1

   f. average this friction slope and the preceding friction slope

   g. subtract the average friction slope from the actual slope of unit 3 (negative value for M3 curve)

   h. find increment in length (distance between this d_1 and the preceding d_1) by dividing the change in energy head by the difference in slopes (from (g) above)

   i. find L_1 by adding the increment of length to the preceding value of L_1

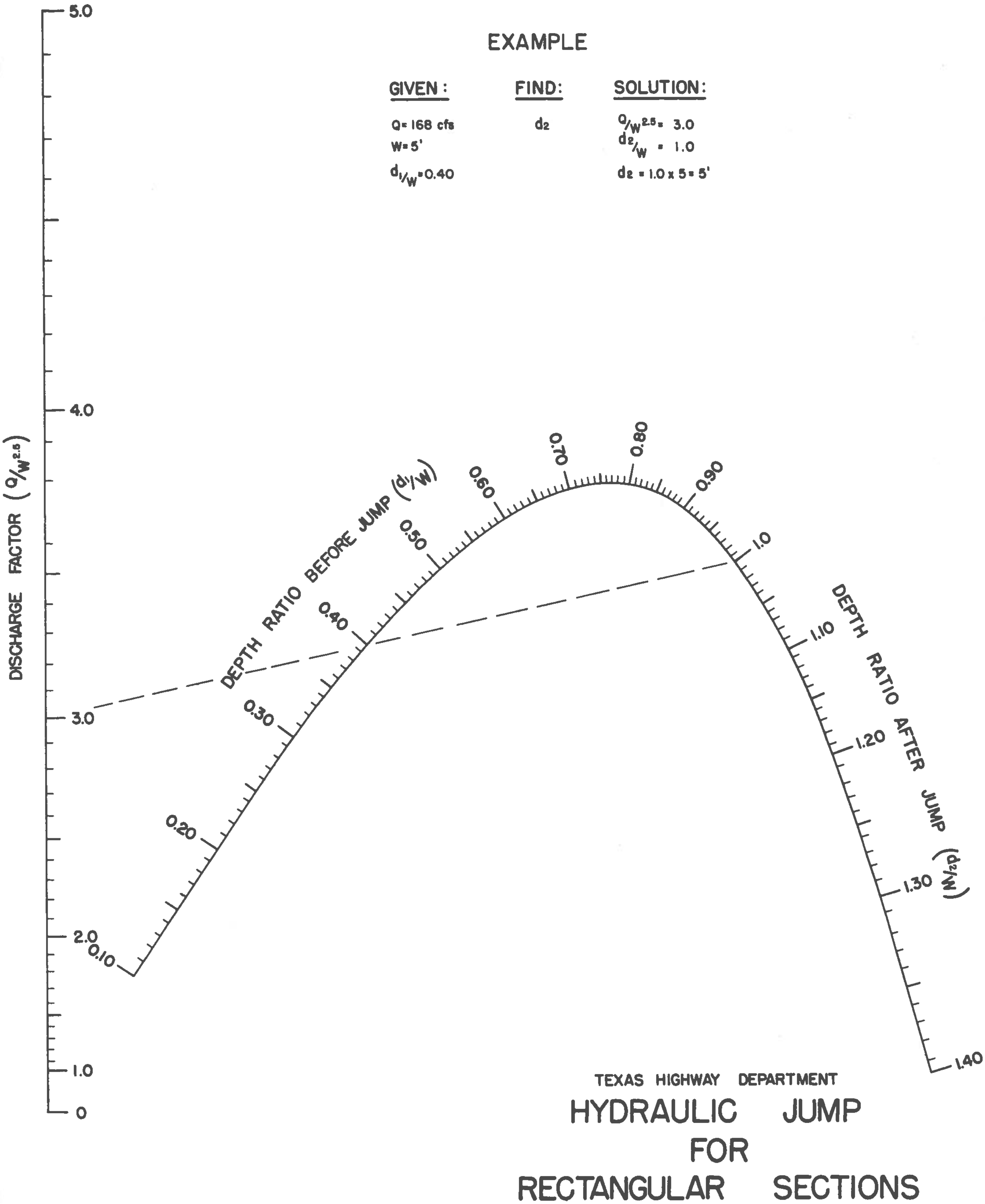
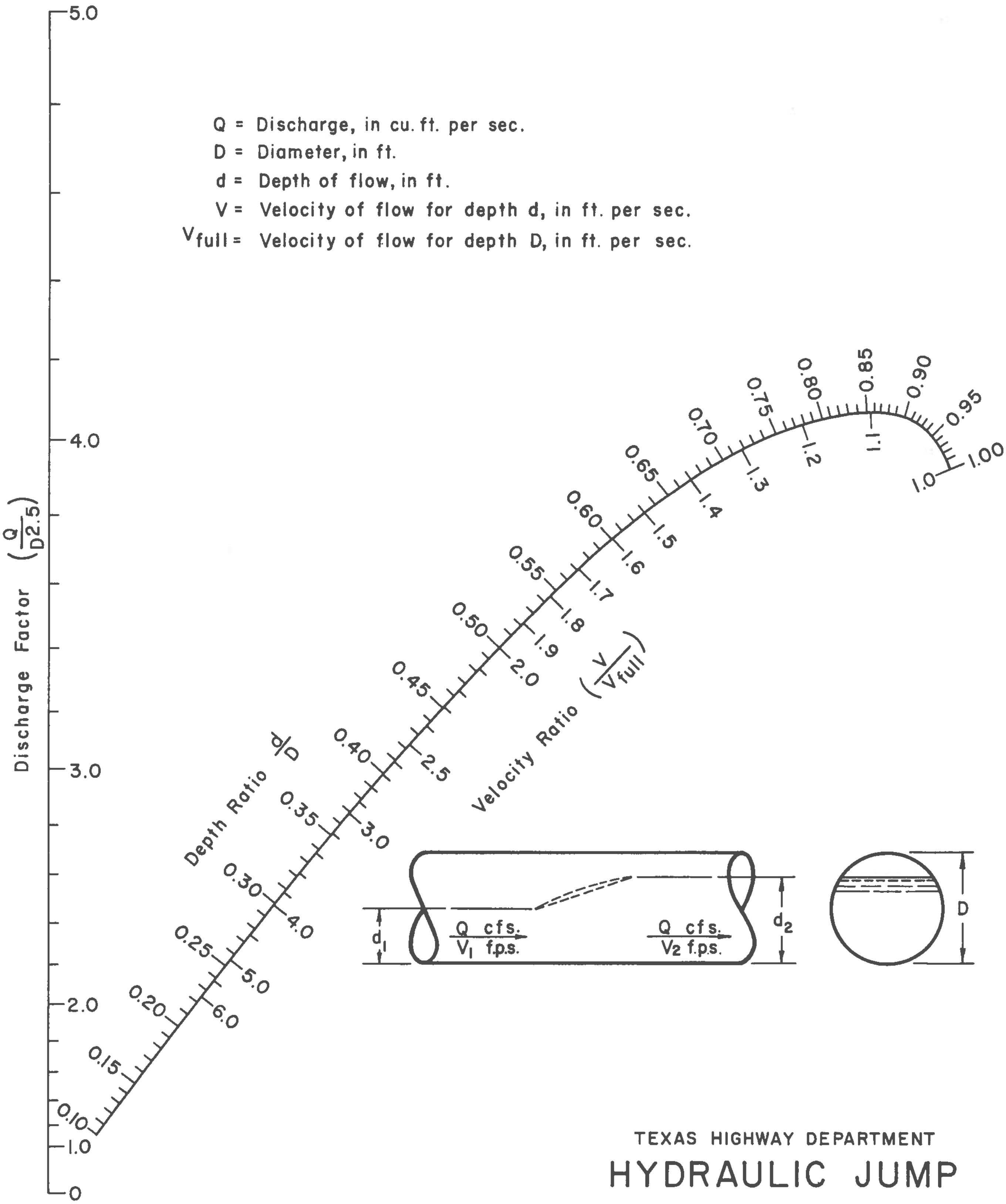

EXAMPLE
GIVEN:
Q= 168 cfs
W= 5'
d1/W=0.40
FIND:
d2
SOLUTION:
Q/W2.5= 3.0
d2/W = 1.0
d2 = 1.0 x 5 = 5'
DISCHARGE FACTOR (Q/W2.5)
5.0
4.0
3.0
2.0
1.0
0
DEPTH RATIO BEFORE JUMP (d1/W)
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.0
DEPTH RATIO AFTER JUMP (d2/W)
1.10
1.20
1.30
1.40
TEXAS HIGHWAY DEPARTMENT
HYDRAULIC JUMP
FOR
RECTANGULAR SECTIONS

Figure 33



TEXAS HIGHWAY DEPARTMENT

HYDRAULIC JUMP FOR CIRCULAR SECTIONS

Figure 34

j. obtain conjugate depth d_2 from Figure 33 for box culverts and Figure 34 for circular pipe

k. compare d_2 to TW. If d_2 is less than or equal to TW, skip to 6, otherwise;

m. compare d_1 to critical depth. If d_1 is greater than or equal to d_c, skip to 8, otherwise;

n. compare L_1 to unit 3 length. If L_1 is greater than or equal to unit 3, skip to 9, otherwise;

p. increase d_1 by some increment and go back to a. above

6. Hydraulic jump forms.

   If TW is greater than barrel depth D, skip to 7, otherwise; outlet velocity should be determined by dividing design discharge by the area of flow under TW depth, skip to 10.

7. Hydraulic jump forms.

   Outlet velocity should be determined by dividing design discharge by the area of full flow for the barrel. Skip to 10.

8. In this case, hydraulic jump will occur but outlet depth will be equal to critical depth and outlet velocity equals critical velocity. Skip to 10.

9. In this case, the hydraulic jump does not form within the length of unit 3 and exit depth is the present value of d_1. Therefore, outlet velocity is design discharge divided by area of flow under d_1. Skip to 11.

10. If a sill has been employed to force an artifical tailwater and the hydraulic jump has formed, the outlet velocity computed represents the velocity of water as it exits the barrel. However, the velocity at which water re-enters the channel is the crucial velocity. This velocity would be the critical velocity of sill overflow. After this is determined, the design is complete.

11. If this step is reached, the hydraulic jump has not formed and the broken-back culvert configuration is ineffective and should be changed in some manner. Possibilities are rearrangement of the culvert profile or addition of a sill.

    If sills are used, the addition of some length of downstream concrete riprap is always in order. Usually, any changes made must be accompanied by new computations. Go back to 5.

4-702.2 RIPRAP

Riprap, when used as an outlet velocity control measure, should be applied to the channel area immediately downstream of the culvert outlet for a distance of no less than 20 feet or to the ROW, whichever is less. This arbitrary limit may be tempered by engineering judgement based on the severity of the velocity and the potential for erosion or scour. Most of the various types of riprap have proven effective as erosion and scour preventatives and soil protectors. Also see Chapter III, Section 3-301, Channel Lining.

4-702.3 SILLS

Limited research has shown that the use of the sill (as depicted in Figure 35) is very effective in aiding the formation of the hydraulic jump in broken-back culverts and in spreading the water back to the natural stream

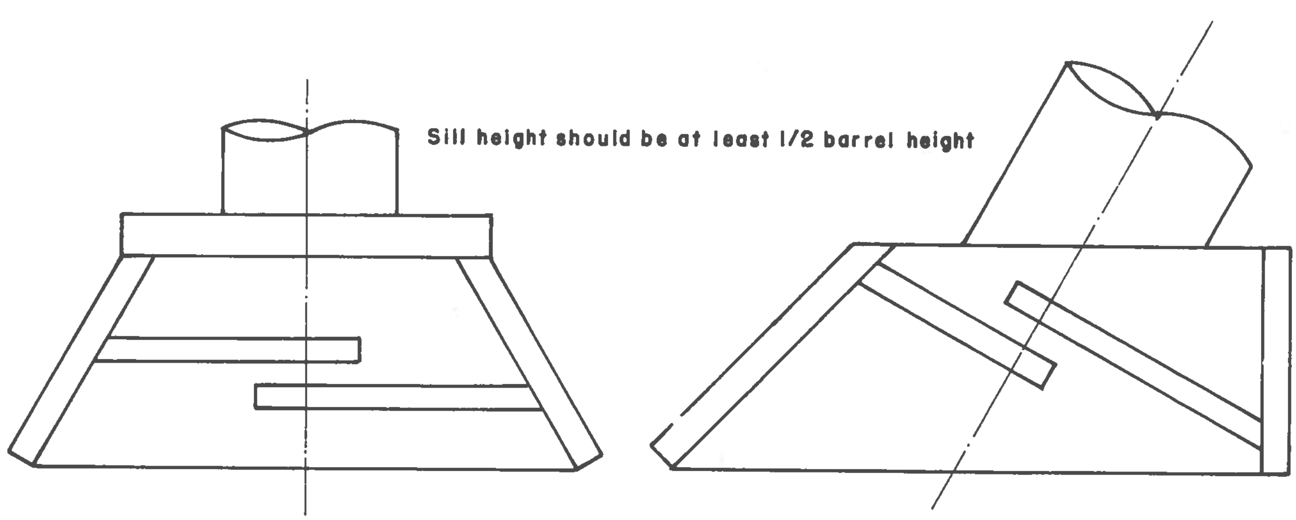


Figure 35

width. The use of sills should not be limited to broken-back culverts as they may effectively retard excessive velocities in any type culvert. Some disadvantages of sills are the possible danger of siltation and the waterfall effect which they usually cause. In certain areas, the sill must be closely maintained to keep it clear of silt. It is recommended that riprap be installed immediately downstream of the sill for a minimum distance of 10 feet to control the turbulence from the waterfall effect.

Until further notice, sills should be located at the midpoint in the downstream culvert wingwall and should be at least 1/2 the depth of the culvert barrel.

4-702.4 IMPACT BASIN

Impact basins are effective energy dissipators (shown in Figure 36) but are relatively expensive facilities. Their design should be made in consulation with the D-5 Hydraulic Section.

4-702.5 STILLING BASINS

Stilling basins are hydraulically similar to sills. However, they are more expensive in construction and could present serious siltation problems. A chief advantage in stilling basins is the lack of a waterfall effect. (See Figure 37). Design of stilling basins should be made in consultation with the D-5 Hydraulic Section.

4-702.6 RADIAL ENERGY DISSIPATORS

Radial energy dissipators are depicted in Figure 38. This facility is quite effective but obviously more expensive to construct. It functions on the principle of a circular hydraulic jump. Curves and procedures for its design are available on request from the D-5 Hydraulic Section since this work is presently being researched.

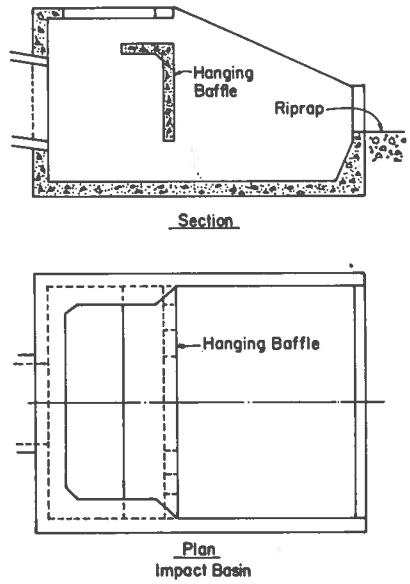


Figure 36

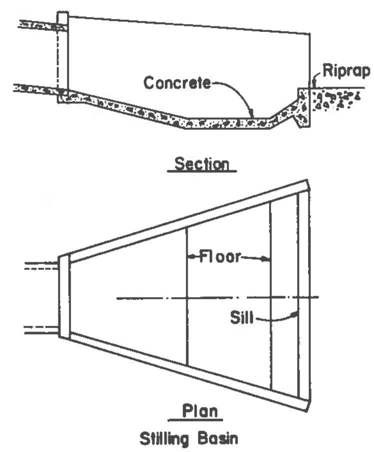


Figure 37

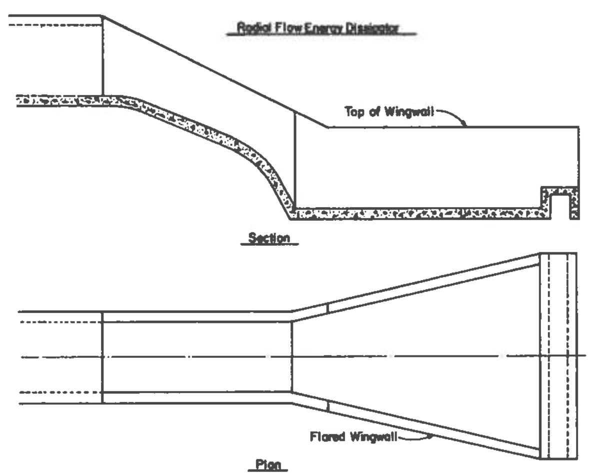


Figure 38

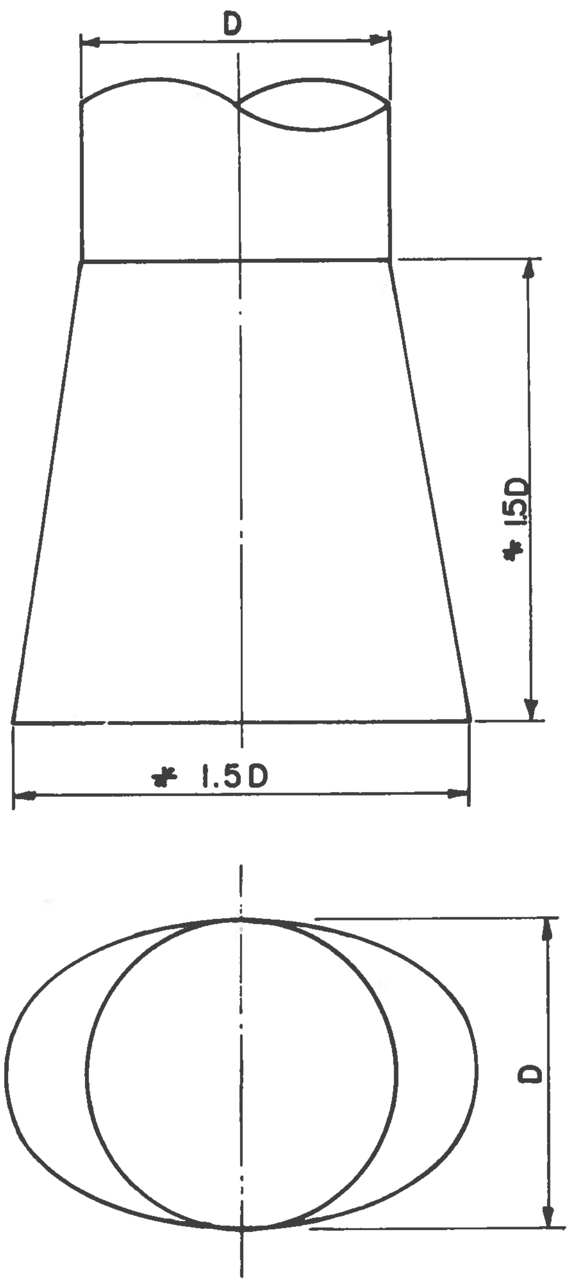


Figure-39

D- pipe diameter

Inside dimensions shown.

Inlet material has no bearing on the hydraulic requirements. These requirements are satisfied by the indicated dimensions.

Face shape should be elliptical.

* No variation is permitted in the indicated dimensions.

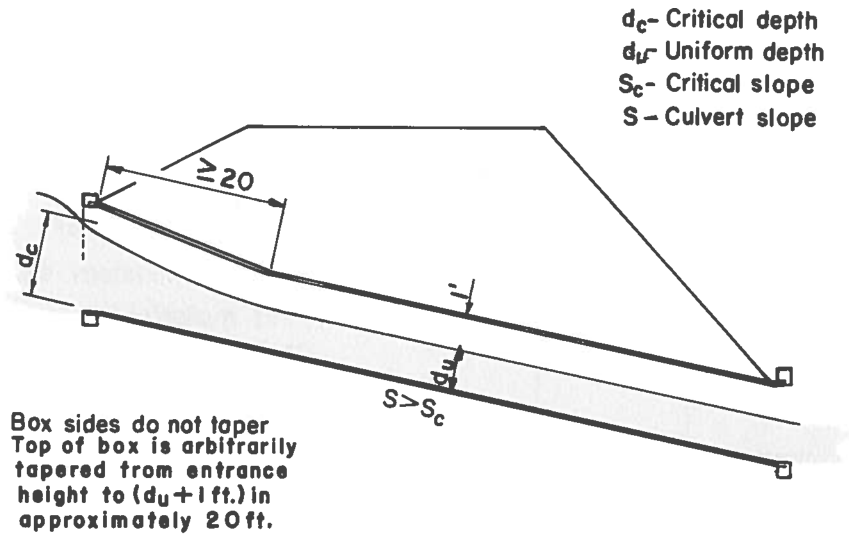


Figure 40a

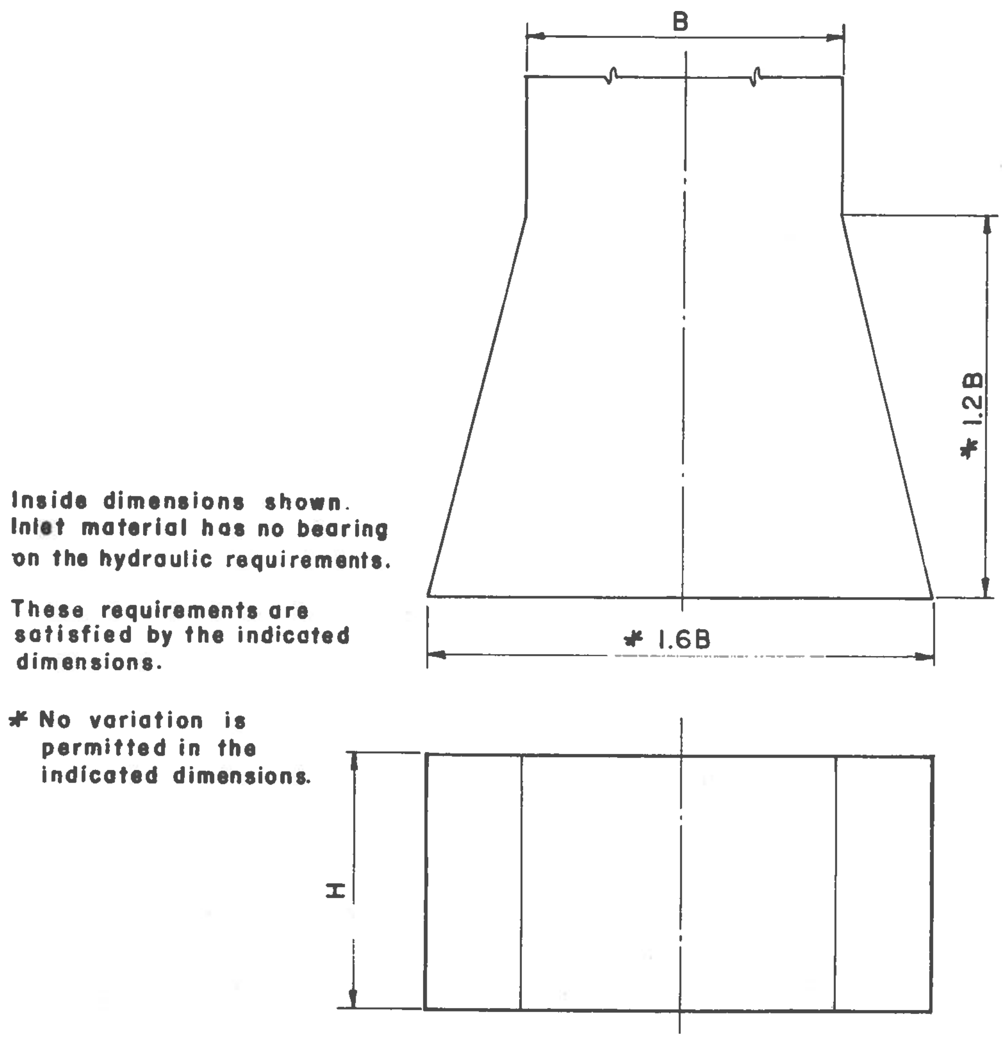
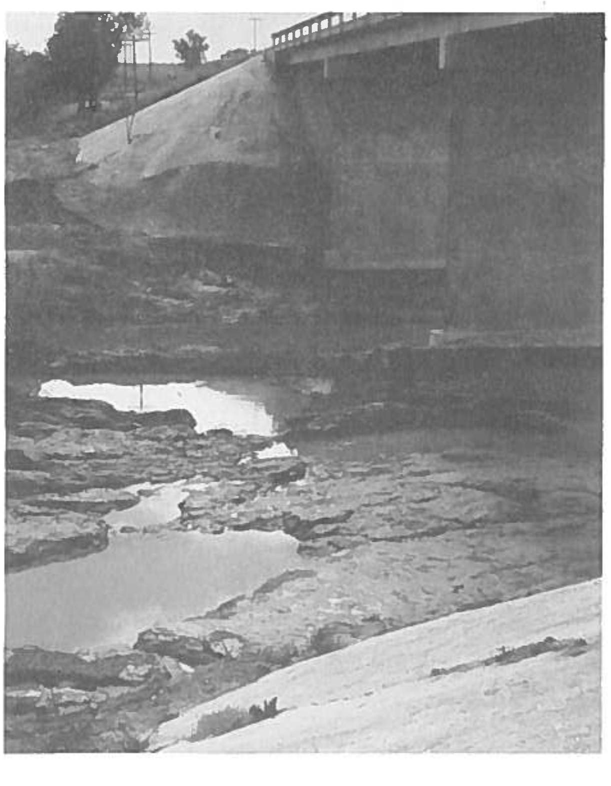
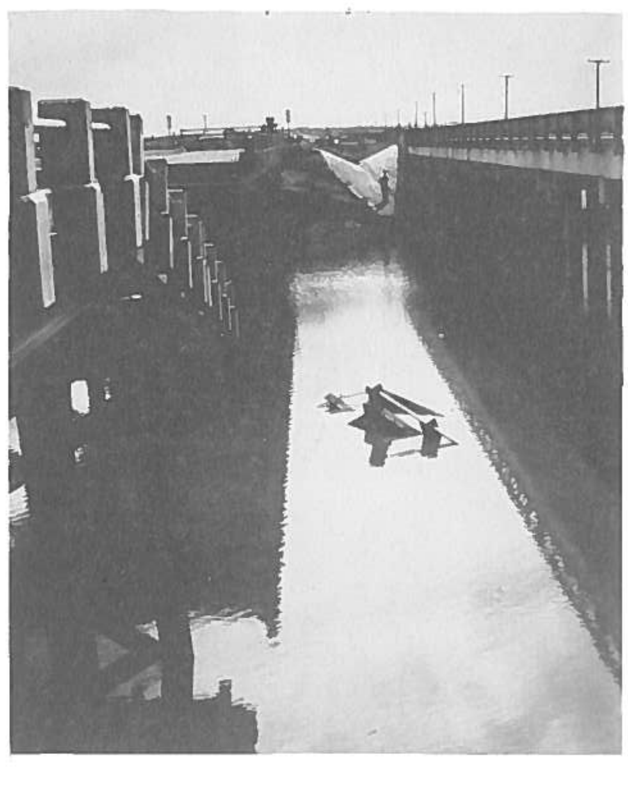


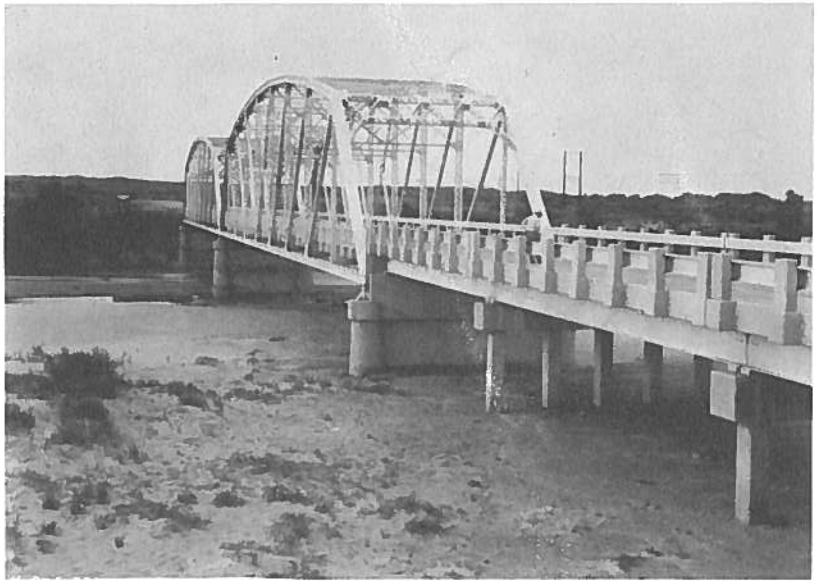
Figure- 40b

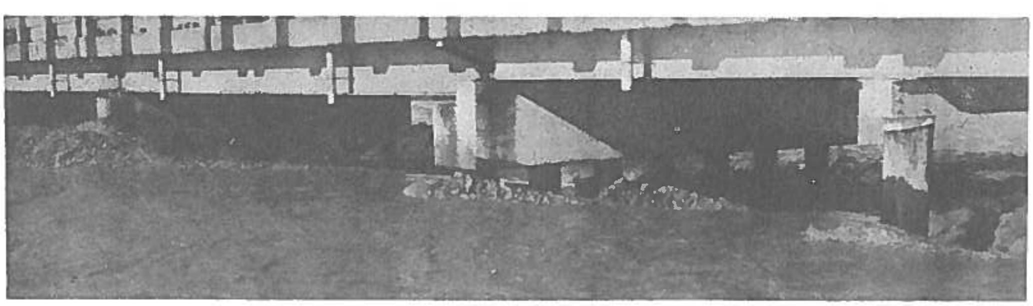
CHAPTER V

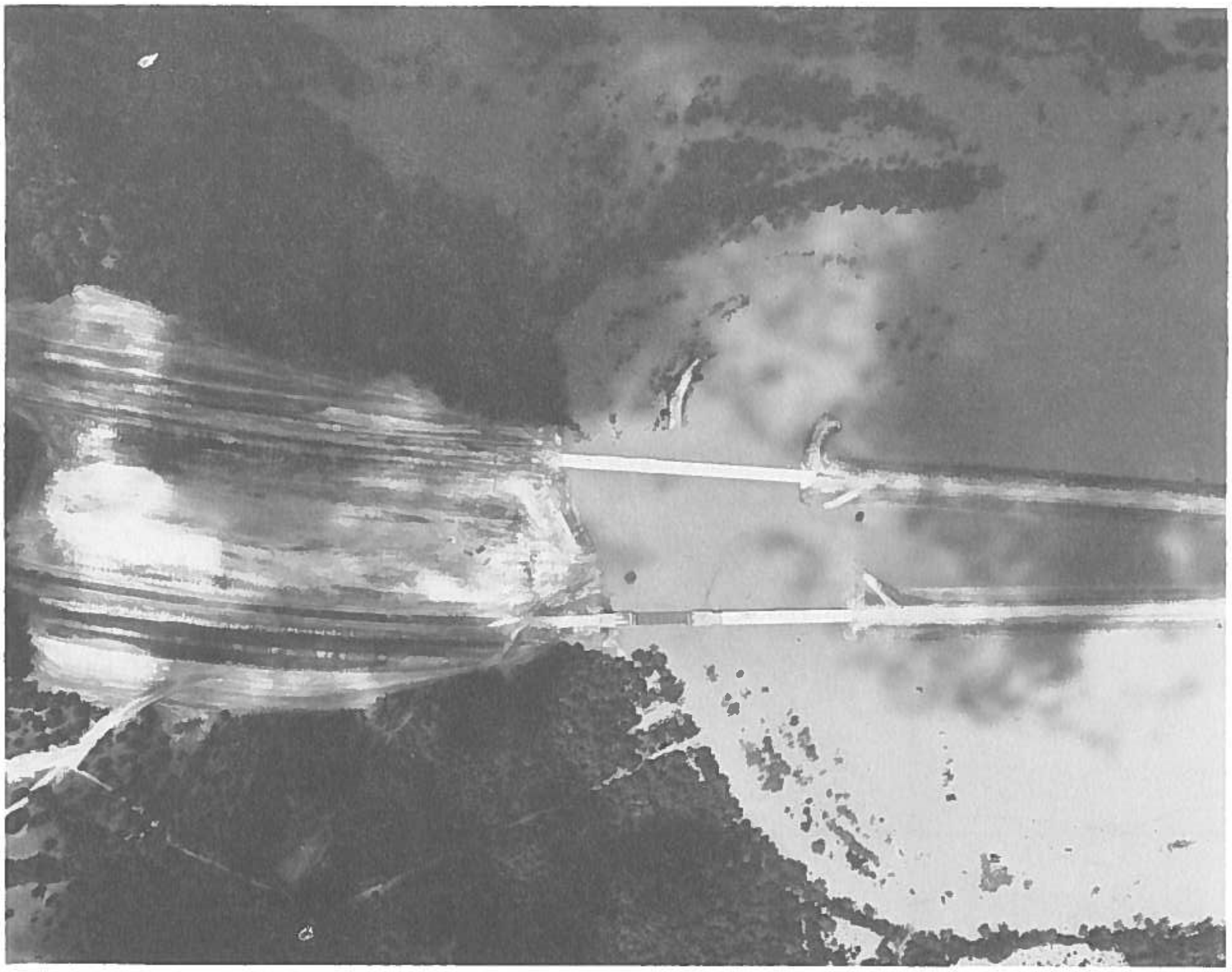
BRIDGES

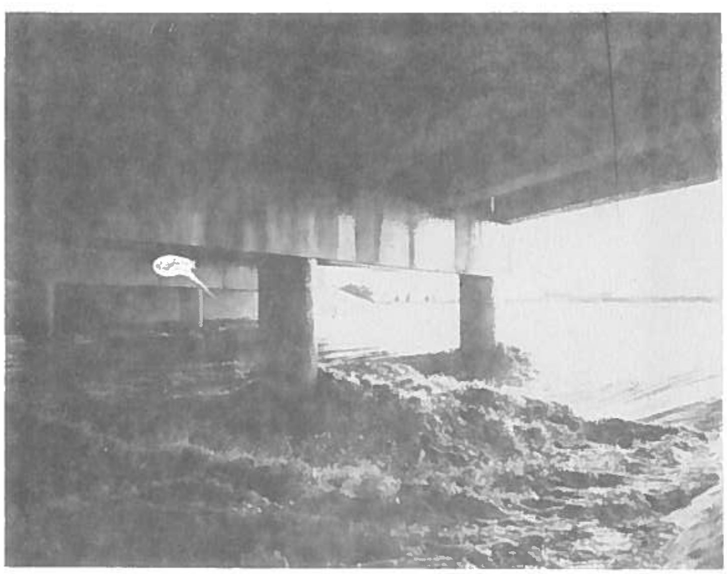












BRIDGES

5-100. GENERAL

Once a design discharge and a design highwater elevation and its location in the stream reach are established (Chapters II and III respectively), proceed to the design of the bridge opening (or openings in the case of wide flood plains requiring relief structures).

It should be noted that columns, piers, etc., have not been considered in any of the following discussion. Usually, they may be neglected. However, their effect in reducing the waterway opening should not be neglected if they constitute a substantial cross-sectional area themselves. This is particularly true in the case of skewed crossings with normal bents.

Throughout the discussions of bridge design, it is assumed that normal cross sections and lengths are used; that is, cross sections and lengths which are at 90 degrees to the direction of stream flow at flood stage. If the crossing is skewed to the stream flow at flood stage, all cross sections and lengths should be normalized before proceeding with bridge length design. If the skew is severe and the flood plain is wide, elevations in the normalized section may need to be adjusted to offset the effects of elevation changes in the point displacement between the skewed section and the normalized section.

Often structural and other considerations will cause a bridge opening to be larger than the bridge opening that would be required by hydraulic design. For instance a header might be placed in a certain location due to soil instability, or a high bank, or bridge costs might be cheaper than embankment costs, or a grade line might dictate an excessive freeboard allowance. These and others are valid considerations that affect bridge waterway openings; however, hydraulic computations are necessary in order to predict the operation of the waterway opening at flood stages. Hydraulic design is not to be neglected, and the reasons should be noted for any opening in excess of the opening determined by hydraulic design.

5-101. TERMINOLOGY

There follows a limited explanation of certain terms used in this chapter.

a. Design Highwater Elevation - A calculated water surface elevation in the natural channel for the design discharge. This is the elevation that is the basis for all bridge length calçulations.

b. Backwater = BW = Depth of water that ponds above the design highwater elevation in order to force the design discharge through a restricted opening. This term is used primarily for bridge span type structures. (See Figure 3)

c. Conveyance = K = Carrying capacity of a specific cross section as defined by $K = \frac{1.486}{n} AR^{2/3}$ Natural stream conveyance is the sum of all subsection conveyances throughout a cross section of the natural stream for a specific highwater elevation. Structure conveyance is the sum of the conveyances for each individual opening for a specific highwater elevation. $K_T = K_1 + K_2$ etc. The two conveyances (natural K and structure K) must be designated to keep them separate, because they are not to be related to each other.

d. Freeboard - That clearance between water surface elevation and low superstructure. The usual minimum freeboard is 2 feet above observed highwater or design highwater, whichever is greater.

5-200. FLOW THROUGH BRIDGES

Compared to culvert flows, flow through bridge openings requires that more emphasis be put on viewing the entire pattern of flow in the flood plain, especially the flow approaching the bridge (or bridges). This is necessary because the relationship of flood plain width to opening width is more complex than for the usual culvert opening. (See Figure 1)

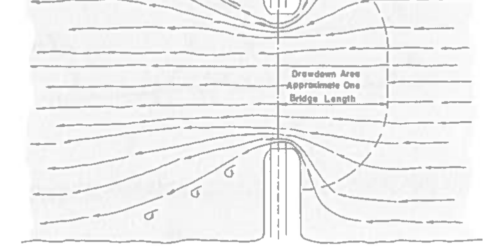


Figure 1

When flood flow encounters a restriction in the natural stream, certain natural adjustments take place in the vicinity of the restriction. That portion of the flow not directly approaching the bridge opening has no convenient way to continue downstream. The flow, therefore, will pond until sufficient head is built up to force it to turn toward the bridge, and generally move parallel to the highway embankment. As the flow moves toward the bridge opening the velocity increases above the natural flood plain velocity. This increased velocity can cause scour, sometimes severe, along the highway embankment and particularly at the bridge header. At the header the intersecting velocity vectors cause high turbulence and eddies which often result in the failure of an interior bent near this turbulence. See Figure 2. If there is only a single structure the flow will find its way to the single structure. If two or more structures are available, the flow, after accumulating a head, will divide and flow to the structure offering the least resistance. This point of division is called a flow divide.

In usual practice it is recommended that the flood discharge be forced to flow parallel to the highway embankment for no more than 600 to 800 feet. If flow distances along the embankment are greater than recommended, then a relief structure to provide additional opening should be investigated, or a spur dike is recommended to control the turbulence at the header. Also, natural vegetation remaining between the toe of slope and the right-of-way line is advantageous in controlling flow along the embankment.

Multiple structures and spur dikes are discussed elsewhere in the chapter.

The backwater created by the restriction in the natural stream is depicted in Figure 3, and is discussed elsewhere in the chapter.

The waterway opening defined by design highwater, the left and right header slopes, and the natural ground

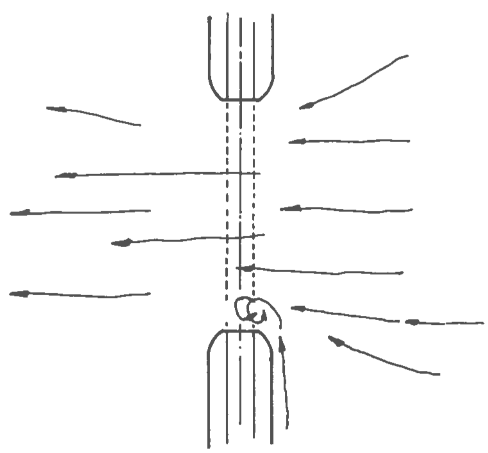


Figure 2

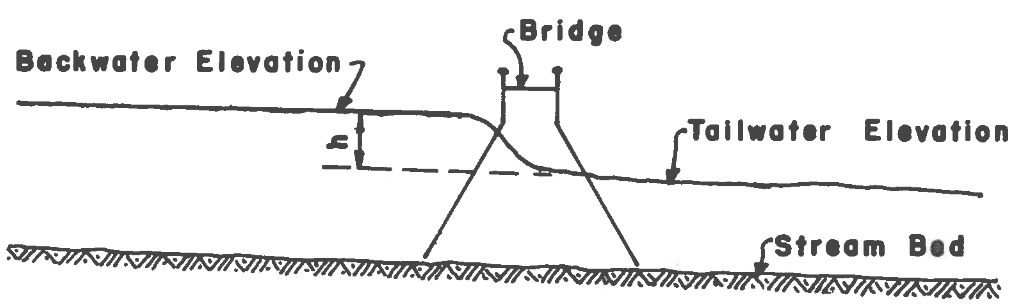


Figure 3

profile (or proposed through-bridge channel section) should be such as to cause an average through-bridge velocity of from 4 to 6 feet per second. This can be accomplished by simply moving the header slopes nearer together or further apart as necessary. Six ft./sec. is arbitrary and the maximum allowable average through-bridge velocity may vary across the State. This usually involves an engineering judgement.

The average through-bridge velocity used throughout this chapter is calculated by the formula

$$V = Q/A$$

where

V = Average velocity in feet per second.

A = The normal cross-sectional area of the water, in square feet.

Q = Design discharge in cubic feet per second

5-300. SINGLE STRUCTURE

Single structures refers to the crossing of a flood plain which requires only one opening in the highway embankment.

The following is the recommended procedure for establishing a single structure length and elevation of low superstructure after obtaining an accurate cross section and determining the design highwater elevation (Chapters II and III). Also necessary is a compilation of past flood history, existing structures, etc. (Chapter I).

Step 1. Assume average trial through-bridge velocity (V_t) that is less than the maximum allowable velocity and more than 4 fps .

Step 2. Find water cross-sectional area (A_t) required for trial velocity.

$$A_t = Q/V_t$$

Step 3. Estimate average depth of water (D_t) in cross sections.

Step 4. Find approximate trial length (L_t) of waterway opening.

$$L_t = A_t/D_t$$

Step 5. Position headers in stream cross section so that they average approximately L_t feet apart.

Step 6. Find exact waterway area (A) below design highwater within structure limits.

Step 7. Find average through-bridge velocity (V_{struct}) for the actual waterway area

$$V_{struct} = Q/A\ .$$

Step 8. If V_{struct} is greater than 4 fps and less than the allowable maximum velocity, the bridge length sizing is complete. This length can usually be adjusted slightly to fit span length requirements.

If V_{struct} is not greater than 4 fps or less than the allowable maximum velocity, the length should be adjusted as necessary and return to Step 5. This routine should be repeated as often as necessary until average through-bridge velocity is within the prescribed velocity range.

Step 9. The low steel or concrete should be placed at an elevation that provides at least 2 feet of freeboard over the design highwater. Often, some observed highwater exceeds the design highwater. In this case, it is usual practice to clear the higher of the highwaters. In any event, the design highwater should be used in the bridge length determination.

Step 10. The backwater caused by the constriction of the bridge opening should be approximated by the following formula.

$$h = \frac{V_{struct}^2 - V_2^2}{2g}$$

where

h = The increase in depth over design highwater

V_{struct} = Average through-bridge velocity

V_2 = Average unrestricted natural stream velocity

g = Acceleration of gravity = 32.2 ft./sec.^{2}

5-400. MULTIPLE STRUCTURES

5-401. GENERAL

Due to a relatively wide flood plain or multiple discharge concentrations (as opposed to a single concentration), it may be necessary to design multiple openings. This is usually referred to as a main channel bridge with relief openings. This type of crossing is simply to provide openings at or near the flow concentrations so as to reduce along-embankment flow and reduce backwater effects.

Multiple structures should be designed so that their relative carrying capacities (or structure conveyances) match the predicted relative discharges approaching them. This is necessary because each bridge must have the capacity to carry its share of the discharge. Otherwise, those bridges whose capacity is less than necessary will be overloaded with resultant high velocities and, in effect, will cause a reapportionment of the approach discharges. This, in turn will cause high backwaters and high along-embankment velocities. In addition to making this balance in proportion, the designer must satisfy average through-bridge velocity requirements.

5-402. FLOW DISTRIBUTION

After determination of the design highwater at a multiple structure location, it is still necessary to determine just how the flow divides itself across the flood plain at flood stage. The flow divides, in the case of multiple structues, determine the portion of the total flood discharge that will be carried by each structure as previously stated in Section 5-200., Flow Through Bridges.

One of the best methods for accomplishing this is by actual observation of the flow at design discharge and highwater at the proposed site. However, it is rare that the engineer is able to make such an observation when the proper set of circumstances occur. Therefore, the following method has been devised for determining flow distribution and establishing flow divides.

Inspection of incremental discharges or conveyances across a flood plain cross section usually reveals where the relatively heavier concentrations of flow are located. By determining these heavier concentrations of flow, the engineer usually can find the proper locations for each of the bridges. The most straight forward method of spotting these heavy concentrations of flow is by use of a cumulative conveyance (or discharge) curve. This curve is constructed as follows:

Step 1. Apply the design highwater to a natural stream cross section in the immediate vicinity of the proposed design section. The section chosen for this highwater application should be a typical section which will definitely control the flow distribution in the reach of the stream wherein the structure is located.

Step 2. Calculate cumulative natural conveyances for each subarea across the section from left bank to right bank (Figure 4). (Discharges may be calculated, but this only requires multiplication of conveyances by the square root of the stream slope. This is an extra step and nothing is actually gained by it since the final shape of the curves would be the same.)

Step 3. These cumulative natural conveyance values are plotted versus the cross section stationing creating a curve as shown in Figure 4 , the value of the last plot is equal to the total conveyance of the stream section.

Step 4. By inspection of this curve, it is seen that, in the vicinity of points 1, 2 and 3, the slope of the curve is relatively steep. This is caused by a rapid increase in natural conveyance with respect to distance across the section. These points define the approximate best locations for bridges. The points on the curve where the curve has the flattest slopes define the approximate locations of flow divides.

That portion of the design discharge carried between the flow divides should be determined by direct calculation or by proportion of relative natural conveyances. (Figure 4) At this point the relative discharges have been determined.

5-403. DESIGN OF MULTIPLE STRUCTURES

The design procedure to be followed in establishing the size of multiple waterways is outlined below. Q_D is the design discharge and subscripts 1, 2, and 3 refer to left flood plain, main channel, and right flood plain respectively. This example makes use of 3 bridges. However, there may be more than 3 bridges or only 2 bridges. The procedure is the same in any case.

Step 1. Using 4 fps as minimum allowable velocity, assign a maximum allowable velocity and determine the approximate average velocity (V_D) of all 3 openings

$$V_D = \frac{V_{max} + V_{min}}{2}$$

Step 2. Determine approximate total waterway opening A_D

$$A_D = Q_D/V_D$$

Step 3. Determine approximate waterway opening (A_1) for bridge 1, bridge 2, and bridge 3.

$$A_1 = \frac{Q_1}{Q_D} \text{ X } A_D$$

$$A_2 = \frac{Q_2}{Q_D} \text{ X } A_D$$

$$A_3 = \frac{Q_3}{Q_D} \text{ X } A_D$$

Step 4. With A_1, A_2, and A_3, and assuming average water depths (D_1, D_2, and D_3),

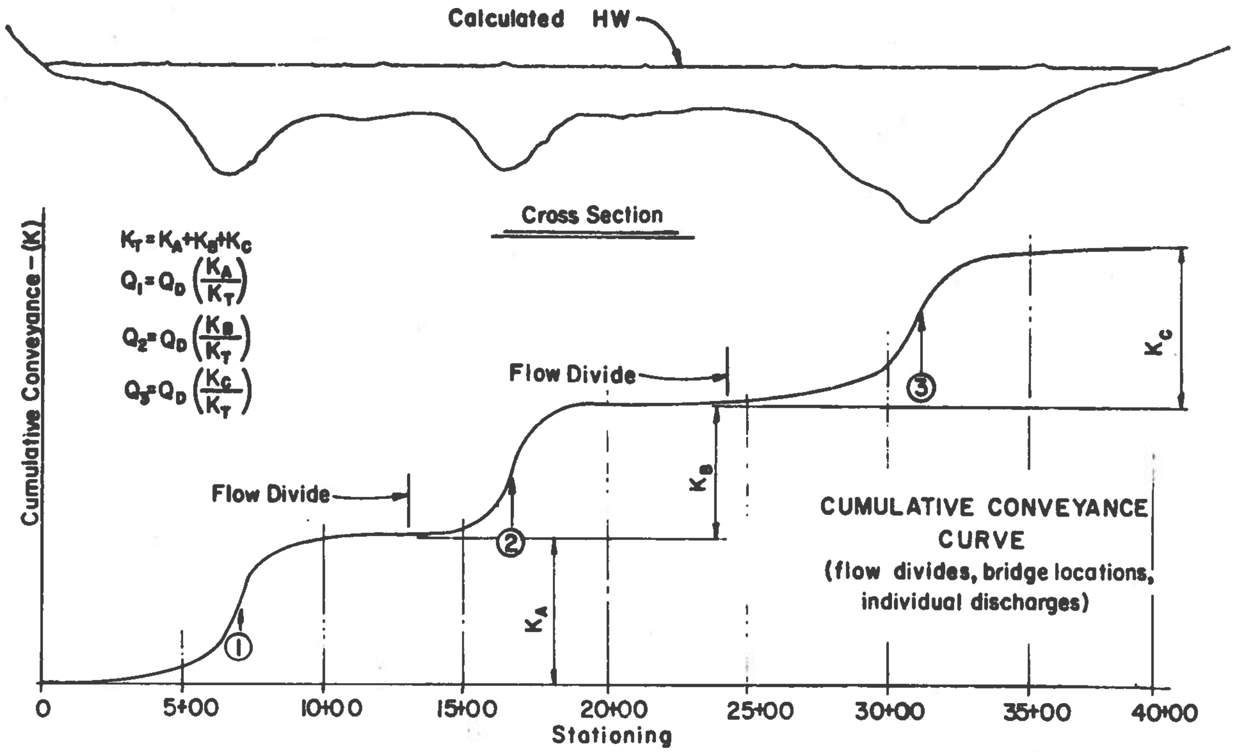


Figure 4

determine approximate structure lengths (L_1, L_2, and L_3).

$$L_1 = \frac{A_1}{D_1}, \text{etc.}$$

Step 5. Determine structure conveyances K_1, K_2, and K_3

$$K = \frac{1.486}{n} AR^{2/3}$$

Since 1.486 is always constant,

$$K = \frac{AR^{2/3}}{n}.$$

In the case where under-bridge terrain is expected to be similar for each bridge, "n" is constant and

$$K = AR^{2/3}.$$

Then $K_T = K_1 + K_2 + K_3$. If the values K_1/K_T, K_2/K_T, and K_3/K_T are approximately equal to Q_1/Q_D, Q_2/Q_D, and Q_3/Q_D, respectively, the structure conveyances are said to be balanced. Otherwise, adjustments in individual bridge lengths should be made in order to effect this balance of proportions.

Step 6. Determine velocity requirements that must be met in addition to the proportion balancing of conveyances. With the trial waterway openings, the velocity requirements should be checked. Velocity through each structure should approximately be not less than 4 feet per second or greater than the allowable maximum velocity set in Step 1. $V_{struct} = Q/A$. Note that a change in one or more bridge lengths affects the total conveyance. Therefore, any change made should be accompanied by recalculation of relative structure conveyances.

Step 7. Determine backwater head which can be approximated by the formula

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

where

V_1 = The average through-bridge velocity for all openings

V_2 = Approach velocity in natural stream

g = 32.2 feet per second per second

5-500. SPUR DIKES

Spur dikes are beneficial at sites where the distance of the flow travel along the embankment is is excess of 600 feet. Spur dikes are also advantageous for improving the characteristics of flow through a single structure.

Spur dikes have the advantage of initial economy and the fact that they are convenient and inexpensive to maintain. They function by directing along-embankment flow out away from the bridge opening. This causes any parallel-embankment velocity to be removed from the embankment itself and serves to greatly reduce the under-bridge turbulence usually caused by intersecting flow vectors. If scour is still a problem, the end of the spur dike is eroded and not the bridge header. Figure 5 shows the recommended geometry of spur dikes and the affected stream flow. The top of spur dike elevation should be above design highwater elevation a minimum of 2 feet. Also, it is recommended that natural vegetation be left around the end of the spur dike.

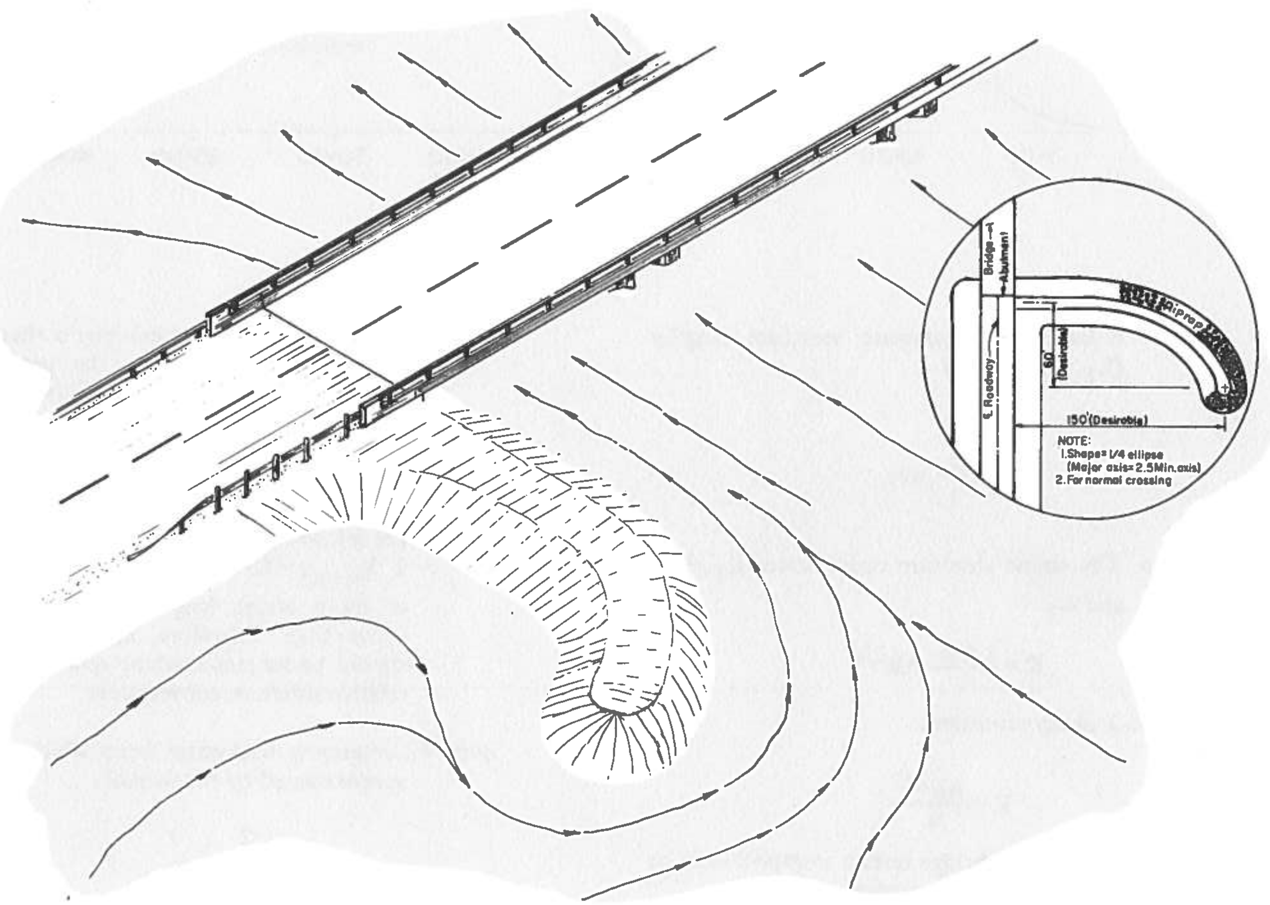
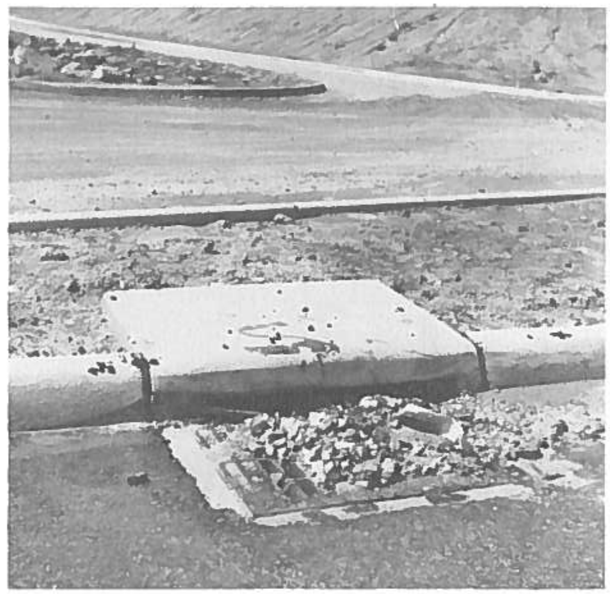
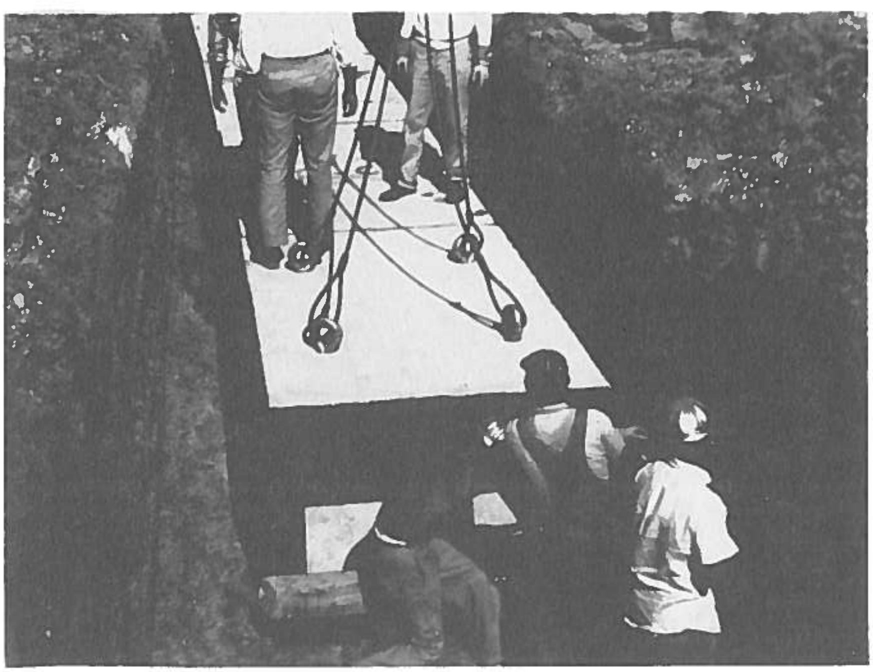


Figure 5

CHAPTER VI

STORM SEWERS





STORM SEWERS

6-100. GENERAL

Provision for the proper drainage of a roadway traversing a built-up municipality generally is a more difficult and more important problem of design than for roadways traversing sparcely settled rural areas. This is attributable to the wide roadway sections, flat grades, both in longitudinal and transverse directions, shallow water courses, absence of side ditches and concentration of all flow; to the large property damages which may occur from ponding of water or from flow of water through built-up areas, and to the fact that the roadway section must act not only as a medium to carry traffic, but also as a channel to carry the water to some disposal point. Unless proper precautions are taken, this flow of water along the roadway will interfere with or possibly halt the passage of highway traffic. These conditions call for the application of sound engineering principles as well as the utilization of all available data in arriving at a satisfactory solution.

The primary problem of municipal drainage design is to limit the amount of water flowing along the gutters or ponding at the sags to quantities which will not interfere with the passage of traffic. This is accomplished by placing inlets at such points and at such intervals to prevent very large accumulations of drainage water.

The most serious effects of an inadequate drainage system are: damage to surrounding property resulting from water overflowing the roadway curbs and entering such property, hazard and delay to traffic caused from excessive ponding in sags or excessive flow along roadway grades, with subsequent weakening of base and subgrade due to saturation from ponding of long duration.

6-200. DETERMINATION OF RUN-OFF

The first step to be considered in the design of a storm sewer system is the determination of the run-off. The Rational Method, as described in Chapter II is the method that applies to the vast majority of run-offs that are to be handled by storm sewers. Chapter II also covers the applicability of other methods when warranted.

In the Rational Method formula the time of concentration is the time required for water to flow from the most distant point of the area drained to the point of the sewer system under consideration, and is generally made up of two components: the time required for water to flow from the most distant point of the drainage area to the inlet, which is known as the inlet time, and the time required for the water to flow through the sewer from that inlet to the point of the sewer line under consideration. In other words the time of concentration for any point on a sewer line is the inlet time for the inlet at the upper end of the line plus the time of flow through the sewer from the upper end of the sewer to the point in question, unless the time for another branch or inlet at that point is greater. In municipal areas the time of concentration is seldom less than five minutes or more than twenty minutes. A minimum time of concentration of ten minutes is recommended for general use.

6-300. PONDING ON ROADWAY

As a general rule inlets should be placed at all low points in the roadway surface and at suitable intervals along long slopes as necessary to prevent excessive ponding on any section of the roadway.

The flow of water in the gutter should be restricted to a depth, and corresponding width, which will not cause the water to spread out over the travelled portion of the roadway in such amount and depth as to obstruct or cause a definite hazard to traffic. The depth of flow naturally depends upon the quantity of water involved, the gutter gradient, the coefficient of roughness or frictional coefficient of the gutter and paving material, the cross slope of the roadway, and inlet spacing.

The following limiting widths are recommended for use:

For Interstate and Controlled Access Highways: Limit ponding to one-half the width of the outer lane.

For Major Highways: Limit ponding to the width of the outer lane.

For Minor Highways: Limit ponding to a width and depth which will allow passage of one lane of traffic with safety.

6-400. INLETS AND MANHOLES

6-401. GENERAL

Either curb opening inlets, grate inlets, or a combination of curb opening and grate inlets may be used for intercepting run-off. Curb opening inlets are to be preferred because of their self-cleansing ability as grate inlets clog easily. In some instances however, the use of grates will be found necessary either with or without curb openings in combination. Where grates are used, the design and placing of the grates should be such that the grate bars will be parallel to the direction of flow of the water rather than perpendicular to the flow. Experiments have shown that this minimizes the clogging by small debris and increases the capacity of the grate. Sketches of inlets of the various types are included in Figures 9 and 10.

6-402. INLETS ON GRADES

The amount of water which will be intercepted by a curb inlet of given length, given depression and on a given gradient may be determined directly from the graphs included in this chapter. The illustrative problem explains the use of these graphs. It should be noted that it is not always necessary or desirable to intercept all the water in the gutter at any given point. It will often be found satisfactory and economical to allow a portion of the water to flow past an inlet to succeeding inlets. This quantity of water which passes an inlet is called "Carry-over" and the proper handling thereof is covered in the illustrative problem included herein.

6-403. INLETS IN SAGS

Under all ordinary conditions the flow of water through a curb opening inlet located at a sag or low point in the grade may be computed by the weir formula:

$$Q = 3.087\ Lh^{3/2}$$

where

Q = the discharge in cubic feet per second

L = the length of opening in feet, and

h = the head or depth of water at the opening in feet

Where the depth of water is such that the curb inlet is completely submerged the proper orifice formula should be used in computing the discharge rather than the weir formula. This is a very rare condition, as the proportioning of inlets should be such as to preclude ponding in sufficient depth to submerge the inlet.

6-404. GRATE INLETS

The flow of water through grate openings may be treated in the same manner as flow of water through rectangular orifices. The formula in most general use for flow through orifices is stated as follows:

$$Q = CA\sqrt{2gh}$$

where

Q = the discharge in cubic feet per second

C = the coefficient of discharge (approximately 0.7)

A = the area of orifice (the net area of the openings in the grate) in square feet

g = acceleration due to gravity (32.2 feet per second per second)

h = head on grate in feet

This formula gives the theoretical capacity of the grate inlet. Since grate inlets are subject to considerable clogging it is recommended that for practical purposes the capacity of the grate inlet be taken as 1/2 of the value given by this formula, or conversely that the net area of the grate be twice as large as the theoretical area required when calculated by the above formula.

6-405. COMBINATION INLETS

Combinations of curb slots and grate inlets may be used to advantage under certain special conditions. Wherever the curb opening type of inlet can be used it will generally prove to be more economical and more desirable than the grate type of inlet, and the use of grates should ordinarily be confined to those instances where it is impracticable to provide any curb opening at all or where the length of opening which can be provided is not sufficient to intercept all the water which must be cared for. The theory commonly advanced by proponents of combination curb slot-grate inlets is that floating debris will be carried on the surface of the water through the curb slot leaving the grate unclogged and hence operating at maximum efficiency. Investigations conducted by various hydraulic research laboratories however, have not revealed any great gain resulting from such combinations. Authentic data on the true capacities of such combinations are insufficient to allow the establishment of any very accurate factors for determining the true capacity of a combination inlet. For design purposes, however, it is believed reasonable to assume that the capacity of the combination inlet will run about 50% of the sum of the individual capacities of the grate and the curb slot, computed in the manner described in the preceding paragraphs. In other words, it is recommended that the capacity of the curb slot inlet and the capacity of the grate inlet (without reduction) be computed separately, added together, and the working capacity of the combination be taken as 50% of this figure.

6-406. MANHOLES

Manholes or combination manholes and inlets should be placed wherever necessary for clean-out and inspection purposes. It is good engineering practice to place manholes at changes in pipe gradients, changes in pipe sizes, changes in direction, junctions of pipe runs, and at intervals of from 300 to 500 feet in long runs where the size or direction is not changed. The invert of the manhole section should be rounded to match the inverts of the pipes entering the manhole in order to reduce eddying and resultant head losses. At junctions of sewer lines, right angle intersections should be avoided if possible and the two lines should be brought together at an acute angle to minimize head losses. (See Figure 11 for standard manhole details.)

6-407. DESIGNING SEWER RUNS

After the locations of inlets, sewer runs, and outfalls with tailwaters have been determined and the inlets proportioned, the next step will be the computation of the quantity of water to be carried by each pipe run and the determination of the size and gradient of pipe required to care for this water. It should be borne in mind that the quantity of water to be carried by any particular section of pipe is not necessarily the sum of the inlet design quantities of all inlets above that section of pipe, but as a general rule is somewhat less in amount than this total. It is well to restate that the time of concentration enters into the picture and as the time of concentration grows larger the proper rainfall intensity to be used in the design grows smaller. In determining the quantity of flow in the design of any particular run of pipe, the time required for the water to flow from the most remote point on the drainage area should be computed and the corresponding value of rainfall intensity derived. The quantity is then calculated from the Rational Formula

$$Q = CIA$$

For all ordinary conditions, runs should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. Manning's formula is recommended for use. (Figures 10, and 14, Chapter IV.) If a pressure run is necessary (all sides wetted, with the hydraulic gradient above the top of the pipe or box), then Figure 5 in this chapter is applicable for boxes, and Figure 14, Chapter IV is applicable for pipe (use full flow point on d/D ratio line when solving for full pipe flow friction slope).

6-407.1 HYDRAULIC GRADIENT

The hydraulic gradient is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a sewer run. The difference in elevation for the water surfaces in the successive tubes represents the friction loss for that length of sewer, and the slope of the line between water surfaces is the friction slope. Therefore, if a sewer run were placed on a calculated friction slope corresponding to a certain quantity of water, cross-section, and roughness factor, the surface of flow (hydraulic gradient) would be parallel to the top of the conduit, and the sewer run would not be under pressure. This is desirable as previously stated. If there is reason to place the sewer run on a slope less than friction slope, then the hydraulic gradient would be steeper than the slope of the sewer run. Depending on the elevation of the hydraulic gradient at the downstream end of the run in question, it is possible to have the hydraulic gradient go above the top of the conduit which would mean the sewer is under pressure until, at some point upstream, the hydraulic gradient is once again at or below the top of the conduit.

It will not be necessary to compute the hydraulic grade line of a sewer run, where the slope and the run sizes are chosen so that the slope is equal to or greater than friction slope and the top surfaces of successive runs are lined up at changes in size rather than the bottom surfaces, and the surface of the water at the point of discharge does not lie 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. There will be small head losses at inlets, manholes, etc., but if these are properly designed these losses may be neglected.

Whenever all of these conditions do not exist however, and particularly in those instances where the inverts in the pipes are placed on the same grade at changes in pipe size (which forces the smaller pipe to discharge against head) or when it is desired to check the sewer system against a larger flood than that used in the proportioning of the pipes, it will be necessary to compute the hydraulic grade line of the entire sewer system. This is done by starting with the tailwater elevation at the point where the sewer is finally discharged and working back up along the entire length of the sewer, computing the friction loss for each run, and plotting the elevation of the total head thus computed at each manhole and inlet.

If the hydraulic grade line as thus plotted does not rise above the top of any manhole or above the lip of any inlet the sewer system is considered satisfactory. Wherever it does rise above these points, however, blow-outs through inlet slots and manhole covers will occur and pipe sizes or gradients should be increased as necessary to eliminate such blow-outs.

Note that any hydraulic gradient must have an original base elevation no lower than the outlet tailwater elevation. Therefore, the backwater effects of a significant tailwater elevation should be checked very carefully.

6-407.2 GENERAL RULES FOR SIZING SEWER RUNS

a. Do not use pipe sizes less than 18" diameter for main sewers or for long laterals and not less than 12" diameter for short laterals.

b. Place all pipes or boxes on such slope that the velocity of flow when full will be not less than 2' per second. Where slopes are comparatively flat it is desirable that the sewer sections and slopes be so designed that the velocity of flow will increase progressively, or at least will not appreciably decrease, in passing from the first inlet to the outlet of the sewer so that solids washed into the sewer and transported by the flowing stream will be carried on through and out of the sewer and will not be dropped at some point due to a sharp decrease in velocity.

c. Do not use non-standard sizes of pipe.

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

e. At changes in size of pipe or box always place the soffits or top inside surfaces of the two pipes at the same level rather than placing the flow lines at the same level. Where flow lines are placed at the same level the smaller pipe must discharge against head and it will be necessary to plot hydraulic grade lines in order to determine the true discharge. Naturally this rule cannot be followed in every instance, but it should be adhered to wherever it is practicable to do so.

f. Laterals should be checked at the time of their design with reference to critical slope. If the slope of the lateral is greater than critical slope, (technically defined as steep slope) the unit will likely be operating under entrance control instead of the originally assumed uniform flow. If so, it may be found that the headwater caused at the upstream inlet for the lateral in question is intolerable. This may be determined by the use of the proper entrance control nomograph in Chapter IV. It may be necessary to resize the lateral to overcome any unfavorable effects of entrance control. Or, the entrance control effects may be found to be acceptable.

Usually, steep units in the trunk lines are not affected by entrance control as the velocity head loss is relatively negligible.

g. Determine a tailwater depth (TW) for the outfall channel and always calculate the hydraulic gradient when the tailwater surface at the outlet is greater than the height of the outlet pipe or boxes.

6-408. EXAMPLE PROBLEM

Given: The drainage area map (see Figure 6) shows the layout of the highway and cross streets to be drained, the typical cross sections of the highway and of the cross streets, as well as the type and area of the surfaces to be drained.

Design frequency:
5 years

Minimum time of concentration:
10 minutes

Minimum inlet length:
5 feet

Gutter depression on Monroe Avenue:
3 inches

On cross streets:
3 inches to 5 inches

Permissible width of ponding:
10 feet on Monroe Avenue in Business Areas

15 feet on Monroe Avenue in Residential Areas

full roadway width on cross streets

Required: Design a storm sewer system that will adequately dispose of the design discharge.

Solution:

Step 1 - Run-Off Computations: The run-off computations are tabulated on Figure 6.

The first six columns of this tabulation are believed to be self-explanatory. The total CA as shown in the seventh column is computed by multiplying each incremental area by its corresponding coefficient of run-off and totaling the sum of these increment products. As an example, the total CA for Drainage Area No. 1 is computed as follows:

Type	Acreage		C		CA
Paved	0.32	x	0.9	=	0.288
Commercial	0.21	x	0.7	=	0.147
Gravel	0.27	x	0.4	=	0.108
					0.543
				use	0.54

The time of concentration is computed by dividing the distance from the most remote point of the drainage area by the assumed velocity of approach flow. Since we have already established that a minimum time of concentration of 10 minutes will be used, this value shall be inserted in the table. Whenever this time is in excess of 10 minutes, the derivation of the time should be shown in the same manner as for drainage area No. 16 in the sample tabulation.

The intensity of rainfall can either be calculated or determined from the nomograph at Figure 4, Chapter II.

The total run-off Q is determined by multiplying CA by I.

Step 2 - Inlet Computations: The inlet computations are shown in Table I. A detailed explanation of this tabulation follows:

Column 1. All inlets should be properly classified and given a designation number in the following manner:

CI-3 = Curb inlet No. 3
DI-1 = Drop inlet No. 1
CGI-1 = Curb and grate inlet No. 1

Column 2. The location of the inlets as established on plan profile sheets should be listed for cross reference and ready identification.

Columns 3 and 4. The respective drainage area numbers and corresponding discharges are taken from the run-off computation tabulation which appears on the drainage area map.

Column 5. The carry-over in this column is the quantity of water which has passed by the last preceding inlet to the inlet under consideration.

Column 6. The total run-off, Q_a, is the run-off from a specific drainage area plus the carry-over from preceding drainage areas.

Column 7. z, is the reciprocal of the cross slope, and in the example problem is determined from the cross sections in the following manner:

Slope 3/16 inch per foot: $z = \frac{12}{3/16} = 64$

Slope 3 inches in 8 feet: $z = \frac{8 \times 12}{3} = 32$

Slope 6 inches in 15 feet: $z = \frac{15 \times 12}{6} = 30$

For circular or parabolic crowns it will be sufficiently accurate to calculate the average slope for the desired width of ponding and use this value in determining z.

Column 8. The ratio $\frac{z}{n}$, is self-explanatory. In the example problem, a roughness coefficient n of .015 is used. This coefficient may vary from about .015 to about .025 depending upon the roughness of the gutter.

Column 9. The slope, s, expressed in feet per foot, is to be obtained from established grade lines as shown on the plan-profile sheets.

Column 10. The value of y is the depth of flow in feet in the gutter for a certain discharge, $\frac{z}{n}$ratio, and longitudinal slope. It is determined from Figure 1. The procedure to be used in determining the value of y is clearly explained on that chart. It will be noted that two z ratios have been indicated for several inlet computations. This is due to the fact that the pavement slope changes at a point 8 feet distant from from the curb line. See typical cross section of Monroe Avenue.

Column 11. The width of ponding must be kept within the previously determined acceptable limits. In the example problem, the maximum permissible ponded width is 10 feet for the section of Monroe Avenue within the business area and 15 feet for the section of Monroe Avenue within the residential area.

The ponded width is the product of y times z as determined in columns 10 and 7 respectively. Attention is called to inlet CI-11 wherein two values of z have been shown because of the change in transverse cross slope. The ponded width in this instance was calculated in the following manner:

> The total depth of water in gutter, y (Column 10) = 0.30 feet. In the first 8 feet of width from the face of curb, there is a rise of 3 inches. This leaves a depth of water y = 0.30-0.25 = 0.05 feet; hence the width of ponding beyond this point of change in transverse slope will be .05 x 64 = 3.2; or the total width of ponding will be 8.0' + 3.2' = 11.2'.

Column 12. The dimension "a" is the gutter depression, or the vertical distance from the normal gutter line to the throat of the inlet. Note that this distance is to be expressed in feet. The greater the dimension "a" is made the more water can be carried by a given length of inlet. On the other hand, the deeper this depression is made the more objectionable it is to traffic moving along close to the curb. Hence, in selecting the most desirable depth, a compromise between these two features must be made. As a general guide the depth of depression may vary from 0 to 1 inch where the gutter is within the theoretical traffic lane; from 1 inch to 3 inches where the gutter is outside the traffic lane or in the parking lane; and up to a maximum of about 5 inches for unimportant city streets not on the highway route. In the typical example, 3 inch depressions have been used on inlets facing the main highway and 3 inch and 4 inch depressions on inlets located on the cross streets.

Column 13. The value, q_L, is the flow in cubic feet per second which will be intercepted by an inlet one foot in length for a given depth of gutter flow, y, and an inlet depression, a. This value is determined directly from Figure 2.

Column 14. The value, L_a, is the length of inlet in feet which is necessary to intercept a given discharge Q_a. Inlet No. CI-3 will be used as an example.

The total discharge Q_a is 2.5 cfs; the interception per foot of inlet, q_L (Column 13), is 0.395 cfs; hence, the length of inlet required (L_a) is $\frac{2.5}{.395}$ = 6.3 feet. In other words, if it were necessary that all of the water be intercepted in the gutter at this inlet, an inlet not less than 6.3 feet in length would be required. In this case, however, there is no objection to a small carry-over, that is, to a portion of the water passing the inlet; hence, the standard 5 foot inlet was used.

Column 15. The dimension L is the actual length in feet of inlet which is to be provided. Under ordinary conditions it is recommended that no inlet be less than 5 feet in length and that standard inlets in multiples of 5 feet be used.

Column 16. The ratio of the length of inlet provided to the length of the inlet required is used in determining the amount of the water that the inlet will actually intercept. For example, at inlet No. CI-3, the value of this ratio

$\frac{L}{L_a}$ is $\frac{5.0}{6.3} = 0.79$.

Column 17. $\frac{a}{y}$, which is the gutter depression at inlet divided by the depth of flow in the normal gutter, is another ratio which is used in determining the percent of interception as shown in Column 18. For inlet No. CI-3, this value $\frac{a}{y}$ is $\frac{0.25}{0.23} = 1.1$.

Column 18. This column shows the percentage of run-off which is intercepted by the inlet in question. It is determined from Figure 3, using the values determined in Columns 16 and 17. For inlet No. CI-3, the $\frac{L}{L_a}$ ratio (from Column 16) is 0.79, and the $\frac{a}{y}$ ratio (from Column No. 17) is 1.1. Upon entering the chart with these values it is found that the percentage of interception, $\frac{Q}{Q_a}$, is 0.86 or 86.0 percent.

Column 19. The value Q shown in this column is the amount of water in cubic feet per second which the inlet in question actually intercepts. It is the product of Q_a from Column 6 and the percentage of interception from Column 18. For inlet No. CI-3, as an example, the value of Q equals 2.5 x 0.86 = 2.1 cfs.

Column 20. The carry-over is the amount of water which passes an inlet, and is the difference between the total run-off, Q_a (Column 6), and the intercepted flow, Q (Column 19). For inlet No. CI-3 the carry-over is $Q_a - Q = 2.5 - 2.1 = 0.4$ cfs.

The preceding method of proportioning inlets applies only to inlets on grade. For low point inlets the method previously described for "Inlets in Sags" should be applied. As an example, consider the inlet at Station 21+00 left which is to carry 11.4 cfs of water.

This is first designed as a straight curb inlet. Allowing a 10 foot width of ponding, it is found that the allowable depth of water above normal gutter grade will be .28 feet. In this example, a 3 inch depression below normal gutter grade is used. This gives a total head of water, from surface of ponded water to throat opening of inlet, of .28 feet + .25 feet = .53 feet. Reference is then made to Figure 4, "Low Point Inlets". Entering this chart with a head of 0.53 feet or 6.4 inches it is found that the inlet will discharge 12.0 cfs as compared to the calculated 11.4 cfs.

It is then assumed that the 3 inch depression at this point is objectionable and the design will be for a combined curb and grate inlet with a gutter depression of only 1-3/4 inches. As explained previously it is assumed that this combined inlet will operate at 50% of theoretical efficiency. Grates of the types shown in the recommended standards included in Figure 10 will be used. They are composed of 14 slots 1-1/4" x 15-3/8", giving a total opening per grate of 1.87 square feet.

Using the allowable ponding width of 10 feet it is found that the allowable depth of water above the normal gutter line will be 3.36 inches, which, when added to the allowable gutter depression of 1.75 inches, gives a total head of the curb opening of 5.11 inches. The corresponding head over the center of the grate will be 4.61 inches.

Three grates with a total opening of 5.61 square feet will be tried. The length of curb slot corresponding to these three grates will be 8.34 feet. Referring to Figure 4, it is found that the curb slot will carry about 0.84 cfs of water per foot length of opening. Hence, the curb slot which is 8.34 feet in length will take a total of 8.34 x 0.84 = 7.00 cfs.

To determine the theoretical discharge of the grate openings, the Orifice Formula $Q = CA\sqrt{2\,gh}$ will be used. Inserting the values explained above, $Q = 0.7 \times 5.61 \times \sqrt{2 \times 32.2 \times 0.384} = 19.5$ cfs.

The total theoretical capacity of the combined curb slot and grates then becomes 7.0 + 19.5 = 26.5 cfs. Applying the 50% efficiency factor, the working capacity of the combination is .50 x 26.5 = 13.2 cfs, as compared with 11.4 cfs of water to be carried by this inlet. Hence, the design as assumed will be adequate.

As an example of the use of grates alone, consider Drop Inlet No. 1 at Station 25+00 left. This inlet is to be designed to carry 4.4 cfs of water. $Q_a = 4.4$ cfs.

Grate Opening: 14 slots, 15-3/8" x 1-1/4"
Gross Area One Grate = 1.87 sq. ft.
Assumed Efficiency of Grate = 50%
Effective Net Area One Grate = .50 x 1.87 = 0.93 sq. ft.
Assumed Allowable Head = 4" = .33'

$Q = CA\sqrt{2\,gh}$ (Orifice Formula)

$$A = \frac{Q}{C\sqrt{2\,gh}} = \frac{4.4}{.7\sqrt{64.4 \times 0.33}} = 1.36 \text{ sq. ft.}$$

Use two grates. Effective area = 2 x 0.93 = 1.86 sq. ft.

Step 3 - Proportioning Sewer Pipes: The computations involved in the proportioning of the various runs of sewer pipe are summarized in the tabulation sheet titled "Storm Sewer Computations", (Table II). A detailed explanation of this tabulation follows:

Columns 1 through 5. Figures shown in these columns are believed to be self-explanatory.

Column 6. The length of each run as shown in this column is the length center to center of inlets or manholes. This length is used in determining the time of flow from one inlet or manhole to another. (Note that these lengths are not to be used as pay lengths of pipe since the standard specifications provide that pay lengths shall include only the actual net length of pipe and shall not include the distance across inlets or manholes where no pipe actually is placed.)

Columns 7, 8, and 9. The time of concentration is the time required for water to flow from the most remote part of the drainage area or areas involved to the upper end of the pipe run under consideration. For the first run the time of concentration is the inlet time for the first inlet. For all succeeding runs the time of concentration may be either the time as computed along the sewer line or the inlet time of the inlet at the beginning of the run under consideration, depending upon which of these two periods is the longer. Accordingly, both times are shown in the tabulation for purposes of comparison and the larger of the two is used in determining I and Q, unless this larger value is less than 10 minutes in which case the established minimum time of 10 minutes is used.

The time of concentration shown in column 7 is computed by taking the time of concentration for the preceding run and adding to it the time required for water to flow through the preceding run to the beginning of the run under consideration. At junctions of lines, the larger value of the time of concentration is used. In the tabulation, for example, lines A & B join to discharge into line C. Since the time of concentration for line A is greater than that for line B, this larger value is carried through in figuring times for line C.

Columns 10 and 11. I and Q are computed in the same manner as explained under "Inlet Computations."

Columns 12, 13, and 14. The size and gradient of pipe as shown in columns 12 and 13 must be chosen in such manner that the pipe when flowing full, but not under head, will carry an amount of water approximately equal to or greater than the computed discharge, Q. In other words, the "Capacity" shown in column 14 must be approximately equal to or greater than the value Q shown in column 11.

The capacity may be calculated by Manning's formula,

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

or the capacity can be taken directly from the appropriate uniform flow nomograph in Chapter IV. If the sewer run is to be designed to flow under pressure, use Figure 14, Chapter IV, or Figure 5, this chapter.

Wherever a pipe run is designed in such a manner that the capacity of a pipe as shown in column 14 is less than the computed discharge shown in column 11, a check of the hydraulic gradient above this run should be made to make sure that the backwater head created by such a design is not large enough to cause blowouts at inlets or manholes above the run. The same is true for pipe runs of larger diameter discharging into runs of smaller diameter or in such instances where it is impossible to line up the soffits of pipe runs at changes of pipe size.

In our example, it will be noted on the plan profile sheet, (Figure 8), that line C is to be connected to an existing 54" sewer and will theoretically discharge against a 12' head. Checking back on the pipe run from manhole 3 to manhole 2 we find that an effective slope of 0.33% will be required to pass the run-off of 40.3 cfs. Projecting this grade from elevation 84.0, the soffit of the 54" diameter pipe, we obtain a backwater elevation 85.58 feet at manhole 2. This procedure is to be repeated, until the hydraulic gradient drops below the top of a pipe run, in this instance between CI-8 and CI-7 on line A. Since the backwater elevations are well below the throats of the inlets the system as designed is satisfactory.

The above method of checking the hydraulic gradient does not take into account all items such as head losses in inlets, head losses or gains due to changes in velocity, etc., but it is considered accurate enough for all practical purposes so long as the hydraulic gradient is kept well below the throats of inlets.

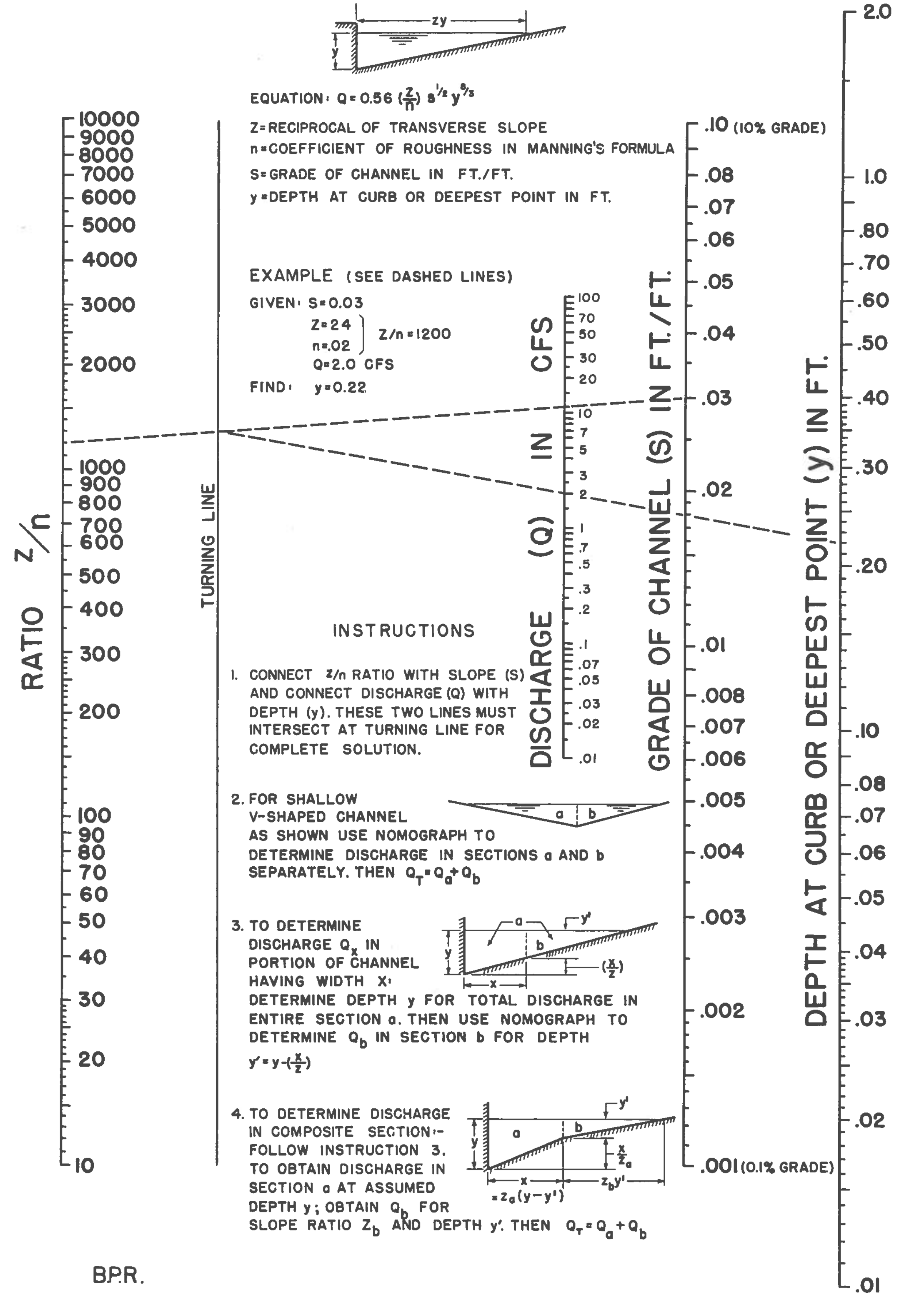
Column 15. The velocities shown in this column can be obtained from the appropriate nomograph in Chapter IV or calculated, and are used in column 7 in determining the time of flow through each run of pipe.

TABLE I

Inlet No. 1	Inlet Station 2	D.A. No 3	Q_a cfs 4	Carry-over cfs 5	Total Q_a cfs 6	z 7	z/n 8	s ft/ft. 9	y ft. 10	Ponded Width y x z ft. 11	a ft. 12	q_L cfs 13	$L_a=Q_a/q_L$ ft. 14	L ft. 15	L/L_a 16	a/y 17	Q/Q_a 18	$Q=Q_a \times Q/Q_a$ cfs 19	Carry-over cfs 20	Remarks 21
C1-1	3+80 Lt.	1	3.1	-	3.1	32	2100	.0125	0.25	8.0	0.25	.405	7.6	10	-	-	-	-	-	
C1-2	6+70 Lt.	2	3.5	-	3.5	32	2100	.0125	0.26	8.3	0.25	.415	8.5	5	0.59	0.96	0.71	2.5	1.0	
C1-3	8+31 Lt.	4	1.5	1.0	2.5	32	2100	.0125	0.23	7.4	0.25	.395	6.3	5	0.79	1.10	0.86	2.1	0.4	
C1-4	8+60 Lt.	3	2.1	0.4	2.5	30	2000	.0025	0.31	9.3	0.33	.600	4.2	5	-	-	-	-	-	
C1-5	10+75 Lt.	5	3.2	-	3.2	32	2100	.0125	0.25	8.0	0.25	.405	7.9	10	-	-	-	-	-	
C1-6	12+80 Lt.	6	3.4	-	3.4	32	2100	.0125	0.26	8.3	0.25	.415	8.2	10	-	-	-	-	-	
C1-7	13+68 Lt.	7	2.2	-	2.2	32	2100	.0125	0.22	7.0	0.25	.380	5.8	5	0.86	1.10	0.91	2.0	0.2	
C1-8	17+31 Lt.	8	3.3	0.2	3.5	32	2100	.0125	0.26	8.3	0.25	.415	8.5	5	0.59	0.96	0.71	2.5	1.0	
C1-9	17+60 Lt.	9	1.0	1.0	2.0	30	2000	.005	0.25	7.5	0.25	.405	5.0	5	-	-	-	-	-	
C1-10	18+18 Lt.	10	1.2	-	1.2	32	2100	.0125	0.17	5.4	0.25	.340	3.5	5	-	-	-	-	-	
C1-13	24+18 Lt.	15	1.3	0.5	1.8	30	2000	.001	0.33	9.9	0.33	.575	3.1	5	-	-	-	-	-	Carry-Over from C1-15 = 0.5 c.f.s.
C1-12	23+75 Lt.	14	3.9	-	3.9	30	2000	.001	0.43	12.9	0.33	.720	5.4	5	0.93	0.77	0.96	3.7	0.2	
C1-11	22+60 Lt.	13	2.6	0.2	2.8	32 64	2100 4200	.003	0.30	11.2	0.25	.448	6.2	5	0.81	0.83	0.89	2.5	0.3	
CG1-1	21+00 Lt.	11&12	11.1	0.3	11.4							Curb and Grate Inlet								3 Grates
C1-14	21+00 Rt.	20	1.6	-	1.6							Low Point Curb Inlet								
C1-17	33+18 Lt.	17	5.3	-	5.3	32 64	2100 4200	.005	0.36	15.0	0.25	.500	10.6	10	0.94	0.70	0.97	5.1	0.2	
C1-16	29+00 Lt.	18	3.1	0.2	3.3	32 64	2100 4200	.005	0.30	11.2	0.25	.450	7.3	5	0.68	0.83	0.80	2.6	0.7	
C1-15	24+35 Lt.	19	2.5	0.7	3.2	32 64	2100 4200	.003	0.33	13.1	0.25	.472	6.8	5	0.74	0.76	0.85	2.7	0.5	Drain Carry-Over to C1-13
D1-1	25+00 Lt.	16	4.4	-	4.4							Drop Inlet								2 Grates
C1-18	29+00 Rt.	22	3.8	-	3.8	32 64	2100 4200	.005	0.31	11.8	0.25	.456	8.3	5	0.60	0.81	0.73	2.8	1.0	
C1-19	25+60 Rt.	21	1.5	1.0	2.5	32 64	2100 4200	.003	0.30	11.2	0.25	.450	5.5	5	0.91	0.83	0.95	2.4	0.1	Drain Carry-Over along 7th Street

FORM 1020

INLET COMPUTATIONS



NOMOGRAPH FOR FLOW
IN TRIANGULAR CHANNELS

Figure 1

From 1	To 2	Drainage Area No. 3	Total D. A. Ac. 4	Total C. A. 5	Length ft. 6	Time of Concentration — Minutes: Along Sewer Line 7	Inlet Time 8	Used in Design 9	I In./Hr. 10	Q cfs 11	Design: Dia. In. 12	Slope % 13	Cap. cfs 14	Vel. ft/sec. 15	Remarks
Line A															
C1-1	C1-2	1	0.8	0.54	290	*470/60 = 7.8	10	10	5.8	3.1	18	1.25	12.5	5.9	
C1-2	C1-3	1&2	1.7	1.15	161	7.8+290/5.9x60 = 8.6	10	10	5.8	6.7	18	1.25	12.5	7.5	
C1-4	C1-3	3	0.5	0.36	44	* 3	10	10	5.8	2.1	12	0.35	2.2	3.2	Lateral A
C1-3	C1-5	1 to 4	2.6	1.77	244	8.6+161/7.5x60 = 8.9	10	10	5.8	10.3	18	1.25	12.5	8.0	
C1-5	C1-6	1 to 5	3.6	2.33	205	8.9+244/8.0x60 = 9.4	10	10	5.8	13.5	21	1.25	19.0	8.8	
C1-6	C1-7	1 to 6	4.6	2.92	88	9.4+205/8.8x60 = 9.8	10	10	5.8	16.9	21	1.25	19.0	9.1	
C1-7	C1-8	1 to 7	5.14	3.30	363	9.8+ 88/9.1x60 = 10	10	10	5.8	19.1	24	1.25	27.5	9.5	
C1-8	C1-10	1 to 9	6.48	4.04	87	10 +363/9.5x60 =10.6	10	10.6	5.7	23.0	24	1.25	27.5	9.7	
C1-9	C1-10	9	0.24	0.16	68	* 3	10	10	5.8	1.0	12	0.20	1.7	2.3	Lateral B
C1-10	C&GI-1	1 to 10	6.87	4.25	282	10.6+87/9.7x60 =10.7	10	10.7	5.7	24.2	24	1.25	27.5	9.9	
Line B															
C1-13	C1-12	15	0.35	0.23	46	*200/0.5x60 = 6.7	10	10	5.8	1.3	12	0.197	1.65	2.3	
C1-12	C1-11	15&14	2.28	0.91	125	6.7+46/2.3x60 = 7.0	10	10	5.8	5.3	18	0.255	5.6	3.7	
C1-11	C&G1-1	15,14&13	3.20	1.35	160	7.0+125/3.7x60 = 7.6	10	10	5.8	7.8	18	0.550	8.5	5.5	
Line C															
C&G1-1	C1-14	1 to 15	13.47	7.52	70	10.7+282/9.9x60=11.2	10	11.2	5.7	42.8	30	1.10	45.0	10.7	
C1-14	MH-2	1 to 15&20	13.78	7.80	145	11.2+70/10.7x60=11.3	10	11.3	5.7	44.5	30	1.15	46.0	11.0	
MH-2	MH-3	1 to 22	22.71	11.87	480	34.0+240/7.3x60=34.5	-	34.5	3.4	40.3	36	0.40	45.0	7.4	
Line D															
C1-17	C1-16	17	2.0	0.92	418	*620/60 = 10.3	10	10.3	5.8	5.3	18	0.255	5.6	3.7	
C1-16	C1-15	17&18	3.0	1.46	465	10.3+418/3.7x60=12.2	10	12.2	5.6	8.2	18	0.57	8.5	5.5	
D1-1	C1-15	16	4.2	1.26	100	*1000/0.5x60 = 33	33	33	3.5	4.4	18	0.175	4.7	3.1	Lateral C
C1-15	MH-1	16,17,18,19	7.9	3.15	160	33+100/3.1x60 = 33.5	10	33.5	3.4	10.7	21	0.50	12.2	5.8	
MH-1	MH-2	16 thru 22	8.93	4.07	240	33.5+160/5.8x60=34.0	-	34.0	3.4	13.8	21	0.82	15.5	7.3	
Line E															
C1-18	C1-19	22	0.74	0.66	340	*450/60 = 7.5	10	10	5.8	3.8	18	0.57	8.5	4.8	
C1-19	MH-1	21&22	1.03	0.92	250	7.5+340/4.8x60 = 8.7	10	10	5.8	5.3	18	0.57	8.5	5.2	

* Overland Flow

FORM 1021

TABLE II

STORM SEWER COMPUTATIONS

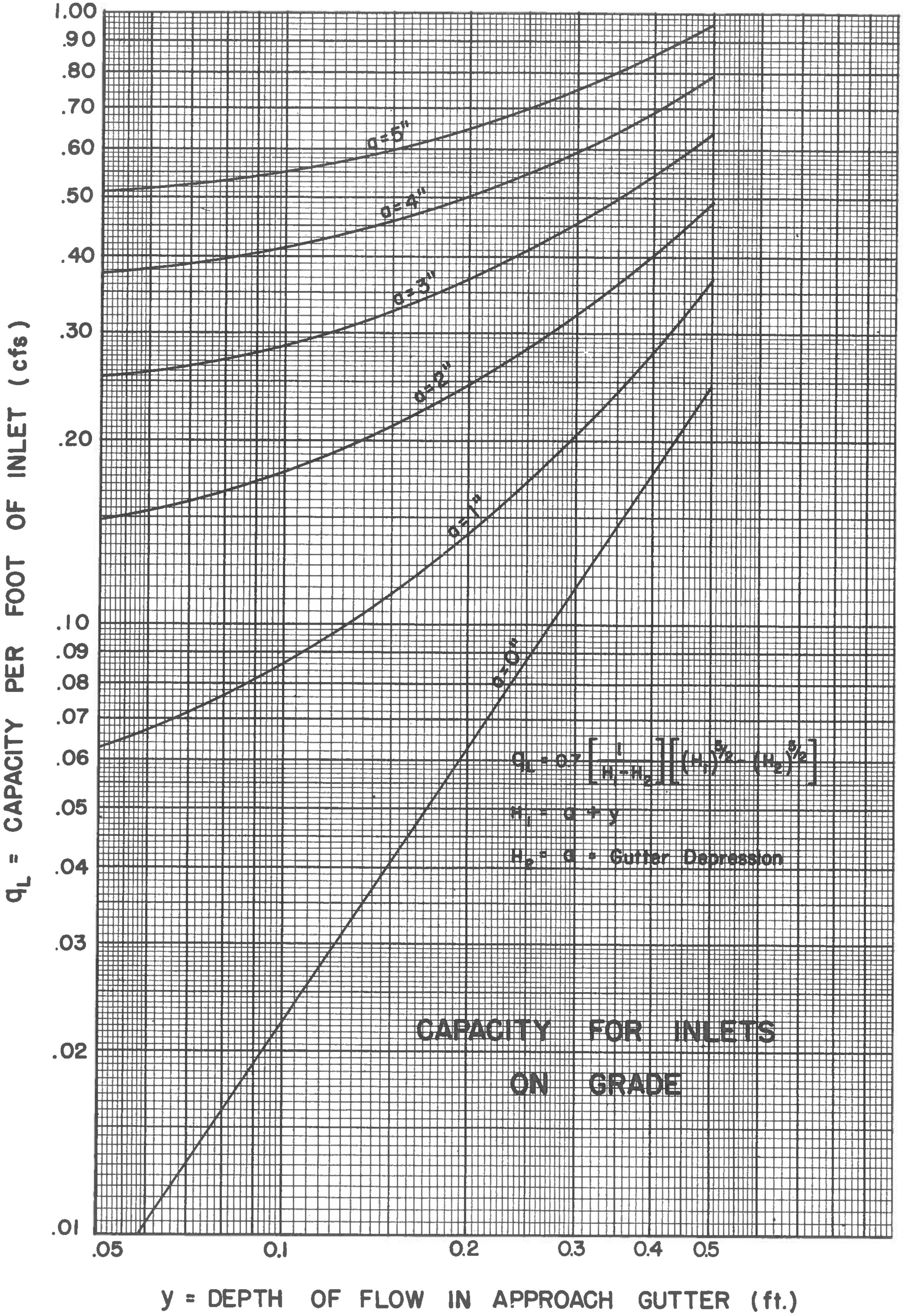

1.00
.90
.80
.70
.60
.50
.40
.30
.20
.10
.09
.08
.07
.06
.05
.04
.03
.02
.01
.05
0.1
0.2
0.3
0.4
0.5
a=5"
a=4"
a=3"
a=2"
a=1"
a=0"
$q_L = 0.7\left[\frac{1}{H_1 - H_2}\right]\left[(H_1)^{5/2} - (H_2)^{5/2}\right]$
$H_1 = a + y$
$H_2 = a$ = Gutter Depression
CAPACITY FOR INLETS ON GRADE
q_L = CAPACITY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

Figure 2

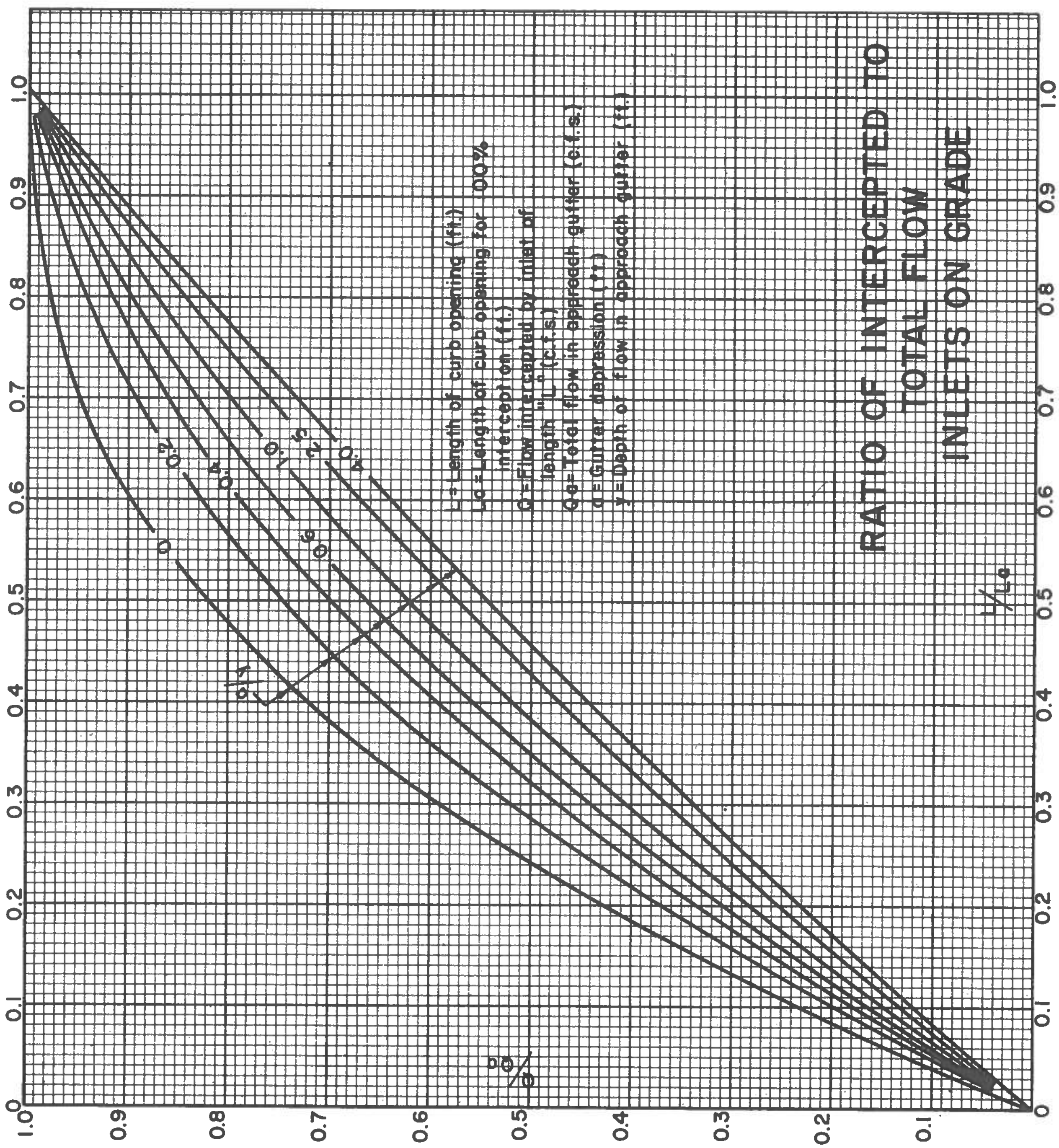
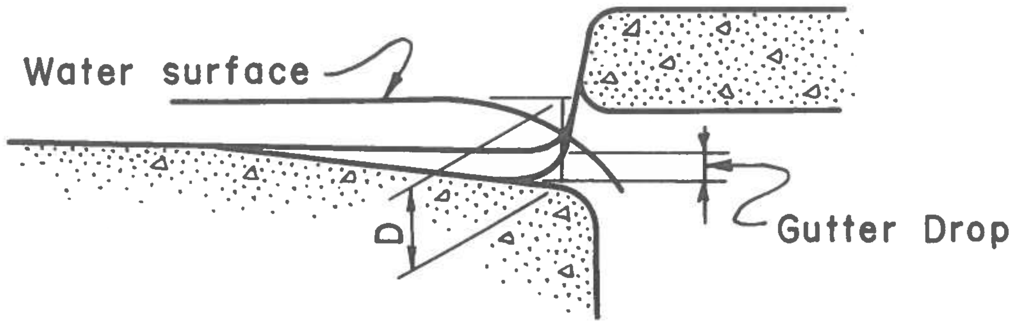
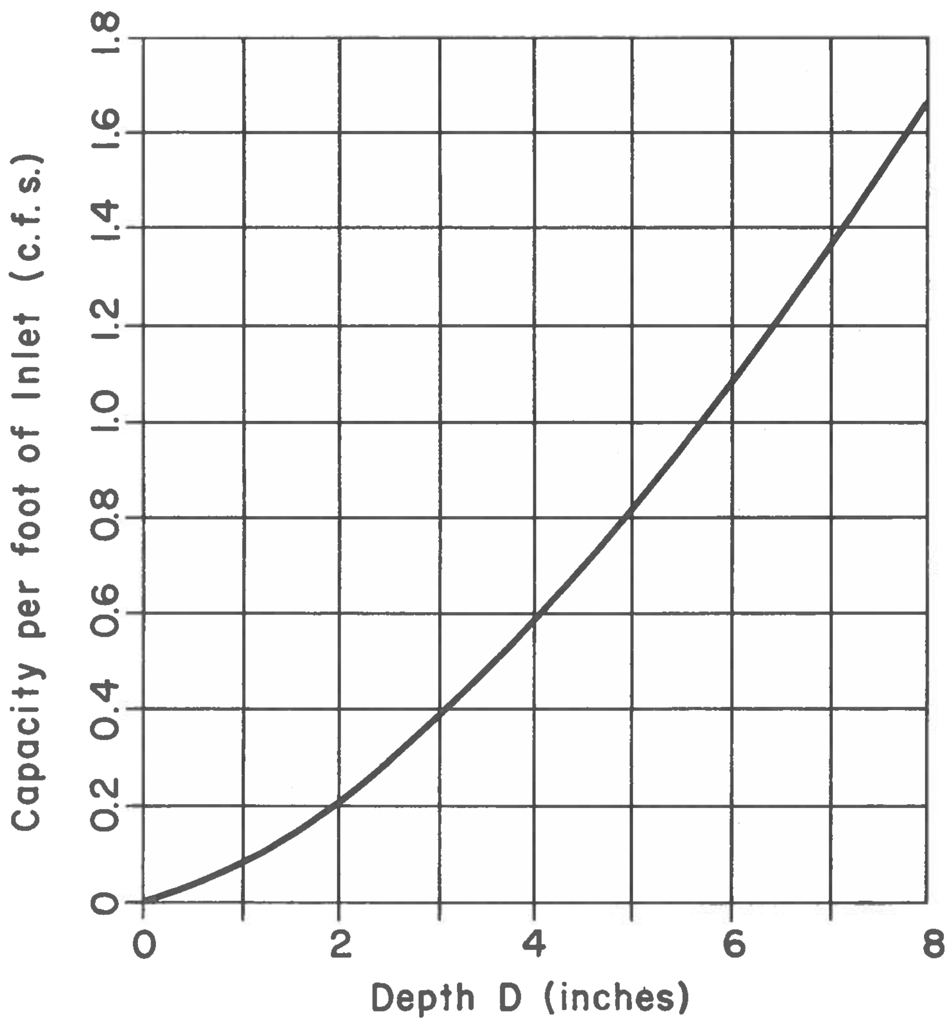

RATIO OF INTERCEPTED TO TOTAL FLOW
INLETS ON GRADE
L = Length of curb opening (ft.)
Lc = Length of curb opening for 100% interception (ft.)
Q = Flow intercepted by inlet of length "L" (c.f.s.)
Qa = Total flow in approach gutter (c.f.s.)
a = Gutter depression (ft)
y = Depth of flow in approach gutter (ft)
a/y
Q/Qa
L/Lc

Figure 3



SECTION



INLET CAPACITY
FOR
LOW POINT INLETS

Figure 4

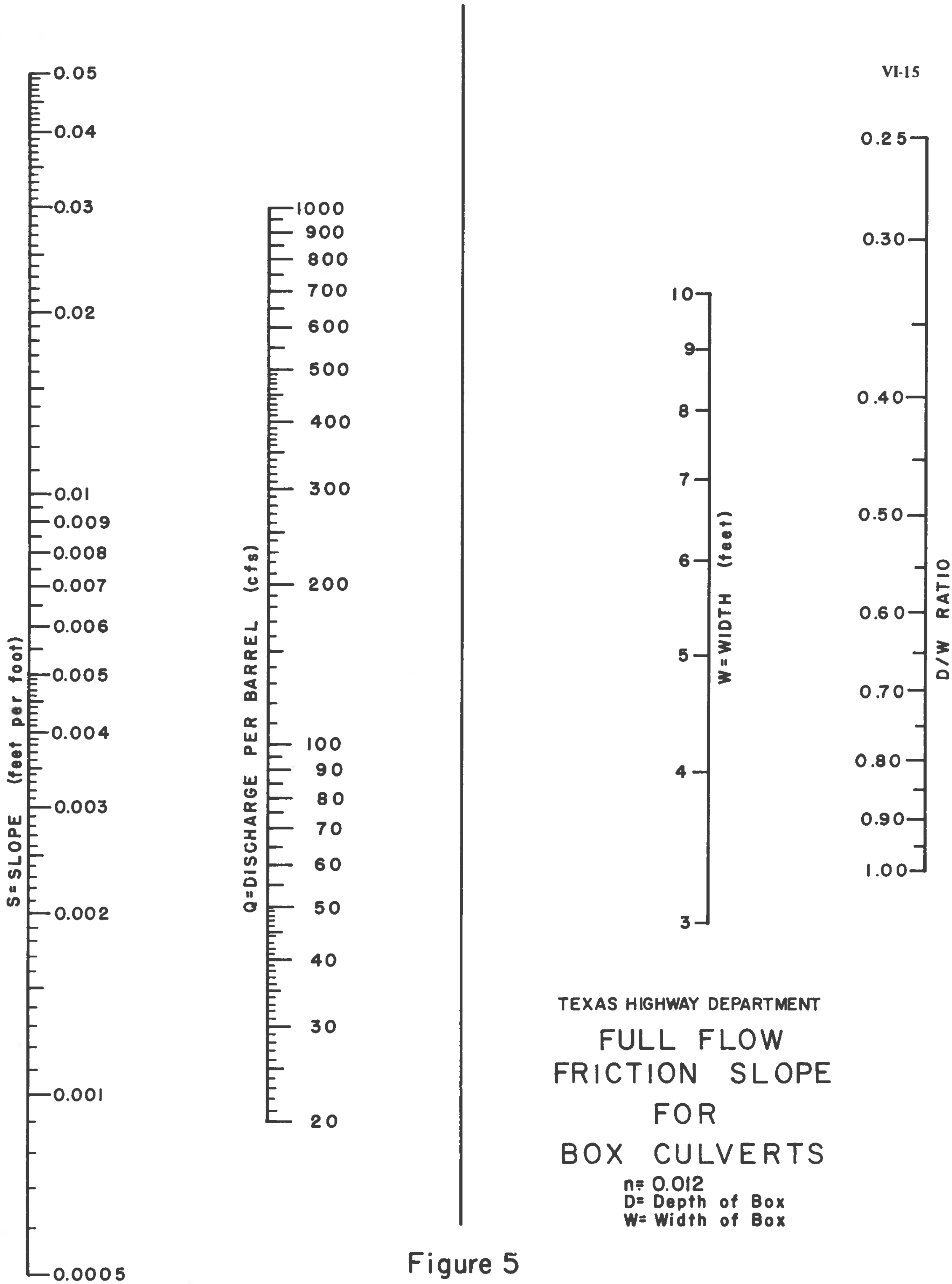
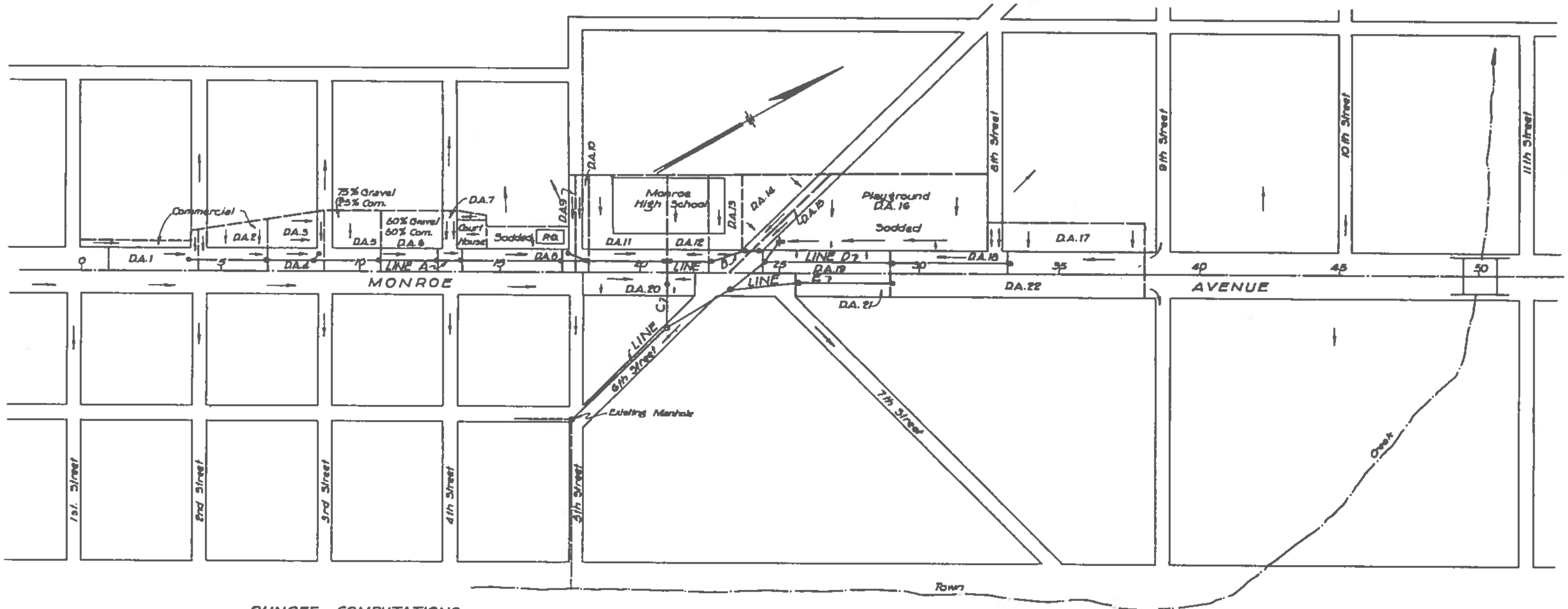

S = SLOPE (feet per foot)
0.05
0.04
0.03
0.02
0.01
0.009
0.008
0.007
0.006
0.005
0.004
0.003
0.002
0.001
0.0005
Q = DISCHARGE PER BARREL (cfs)
1000
900
800
700
600
500
400
300
200
100
90
80
70
60
50
40
30
20
W = WIDTH (feet)
10
9
8
7
6
5
4
3
D/W RATIO
0.25
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
TEXAS HIGHWAY DEPARTMENT
FULL FLOW
FRICTION SLOPE
FOR
BOX CULVERTS
n= 0.012
D= Depth of Box
W= Width of Box

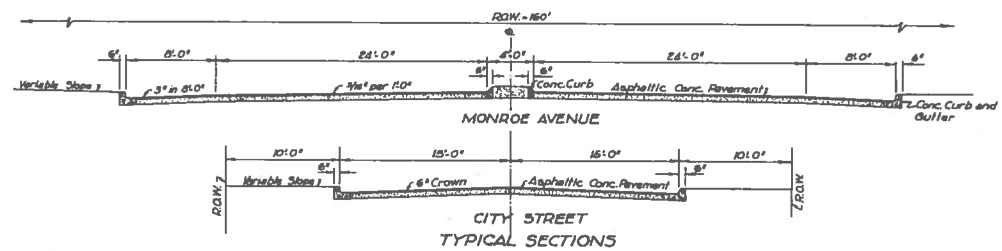
Figure 5



RUNOFF COMPUTATIONS

Drainage Area No.	Drainage Area Ac.	Acres – Drained: Rev.'l C=0.9	Comm. C=0.7	Resid. C=0.35	Gravel C=0.4	Imperv. Sodded C=0.3	Total CA	Time of Concentration - minutes -	Is inches per hour	Q5 cfs
1	0.8	0.32	0.21		0.27		0.54	10	5.8	3.1
2	0.9	0.24	0.43		0.23		0.61	10	5.8	3.5
3	0.5	0.03	0.47				0.36	10	5.8	2.1
4	0.4	0.19			0.21		0.26	10	5.8	1.5
5	1.0	0.24	0.16		0.60		0.56	10	5.8	3.2
6	1.0	0.18	0.33		0.49		0.59	10	5.8	3.4
7	0.54	0.20	0.23		0.06	0.05	0.38	10	5.8	2.2
8	1.1	0.30	0.11		0.30	0.39	0.58	10	5.8	3.3
9	0.24	0.12			0.12		0.16	10	5.8	1.0
10	0.39	0.15			0.08	0.16	0.21	10	5.8	1.2
11	2.2	0.24	0.92		0.26	0.78	1.19	10	5.8	6.9
12	1.2	0.13	0.69		0.14	0.24	0.73	10	5.8	4.2
13	0.92	0.11	0.23		0.11	0.47	0.44	10	5.8	2.6
14	1.93	0.14			0.09	1.70	0.68	10	5.8	3.9
15	0.35	0.2				0.15	0.23	10	5.8	1.3
16	4.2					4.20	1.26	1000/1.5x60=33	3.5	4.4
17	2.0	0.44		1.12		0.44	0.92	10	5.8	5.3
18	1.0	0.40				0.60	0.54	10	5.8	3.1
19	0.7	0.36				0.34	0.43	10	5.8	2.5
20	0.31	0.31					0.28	10	5.8	1.6
21	0.29	0.29					0.26	10	5.8	1.5
22	0.74	0.74					0.66	10	5.8	3.8

LEGEND
- • Curb Inlet
- ▪ Curb & Grate Inlet
- + Drop Inlet
- • Manhole



Design Frequency: 5 Years
Time of Concentration: Minimum 10 minutes
Gutter Depression: Monroe Ave. – 3" for Curb Inlets
1" for Curb & Grate Inlets
City Streets – 3" to 5" for Curb Inlets
Permissible Ponding: Monroe Ave. – 10' in business district
15' in residential district
City Streets: Unlimited
Minimum Inlet Length: 5'-0"

MAP OF DRAINAGE AREAS
SCALE: 1" = 200'

FED. ROAD DIV. NO.	STATE	FEDERAL AID PROJECT NO.	SHEET NO.
6	TEXAS		
STATE DIST. NO.	COUNTY		

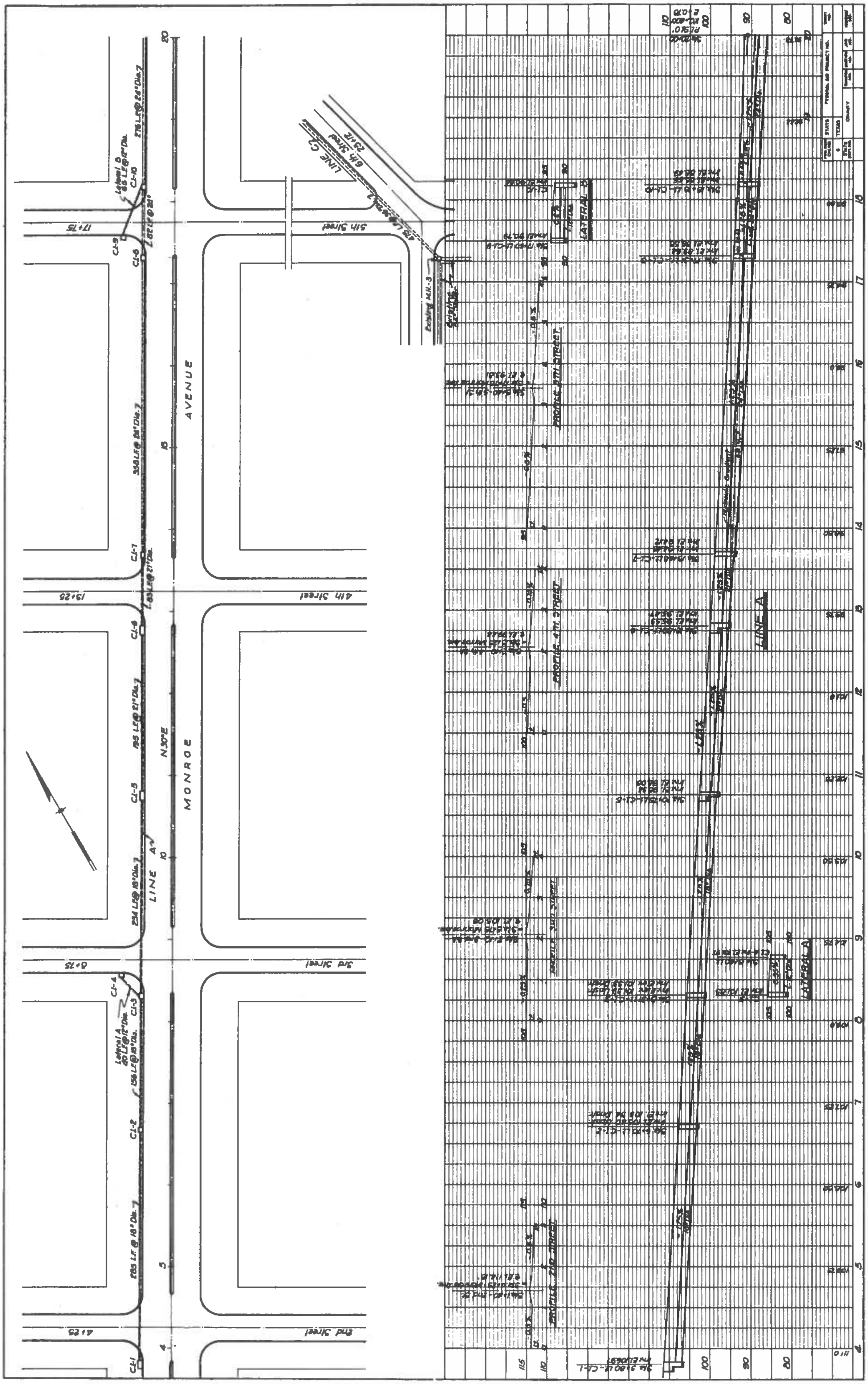
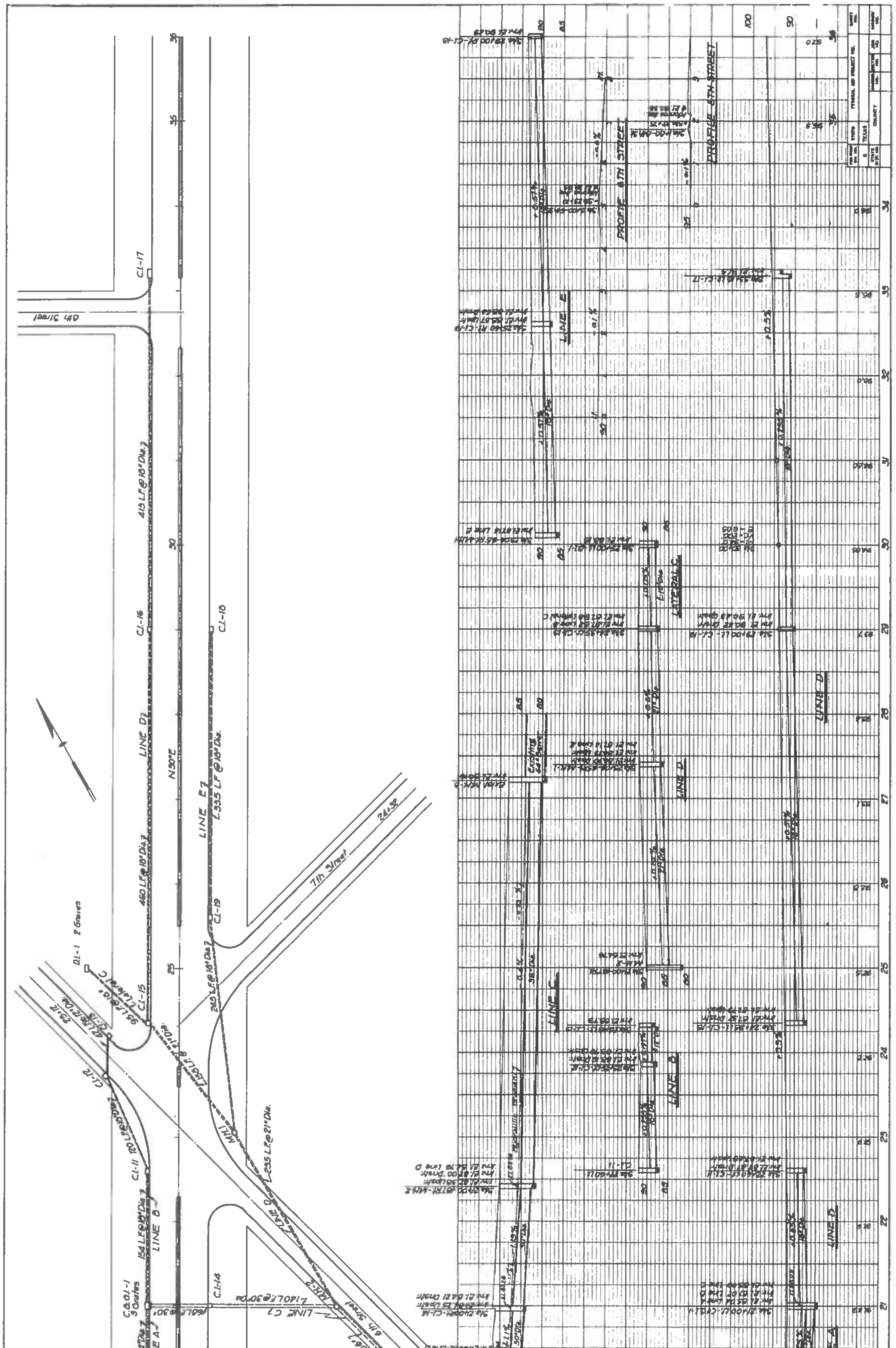
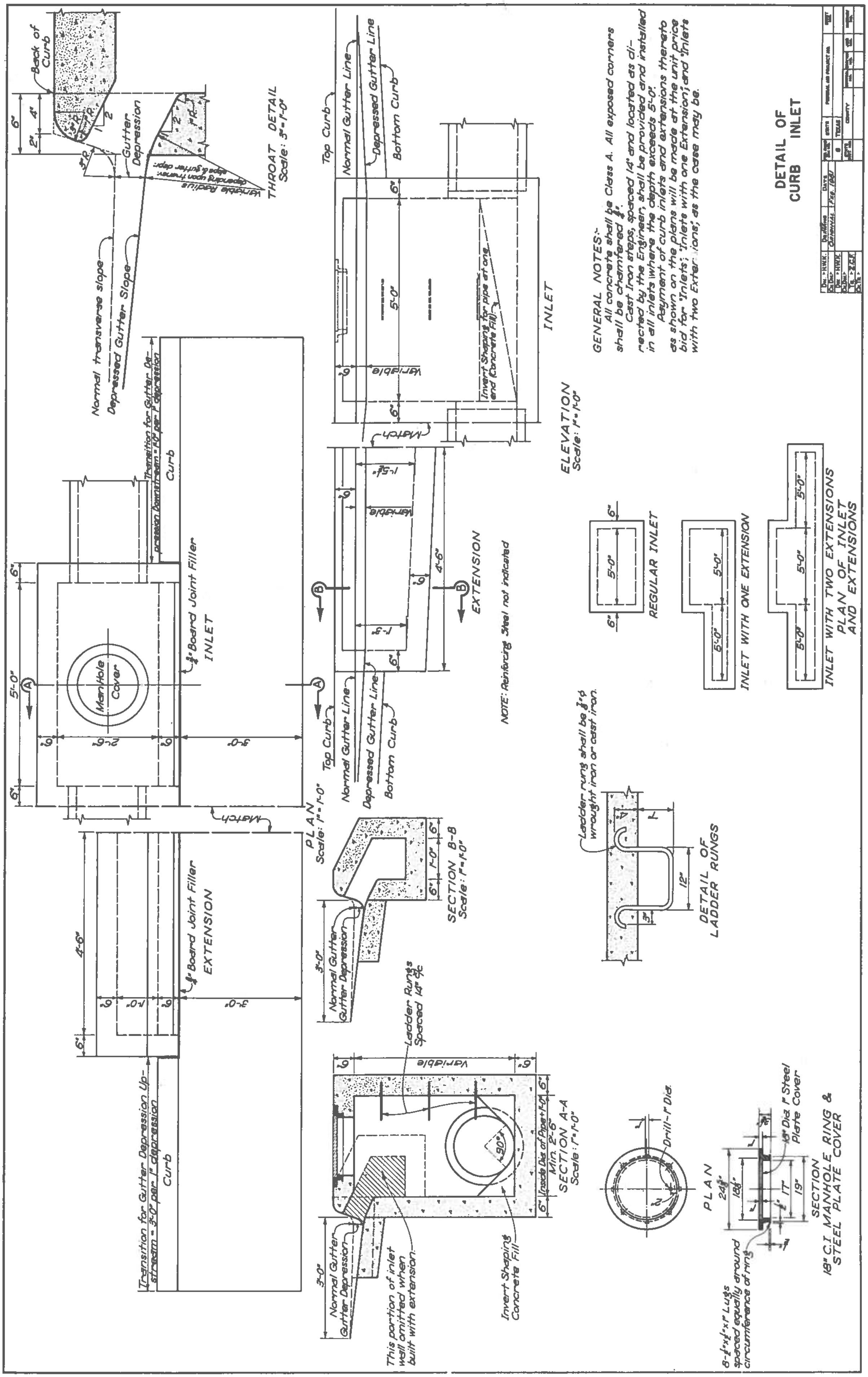

MONROE AVENUE
LINE A
2nd Street
3rd Street
4th Street
5th Street
6th Street
LATERAL A
LATERAL B

Figure 7



6th Street
7th Street
LINE A
LINE B
LINE C
LINE D
LINE E
LATERAL C
PROFILE 6TH STREET

Figure 8



DETAIL OF CURB INLET
GENERAL NOTES:-
All concrete shall be Class A. All exposed corners shall be chamfered ¾".
Cast Iron steps, spaced 14" and located as directed by the Engineer, shall be provided and installed in all inlets where the depth exceeds 5'-0".
Payment of curb inlets and extensions thereto as shown on the plans will be made at the unit price bid for "Inlets", "Inlets with one Extension", and "Inlets with two Extensions", as the case may be.
THROAT DETAIL
Scale: 3" = 1'-0"
Back of Curb
Gutter Depression
Variable Radius depending upon transv. slope & gutter depr.
Normal transverse slope
Depressed Gutter Slope
Transition for Gutter Depression Downstream = 1'-0" per 1" depression
Curb
Transition for Gutter Depression Upstream = 3'-0" per 1" depression
¾" Board Joint Filler
INLET
EXTENSION
Manhole Cover
Match
PLAN
Scale: 1" = 1'-0"
Top Curb
Normal Gutter Line
Depressed Gutter Line
Bottom Curb
ELEVATION
Scale: 1" = 1'-0"
Invert Shaping for pipe at one end (Concrete Fill)
Variable
NOTE: Reinforcing Steel not indicated
SECTION B-B
Scale: 1" = 1'-0"
Normal Gutter
Gutter Depression
SECTION A-A
Scale: 1" = 1'-0"
Inside Dia. of Pipe + 1'-0" Min. 2'-6"
Ladder Rungs Spaced 14" c/c
This portion of inlet wall omitted when built with extension.
Invert Shaping Concrete Fill
Ladder rung shall be ¾" ϕ wrought iron or cast iron.
DETAIL OF LADDER RUNGS
REGULAR INLET
INLET WITH ONE EXTENSION
INLET WITH TWO EXTENSIONS
PLAN OF INLET AND EXTENSIONS
PLAN
Drill-1" Dia.
SECTION
18" C.I. MANHOLE RING & STEEL PLATE COVER
18" Dia. 1" Steel Plate Cover
6-¾"x¾"x1" Lugs spaced equally around circumference of ring

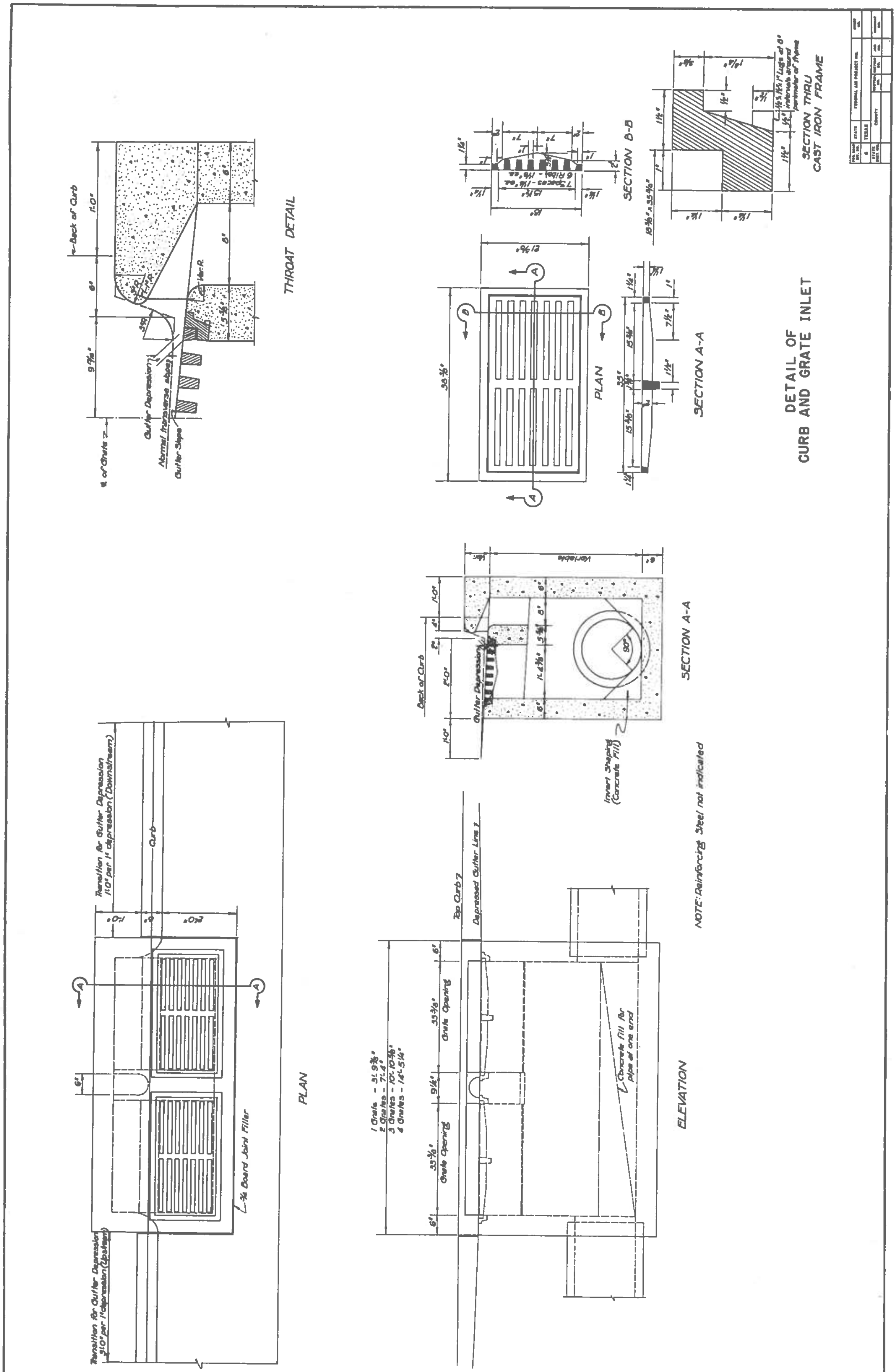

DETAIL OF
CURB AND GRATE INLET
PLAN
ELEVATION
SECTION A-A
THROAT DETAIL
SECTION B-B
SECTION THRU
CAST IRON FRAME
NOTE: Reinforcing Steel not indicated
Transition for Gutter Depression
1'-0" per 1" depression (Downstream)
Transition for Gutter Depression
3'-0" per 1" depression (Upstream)
Curb
3/4" Board Joint Filler
Top Curb
Depressed Gutter Line
Grate Opening
1 Grate - 3'-9 3/8"
2 Grates - 7'-4"
3 Grates - 10'-10 5/8"
4 Grates - 14'-5 1/4"
Concrete Fill for pipe at one end
Back of Curb
Gutter Depression
Invert Shaping (Concrete Fill)
Variable
Normal transverse slope
Gutter Slope
℄ of Grate
TEXAS

Figure 10

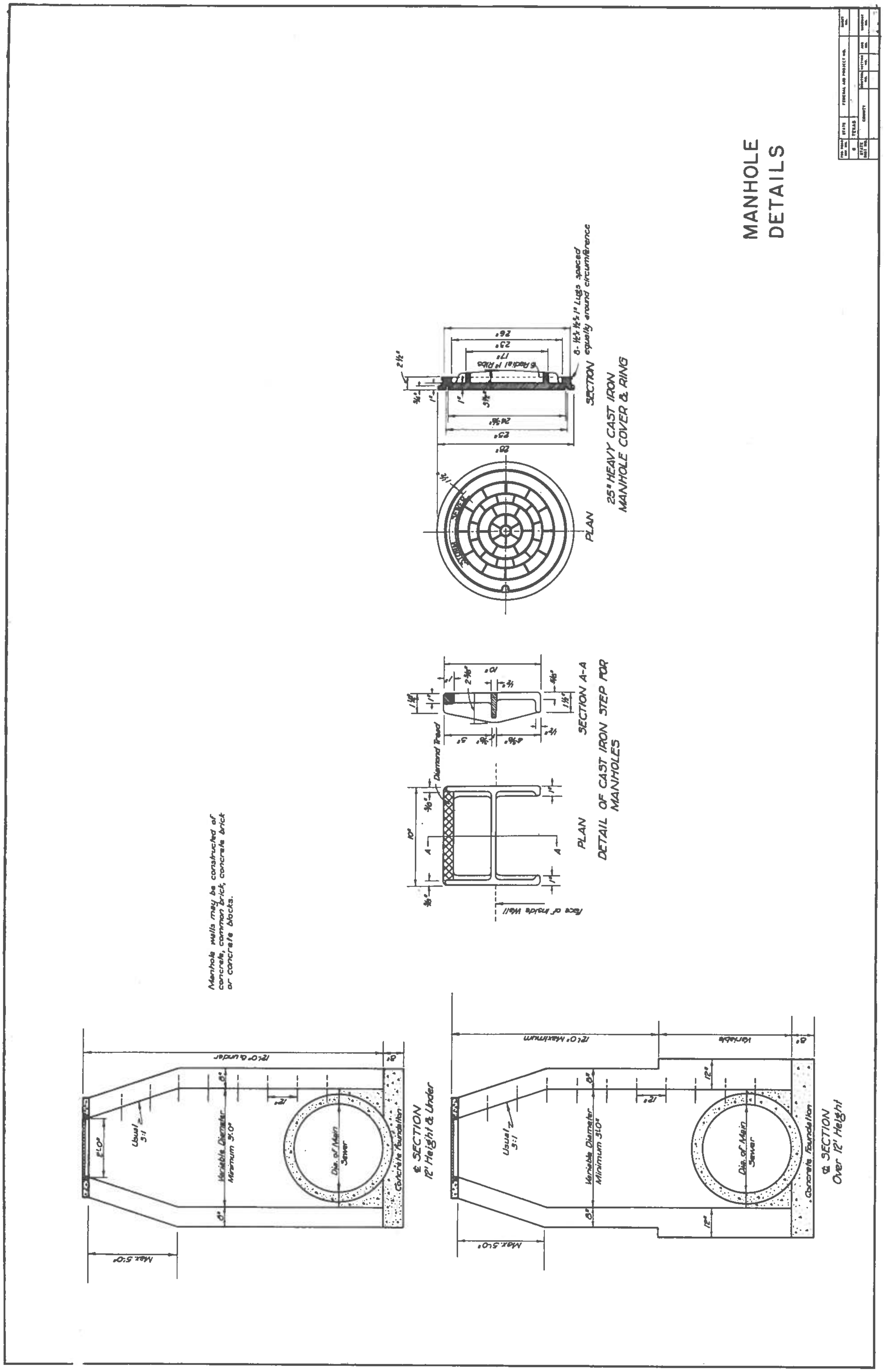
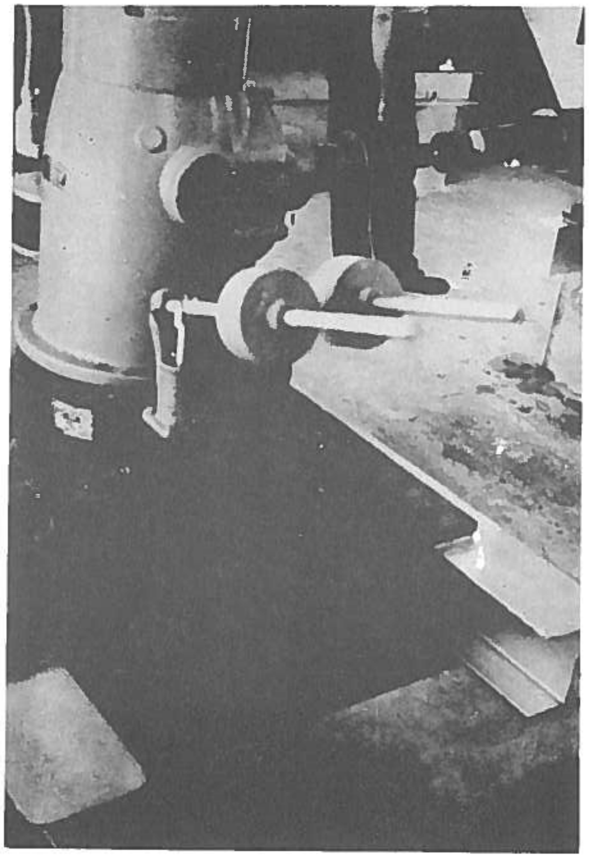
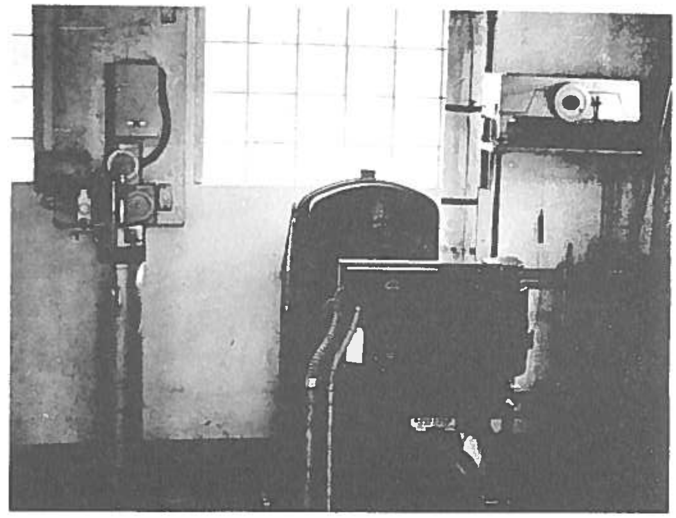

MANHOLE DETAILS
TEXAS
PLAN
SECTION
25" HEAVY CAST IRON MANHOLE COVER & RING
8 - 1½ x 1½ x 1" Lugs spaced equally around circumference
PLAN
SECTION A-A
DETAIL OF CAST IRON STEP FOR MANHOLES
Diamond Tread
Face of Inside Wall
Manhole walls may be constructed of concrete, common brick, concrete brick or concrete blocks.
℄ SECTION 12' Height & Under
℄ SECTION Over 12' Height
Usual 3:1
Variable Diameter Minimum 3'-0"
Dia. of Main Sewer
Concrete Foundation
12'-0" & under
12'-0" Maximum
Variable
Max. 5'-0"

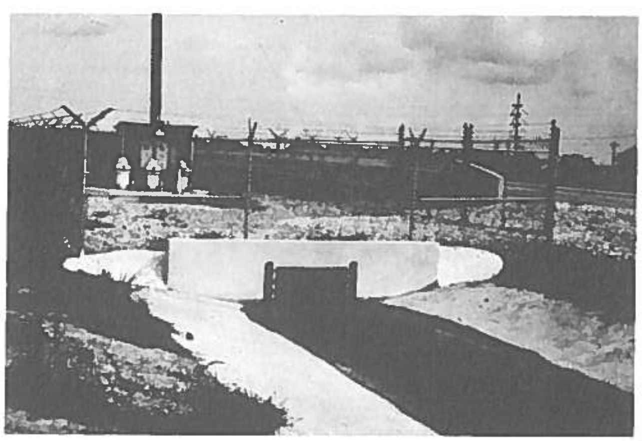
Figure 11

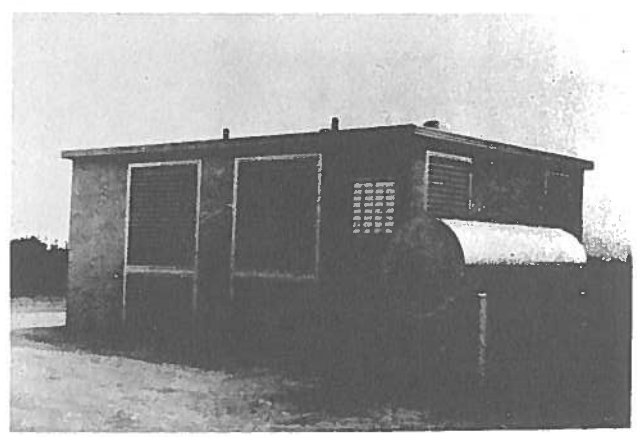
CHAPTER VII

PUMP STATIONS









PUMP STATIONS

7-100. GENERAL

The mechanical lifting of storm water for roadway drainage becomes economically advantageous when long, deep gravity outfalls are necessary. In general, gravity outfalls are desirable but the possible economic gain of using mechanical lift in lieu of gravity drainage is such that it certainly behooves the designer to investigate the mechanical lift design.

This chapter is intended to guide the designer toward an acceptable pump station design. In addition, pump manufacturer's and electrical contractor's representatives can be very helpful in the planning stages. Also, contacts are necessary with representatives of the utility firms that might supply power to the station (electricity, natural gas or diesel fuel.) Upon request, D-5 personnel will furnish samples of pump station designs and specifications.

7-200. PUMP STATION

This usually refers to the entire facility required for mechanically lifting the storm water a predetermined distance from a gravity inflow to a gravity outfall. The basic features of a pump station (Figure 1) might include a wet well sump, pumps, appurtenances, pump house, and motors and/or engines.

The entire pump station should be protected from vandals and children by use of fences, locks, etc. When planning the fencing, adequate provision should be made for moving trucks and equipment in and out of the station.

a. Wet Well Sump - The wet well sump receives the inflow of storm water prior to pumping. The wet well sump must be designed with provisions for screening trash and easy access for removing accumulated trash and silt.

   The wet well sump has been constructed generally in three different ways, 1) as a concrete caisson formed in sections and sunk into place by excavating within the limits of the caisson, 2) a rectangular sump constructed in conjunction with a retaining wall, where the retaining wall constitutes one wall of the wet well sump and the three other walls are added, 3) by excavating and constructing a rectangular wet well sump in place.

   The concrete slab at the top of the wet well sump is usually referred to as the pump floor since the pumps are suspended from this slab.

   The flow line of the inflow pipe or box should be above the cut-off elevation (minimum water elevation in sump).

b. Pumps - The type of pump successfully used by the Texas Highway Department is an axial flow, propeller or mixed flow pump. Positive lubrication is a must, so it is recommended that the shaft that transmits power to the impellers be enclosed and flooded with oil at all times. This prevents infiltration of water that might attack the shaft and bearings. Lubrication for the lower bearings, etc. shall be provided by a separate lubrication line with a lubricant reservoir located on the pump floor for easy access and observance.

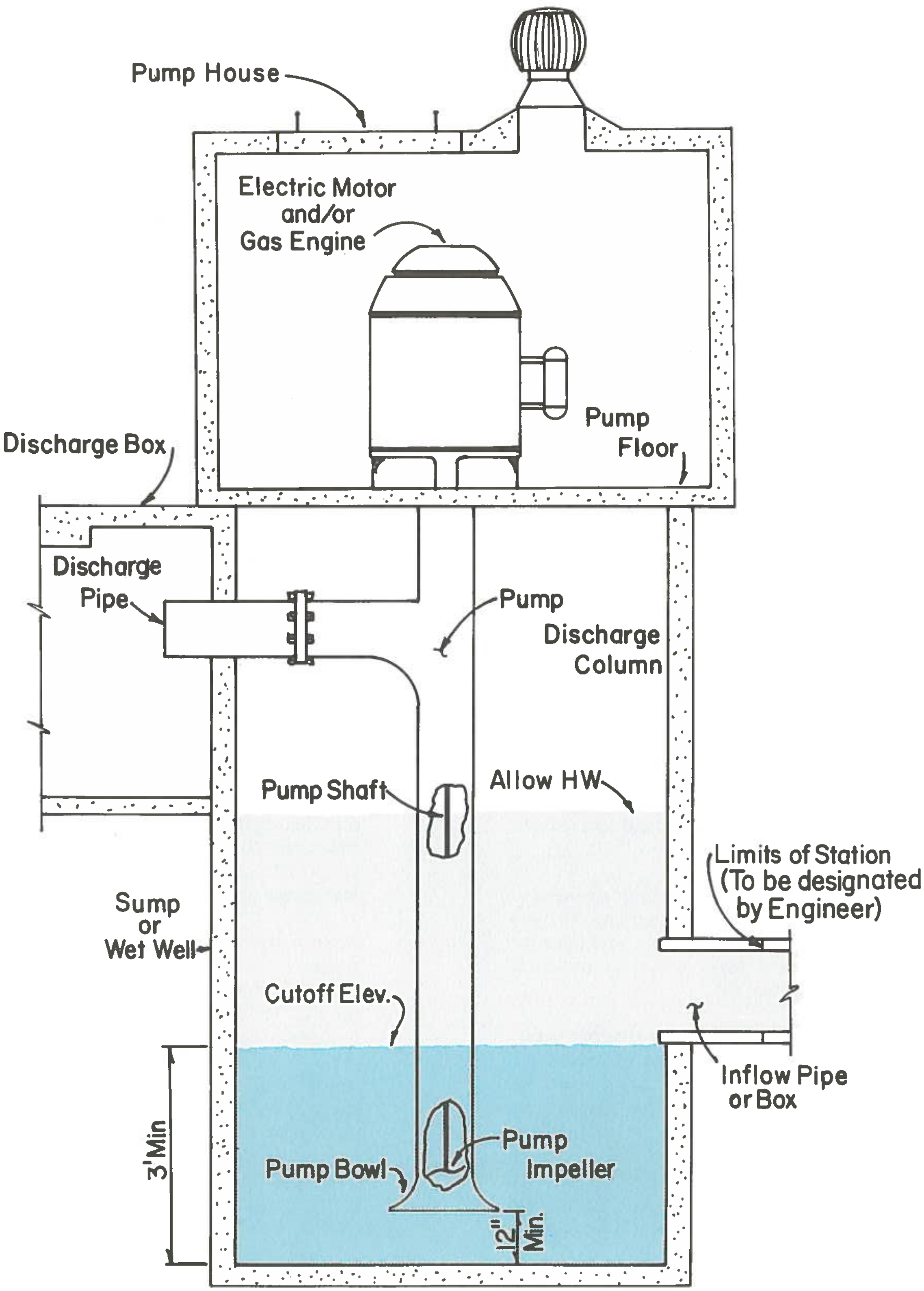
   Two or more pumps are required because of the factor of safety afforded by multiple pumps.

c. Appurtenances - Some device for activating the motors and/or engines is necessary. Two types of controls are used. One type utilizes electrodes mounted on the sump wall at various elevations, and, as the water rises, contact is made and the starter relays are activated. The second type, and the type most frequently used, is a float control which consists of a float connected to one end of a steel tape that has a counterweight on the opposite end. The tape runs over a pulley, and as the water rises in the sump the float rises and the pulley rotates to activate the starter relays.

d. Pump House - The pump house provides shelter for the array of electrical equipment, the motors and/or engines, etc. It is of great importance to plan for openings of sufficient size to remove and repair the pumps, engines, motors, etc. and to provide an adequate number of electrical outlets and lights for maintenance of the station.

   Pump houses are not always provided when only electric motors are specified. Since weatherproof electric motors are available, they can be left in the weather and a simple shelter can be provided to protect the electrical equipment.

e. Motors and/or Engines - The use of electric motors and/or internal combustion engines (natural gas or diesel) depends on available power and its reliability. Many stations operate with only electric motors, others with internal combustion engines and still others with both an electric motor and an internal combustion engine for each pump. Critical situations often require the consideration of two power sources. One type might be electrical power to the station from two separate electrical sub-stations with an automatic switch that selects the "hot line".



PUMP
STATION
Figure-1

Another type is a combination motor and engine driven pump with automatic provisions for using either the motor or engine.

7-300. RUNOFF

The maximum rate of runoff is to be determined in accordance with the procedures outlined in Chapter II. For the most part small drainage area sizes are associated with mechanical lifting of roadway drainage, therefore the method for determining runoff is usually the Rational Method. It is desirable to limit, as much as possible, the watershed that will contribute to the depressed section of roadway. This will hold to a minimum that runoff which will subsequently be pumped.

7-400. ALLOWABLE HIGHWATER

This is the maximum elevation that storm water is allowed to pond in the low point of the roadway section. Properly sized pumps (for the design storm) should keep the ponded elevation of the storm water equal to or below the allowable highwater.

7-500. STORAGE

This is the volume of water that can pond in the system below allowable highwater. This includes surface water that would be ponded in roadway ditches, in gutter and travel lanes, pipes, boxes, inlets, wet well sump, etc.

7-600. EXAMPLE PROBLEM

It is required to determine the necessary number and sizes of pumps to drain a depressed roadway section in Mud County.

Given: 12.6 Acres in Mud County.

Plan, profile, and low point cross section of roadway as shown in Figures 2 and 3.

Maximum highwater elevation equals 100.0 feet.

2 or more pumps are required.

Wet Well Sump to be circular, with an inside diameter of 15 feet.

Lift = 20 feet.

50 year design frequency.

Step 1 - Determine the estimated peak rate of runoff from the drainage area in question. This does not represent the total volume of run-off, only the maximum rate. Q = CIA (see Chapter II) (Also see Table I).

Step 2 - Determine the available storage in the system. This storage represents available space, below the allowable highwater elevation and above pump cut-off elevation, where storm water can be stored before flooding occurs.

Sump storage
From Elev. 91.5 to Elev. 100 = 1,500 CF

Pipe storage
200 feet of 48'' dia. pipe = 2,514 CF

Ditches
4 ditches approximately 60 feet long* with water 3 feet deep = 8,650 CF

Total effective ponded volume below elevation 100.0' = 12,664 CF

*This is the average length of ditch in which water will be ponded below elevation 100.0' feet. The depth of water in the ditch at the roadway low point is 3 feet.

Step 3 - Select pump size or sizes. Even though the 50 year design discharge is based upon a storm duration equal to the time of

TABLE I

ACRES DRAINED		C X A		Total CA	t_c Min	I_{50} in/hr	Q_{50} cfs
Pvmt. Ac.	Sod Ac.	Pvmt. (C = .9)	Sod (C = .4)				
4.1	8.5	3.7	3.4	7.1	10	8.5	60.4

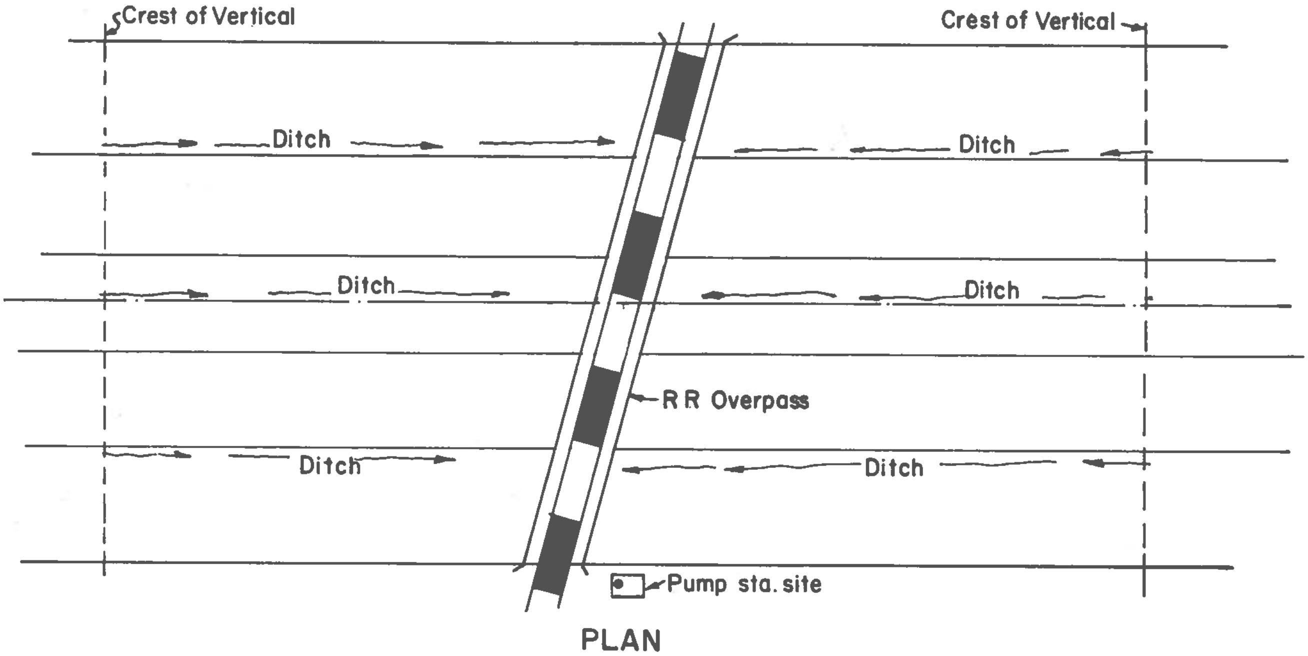


Figure 2

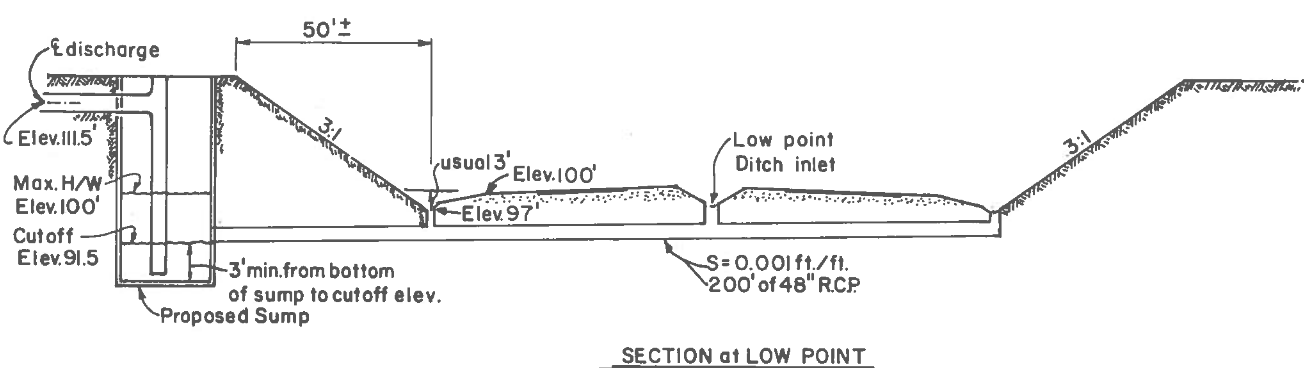


Figure 3

concentration (10 minutes), the area can experience 50 year storms of various durations. Each duration is associated with a different intensity. With storage added as another factor, the system must be evaluated to determine the adjusted peak. The adjusted peak will represent the average pump capacity required.

It is assumed that the rain occurs at a constant intensity (inches per hour) for a certain duration (minutes). See Figure 4. The inflow into the storage area is assumed to vary in a straight line from zero rate (cubic feet per second) at the beginning of the rain to the point of maximum runoff rate at a time equal to the time of concentration. At a time equal to the time of concentration, the rain ceases (if duration equals time of concentration), and runoff rate varies from maximum rate to zero in a period of time also equal to the time of concentration. See Figure 4. If the storm duration is longer than the time of concentration, the maximum rate of runoff is not momentary, but continues at a constant rate until the storm ceases. See Figure 5. The area under the rate versus time curve yields the volume of water to be handled.

Since the maximum rate of runoff for a specific intensity is

$$Q = CIA$$

Then

$$\text{Total runoff volume} = CIA \times \text{Duration}$$

where

$$\text{Duration} = \text{Storm Duration in seconds}$$

and

the total volume (cubic feet) that will flood above elevation 100.0 feet is

$$\text{Excess Volume} = (CIA \times \text{Duration}) - (\text{Storage})$$

and

the average pump capacity (cubic feet per second) required to remove the flood volume is

$$\text{Avg Pump Capacity} = \frac{(CIA \times \text{Duration}) - (\text{Storage})}{\text{Duration}}$$

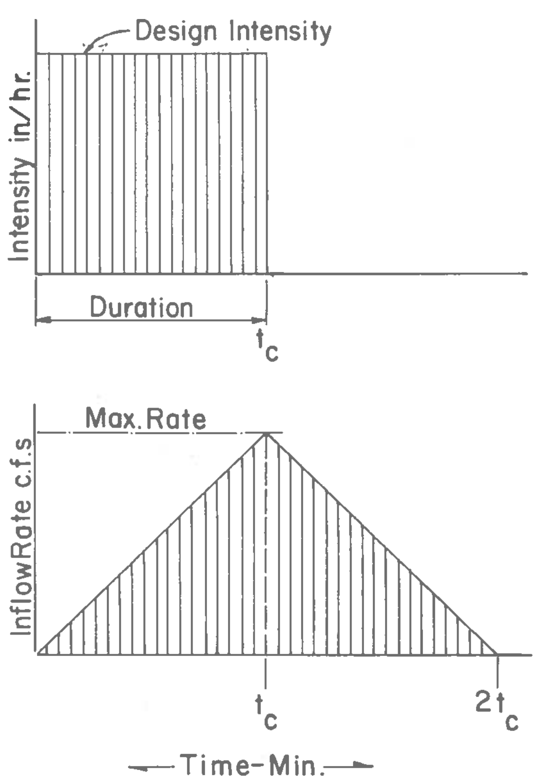


Figure 4

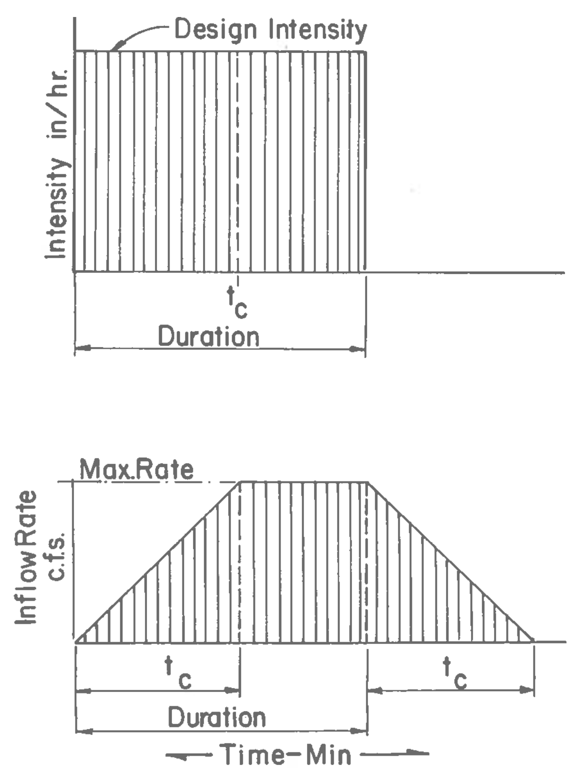


Figure 5

TABLE II

DURATION (1) SECS.	I (2) In./Hr.	CA (3)	CIA (2) x (3) (4) CFS	D x CIA (1) x (4) (5) CF	STORAGE VOL. (6) CF	FLOOD VOL. (5) – (6) (7) CF	AVG. PUMP CAP. (7) – (1) (8) CFS
5 Min. = 300 Sec.	10.0	7.1	71.0	21,300	12,664	8,636	28.8
10 Min. = 600 Sec.	8.5	7.1	60.4	36,200	12,664	23,536	39.2
15 Min. = 900 Sec.	7.5	7.1	53.2	47,900	12,664	35,236	39.2*
20 Min. = 1,200 Sec.	6.7	7.1	47.6	57,100	12,664	44,436	37.0
30 Min. = 1,800 Sec.	5.7	7.1	40.5	72,900	12,664	60,236	33.5
40 Min. = 2,400 Sec.	5.0	7.1	35.5	85,200	12,664	72,536	30.2
50 Min. = 3,000 Sec.	4.5	7.1	32.0	96,000	12,664	83,336	27.5
60 Min. = 3,600 Sec.	4.1	7.1	29.1	104,800	12,664	92,136	25.6
70 Min. = 4,200 Sec.	3.8	7.1	27.0	113,400	12,664	100,736	24.0

*Design Peak

TABLE III

ELAPSED TIME (1) MIN	INFLOW RATE @ EACH MINUTE (2) CFS	INCRE-MENTAL INFLOW VOLUME (3) CF	ACCUM. INFLOW VOLUME (4) CF	ACCUM. OUTFLOW PER PUMP: PUMP #1 792 CFM (5) CF	PUMP #2 792 CFM (6) CF	PUMP #3 792 CFM (7) CF	REMAINING IN SUMP: 1 PUMP ON (4)–(5) (8) CF	2 PUMP ON (4)–(5)–(6) (9) CF	3 PUMP ON (4)–(5)–(6)–(7) (10) CF
0									
1	5.3			Begin #1 @ 2 Min.					
2	10.6	636	636		Begin #2 @ 4 Min.	Begin #3			
3	15.9					@ 6 Min.			
4	21.2	1,908	2,544	1,584			960		
5	26.5								
6	31.8	3,180	5,724	3,168	1,584		2,556	972	
7	37.1								
8	42.4	4,452	10,176	4,752	3,168	1,584	5,424	2,256	672
9	47.7								
10	53.3	5,724	15,900	6,336	4,752	3,168	9,564	4,812	1,644
11	53.3								
12	53.3	6,396	22,296	7,920	6,336	4,752	14,376	8,040	3,288
13	53.3								
14	53.3	6,396	28,692	9,504	7,920	6,336	19,188	11,268	4,932
15	53.3	3,678	32,370	10,296	8,712	7,128	22,074	13,362	6,234
16	47.7								
17	42.4	5,724	38,094	11,880	10,296	8,712	26,214	15,918	7,206*
18	37.1								
19	31.8	4,452	42,546	13,464	11,880	10,296	29,082	17,202	6,906
20	26.5								
21	21.2	3,180	45,726	15,048	13,464	11,880	30,678	17,214	5,334
22	15.9								
23	10.6	1,908	47,634	16,632	15,048	13,464	31,002	15,954	2,490
24	5.3								**
25	0	636	48,270	18,216	16,632	15,048	30,054	13,442	

* Peak remaining @ 17 Min. is less than 12,664 CF storage, so this combination of pumps will work satisfactorily.

** All pumps off at approximately 24 minutes elapsed time.

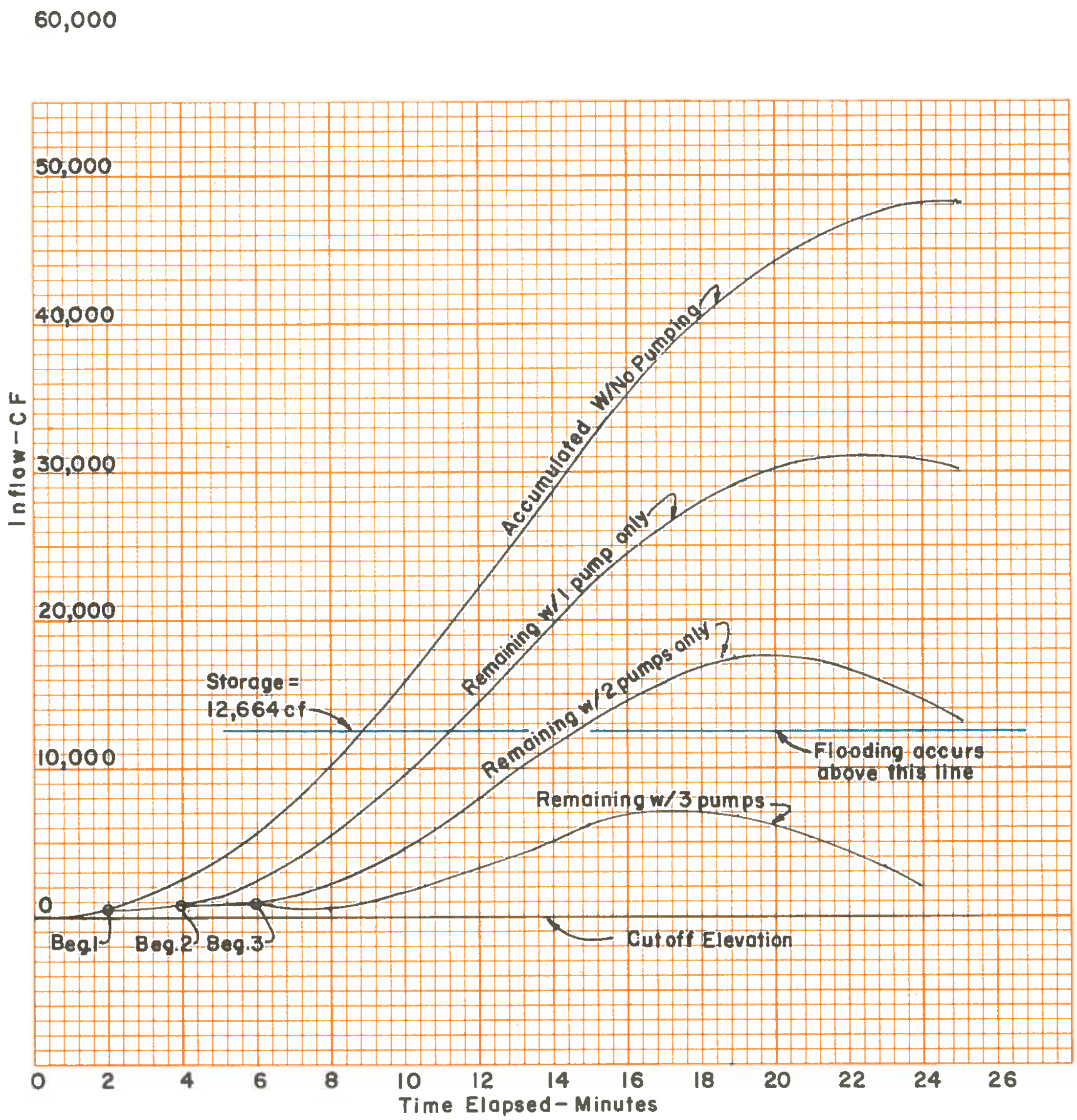


Figure 6

Now proceed to solve for the maximum required average pump capacity as shown in Table II.

It is found that the average pump capacity peaks for a 15 minute duration storm. This example with large storage was chosen to illustrate the storage effect. There will be sites where storage is small and will not offer any adjustment in the peak. When this occurs use average pump capacity = Maximum rate from Step 1. Since pumps are usually rated in gallons per minute, convert several pump sizes to cubic feet per second and select 2 or more pump sizes that will furnish the desired average pump capacity determined from Table II.

The following various pump sizes are arbitarily chosen:

5000 gpm = 11.0 cfs
6000 gpm = 13.2 cfs
7000 gpm = 15.4 cfs
8000 gpm = 17.6 cfs
9000 gpm = 19.8 cfs
10000 gpm = 22.0 cfs

It appears that 3-6000 gpm pumps (39.6 cfs) will satisfy the average pump capacity.

Step 4 - Make an analysis of the proposed pump operation using the selected pump sizes, and the storm duration requiring the largest average pump capacity. Figure 6 and Table III is a method for compiling the necessary information. Figure 6 represents a plot of the pump operation determined in Table III. The inflow rate, Table III, is the rate of inflow in cubic feet per second. The inflow volume is the area under the rate versus time curve (Figures 4 and 5) for the increment of time being considered. The starting time of the pumps can be varied if necessary, and the objective is to arrange the pump sizes and operation so that for the design storm, the pumps will operate such that at no time will the volume in the storage area (below elevation 100.0 feet) be greater than the storage volume determined in Step 2.

Step 5 - Determine the required minimum horsepower for the motors and/or engines to be used.

The following equation is used to determine the minimum horsepower which is based upon 80% efficiency.

$$\text{Req'd Min. Horsepower} = \frac{wQh}{.8 \times 550}$$

where

w = Unit wt. of water = 62.4 lbs/cu. ft.

Q = Quantity of water to be pumped, cubic ft./second

h = Height of lift, feet

550 ft.lbs./sec/hp = conversion factor

HP for 1-6000 Gallon per minute pump

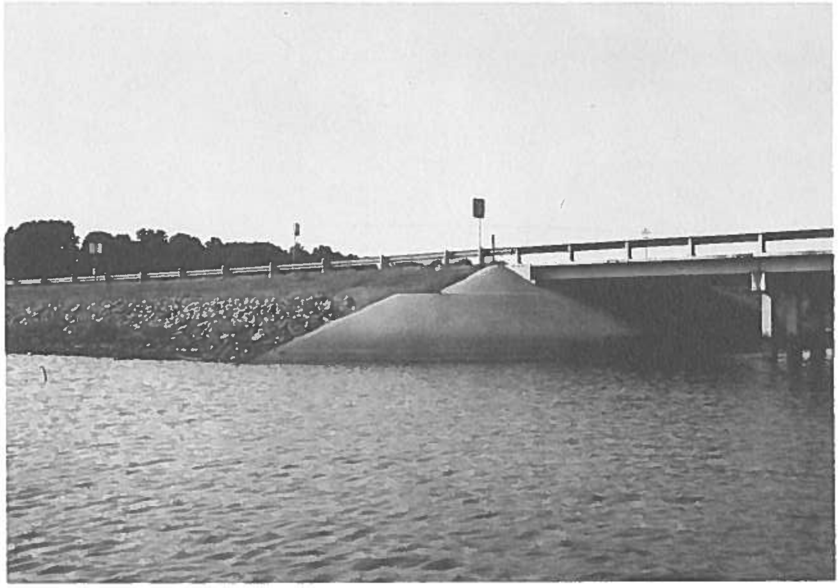
$$HP = \frac{(62.4)\,(13.2)\,(20)}{.8 \times 550} = 37.4$$

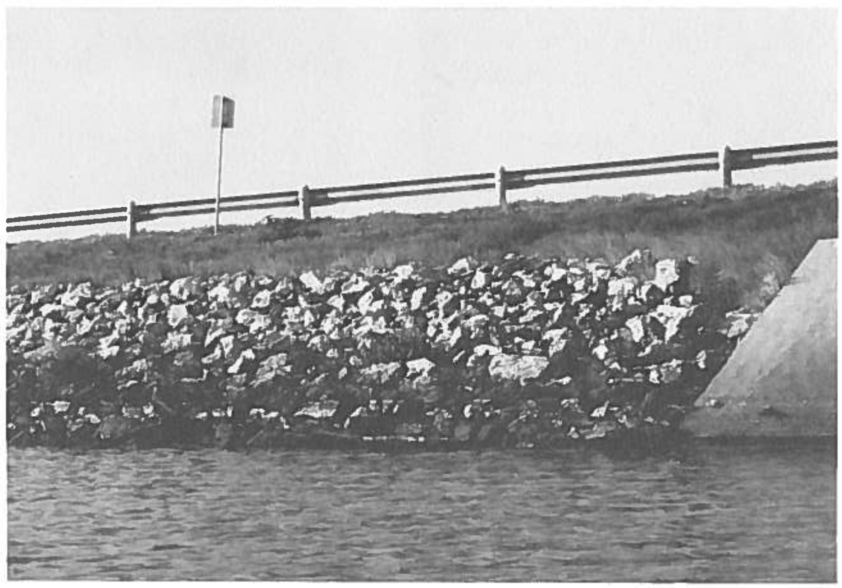
Use 40 Hp. minimum.

It is not intended that 3-6000 gpm are the only combination. There are other combinations that would also operate satisfactorily. This does, however, complete the example problem in which it was determined that 3-6000 gpm pumps driven by 40 horsepower engines and/or motors would maintain the maximum highwater elevation below the allowable highwater elevation chosen.

CHAPTER VIII

RESERVOIRS





RESERVOIR CROSSINGS

8-100. GENERAL

The following is a description of terms and procedures that are to be applied in establishing grade lines for highway embankments, bridges, top of riprap, etc., for highways that cross or border reservoirs.

8-200. ESTABLISHING STRUCTURE LENGTH

For the completed reservoir condition, a minimum structure length shall be established that will accommodate a 50 year design discharge, and the associated 50 year design flood surface elevation at the crossing shall be based upon a calculated backwater curve which begins at the reservoir conservation pool elevation (Figure 1). Velocity determination through the openings shall be based upon waterway area below the above described 50 year flood surface elevation (without wind effects).

The 50 year design discharge, based on the drainage area above the highway site (Figure 2) shall be determined in accordance with Chapter II and the 50 year flood stage shall be determined in accordance with Section 3-204., Chapter III.

Since the structures and embankment may be required to accommodate a 50 year flood prior to impounding water in a new reservoir, highwater and velocity determination should be checked based on the natural conditions. Often natural conditions will produce a greater highwater elevation than when the reservoir is completed. If the facility required for a "state of nature" condition is considerably more costly than that which would be required for the condition for a completed reservoir, then consideration should be given to taking a calculated risk where adjustment of the highway does not precede reservoir impoundment by an unreasonable length of time.

The reservoir engineers are required to furnish the water surface elevations at the various affected highways. However, these water surface elevations required for establishing structure length should be checked in accordance with the procedure outlined in Section 3-204., Chapter III. The cross sections required in this procedure can be obtained from accurate contour maps which are usually available for the proposed reservoir area. In order to satisfactorily check for flood surface elevation and velocity for natural conditions, at least two sets of "n" values are required. One set of "n" values should reflect completed reservoir conditions. Clearing of timber within the reservoir basin may greatly affect these values. This will provide data for obtaining a 50 year flood surface elevation with natural conditions and a 50 year flood surface elevation for completed reservoir conditions. A computer program is available for calculating the above water surface elevations.

8-300. STRUCTURE LOCATION ACROSS FLOOD-PLAIN

Location of the structure or structures will be in accordance with procedures outlined in Chapter V which generally bases the location of the structures upon flow distribution across the channel section. There are cases however, when additional openings in the highway embankment may be needed nearer the borders of the reservoir to insure reservoir circulation. The location and financing of these additional openings are to be the responsibility of the reservoir sponsoring agency.

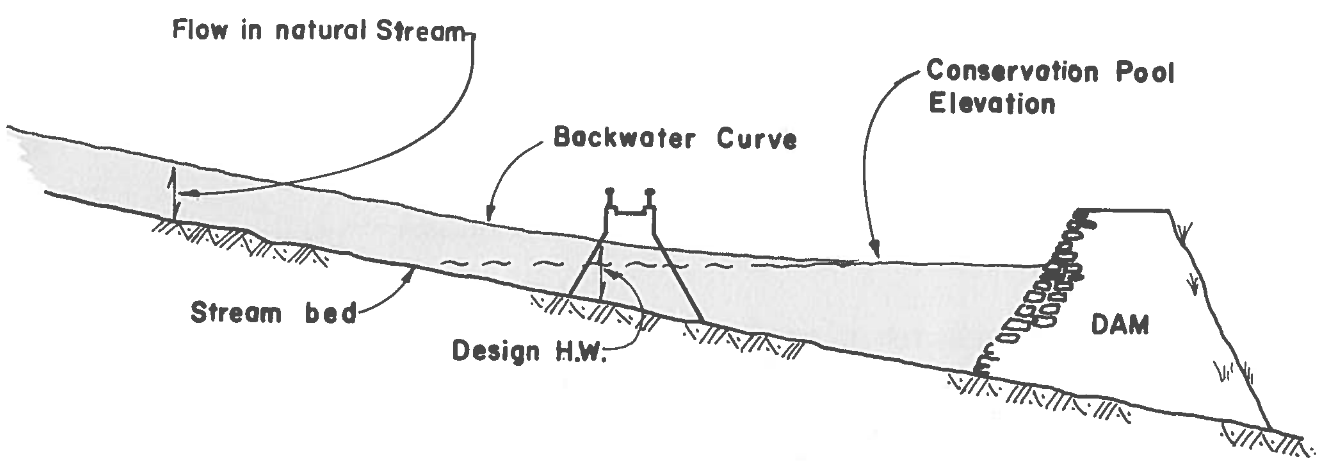


Figure 1

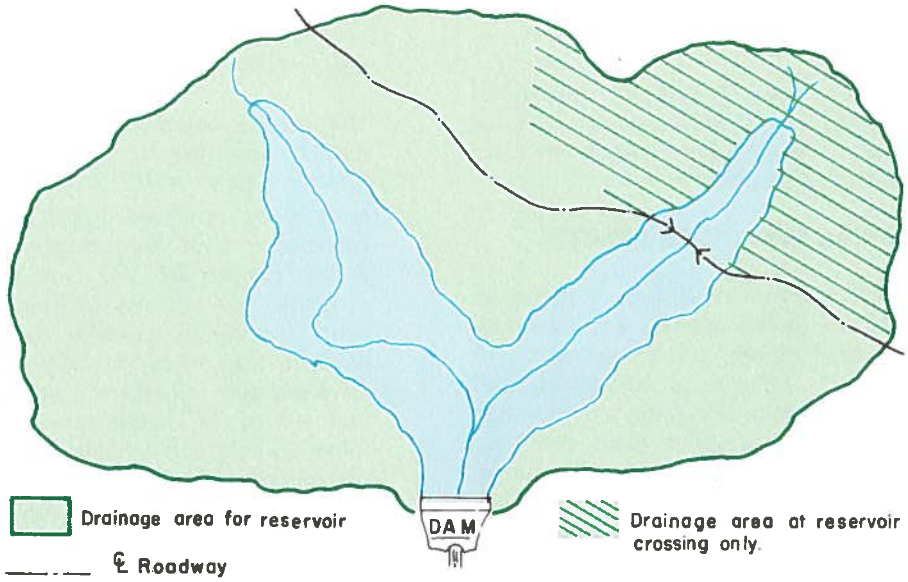


Figure 2

8-400. ECONOMICS

Economics of bridge versus embankment is, as always, a factor to be considered. Cost of embankment (usually ripraped) shall be compared to the cost of a bridge. If the bridge costs are approximately equal to or less than the cost of the embankment, a bridge is more desirable.

Further, after becoming familiar with wave runup and its relationship to various embankment slopes (Figure 5) it can be seen that by flattening the side slope the wave runup is decreased, thereby decreasing the minimum top of riprap elevation, whereas steeper slopes will save embankment quantity but will increase riprap height. Therefore it is necessary to study the various relationships to determine the most economic combination of side slope and minimum top of riprap elevation. The choice of embankment protection will further affect the above relationships and costs. The recommended maximum side slope for application of riprap is 2.5:1.

Additional economic gain might be realized by varying the side slope, i.e., flatter slopes could be provided for the range of elevations in which wave runup might be encountered, and below this range, the slopes could be much steeper.

8-500. CRITERIA FOR MINIMUM TOP OF RIPRAP ELEVATION

As a general criteria for establishing minimum top of riprap elevation, the top of riprap shall be set no lower than the elevation created by the following condition:

> The 50 year reservoir surface elevation for the entire reservoir plus wind effects (wind tide, wave runup) plus a minimum freeboard of 2 feet (Figure 3). The 50 year reservoir surface elevation shall be based upon the entire reservoir watershed (Figure 2) contributing at the dam site. The 50 year reservoir surface elevation will be requested from the reservoir engineers and shall be based upon an inflow hydrograph having a peak rate of inflow approximately equal to the 50 year discharge determined in accordance with Chapter II for a drainage area equal to the total contributing reservoir watershed. The reservoir shall be at conservation pool elevation when the 50 year flood begins. The wind effects will be determined as outlined below.

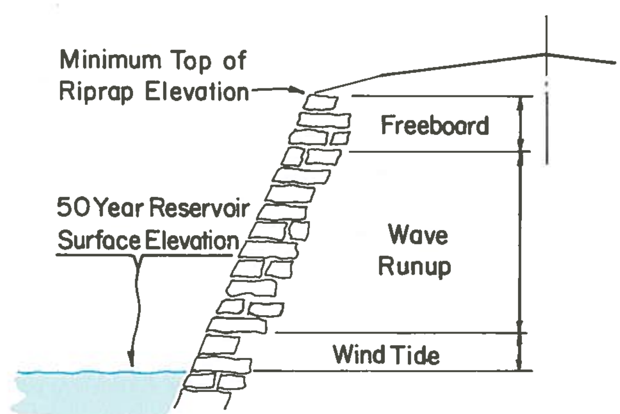


Figure- 3

Since the minimum top of riprap elevation on the upstream and the downstream side can be different, the minimum top of riprap elevation must be individually determined for each side.

8-600. CRITERIA FOR MINIMUM LOW SUPERSTRUCTURE ELEVATION

As a general criteria for bridges, the minimum low superstructure elevation shall be no lower than the highest elevation created by either of the following:

Condition I - The 50 year design flood surface elevation based upon the backwater profile after reservoir is completed, as described in Section 8-200., Establishing Structure Length, plus a minimum freeboard of 2 feet. No wind effects are to be included (Figures 1 and 4).

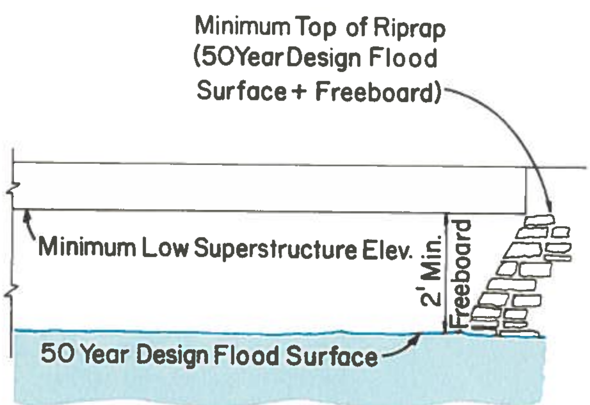


Figure-4

Condition II - The 50 year reservoir surface elevation for the entire reservoir plus wind effects (wind tide, wave height) plus a minimum freeboard of 2 feet (Figure 5). The 50 year reservoir surface elevation is to be based upon the entire reservoir watershed (Figure 2) contributing at the dam site. The 50 year reservoir surface elevation will be requested from the reservoir engineers and shall be based upon an inflow hydrograph having a peak rate of inflow approximately equal to the 50 year discharge determined in accordance with Chapter II for a drainage area equal to the total contributing reservoir watershed. The reservoir shall be at conservation pool elevation when the 50 year flood begins. The wind effects will be determined as outlined below.

If the sponsoring reservoir agency requires a clearance for the boat traffic in excess of the requirements of

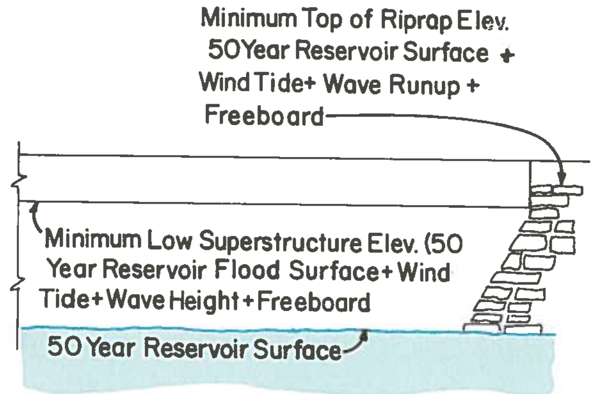


Figure-5

Conditions I and II this is usually provided near the center of the structure, and the low points of the superstructure near the header usually are the critical areas from the standpoint of meeting the criteria for minimum low superstructure elevation (Figures 6a & 6b). Any additional cost required for boat clearances will be borne by the sponsoring reservoir agency.

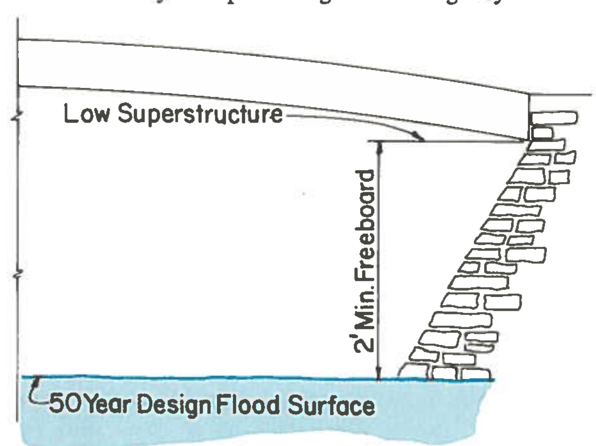


Figure-6 a

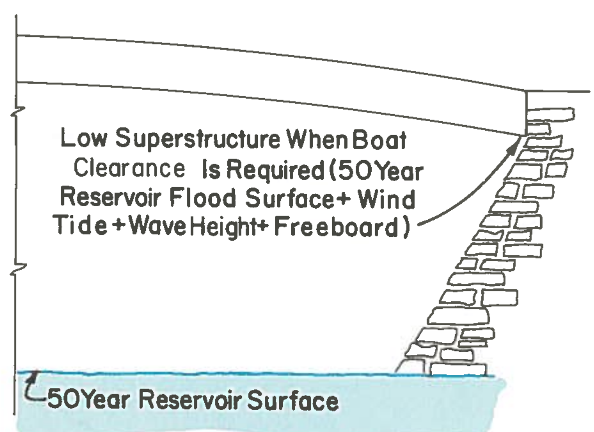


Figure-6 b

SITE LOCATION	Effective fetch Ft. or Mi.	Wave Height H_o Fig. 8 Ft.	Wave Length L_o Fig. 8 Ft.	Avg. Water Depth D 200' from Emb. Ft.	Wave Breaking Height .78x④ .78 D Ft.	Design Wave Height Col. ② or Col. ⑤ Ft.	Wave Steepness ⑥/③	Rough or Smooth Slope	Emb. Slope Ratio	Runup Ratio Fig. 9	Wave Runup ⑥x⑩ Ft.	Wind Tide Fig. 10 Ft.
	①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪	⑫
SH. I U.S.	7.5	5.7	119	6.5	5.1	5.1	0.0429	Rough	3:1	1.21	6.17	0.80
SH I D.S.	3:1	3.8	72	5	3.9	3.8	0.0528	Rough	3:1	1.67	6.30	1.1

TABLE I

8-700. DETERMINING WIND EFFECTS

All wind effects determined herein are based upon a 50 mile per hour over water velocity. The 50 mile per hour wind velocity should be considered minimum and usual. For unusual wind conditions, a modification will be required of the method outlined below.

The U. S. Corps of Engineers' method for determining wind effects on inland reservoirs, as outlined in the publication ETL 1110-2-8, was modified for Texas Highway Department use.

Wind effects must be determined for both the upstream side and the downstream side of the highway crossing.

The procedural order for establishing wind effects follows the numerical order in which the columns appear in Table I. For this reason the appropriate explanations are given under the numerical column heading as follows:

Col. 1 - The effective fetch is the theoretical distance that wind moves over the water surface in the "fetch" direction. Because of varying shoreline and complicated wind patterns, the effective fetch is determined from a series of radials drawn from the crossing site both in the upstream and downstream directions. The assumption is made, for this area of the country, that it is possible to get the 50 mile per hour wind from any direction.

The procedure for determining affective fetch is as follows:

a. Secure an accurate plan of the reservoir with highway locations shown.

b. Place the central point of the transparency (Figure 7a) on the highway centerline that crosses the reservoir and record in tabular form (Table II) the length of each radial from the central point to where the radial touches a land surface (extension of lines may be necessary). Calculate an effective fetch as illustrated by Table II. If another highway crossing lies in the path of the radials, the radials that intersect at the bridge openings would go on through to a land surface while those that intersect the embankment will terminate (Figure 7b).

c. Rotate the radials to another position and repeat step b. Various positions of the central point on the highway centerline as well as various locations rotated about the central point will yield several effective fetch distances. The greater of these is the design effective fetch.

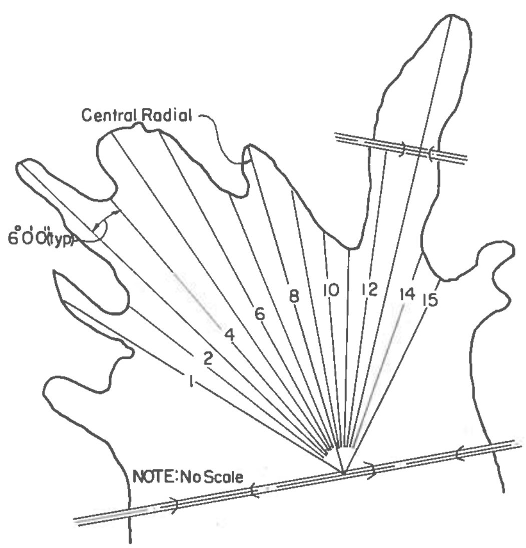


Figure 7b

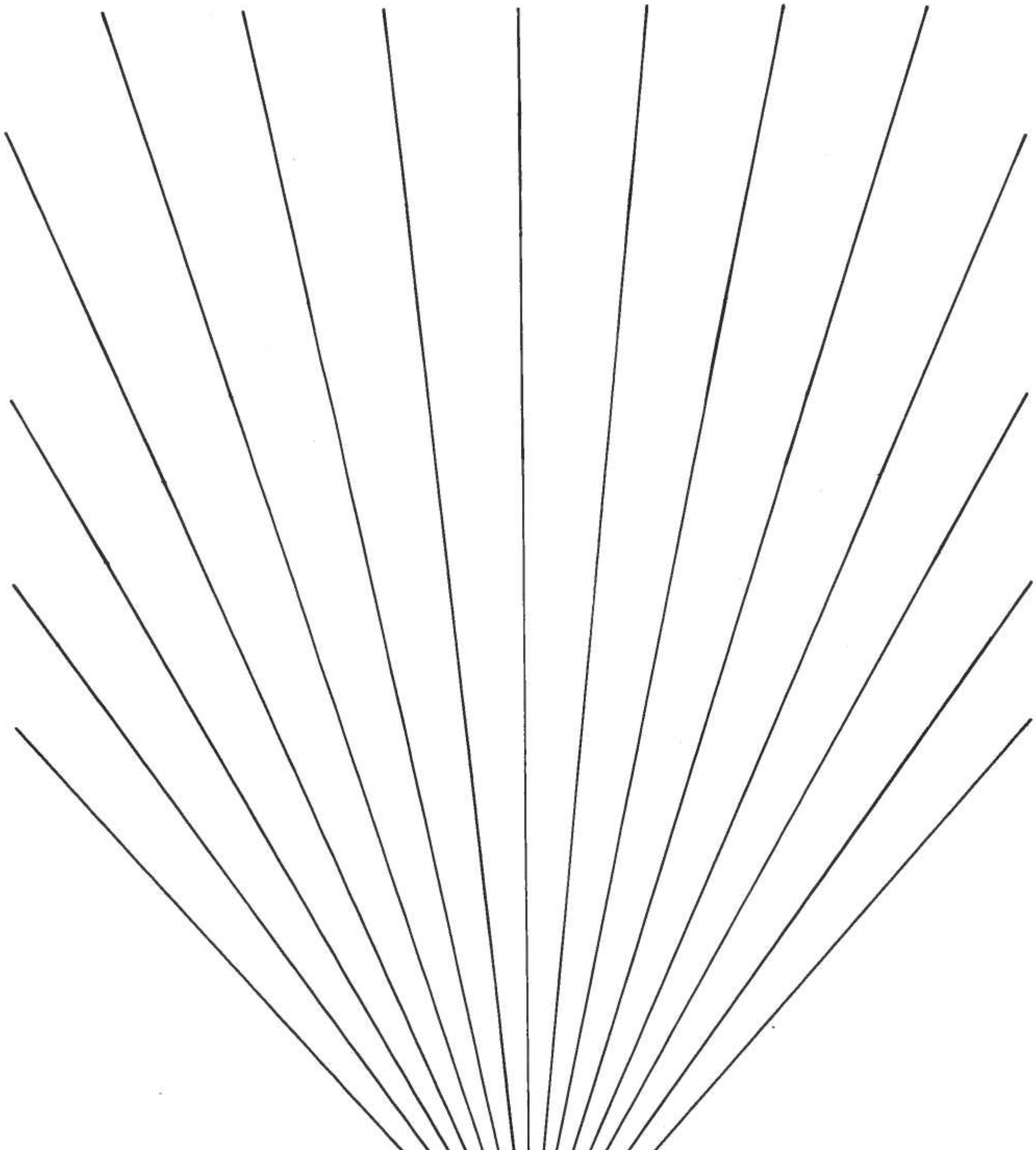


Figure 7a

TABLE II

TABULATION FOR CALCULATING EFFECTIVE FETCH

Radial number	Angle from Central radial	Cosine of Angle	Length of each radial (Miles)	Radial length parallel to central radial (Miles)	Product of (3) x (5)
(1)	(2)	(3)	(4)	(5)	(6)
1	42°	.743	1.33	0.99	0.736
2	36°	.809	2.70	2.18	1.764
3	30°	.866	2.89	2.50	2.165
4	24°	.914	3.22	2.94	2.687
5	18°	.951	6.49	6.17	5.868
6	12°	.978	12.64	12.36	12.088
7	6°	.995	16.95	16.87	16.781
8	0°	1.000	16.71	16.71	16.710
9	6°	.995	12.31	12.25	12.189
10	12°	.978	11.74	11.48	11.227
11	18°	.951	6.39	6.08	5.782
12	24°	.914	5.87	5.37	4.908
13	30°	.866	5.59	4.84	4.191
14	36°	.809	3.46	2.80	2.265
15	42°	.743	3.41	2.53	1.880
		13.512			101.241

Effective fetch $F_{Eff.} = \frac{101.241}{13.512} = 7.5$ miles.

d. Steps b. and c. are to be repeated for the opposite side of the crossing in order to determine the design effective fetch for both upstream and downstream directions.

Col. 2 - Wave heights (H_o) are determined from Figure 8.

Col. 3 - Wave length (L_o) is also determined from Figure 8. This wave length is measured from trough to trough or crest to crest.

Col. 4 - Average water depth (D) approximately 200 feet from highway embankment. D is measured from the bottom of the reservoir to the 50 year reservoir surface plus wind tide.

Col. 5 - Maximum height of wave before breaking. Waves are generated by wind over open water areas. The depth in the generating area is not a parameter for determining the wave height generated; however, as the waves reach areas of less depth closer to the highway embankment, the wave height characteristics tend to change. Theoretically, the maximum wave height cannot exceed 0.78D, where"D" is the depth of water (from the 50 year reservoir surface plus wind tide to the bottom) without wave action. After breaking, the waves tend to reform, with lower heights equal to .78D, within a distance equal to a few wave lengths.

Col. 6 - Design wave height - Either Col. 2 or Col. 5 whichever is less.

Col. 7 - Wave steepness - Design wave height (Col. 6) divided by wave length (Col. 3).

Col. 8 - The embankment slope must be classified rough or smooth in order to determine the proper wave runup. Also, the type of proposed embankment protection must be known. Rock type riprap is considered rough, while soil cement riprap, concrete riprap, etc. is considered as smooth.

Col. 9 - Embankment slope ratio. (2 1/2:1, 3:1, etc.)

Col. 10 - The runup ratio is determined from Figure 9, using the wave steepness (Col. 7) and the embankment slope ratio (Col. 9).

Col. 11 - Wave runup is determined by multiplying the runup ratio (Col. 10) by the design wave height (Col. 6).

Col. 12 - Wind tide or "setup" is the "piling up" of water in the face of a sustained wind blowing across a water surface. Wind tide is determined from Figure10, which is based upon the Zuider Zee formula developed by Dutch engineers. Note that the depth for determining wind tide is the average depth along the wind tide fetch line. For direction of the wind tide fetch line, draw a line outward from the highway crossing, in the direction being investigated, that would represent the maximum single fetch distance (independent of radials used for determining effective fetch). The length of the wind tide fetch is twice the calculated effective fetch.

8-800. RIPRAP

For highway embankments crossing reservoirs, it is the usual practice to provide embankment protection. The design thickness for the three most commonly used types of embankment protection are reviewed below. The determination of the minimum top of riprap elevation is covered elsewhere in this chapter.

8-801. ROCK RIPRAP

This type of riprap consists of loose rock, varying in size depending on design, placed on a bedding usually not less than 6 inches in thickness (Figure 11). The rock riprap thickness may be determined from the nomograph shown in Figure 12. The nomograph was developed by the U. S. Corps of Engineers.

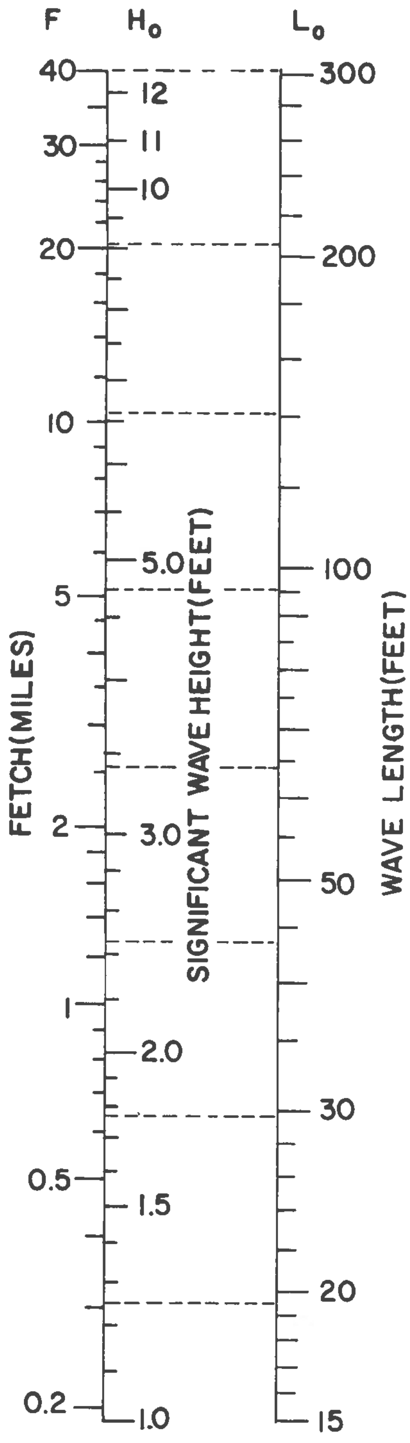
8-802. SOIL CEMENT RIPRAP

Soil Cement riprap has been successfully and economically used where the proper materials are available locally. It is installed in horizontal layers usually 6 inches in thickness and a usual width of 8 feet (Figure 13).

One advantage to soil cement riprap is the fact that it can be incorporated as a part of the embankment, which in turn reduces the amount of embankment material that would ordinarily be required when riprap is used.

8-803. CONCRETE RIPRAP

This consists of a reinforced concrete blanket on the side slopes of the highway embankments. Concrete riprap has a usual thickness of 5 inches. Concrete riprap, although less commonly used than rock riprap or soil cement riprap, has its place, in certain cases, as an effective means of protecting the embankment.



To use: Hold straight-edge horizontally at appropriate value for fetch. Read H_o, and L_o directly.

Example: Fetch = 7.5 Mi.
Read: H_o = 5.7 ft.
L_o = 119 ft

NOTE: H_o and L_o are based on Exhibit 3 and 4 respectively, U.S. Engr. E.T.L. 1110-2-8, using a constant 50 MPH over water wind velocity.

WAVE LENGTH AND HEIGHT VS. FETCH

(Wind velocity = 50 M.P.H.)

Figure-8

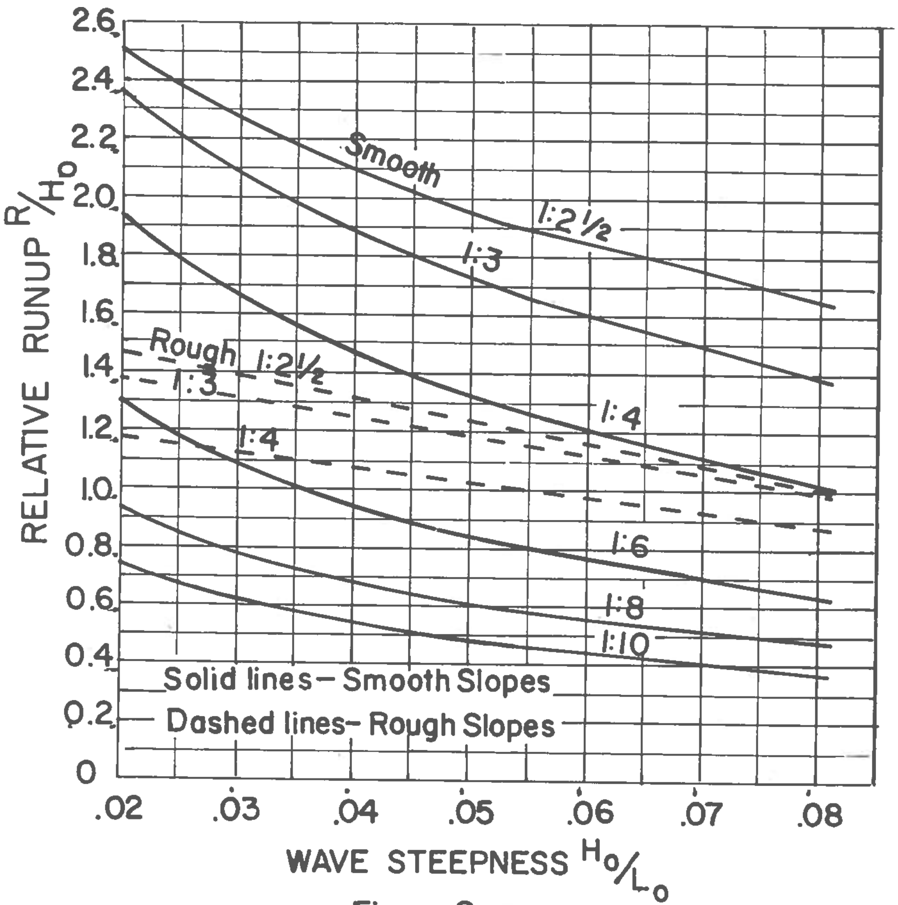
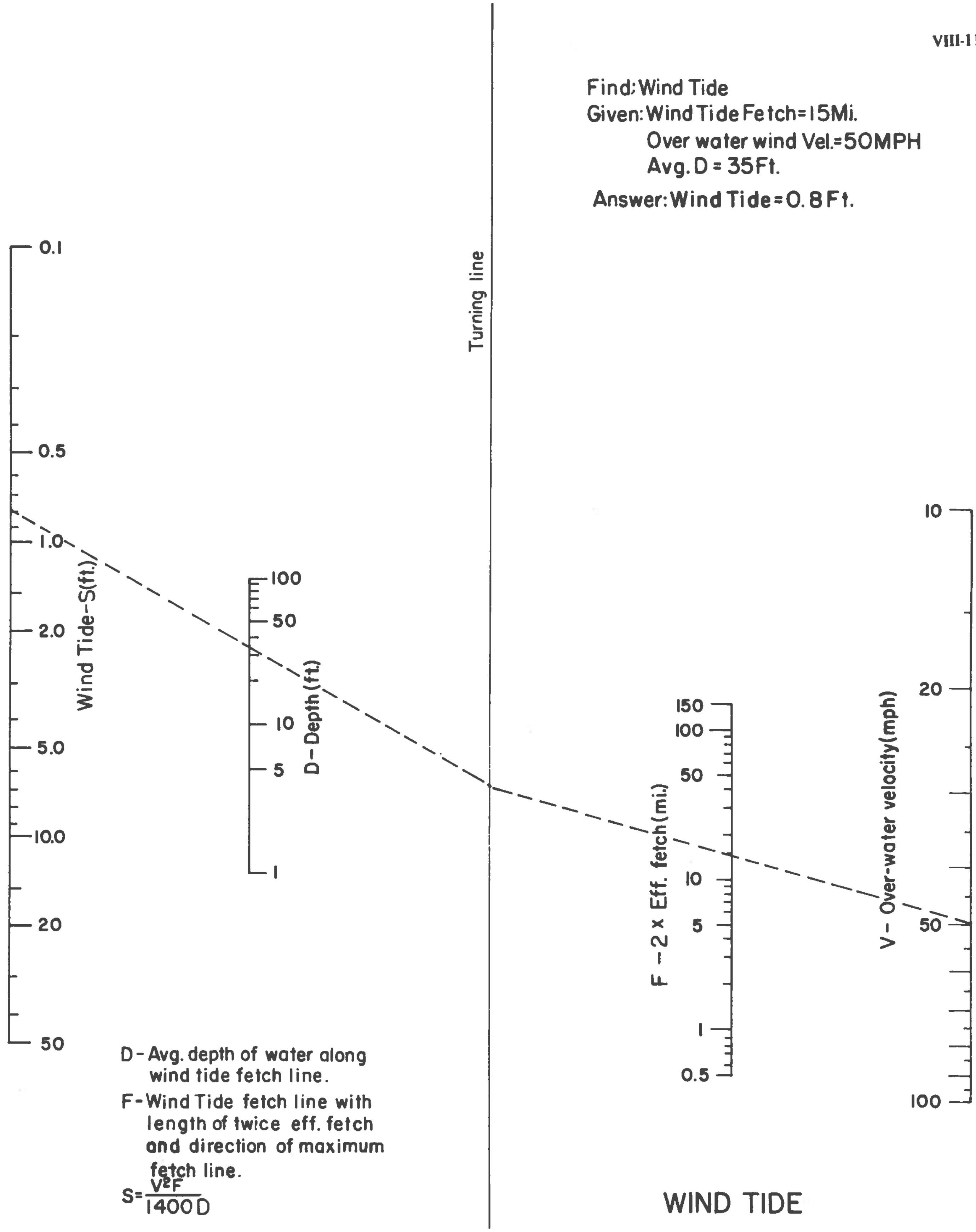

2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
RELATIVE RUNUP R/Ho
Smooth
1:2 1/2
1:3
Rough 1:2 1/2
1:3
1:4
1:4
1:6
1:8
1:10
Solid lines – Smooth Slopes
Dashed lines – Rough Slopes
.02
.03
.04
.05
.06
.07
.08
WAVE STEEPNESS Ho/Lo

Figure 9

Find: Wind Tide
Given: Wind Tide Fetch = 15 Mi.
Over water wind Vel. = 50 MPH
Avg. D = 35 Ft.
Answer: Wind Tide = 0.8 Ft.



D - Avg. depth of water along wind tide fetch line.

F - Wind Tide fetch line with length of twice eff. fetch and direction of maximum fetch line.

$$S = \frac{V^2F}{1400D}$$

WIND TIDE

Figure 10

EXAMPLE:

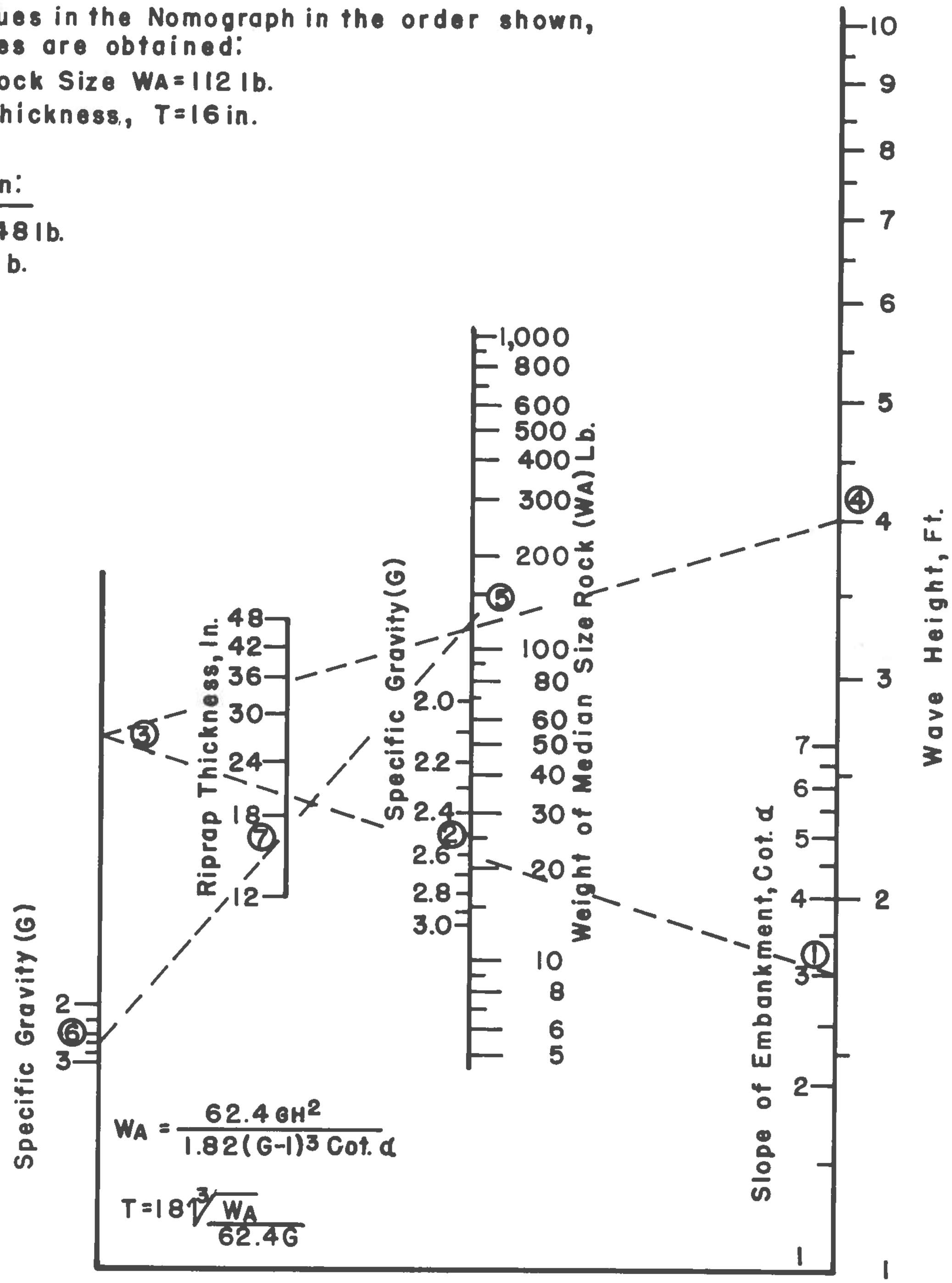
GIVEN: 1 Embankment Slope, Cot. α=3
2, 6 Specific Gravity of Rock, G=2.6
4 Wave Height, H=4 ft.

Entering these values in the Nomograph in the order shown, the following values are obtained:

5 Median Rock Size W_A=112 lb.
7 Riprap Thickness, T=16 in.

Riprap Graduation:

$W_{max.} = 4\,W_A = 448$ lb.
$W_{min.} = \frac{W_A}{8} = 14$ lb.



Corps of Engineers

THICKNESS OF ROCK RIPRAP

FIGURE 12

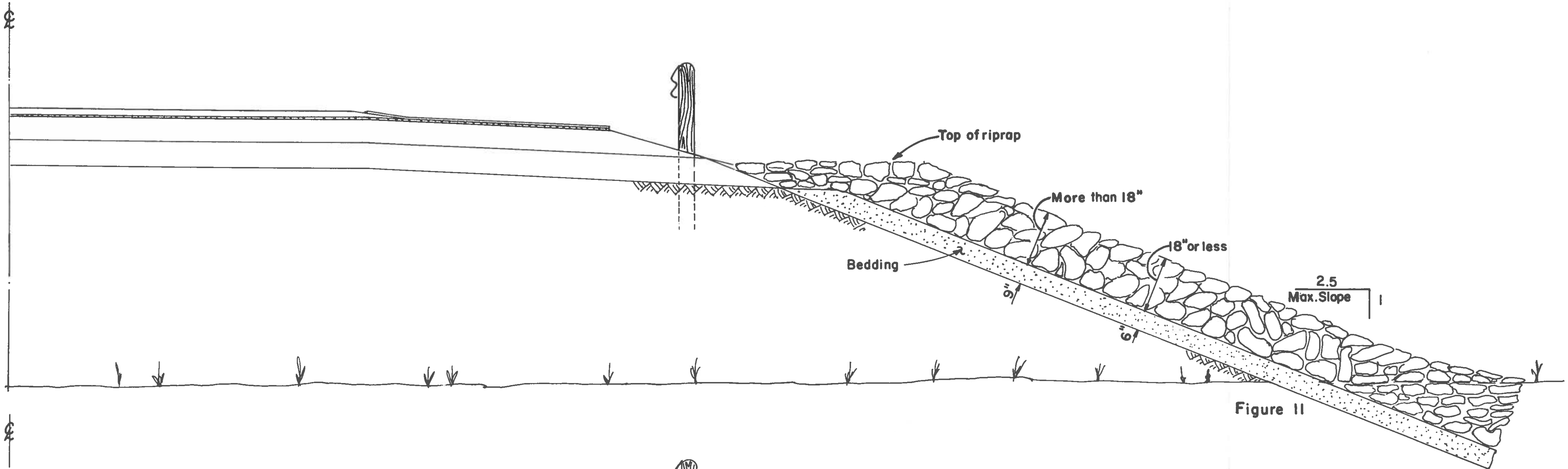


Figure 11

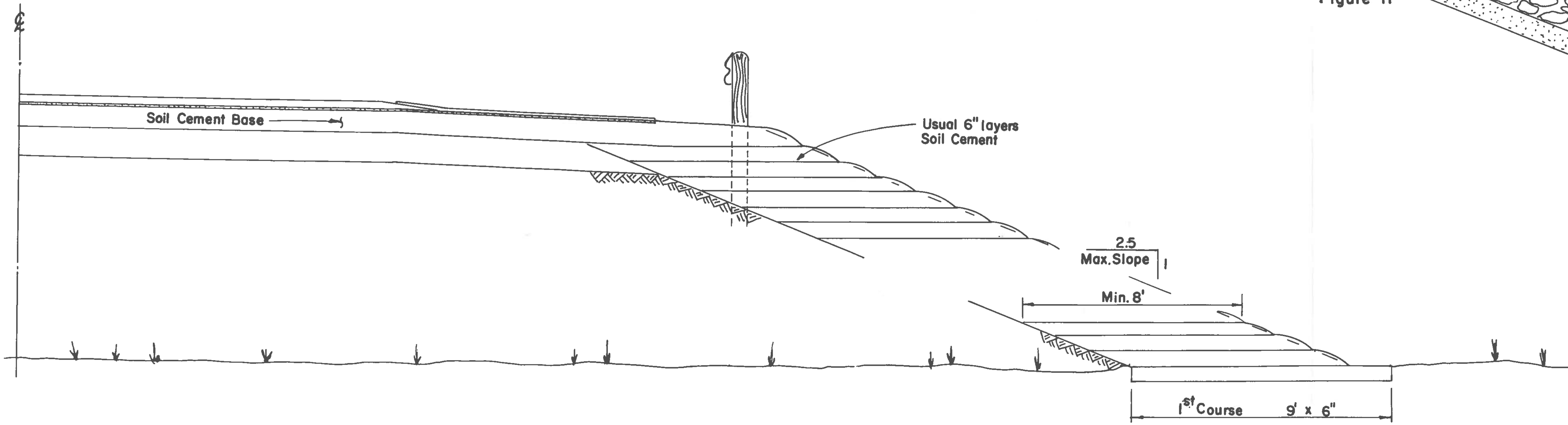


Figure 13

CHAPTER IX

PIPE STRENGTH

9-100 GENERAL

This chapter comprises a general discussion on pipe culvert strengths. Also, there are tables included for determining design loads for concrete pipe, and wall thickness for metal pipe. The tables are based upon fill heights.

9-200 METAL PIPE

Metal pipe strength is a function of the metal wall thickness or gage of metal required to support a certain fill height over the pipe. Tables I, II, III and IV directly provide the required gages for different fill heights and for various corrugations of aluminum and steel full-circle pipe. Fill height tables are not provided for field assembled structures such as structural plate pipes, special structural plate shapes, and long-span structural plate structures. The design of the field assembled structures is not readily represented by tables; therefore, D-5 will provide design information upon request.

The fill height tables for metal full-circle pipe are based on a soil weight of 120 pounds per cubic foot. The tables are also based upon full-circle steel and aluminum pipes that have either riveted seam, lock seam, or welded seam, except for the 5″×1″ (125mm × 25mm) corrugated steel pipe. The 5″×1″ (125mm × 25mm) gages shown in the fill height table (Table II) are based upon *only* lock seam or welded seam construction, *not* riveted seam. For 5″×1″ (125mm × 25mm) pipe with riveted seams, gages must be computed. A maximum allowable deflection, measured along the vertical axis, of the pipe is 5% ± of the full circle diameter. This 5% ± deflection will also be allowed during construction but shall not be exceeded.

Steel and aluminum pipes are considered to be flexible conduits, and the required earth loads and wall thicknesses for corresponding fill heights are determined in accordance with the appropriate sections of the AASHTO Standard Specifications for Highway Bridges.

9-300 CONCRETE PIPE

Concrete pipe is a rigid conduit consisting of both concrete and reinforcing steel and therefore the final design of the pipe walls is not specified on the plans. The required strength of the concrete pipe is specified on the plans by giving the D-load that the pipe will be required to support in the test for acceptance. With this designated loading the manufacturer can determine the most economical structural design of the pipe walls and reinforcement which comply with the applicable ASTM specification.

The D-load is defined as the load on the concrete pipe in pounds per linear foot of pipe per foot of diameter. The D-load is written as a number followed by (-D), i.e., 1350-D. It is written in this manner because 1350 is *not* the *total* load, but only the load per foot of pipe length per foot of diameter. The 1350 must be multiplied by the pipe diameter (in feet) to obtain the total loading per foot of pipe length.

The design load (D-load) for concrete pipe is given in Tables V, VI, VII, and VIII, and is primarily dependent on soil weight, the height of fill above the pipe, the installation conditions, trench widths, bedding and live loads. The soil weight used for preparing the tables is 120 pounds per cubic foot. Live loads are determined using current AASHTO methods, and the design loads for the various pipe diameters and corresponding fill heights are based upon the American Concrete Pipe Association Design Manual (Rev. 1978).

There are four basic installation conditions for concrete pipe. Each are represented in Figure 1. The installation conditions usually encountered in highway construction are trench condition, and positive projection condition. In order to establish trench conditions the minimum trench shapes must conform to those shown in Figure 2.

The negative projection condition and the induced trench condition both represent conditions which are ordinarily artificially created, and are therefore more costly than the two more common installations. In fact, induced trench and negative projection normally become cost-effective only when fill heights approach 30 feet. D-5 can assist if it appears there might be some advantage to creating induced trench or negative projection conditions.

Bedding is very influential in determining the required pipe strength. The four recognized classes of bedding are depicted in Figure 3. The most common classes of bedding are Class B and Class C. Class C is the most economical and Class A the most expensive. However, for a given fill height, Class A bedding will require the lowest pipe strength, and Class C the greatest strength. The selection of the bedding should be based upon designing the most cost-effective facility.

The Department's specifications have for many years specified certain excavation widths for measurement and payment. The widths specified are: Outside diameter (O.D.) +2′ for 42″ and less inside diameter (I.D.), and O.D. +4′ for greater than 42″ I.D. These specification widths were used in the preparation of the D-Load Tables. For the trench widths to be effective under field conditions, the trench wall heights must be in accordance with those shown in Figure 2. Positive projection conditions need not conform to any specified width.

If specific site parameters vary by more than 5% ± from the assumptions on which the tables are based, earth loads should be calculated; otherwise, the tables will suffice. File D-5 will assist when requested to do so.

When the required pipe strength exceeds a D-load of 3000-D, the structural design of the pipe can fall in a special design category. This alone can increase the cost because such pipe is not usually a standard stock item. Often, with high strength pipe, refinement of parameters such as bedding, soil weight, and/or trench width is warranted, because the final computed D-loads are much more sensitive to these parameters than when lesser D-loads are encountered. In this area of design even the use of Class A bedding may prove to be cost-effective. The concrete pipe manufacturer should be contacted to assist with estimates for the various design alternatives when earth loads exceed 3000-D.

It is recommended that pipe strengths be specified as indicated below to reduce the number of bid items and simplify the administration of a project:

For D-loads Up to 800-D Use	800-D
Above 800-D to 1000-D	1000-D
Above 1000-D to 1350-D	1350-D
Above 1350-D to 1700-D	1700-D
Above 1700-D to 2000-D	2000-D
Above 2000-D	500-D Increments

The actual computed D-load may be specified for bidding purposes, without adhering to the increments above, but this should occur only when there is a sufficient quantity of a pipe size to warrant the manufacture of a specific D-load. Because conditions vary across the State, the manufacturers should be contacted to confirm any suspected advantage.

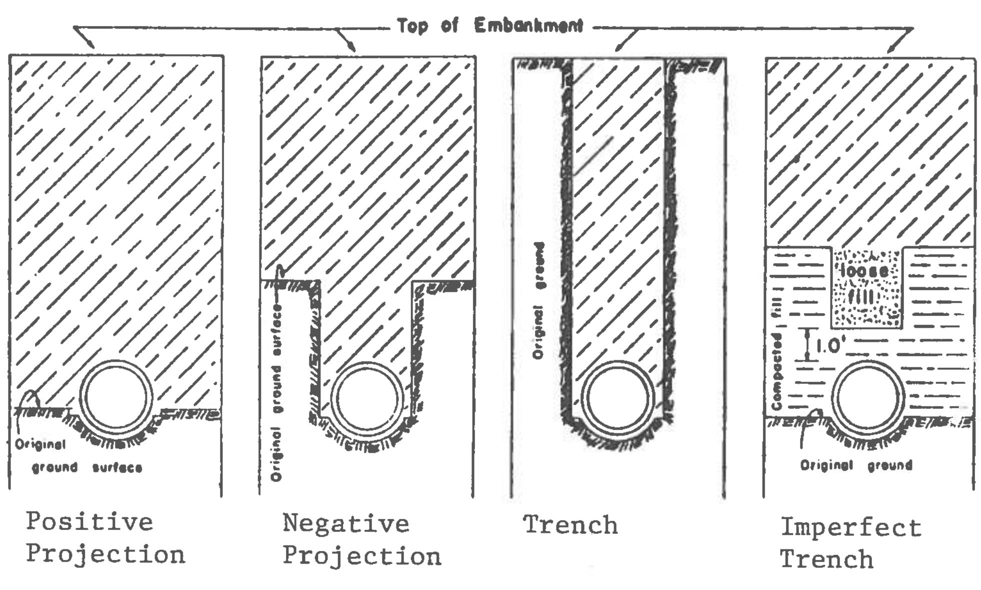
9-400 STRENGTHS FOR JACKED PIPE

For jacked pipe there are two loads to be aware of when establishing strength requirements. One is the axial or thrust load caused by the jacking forces applied during construction and the load due to earth cover and live loads.

The earth and live loads have already been covered in this chapter.

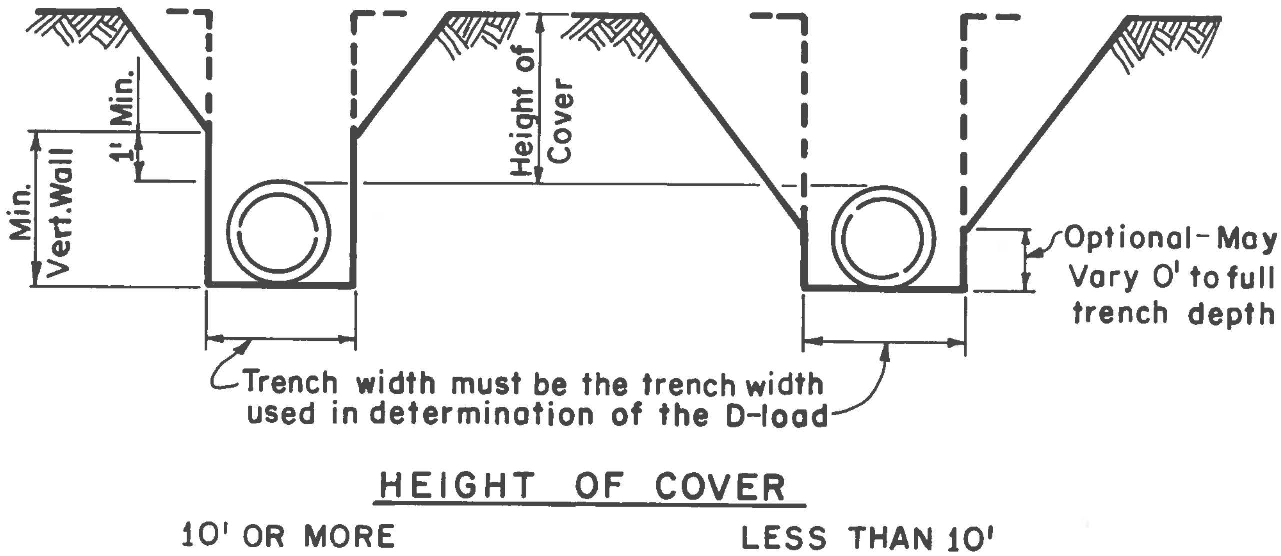
For axial loads, the cross-sectional area of a standard concrete pipe wall is adequate to resist stresses encountered in normal jacking operations, if certain construction techniques are followed. To prevent localized stress concentrations, it is necessary to provide relatively uniform distribution of the axial loads around the periphery of the pipe. This requires that the pipe ends be parallel and square for uniform contact, and that the jacking assembly be arranged so that the jacking forces are exerted parallel to the pipe axis. If excessive jacking pressures are anticipated due to long jacking distances, intermediate jacking stations should be provided.

In summary the strengths required for earth and live loads are more than adequate to withstand the axial loads of a normal jacking operation if the above requirements are met.



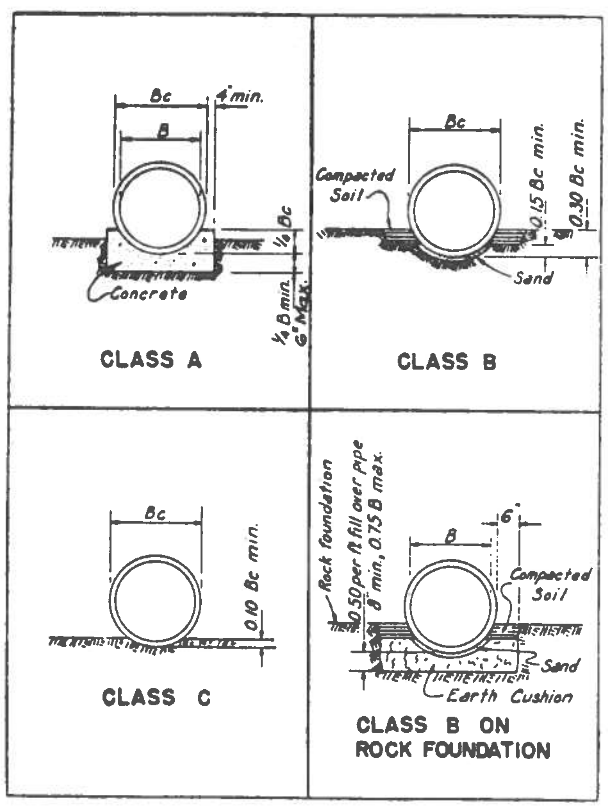
Installation Conditions

Figure 1



PERMISSIBLE TRENCH SHAPES FOR TRENCH CONDITION

FIGURE 2



Bedding Conditions

Figure 3

Corrugated Steel
2 2/3" x 1/2" Corrugations
Full Circle Pipe

Pipe Diameter	Min. Cover, Top of Pipe to Top of Subgrade	Thickness 0.064 In. 16 Gage	0.079 In. 14 Gage	0.109 In. 12 Gage	0.138 In. 10 Gage	0.168 In. 8 Gage
Inches		Maximum Fill Height Above Top of Pipe in Feet*				
12	12	112	122	157	164	171
15	12	90	97	125	131	137
18	12	75	81	104	109	114
21	12	64	70	82	94	98
24	12	56	60	69	78	86
27	12	50	54	60	67	73
30	12	49	51	55	59	65
33	12	41	44	52	55	59
36	12	37	41	49	52	55
42	12	41	45(57)	46(90)	49(94)	50(98)
48	12	-	43(50)	44(78)	46(82)	47(86)
54	12	-	43	43(70)	44(73)	45(76)
60	12	-	-	43(63)	43(66)	44(69)
66	12	-	-	-	43(60)	43(62)
72	12	-	-	-	43(55)	43(57)
78	12	-	-	-	-	42(53)
84	12	-	-	-	-	42(49)

* Fill hts. in ( ) apply when pipe is 5% vertically elongated prior to installation.

Note: Thickness in inches, as shown above, refers to the coated metal after galvanizing.

The bold line in the gage columns is drawn below the maximum diameter allowed for that gage.

TABLE I

Corrugated Steel
3" x 1" or 5" x 1"
Full Circle Pipe

Pipe Diameter	Min. Cover, Top of Pipe to Top of Subgrade	Thickness 0.064 In. 16 Gage	0.079 In. 14 Gage	0.109 In. 12 Gage	0.138 In. 10 Gage	0.168 In. 8 Gage
Inches		Maximum Fill Height Above Top of Pipe in Feet*				
48	12	48	52(60)	56(89)	61(107)	66(118)
54	12	43	48(53)	52(79)	55(95)	58(105)
60	12	38	46(48)	49(71)	51(85)	53(95)
66	12	35	43	47(65)	48(78)	50(86)
72	12	32	40	45(59)	47(71)	48(79)
78	12	29	36	44(54)	45(66)	46(73)
84	12	-	34	43(51)	44(61)	45(68)
90	12	-	32	43	44(57)	44(63)
96	12	-	30	43	43(53)	44(59)
102	24	-	28	42	43(50)	43(56)
108	24	-	-	39	42	43(53)
114	24	-	-	37	42	42(50)
120	24	-	-	35	42	42

* Fill hts. in ( ) apply when pipe is 5% vertically elongated prior to installation.

Note: Thickness in inches, as shown above, refers to the coated metal <u>after</u> galvanizing.

The bold line in the gage columns is drawn below the maximum diameter allowed for that gage.

TABLE II

Corrugated Aluminum
2 2/3" x 1/2" Corrugations
Full Circle Pipe

Pipe Diameter	Min. Cover, Top of Pipe to Top of Subgrade	Thickness 0.060 In. 16 Gage	0.075 In. 14 Gage	0.105 In. 12 Gage	0.135 In. 10 Gage	0.164 In. 8 Gage
Inches		Maximum Fill Height Above Top of Pipe in Feet				
12	12	45	45	77	-	-
15	12	36	37	56	-	-
18	12	28	30	36	43	49
24	12	22	23	25	28	31
27	12	20	21	23	25	27
30	12	18	18	21	23	24
33	12	16	17	20	21	22
36	12	-	15	19	20	21
42	12	-	-	19	19	20
48	15	-	-	18	18	19
54	16	-	-	17	18	18
60	16	-	-	-	16	18
66	18	-	-	-	12	15
72	20	-	-	-	18	11
78	20	-	-	-	-	9
84	20	-	-	-	-	7

Note: Thickness in inches, as shown above, refers to the clad sheet.

The bold line in the gage columns is drawn below the maximum diameter allowed for that gage.

TABLE III

Corrugated Aluminum
3" x 1" or 6" x 1" Corrugations
Full Circle Pipe

Pipe Diameter	Min. Cover, Top of Pipe to Top of Subgrade	Thickness 0.060 In. 16 Gage	0.075 In. 14 Gage	0.105 In. 12 Gage	0.135 In. 10 Gage	0.164 In. 8 Gage
Inches		Maximum Fill Height Above Top of Pipe in Feet				
30	12	27	30	35	41	46
36	12	23	24	28	30	34
42	14	21	22	24	26	28
48	16	20	20	22	23	24
54	18	19	19	20	21	22
60	20	<u>18</u>	19	20	20	21
66	22	-	18	19	19	20
72	24	-	<u>17</u>	19	19	19
78	26	-	-	18	18	19
84	26	-	-	<u>15</u>	18	18
90	24	-	-	-	16	18
96	24	-	-	-	<u>14</u>	16
102	30	-	-	-	-	14
108	30	-	-	-	-	12
114	30	-	-	-	-	10
120	30	-	-	-	-	8

Note: Thickness in inches, as shown above, refers to the clad sheet.

The bold line in the gage columns is drawn below the maximum diameter allowed for that gage.

TABLE IV

D-LOADS FOR
REINFORCED CONCRETE PIPE
POSITIVE PROJECTION
CLASS C BEDDING

		FILL HEIGHT, FEET																		
DIAM	B_c*	.5-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	10	12	14	16	18	20	25	30	35	40	45	50
12	1.33 FT	2556	1473	936	800	800	850	942	1040	1170	1407	1644	1881	2118	2355	2947	3539	4131	4723	5315	5907
15	1.63 FT	2294	1339	878	800	800	825	914	1008	1138	1369	1600	1831	2062	2293	2870	3448	4025	4602	5179	5757
18	1.92 FT	1857	1235	831	800	800	803	890	986	1115	1343	1570	1797	2024	2251	2818	3386	3953	4521	5088	5656
21	2.21 FT	1521	1146	800	800	800	800	876	969	1099	1323	1547	1771	1996	2220	2780	3341	3901	4462	5022	5582
24	2.50 FT	1393	1072	800	800	800	800	861	956	1085	1307	1530	1752	1974	2196	2751	3306	3862	4417	4972	5527
27	2.79 FT	1174	937	800	800	800	800	849	942	1074	1295	1515	1736	1956	2177	2728	3279	3830	4381	4932	5483
30	3.08 FT	988	818	800	800	800	800	839	933	1065	1284	1503	1723	1942	2161	2709	3257	3804	4352	4900	5448
33	3.38 FT	843	800	800	800	800	800	829	925	1056	1275	1493	1711	1929	2147	2693	3238	3783	4328	4873	5418
36	3.67 FT	800	800	800	800	800	800	821	915	1049	1266	1484	1701	1918	2136	2679	3222	3764	4307	4850	5393
42	4.25 FT	800	800	800	800	800	800	805	900	1035	1252	1468	1684	1900	2116	2656	3195	3734	4274	4813	5352
48	4.83 FT	800	800	800	800	800	800	800	884	1023	1239	1455	1670	1885	2100	2637	3174	3710	4247	4784	5321
54	5.42 FT	800	800	800	800	800	800	800	853	1012	1228	1443	1657	1871	2086	2621	3156	3691	4225	4760	5295
60	6.00 FT	800	800	800	800	800	800	800	818	1001	1217	1432	1646	1860	2073	2607	3140	3674	4207	4740	5273
66	6.58 FT	800	800	800	800	800	800	800	800	975	1206	1421	1635	1849	2062	2594	3127	3659	4190	4722	5254
72	7.17 FT	800	800	800	800	800	800	800	800	941	1196	1411	1625	1838	2051	2583	3114	3645	4176	4707	5237
78	7.75 FT	800	800	800	800	800	800	800	800	913	1175	1401	1615	1828	2041	2572	3103	3633	4163	4692	5222
84	8.33 FT	800	800	800	800	800	800	800	800	888	1139	1391	1606	1819	2032	2562	3092	3621	4151	4680	5209
90	8.92 FT	800	800	800	800	800	800	800	800	867	1109	1375	1596	1810	2022	2553	3082	3611	4139	4668	5196
96	9.50 FT	800	800	800	800	800	800	800	800	849	1082	1338	1586	1800	2013	2543	3072	3601	4129	4656	5184
102	10.08 FT	800	800	800	800	800	800	800	800	832	1059	1306	1575	1791	2004	2534	3063	3591	4119	4646	5173
108	10.67 FT	800	800	800	800	800	800	800	800	818	1038	1278	1537	1781	1995	2525	3054	3582	4109	4636	5163
114	11.25 FT	800	800	800	800	800	800	800	800	804	1020	1253	1504	1771	1986	2517	3045	3573	4100	4626	5153
120	11.83 FT	800	800	800	800	800	800	800	800	800	1003	1230	1474	1736	1976	2508	3037	3564	4091	4617	5143

* B_c is the outside diameter of the full circle pipe based upon ASTM C-76 wall B.

Note 1 - When D-loads are greater than 3000-D, consult the text, Sec. 9-300.

Note 2 - Minimum recommended cover is 12".

TABLE V

D-LOADS FOR
REINFORCED CONCRETE PIPE
TRENCH CONDITION
CLASS C BEDDING

FILL HEIGHT, FEET

DIAM	B_d*	.5-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8 **	10	12	14	16	18	20	25	30	35	40	45	50
12	3.33 FT	2577	1510	1008	853	945	1057	1182	1314	1512	1734	2009	2297	2579	2700	2932	3089	3196	3268	3317	3350
15	3.63 FT	2317	1376	949	822	917	1028	1150	1277	1431	1675	1956	2208	2345	2464	2696	2858	2972	3051	3106	3145
18	3.92 FT	1880	1273	900	801	894	1004	1123	1256	1369	1645	1904	2059	2194	2313	2548	2717	2839	2926	2988	3033
21	4.21 FT	1546	1185	859	800	876	986	1110	1234	1350	1623	1803	1955	2090	2209	2449	2626	2756	2851	2921	2972
24	4.50 FT	1419	1111	827	800	863	972	1093	1218	1335	1558	1728	1879	2014	2134	2381	2565	2703	2806	2884	2942
27	4.79 FT	1201	978	800	800	854	961	1078	1202	1315	1503	1671	1822	1958	2079	2331	2523	2670	2781	2867	2932
30	5.08 FT	1016	860	800	800	835	957	1066	1190	1273	1459	1626	1777	1914	2037	2295	2495	2649	2769	2862	2934
33	5.38 FT	872	814	800	800	817	941	1062	1187	1240	1423	1590	1742	1879	2004	2268	2475	2638	2766	2866	2945
36	5.67 FT	800	800	800	800	800	932	1052	1176	1212	1394	1561	1713	1851	1978	2248	2462	2632	2768	2876	2961
42	6.25 FT	800	800	800	800	800	888	1034	1159	1169	1350	1516	1669	1810	1940	2220	2448	2633	2784	2906	3005
48	8.83 FT	800	800	800	800	800	855	998	1146	1347	1624	1853	2058	2320	2582	3127	3520	3860	4153	4406	4624
54	9.42 FT	800	800	800	800	800	828	963	1114	1336	1589	1786	2046	2307	2568	3023	3415	3756	4054	4313	4539
60	10.00 FT	800	800	800	800	800	806	934	1078	1326	1538	1733	2035	2295	2495	2941	3333	3677	3979	4244	4477
66	10.58 FT	800	800	800	800	800	800	913	1048	1291	1498	1763	2024	2239	2432	2876	3268	3614	3921	4192	4432
72	11.17 FT	800	800	800	800	800	800	894	1024	1260	1464	1743	1989	2189	2381	2822	3214	3564	3875	4152	4398
78	11.75 FT	800	800	800	800	800	800	878	1004	1234	1436	1704	1949	2147	2337	2777	3171	3523	3838	4121	4374
84	12.33 FT	800	800	800	800	800	800	864	987	1210	1411	1672	1914	2111	2300	2739	3134	3489	3809	4097	4356
90	12.92 FT	800	800	800	800	800	800	853	972	1189	1391	1644	1885	2080	2268	2707	3103	3461	3785	4078	4343
96	13.50 FT	800	800	800	800	800	800	842	960	1171	1372	1621	1859	2054	2241	2679	3076	3437	3765	4063	4334
102	14.08 FT	800	800	800	800	800	800	833	949	1155	1357	1600	1836	2030	2217	2654	3053	3417	3749	4051	4327
108	14.67 FT	800	800	800	800	800	800	824	939	1141	1343	1582	1816	2009	2195	2633	3033	3400	3735	4042	4323
114	15.25 FT	800	800	800	800	800	800	817	930	1129	1330	1566	1798	1991	2176	2614	3016	3385	3724	4035	4321
120	15.83 FT	800	800	800	800	800	800	810	922	1118	1319	1552	1782	1974	2159	2597	3000	3372	3714	4030	4320

* B_d is the width of trench based upon pipe O.D. +2' for 42" I.D. or less, and O.D. +4' for greater than 42" I.D. Pipe O.D. is based upon ASTM C-76 wall B.

** When the height of cover over the pipe is less than 10', it is not necessary to maintain vertical trench sides. When the height of cover is 10' or greater, the sides of the trench installation must be vertical at least to a height of 1' above the top of the pipe. For this table to apply, the trench widths are not to exceed B_d.

Note 1 - When D-loads are greater than 3000-D, consult the text, Sec. 9-300.

Note 2 - Minimum recommended cover is 12".

TABLE VI

D-LOADS FOR
REINFORCED CONCRETE PIPE
POSITIVE PROJECTION
CLASS B BEDDING

		FILL HEIGHT, FEET																	
DIAM	B_c*	.5-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	10	12	14	16	18	20	25	30	35	40	45	50
12	1.33 FT	2548	1456	896	800	800	800	800	872	960	1154	1349	1543	1738	1932	2418	2905	3391	3877	4363	4849
15	1.63 FT	2286	1322	841	800	800	800	800	844	933	1122	1312	1502	1691	1881	2355	2829	3303	3777	4251	4726
18	1.92 FT	1849	1219	800	800	800	800	800	825	914	1101	1287	1473	1660	1846	2312	2778	3244	3710	4176	4642
21	2.21 FT	1514	1130	800	800	800	800	800	811	900	1084	1268	1452	1636	1821	2281	2741	3201	3661	4122	4582
24	2.50 FT	1385	1056	800	800	800	800	800	800	889	1071	1254	1436	1618	1801	2257	2713	3168	3624	4080	4536
27	2.79 FT	1167	922	800	800	800	800	800	800	879	1060	1242	1423	1604	1785	2237	2690	3142	3595	4047	4500
30	3.08 FT	981	803	800	800	800	800	800	800	871	1051	1231	1411	1591	1771	2221	2671	3121	3571	4021	4470
33	3.38 FT	836	800	800	800	800	800	800	800	864	1043	1223	1402	1581	1760	2208	2655	3103	3551	3998	4446
36	3.67 FT	800	800	800	800	800	800	800	800	857	1036	1215	1393	1572	1750	2196	2642	3088	3533	3979	4425
42	4.25 FT	800	800	800	800	800	800	800	800	846	1024	1201	1379	1556	1733	2176	2619	3062	3505	3948	4391
48	4.83 FT	800	800	800	800	800	800	800	800	835	1013	1190	1366	1543	1719	2160	2601	3042	3483	3924	4364
54	5.42 FT	800	800	800	800	800	800	800	800	826	1003	1179	1356	1532	1707	2147	2586	3025	3464	3903	4342
60	6.00 FT	800	800	800	800	800	800	800	800	816	993	1170	1346	1521	1697	2135	2573	3011	3449	3886	4324
66	6.58 FT	800	800	800	800	800	800	800	800	800	984	1161	1336	1512	1687	2124	2561	2998	3435	3871	4308
72	7.17 FT	800	800	800	800	800	800	800	800	800	975	1152	1327	1503	1678	2114	2550	2986	3422	3858	4294
78	7.75 FT	800	800	800	800	800	800	800	800	800	957	1143	1319	1494	1669	2105	2540	2976	3411	3846	4281
84	8.33 FT	800	800	800	800	800	800	800	800	800	927	1134	1310	1486	1660	2096	2531	2966	3400	3835	4269
90	8.92 FT	800	800	800	800	800	800	800	800	800	901	1120	1302	1477	1652	2088	2522	2956	3390	3824	4258
96	9.50 FT	800	800	800	800	800	800	800	800	800	878	1089	1293	1469	1644	2079	2514	2948	3381	3815	4248
102	10.08 FT	800	800	800	800	800	800	800	800	800	858	1061	1283	1461	1636	2071	2506	2939	3372	3805	4238
108	10.67 FT	800	800	800	800	800	800	800	800	800	840	1037	1251	1452	1628	2064	2498	2931	3364	3797	4229
114	11.25 FT	800	800	800	800	800	800	800	800	800	824	1016	1223	1443	1620	2056	2490	2923	3356	3788	4220
120	11.83 FT	800	800	800	800	800	800	800	800	800	810	996	1197	1414	1611	2048	2482	2915	3348	3780	4212

* B_c is the outside diameter of the full circle pipe based upon ASTM C-76 wall B.

Note 1 - When D-loads are greater than 3000-D, consult the text, Sec. 9-300.

Note 2 - Minimum recommended cover is 12".

TABLE VII

D-LOADS FOR
REINFORCED CONCRETE PIPE
TRENCH CONDITION
CLASS B BEDDING

DIAM	B_d*	.5-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8 **	10	12	14	16	18	20	25	30	35	40	45	50
		FILL HEIGHT, FEET																			
12	3.33 FT	2565	1484	948	800	800	867	960	1060	1193	1369	1586	1813	2036	2132	2315	2439	2523	2580	2618	2644
15	3.63 FT	2305	1351	893	800	800	843	934	1029	1130	1323	1545	1743	1851	1945	2129	2257	2346	2409	2452	2483
18	3.92 FT	1869	1249	847	800	800	823	911	1012	1080	1299	1503	1625	1732	1826	2012	2145	2241	2310	2359	2394
21	4.21 FT	1535	1161	808	800	800	808	900	994	1065	1281	1423	1544	1650	1744	1934	2073	2175	2251	2306	2346
24	4.50 FT	1408	1089	800	800	800	800	886	981	1054	1230	1364	1484	1590	1685	1879	2025	2134	2216	2277	2323
27	4.79 FT	1190	955	800	800	800	800	874	967	1038	1186	1319	1438	1545	1641	1840	1992	2108	2196	2263	2314
30	5.08 FT	1006	838	800	800	800	800	864	958	1005	1152	1284	1403	1511	1608	1812	1970	2092	2186	2259	2316
33	5.38 FT	862	800	800	800	800	800	860	956	979	1124	1255	1375	1484	1582	1790	1954	2083	2183	2263	2325
36	5.67 FT	800	800	800	800	800	800	851	946	957	1101	1232	1352	1462	1561	1774	1944	2078	2185	2270	2338
42	6.25 FT	800	800	800	800	800	800	837	933	923	1065	1197	1318	1429	1531	1753	1933	2079	2198	2294	2372
48	8.83 FT	800	800	800	800	800	800	808	922	1063	1282	1463	1625	1832	2038	2468	2779	3047	3279	3478	3651
54	9.42 FT	800	800	800	800	800	800	800	895	1055	1254	1410	1615	1821	2027	2386	2696	2966	3200	3405	3583
60	10.00 FT	800	800	800	800	800	800	800	866	1047	1215	1400	1606	1812	1970	2322	2631	2903	3141	3350	3534
66	10.58 FT	800	800	800	800	800	800	800	842	1019	1182	1392	1598	1768	1920	2270	2580	2853	3095	3309	3499
72	11.17 FT	800	800	800	800	800	800	800	824	995	1156	1376	1570	1728	1879	2228	2538	2814	3059	3278	3472
78	11.75 FT	800	800	800	800	800	800	800	807	974	1133	1346	1538	1695	1845	2192	2503	2781	3030	3253	3453
84	12.33 FT	800	800	800	800	800	800	800	800	955	1114	1320	1511	1667	1816	2162	2474	2755	3007	3234	3439
90	12.92 FT	800	800	800	800	800	800	800	800	939	1098	1298	1488	1642	1791	2137	2450	2732	2988	3219	3429
96	13.50 FT	800	800	800	800	800	800	800	800	924	1084	1280	1468	1621	1769	2115	2429	2714	2973	3208	3421
102	14.08 FT	800	800	800	800	800	800	800	800	912	1071	1263	1450	1603	1750	2095	2410	2698	2960	3198	3416
108	14.67 FT	800	800	800	800	800	800	800	800	901	1060	1249	1434	1586	1733	2079	2395	2684	2949	3191	3413
114	15.25 FT	800	800	800	800	800	800	800	800	891	1050	1236	1420	1571	1718	2064	2381	2672	2940	3186	3411
120	15.83 FT	800	800	800	800	800	800	800	800	883	1041	1225	1407	1558	1705	2050	2369	2662	2932	3181	3410

* B_d is the width of trench based upon pipe O.D. +2' for 42" I.D. or less, and O.D. +4' for greater than 42" I.D. Pipe O.D. is based upon ASTM C-76 wall B.

** When the height of cover over the pipe is less than 10', it is not necessary to maintain vertical trench sides. When the height of cover is 10' or greater, the sides of the trench installation must be vertical at least to a height of 1' above the top of the pipe. For this table to apply, the trench widths are not to exceed B_d.

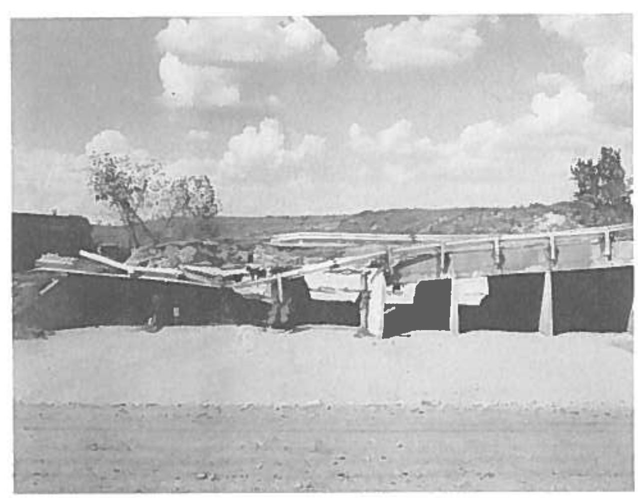
Note 1 - When D-loads are greater than 3000-D, consult the text, Sec. 9-300.

Note 2 - Minimum recommended cover is 12".

TABLE VIII

CHAPTER X

DOCUMENTATION



DOCUMENTATION

10-100. GENERAL

The thoroughness of any hydrologic and hydraulic study will certainly depend upon the nature of the stream being crossed. The preparation of any study, however, shall follow the guidelines outlined in this manual.

There are generally two conditions whereby hydrologic and hydraulic design data is submitted. One involves data that is to accompany all hydraulic structures which are submitted for preliminary review and approval prior to design of the structures. The other involves data to be presented in all plans (not on supporting papers but included in the plans) submitted for PS&E processing. Pertinent hydrologic and hydraulic information, covered elsewhere in this chapter, is to be included in both types of submissions and the inclusion of *data for preliminary review does not relieve submission requirements with PS&E.*

10-200. HYDROLOGIC AND HYDRAULIC DESIGN FILE

The complete hydrologic and hydraulic design data file for all waterway crossings will be maintained by each District. This complete file could include such items as structure investigated, photos, cost estimates, runoff investigations, drainage area maps, general statements concerning historical highwater, vicinity maps, contour maps, history of performance of existing structures, information on upstream control structures, pump station design, etc.

This listing is not all inclusive but is termed "Complete File" to indicate the filing of all minor or major investigations that went into the decision that fixed the size of the waterway opening or openings for a specific site or project. Again, it is the responsibility of the District to maintain the complete design file.

10-300. SUBMISSION DATA

10-301. EXISTING LOCATIONS

Refer to Section 2-204.12, EXISTING LOCATIONS, of the Bridge Division Operation and Planning Manual.

When the proposed highway alignment follows an existing location, each existing structure must be viewed by the Engineer with the thought in mind of improving, paralleling, extending or replacing the existing structure or structures or of leaving the structure unchanged. A portion of the decision should rest upon the determination of whether the existing structure is hydraulically too small, too large, or adequate. For instance, at sites which require extensions of originally overdesigned culverts or the addition of a new bridge structure to parallel an originally overdesigned bridge, the design requirements may be satisfied by extending the old culvert with a smaller size or the new bridge may be shorter than the old bridge. The opposite may be true, in that the existing structure is found, by hydraulic analysis, to be under-designed; in which case, an enlarged structure may be necessary to accommodate the design flood.

Due to the numerous problems requiring solutions, sufficient data must be gathered by the Engineer to justify the action taken at a site with an existing structure. As a minimum, the pertinent hydrologic and hydraulic information required on the plans or in a preliminary submission would include a drainage area size, design frequency, a design discharge, the calculated design highwater, and data concerning observed highwater.

10-302. NEW LOCATIONS

The pertinent data, for new locations, to accompany preliminary structure submissions or to be presented in the plans may or may not include all data contained in the complete design file. Required data for various hydraulic facilities is outlined below:

a. Bridges - Include for bridge type structures, the design discharge with the hydrologic method used, the design frequency, drainage area size, design highwater and an elevation versus discharge curve based upon natural stream cross section and natural conditions, or pertinent information to show how the design highwater was determined, observed highwater with dates (if available), "through-structure" velocity, area below design highwater on a plane normal to stream flow at flood stage within the limits of the bridge type structure, a cross section of the entire flood plain showing Manning's "n" value for various subareas, natural channel velocity, and backwater created by the proposed structure. (Figure 1)

b. Culverts - For culverts the plan requirements can be met by including in the plans, a reproduction of the culvert computation sheet (Figure 2) and a drainage area map (Figure 1) with runoff computations for the design frequency. All culvert cross sections should be plotted to a scale 1" = 5' or 1" = 10' horizontally and vertically. Flow line elevations at upstream and downstream ends should be shown together with such other dimensions as necessary for clarification. For skewed culverts, the skewed length shall be shown, and the skewed length shall be used in all hydraulic computations for skewed culverts.

c. Storm Sewers - For storm sewers, the standard inlet computation sheet, the storm sewer computation sheet, the storm sewer plan-profile sheets, a complete drainage area map with runoff computations, the design frequency, and complete information on outfalls and existing sewers shall be included in the plans and with all preliminary storm sewer submissions. It is desirable that the storm sewer drainage area map show an outline of the trunk line, lateral and inlets, and also the outfalls for the system and the existing sewers affecting the proposed system should be clearly indicated. There should be sufficient arrows to indicate the direction of overland flow and also to indicate direction of flow in the sewer system.

d. Pump Stations - For a preliminary submission, a pump station design should include the data similar to the example problem in Chapter VII. For plan presentation the pertinent data should include drainage area map, design frequency, discharge, maximum allowable highwater, cut-off and cut-on elevations, pump sizes, etc.

The intent here is not to limit the data to only that listed above, but rather it is to establish a minimum requirement consistent with the proper hydraulic design procedures. If extenuating circumstances are such that the hydraulic structure is sized by other than normal design procedures, a narrative summary shall be included in the construction plans outlining the circumstances and reasons the proposed size was selected.

If something other than hydraulic design dictates the roadway grade line and length of opening, this shall also be stated. In this case however, there still should be included a drainage area size, a design frequency, a design discharge, and a calculated highwater.

10-400. MISCELLANEOUS PLANNING DETAILS

Bridge Pier Alignment - When a new structure is to parallel an existing structure, the span, skew angle and shape of the piers or bents of the existing structures should be considered in determining the location of the new piers or bents, so that turbulence and/or obstruction to flow is kept to a minimum.

Improved Culvert Entrances - No improved inlet designed for culverts is to be skewed (Figure 3). The physical dimensions of the improved inlet are critical to the design and must not be altered (also see Section 4-603 and 4-604, Chapter IV).

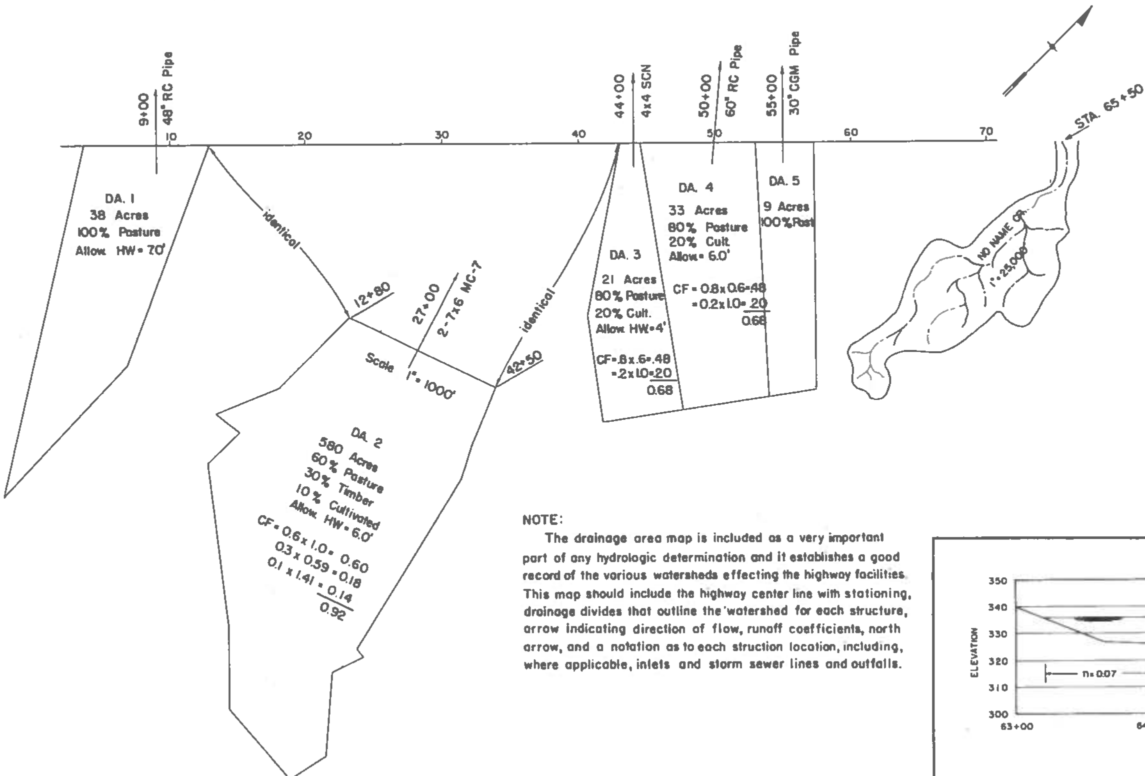
10-500. FORMS

The computation forms listed below are available by ordering from File D-4.

Form 1264	Culvert Computation Sheet
Form 1020	Inlet Computation Sheet
Form 1021	Storm Sewer Computation Sheet
Form 1290	Stream Water Surface Profile Computation Sheet

RUN-OFF COMPUTATIONS

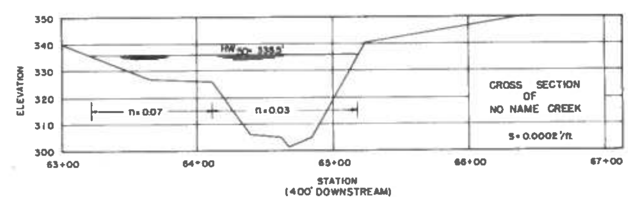
STATION	DRAINAGE AREA	AREA (ac)	T (min)	C	I_{50} (in/hr)	Q (cfs)
9+00	1	38	21	0.50	6.5	124
27+00	2	580	45	0.20	4.6	552
44+00	3	21	16	0.60	7.1	89
50+00	4	33	20	0.58	6.6	127
55+00	5	9	15	0.40	7.3	26

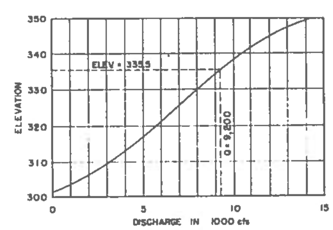


NOTE:
The drainage area map is included as a very important part of any hydrologic determination and it establishes a good record of the various watersheds effecting the highway facilities. This map should include the highway center line with stationing, drainage divides that outline the watershed for each structure, arrow indicating direction of flow, runoff coefficients, north arrow, and a notation as to each struction location, including, where applicable, inlets and storm sewer lines and outfalls.

MAP OF DRAINAGE AREAS

SCALE: 1" = 400'
unless otherwise noted





DA=44 Sq.Mi.
Q_{50}=9200cfs(Bulletin 6311)
HW_{50} @ ℄ =335.58
Area thru struct. below HW_{50}=2,000Sq.Ft.
$V_{STRUCT.} = Q_{50}/A = 4.6 fps$
Area in stream below HW_{50} @400'D.S.=3214Sq.Ft.
$V_{CHANNEL} = Q_{50}/A = 2.8 fps$

TYPICAL CROSS-SECTION & DISCHARGE vs ELEVATION CURVE

Figure 1

STATION	Q	ALLOW. H.W.	OUTLET CHANNEL						CULVERT														
			W	SS	n	S	TW	V	SIZE, LENGTH & DESCRIPTION	dc	Sc	S	$\frac{v^2}{2g}$	Ce	he	hf	H	P	SL	HW	V out.	TYPE OPER.	REMARKS

FORM 1264

Figure 2

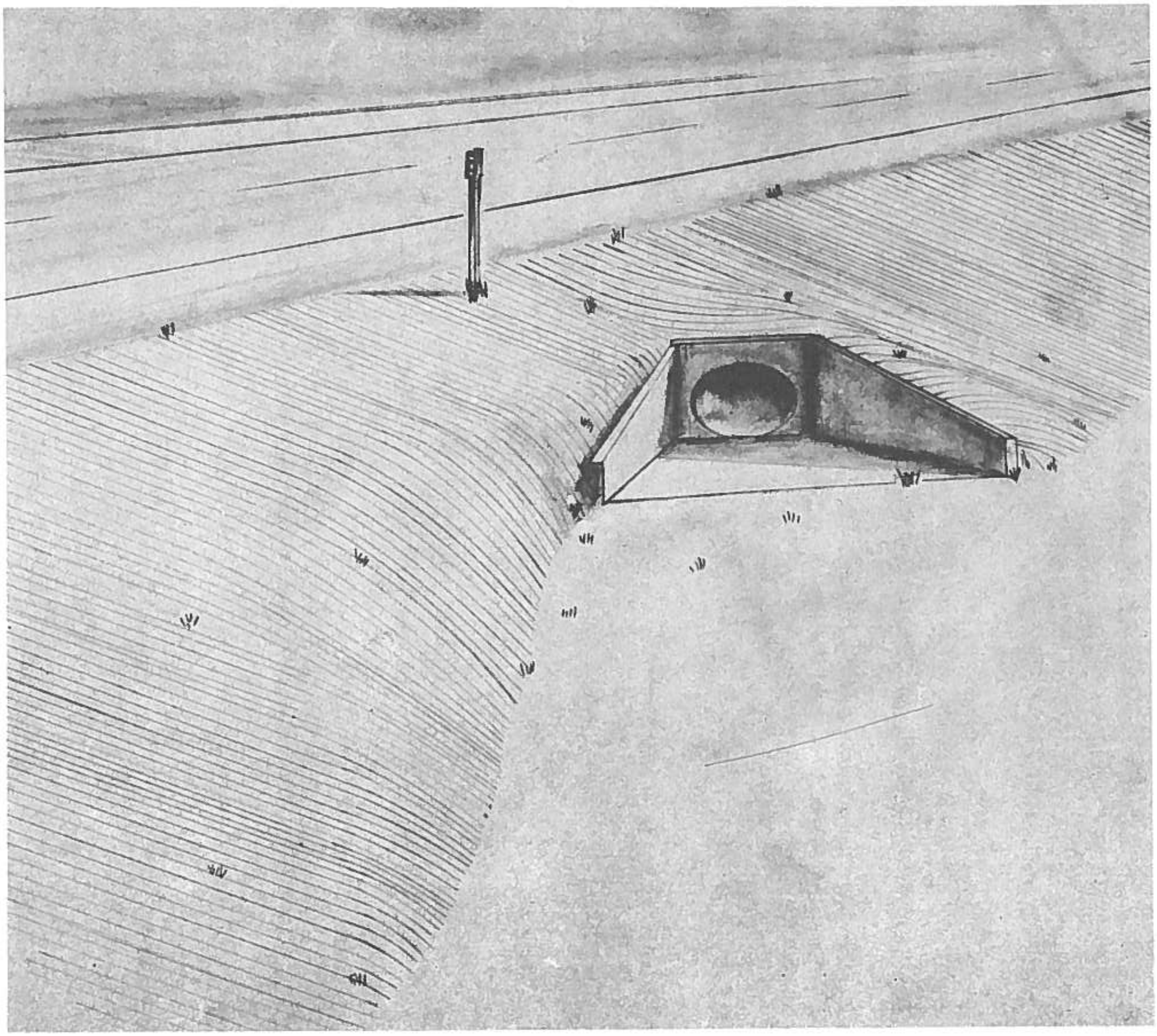


Figure 3

INDEX