DRAINAGE CRITERIA MANUAL

FORT BEND COUNTY

DRAINAGE DISTRICT

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Revised 2011

DRAINAGE CRITERIA MANUAL
FOR
FORT BEND COUNTY, TEXAS

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1.0 INTRODUCTION

This is the first major revision or update of the Fort Bend County Drainage District (FBCDD) Drainage Criteria Manual which was originally dated 1987 with minor revisions in April 1999. Since the 1987 manual was published and adopted, experience with criteria and with constructed facilities and changes in community preferences necessitated updating policy and criteria.

Transition Plan:

The plan for transitioning from the current 1987 FBCDD criteria manual to the updated FBCDD criteria manual dated 2011 is listed in the summary below (all dates apply to the time from adoption by Commissioners' Court):

- New Projects – all new criteria apply
- Drainage or Design Report (feasibility or planning phase)
    - Coordination meeting held but report not yet submitted – Six months
    - Submitted – One year
- Construction Drawings
    - Coordination meeting held but drawings not yet submitted – Three months
    - Submitted – One year
    - Previously approved but construction not commenced – Two years
    - Previously approved and construction commenced – Not applicable
- Construction – Not applicable
- Acceptance – Not applicable

1.1 CREATION AND AUTHORITY OF THE DRAINAGE DISTRICT

The Fort Bend County Drainage was created by Act 1949 of the 51st Legislature of the State of Texas, Page 550, Chapter 303, paragraphs 1 to 6 inclusive, which said Act became effective June 2, 1949; and which said Act became valid and in full force and effect under the laws of the State of Texas on June 2, 1949; and which said Act reads as follows:

"Section 1: That, pursuant to authority conferred by Section 59 of Article XVI of the Constitution of Texas, there is hereby created within the State of Texas, in addition to the districts into which the State has heretofore been divided, a district to be known as "Fort Bend County Drainage District" (hereinafter called the 'District') and shall include therein all of the territory situated within Fort Bend County, Texas, the boundaries of said District to be co-terminus with the boundaries of said county.

Said District is hereby created for the purpose of reclamation and drainage of its over-flowed lands and other lands needing drainage.

Said District shall be a governmental agency and body politic and corporate, with the powers of government and with the authority to exercise the rights, privileges and functions hereinafter specified, the creation and establishment of such District being essential to the accomplishment of the purpose of Section 59 of Article XVI of the Constitution of Texas.

Section 2: The Commissioner's Court of Fort Bend County, Texas, is hereby designated as the governing body of such District and the agency through which the management and control of said District shall be administered; and said Court is hereby empowered to do any and all things necessary to carry out the aims and purposes of this Act. Within Thirty days after this Act becomes effective, said Court shall call an election for the purpose of submitting to a vote of the duly qualified resident electors of the District who are qualified to vote in bond elections the questions of whether or not the creation of the District shall be confirmed. The cost of such an election shall be paid by Fort Bend County out of the county general fund. If a majority of the qualified voters voting at such election vote against confirmation of the District, the District hereby created shall be abolished and this Act shall be of no further force or effect. If a majority vote in favor of the confirmation, said District shall be permanently created. In such case, all other drainage districts situated in Fort Bend County shall be abolished and the order of the Commissioner's Court declaring the result of such election shall, in such case, recite that all such districts are abolished. Title and possession of all property and assets of the abolished districts shall thereupon pass to and be vested in the District hereby created and said District shall assume all functions, duties and obligations of the abolished districts.

Section 3: (A) In addition to the general powers herein granted, said Fort Bend County Drainage District shall be authorized to exercise the following powers, privileges and functions;

(1) To acquire within the District easements, right-of-ways, and any other character of property needed to carry on the work of the District by way of gift, devise, purchase, leasehold or condemnation; and to acquire without the District easements and right-of-ways by condemnation or otherwise; provided, however, such acquisition of easements or right-of-ways acquired without the District by way of condemnation shall be first unanimously approved by the entire Commissioners Court of the County wherein such easements and right-of-ways sought to be condemned, may be situated. The right of eminent domain is hereby expressly conferred on said District and the procedure with reference to condemnation, the assessment of and estimating of damages, payment, appeal, the entering upon the property pending appeal and all other procedure prescribed in Title 52, of the Revised Civil Statutes of Texas, 1925, as heretofore or hereafter amended, shall apply to said District

(2) To dispose of property or rights therein when the same are no longer needed for the purposes for which the District was created or to lease same for purposes which will not interfere with the use of such property by the District.

(3) To devise plans and construct works to reclaim lands in the District; to provide drainage facilities for the reclamation and drainage of the overflowed lands and other lands within the District needing drainage; to acquire or construct properties and facilities beyond the boundaries of the District where in the judgment of the governing body such properties or facilities are necessary to facilitate the drainage and reclamation of lands within the District; and to remove obstructions, natural or artificial, from the streams and water courses, and to clean, straighten, widen and maintain streams, water courses and drainage ditches.

(4) To cooperate with and contract with the United States of America or with any of its departments or agencies now existing, or which may be created hereafter, to carry out any of the powers or to further any of the purposes set forth in this act, and, for such purposes, to receive grants, loans or advancements therefrom; or to contribute to the United States of America or any of its departments or agencies in connection with any project undertaken by it affecting or relating to any of the purposes for which the District is organized.

(5) To cooperate and contract with any department or agency of the State of Texas, or any political subdivision thereof, or any municipal corporation to carry out any purpose for which the District is organized.

(6) To sue and be sued in the name of the District and all courts shall take judicial notice of the establishment of the District.

(7) To construct works, ditches, canals and other improvements over, across and along any public streams, roads, highways, or any lands belonging to the State of Texas, provided that the plans for such improvement on state highways shall be subject to the approval of the State Highway Department and on Prison System lands, shall be subject to the approval of the Texas Prison Board.

(8) To do any and all other acts or things necessary or proper to carry into effect the purposes for which the District is organized.

(B) The County Judge, County Commissioners, the Assessor and Collector of Taxes, the County Treasurer, and the county depository of Fort Bend County are authorized to, and shall be required to, perform all duties in connection with the District required of them by law in connection with official matters for Fort Bend County, and the County Auditor of said county shall be the Auditor for the Fort Bend County Drainage District. Said Court may employ a General Manager for said District and other such agents, attorneys, engineers and employees as may be considered necessary in connection with the purposes of this Act, and all compensation for such persons may be payable from funds herein created for the maintenance and operation of the District.

The governing body shall require the County Tax Assessor-Collector, the County Treasurer and such other officers and employees as the governing body shall designate, to make official bonds payable to the District in such amounts as the governing body shall determine, conditioned upon the faithful performance of their duties and paying over and accounting for all money and other things of value belonging to the District coming into their possession. Such bonds shall be executed by a surety company authorized to do business in Texas and shall be subject to the approval of the governing body and the premiums thereon shall be paid by the District.

<u>Section 4</u>: (A) The Court shall be authorized, from time to time, in behalf of said Fort Bend County Drainage District, to issue the bonds of said District, within the limitations hereinafter stated, for the purpose of acquiring funds with which to accomplish and carry out any one or more of the powers and purposes herein granted to the District, and to provide for the payment of the interest on such bonds as it accrues and to create a sinking fund for the redemption of said bonds as they mature, by levying and causing to be collected a tax on all taxable property within the District, as shown by the then current approved county assessment rolls, sufficient for such purposes. It is expressly provided, however, that the total principal amount of bonds issued by the District at any one time, together with all previously issued bonds then outstanding, shall never exceed a sum equal to five percent (5%) of the assessed valuation of all taxable property within the District, as shown by the then current County Assessment rolls. No such bonds shall be issued until first authorized by a majority of the voters qualified to vote on bond issues under the Constitution of Texas voting at an election called for the purpose of determining whether or not such bonds shall be issued and whether or not taxes shall be levied to pay principal and interest thereon

when due. In the event a majority of the qualified voters at such election shall vote in favor of the issuance of the bonds and the levy of Taxes, the Court shall be authorized to issue, sell and deliver said Fort Bend County Drainage District Bonds and to receive, use and apply the proceeds for the aforesaid purposes of said District, and to levy and assess taxes upon all property subject to taxation in said District, and to arrange for the collection of such taxes. Subject to the limitations contained in this Act, additional bonds may be issued from time to time, in like manner, and under the same procedure. The proposition of the issuance of bonds may be submitted at the election called for confirmation of the creation of said District under Section 2, hereof, or at such later times as the Court shall deem proper. If the proposition of issuing bonds is submitted at the confirmation election, the form of ballot at such first election shall be substantially as follows: "For the confirmation of the District, the issuance of bonds and the levy of taxes in payment thereof", and the contrary of such proposition. The levy of maintenance taxes may be submitted as a further proposition at such election. The provisions of Chapter 1 of Title 22, Revised Civil Statutes of Texas, as amended, shall apply to all bond elections in said District except where in conflict with this Act.

Such bonds shall not be delivered to the purchasers until they have been approved as to legality by the Attorney General of Texas and registered by the Comptroller of Public Accounts of Texas. The cost of issuing, selling and delivering such bonds may be paid out of the proceeds of sale thereof.

Such bonds shall bear interest at a rate not to exceed five (5%) percent per annum and shall mature serially or otherwise over a period of not to exceed thirty (30) years from their date or dates. They shall be sold for not less than par and accrued interest and, after having been approved by the Attorney General and registered by the Comptroller of Public Accounts and sold for not less than par and accrued interest, such bonds shall be held in any suit or proceeding in which their validity may be questioned to be valid, binding obligations of such district, subject only to the defense of fraud, forgery or constitutional violation. No suit shall be brought attacking the validity of such bonds, except upon the grounds stated, after such bonds have been delivered and the proceeds of sale have been received by the District.

The District shall not be authorized to issue time warrants payable from taxes.

(B) All bonds issued under the provisions of this Act shall be issued in the name of the Fort Bend County Drainage District of Fort Bend County, Texas, and shall be signed by the County Judge, attested by the County Clerk, and the seal of the Commissioner's Court of Fort Bend County, shall be affixed to each of them. Said bonds shall be issued in the denominations of not less than One Hundred Dollars ($100) nor more than Ten Thousand Dollars ($10,000), as determined in the order authorizing their issuance, and shall bear interest at a rate not to exceed dive (5%) percent per annum to be evidenced by attached coupons which shall bear the facsimile signatures of the County Judge and of the County Clerk.

Payment of principal and interest may be made at such place or places as may be determined by the Commissioners' Court in the order authorizing the issuance of such bonds.

Section 5: In addition to the levy and collection of taxes to pay bonds, as heretofore, provided, said Court may levy and cause to be collected a tax not exceeding twenty-five (25¢) cents on each \$100.00 valuation of taxable property within the District for the purpose of paying the cost of operating the District and maintaining its properties; provided, however, that no such tax shall be levied of assessed, until authorized at an election called for such purpose by said Court, in the manner provided by Section 4, hereof, at which a majority of the qualified electors, qualified to vote under the Constitution, voting at such election, vote in favor of the levy and collection of such tax. The assessed valuation of taxable property for District purposes shall be the same as that for State and County purposes; and the County Tax Assessor-Collector, of Fort Bend County, is hereby named and appointed Tax Assessor-Collector for said District; and the Board of Equalization of Fort Bend County is hereby named, constituted and appointed the Board of Equalization for said District. All laws of the State of Texas relating to the assessing and collecting of State and County taxes are by this Act made available for, and shall be applied to, the assessing of current taxes and to the collection of both current and delinquent taxes of said District, except where the same are in conflict with the provisions of this Act. The County Tax Assessor-Collector shall be paid such sum, not to exceed two (2%) percent of taxes collected for assessing and not to exceed two (2%) percent for collecting, as may be prescribed by the Commissioner's Court.

Section 6: If any section, sentence, phrase or clause, or any part of this Act, shall, for any reason, be held invalid, such invalidity shall not affect the remaining portions of this Act and it is hereby declared to be the intention of this Legislature to have passed each section, sentence, phrase, clause and part hereof irrespective of the fact that any other section, sentence, phrase, clause or part hereof may be declared invalid.

1.2 PURPOSE

The purpose of this drainage manual is to establish standard principles and practices for the design and construction of drainage systems within Fort Bend County to avoid increases in flood risks or flood hazards or create new flood hazard areas, to ensure the constructed infrastructure within FBCDD ROW performs its intended function with normal maintenance and repair, to provide procedures for the review and approval of proposed infrastructure within FBCDD ROW, and to provide procedures for acceptance by FBCDD. The design factors, formulae, graphs and procedures are intended for use as engineering guides in the solution of drainage problems involving determination of the quantity, quality, rate of flow, method of collection, storage and conveyance of storm water.

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

1.3 SCOPE

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

The past procedures and practices that have been used to design drainage facilities in Fort Bend County, along with numerous drainage criteria manuals for other areas, were reviewed to determine the most appropriate techniques and criteria for drainage design for use in Fort Bend County. This was especially true of Harris County's Criteria Manual for the Design of Flood Control and Drainage Facilities, which was used as the primary guide in selecting drainage criteria and in preparing this Criteria Manual for Fort Bend County. This was done in part so as not to "reinvent the wheel" in developing simplified procedures for applying the complex equations dealing with stormwater drainage. Also, there was the desire for consistency in criteria and methodology, where appropriate, to avoid unnecessary difficulty, confusion and expense in the design of drainage systems by engineers who have been or will be working in Fort Bend County. However, while there was obvious benefit for having consistency in the drainage criteria manuals of these two counties, this drainage criteria manual not only had to be an easy-to-use tool for solving drainage problems in Fort Bend County, but needed to contain standards and methodology that would be applicable to the specific problems and objectives of Fort Bend County. As a result, certain criteria and methodology were changed from those used in Harris County as was considered appropriate.

To assist design engineers in dealing with these two county manuals, the following is a list of the more significant differences in their design criteria:

1. the equations for computation of Clarks TC and R coefficients.
2. the loss rate parameters.
3. application of the ponding adjustment factor.
4. rainfall totals (hyetographs) for various events.
5. the drainage area – discharge curves
6. use of the Rational Method for drainage areas smaller than 200 acres
7. requirements for development within leveed areas.

1.4 DRAINAGE POLICY

The basic objective of the Fort Bend County Drainage District is to construct and maintain facilities intended to minimize the threat of flooding to all areas of the County and comply with the requirements of the National Flood Insurance Program. The ultimate goal is intended to be accomplished by the construction and maintenance of 100-year design drainage facilities and flood control measures to provide 100-year flood protection in all areas of Fort Bend County. The 100-year design drainage facilities are defined as all public channels within dedicated rights-of-way approved and accepted by the Drainage District and all other public flood control structures and facilities dedicated to, approved and accepted by the Drainage District. Additionally, it is the District's intent to insure that adequate facilities are constructed to accommodate new development such that existing property will not be subjected to additional flooding and so as not to increase the limits of the flood plains as shown on the flood insurance rate maps for Fort Bend County and other entities (Cities, Levee Improvement Districts, and Municipal Utility Districts).

It is not economically feasible to construct storm sewer facilities, which are large enough to keep the street systems from becoming inundated during severe storm events. The topographic relief of the coastal prairie is too flat to allow for quick runoff during severe storm events. The net effect of the District's policies will be to insure that for new developments the ponding in the street systems will be of minimum depth and duration, and most importantly, that minimum new building or structure slab elevations are set at least 12 inches above the maximum anticipated ponding levels. The intent of this policy is that there should be no street ponding for minor storm events, minor street ponding for larger events, and major ponding for the 100-year event storms but without water in structures. Every attempt will be made to design major thoroughfares so that they are passable during severe storm events.

To accomplish the goal of eliminating existing flooding conditions and to insure that future drainage problems do not develop, additional drainage improvement measures shall be taken. The measures considered appropriate by the District include further channel improvements to existing watercourses, overflow channels (primarily conveying flood flows directly to the Brazos River), and the construction of storm water detention facilities. The Fort Bend County Drainage District shall be responsible for the review and approval of all plans for 100-year design drainage facilities within Fort Bend County. All new drainage facilities must take into consideration the existing drainage upstream. In addition, new development must provide the ultimate planned right-of-way width to the Fort Bend County Drainage District through the developing property as shown in the various watershed master plans which have been developed by the Fort Bend County Drainage District and are in accordance with the technical criteria contained in this manual for handling the drainage needs of future development upstream. All plans submitted for review and approval will be made available for public inspection.

The District has included in this manual criteria covering the design of storm water systems to serve both existing and new developments. The Fort Bend County Drainage District has quantified the needed improvements for existing development in most of the watersheds in Fort Bend County and is responsible for the approval, and upon acceptance, the maintenance and operation drainage facilities which are in drainage rights-of-way dedicated to the Fort Bend County Drainage District. Upon the completion and acceptance of all new 100-year design drainage facilities the District will maintain, and operate said facilities for flood control purposes as an extension of the District's existing drainage system provided the facilities are constructed in accordance with the requirements of this manual. The Drainage District will not accept storm sewers or detention facilities for maintenance. The criteria in this manual are considered a minimum for Fort Bend County Drainage District approval. Approval from other applicable agencies may be required. Ultimate approval for any variance of the criteria contained in this manual must be given in writing by the Fort Bend County Drainage District.

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2.0 HYDROLOGY

2.1 GENERAL

The planning, design, and construction of drainage facilities are based on the determination of one or more aspects of storm runoff. If the estimate of storm runoff is incorrect, the constructed facilities may be undersized, oversized, or otherwise inadequate. An improperly designed drainage system can be uneconomical, cause flooding, interfere with traffic, disrupt commercial and other activities, and be a general nuisance in the affected area. However, the peak rate, volume and time-sequence of storm runoff related to a certain recurrence interval (frequency) can only be approximated due to the many physical and climatic factors involved.

Continuous long-term records of rainfall and resulting storm runoff in an area provide the best data source from which to base the design of storm drainage and flood control systems in that area. However, it is not possible to obtain such records in sufficient quantities for all locations requiring storm runoff computations. Therefore, the accepted practice is to relate storm runoff to rainfall, thereby providing a means of estimating the rates, timing and volume of runoff expected within local watersheds at various recurrence intervals. Although numerous methods to relate rainfall and runoff have been considered, three methods are recommended for use in Fort Bend County. These methods, discussed below, provide reasonable and consistent procedures for approximating the characteristics of the rainfall-runoff process.

It is generally accepted that urban development has a pronounced effect on the rate and volume of runoff from a given rainfall. Urbanization generally alters the hydrology of a watershed by improving its hydraulic efficiency, reducing its surface infiltration and reducing its storage capacity. This alteration can be intensified in flat areas like Fort Bend County. Figure 2-1 illustrates the effect of improving a watershed's hydraulic efficiency by presenting runoff rate versus time for the same storm with two different stages of watershed development. The reduction of a watershed's storage capacity and surface infiltration results from the elimination of porous surfaces and ponding areas by grading and paving building sites, streets, drives, parking lots, and sidewalks and by constructing buildings and other facilities characteristic of urban development. Zoning maps, future land use maps, and watershed master plans should be used as aids in establishing the anticipated surface character following development. The selection of design runoff coefficients and/or percent impervious cover factors, which are explained in the following discussions of runoff calculation, must be based upon the appropriate degree of urbanization.

Because of its versatility and accuracy, the widely used computer program HEC-HMS version 3.0.1 (or newer) is recommended as the primary tool for modeling storm runoff hydrographs in Fort Bend County for new developed models. Versions of HEC-HMS must be consistent throughout each project. Accordingly, the hydrologic design techniques described in this manual incorporate many of the routines contained in HEC-HMS. The principal routines used for describing runoff in the county as presented in this section are based on the Clark unit hydrograph technique, design storms and rainfall loss rates. A methodology for deriving the parameters used to compute the Clark unit hydrograph was developed from optimization studies utilizing U.S. Geological Survey (USGS) regional rainfall-runoff data and standard unit hydrograph techniques is appropriate for a wide range of drainage area sizes and is the preferred method in all but certain small areas requiring only peak discharge determinations. Appendix A presents an in-depth discussion of the technical development of this methodology. The HEC models from the Fort Bend County Drainage District must be obtained for updating the individual watershed in order to submit any drainage study.

HEC-HMS modeling is required of new development to ensure the development causes no adverse impact to a watershed for events including the 100-year, 25-year and 10-year rainfall events. Section 6 and Section 8 of this Manual define certain conditions under which a development may provide detention without a HEC-HMS analysis.

For areas less than 2000 acres and greater than 200 acres, drainage area-discharge curves have been developed as a means to determine peak discharge.

For certain small drainage areas (generally less than 200 acres in size), the widely used Rational Method provides a useful means of determining peak discharges. In situations requiring determination of a complete flood hydrograph, and not just a peak discharge, the Malcom Small Watershed Method should be utilized. If the engineer wishes to use an alternative design technique, it is recommended that the Fort Bend County Drainage District Engineer be consulted prior to design. For large drainage areas, hydrologic modeling using HEC-HMS is recommended to obtain runoff hydrographs.

The drainage discharge curves, Rational Method and Malcom Small Watershed Method are to be used as tools to assist in the design of internal drainage components of a development, and to assist in HEC-HMS modeling. The results provided by these curves/methods alone do not justify the amount of detention a development must provide to mitigate its impact to the watershed. See Section 6 and/or Section 8 of this Manual for minimum detention requirements.

2.2 RAINFALL-RUNOFF COMPUTATIONS USING HEC-HMS

A stream network model which simulates the runoff response of a river basin to rainfall over that basin can be developed utilizing the HEC-HMS computer program by the appropriate combination of hydrograph and routing computations. The following sections describe the elements required to develop a HEC-HMS computer model.

2.2.1 Design Storm Rainfall

Design storm rainfall can be described in terms of frequency, duration, areal extent and distribution of intensity with time. A design storm's rainfall distribution in time should be handled in the HEC-HMS by offsetting the intensity position of hyetograph by 67% in HEC-HMS if either the watershed is shared with Harris County or there is no existing model for the watershed. If there is an existing model for the watershed with the intensity position at 50%, then for consistency, the updated model for the watershed should have the intensity position at 50%. The engineer's choice for frequency and duration is dependent upon the physical characteristics, location and study objectives. In most cases, design will be based on a 24-hour duration storm event. The HEC-HMS program has the capability to modify runoff hydrographs to account for progressively smaller design storm volumes as areal coverage increases. The HEC-HMS user manual suggests how to model storm rainfall depth versus drainage area relationships, based on Figure 15 in the National Weather Service's Technical Paper No. 40 which presents a means of reducing point rainfall totals as drainage area size increases.

It is often necessary to increment design rainfall hyetographs in five-minute intervals to meet the design needs of small drainage areas having short times of concentration. The TP-40 rainfall isopluvial maps are limited to storm durations no less than 30 minutes. Table 3 of TP-40 then provides a method to calculate the rainfall amounts for shorter duration storms based on national average values. To more accurately define these rainfall quantities on a local basis the National Weather Service issued Technical Memorandum NWS Hydro-35 entitled "Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States". Thus, both TP-40 and Hydro-35 were used to develop Table 2-1 in which depth vs. duration data is presented for a variety of storm frequencies. Table 2-1 is also useful in utilizing the Rational Method.

2.2.2 Design Storm Losses

Only a portion of the rainfall volume which falls on a watershed during a storm event actually ends up as stream runoff. The remainder is intercepted by infiltration, depression storage, evaporation and other mechanisms. The volume of rainfall which becomes runoff is termed the "excess" rainfall. The difference between the observed total rainfall hyetograph and the excess rainfall hyetograph is termed abstractions or losses.

The exponential loss method is one of several loss methods included in HEC-HMS. The exponential loss method is recommended for calculation of abstractions in Fort Bend County. Exponential loss method is an empirical method in which the loss rate is determined to be a function of both the rainfall intensity and accumulated losses. In general, this method should not be used without calibration. It is highly recommended to obtain exponential loss parameters by calibration whenever possible.

Following are the description of the parameters of the exponential loss method:

1) **Initial range:** the amount of initial accumulated infiltration during which the loss rate is increased. This parameter is considered to be a function primarily of antecedent soil moisture deficiency and is usually storm-dependent.

2) **Initial coefficient:** specifies the starting loss rate coefficient on the exponential infiltration curve. It is assumed to be a function of infiltration characteristics and consequently may be correlated with soil type, land use, vegetation cover, and other properties of a sub-basin.

3) **Coefficient ratio:** indicates the rate at which the exponential decrease in infiltration capability proceeds. It may be considered a function of the ability of the surface of a sub-basin to absorb precipitation and should be reasonable constant for large, homogeneous areas.

4) **Precipitation exponent:** reflects the influence of precipitation rate on sub-basin-average loss characteristics. It reflects the manner in which storms occur within an area and may be considered a characteristic of a particular region. It varies from 0.0 up to 1.0.

5) **Impervious %:** percentage of the sub-basin which is directly connected impervious area can be specified. No loss calculations are carried out on the impervious area; all precipitation on that portion of the sub-basin becomes excess precipitation and subject to direct runoff.

Based on the analyses conducted in the original development of the hydrologic methodology (See Appendix A) and a consideration of soil characteristics in Fort Bend County, the following are recommended values for the variables to be used with this methodology:

1) Initial Range (in HEC-HMS) or DLKTR (in HEC-1) = amount in inches of initial accumulated rain loss during which the loss coefficient is increased = 0.0

2) Initial coefficient (in HEC-HMS) or STRKR (in HEC-1) = starting value of the loss coefficient on the exponential recession curve for rain losses = 0.5

3) Coefficient Ratio (HEC-HMS) or RTIOL (in HEC-1) = parameter computed as the ratio of STRKR to a value of STRKR after ten inches of accumulated loss. = 3.0

4) Exponent (in HEC-HMS) or ERAIN (in HEC-1) = exponent of precipitation for rain loss function that reflects the influence of the precipitation rate on the basin-average loss characteristics = 0.6

5) Impervious % (in HEC-HMS) or RTIMP (in HEC-1) = percentage of drainage basin that is impervious= (% Urban Development) x (average % impervious cover of the developed area)/100

Typical values for the percentage of impervious cover corresponding to various types of development in Fort Bend County are given in Table 2-2. These values should be used when only the general type of planned development is known; once the actual level of development has been determined for a specific area, a refined value should be used to reflect the actual percent of impervious cover.

2.2.3 Design Storm Runoff

2.2.3.1 General

Given the design storm excess rainfall, it is necessary to determine the storm runoff hydrograph at the point of interest utilizing the HEC-HMS program. The Clark unit hydrograph for a drainage area is described by three parameters: TC, R and a time-area curve. TC represents the time of concentration and R is a storage coefficient for the area. The time-area curve defines the cumulative area of the watershed contributing runoff to the design point as a function of time.

A statistical analysis of historical rainfall and runoff data taken from selected watersheds in the Fort Bend County vicinity was performed to correlate TC and R to drainage area physiographic characteristics. These characteristics include the length, slope and roughness of the basin's longest watercourse, the average basin slope and the effective imperviousness of the basin. From this analysis, the following equations were derived:

$$TC + R = 128 \, \frac{(L/\sqrt{S})^{0.57} \, (N)^{0.8}}{(S_o)^{0.11} \, (10)^{I}} \qquad (2\text{-}1)$$

And

$$TC = (TC+R) \times 0.38 \, (\log S_o) \qquad (2\text{-}2)$$

$$R = (TC+R) - TC \qquad (2\text{-}3)$$

Where TC = Clark's time of concentration (hrs)

R = Clark's storage coefficient (hrs)

L = length of the longest watercourse within the drainage area (miles)

S = average slope along the area's longest watercourse (ft/mile)

N = Manning's weighted roughness coefficient along the longest watercourse (see Step 4 of Section 2.2.4)

S_o = average basin slope of land draining overland into the longest watercourse (ft/mile)

I = effective impervious ratio

A plot of Equation 2-1, along with the basic data used in it development, is contained in Appendix A.

The effective impervious ratio (I) used in equation (2-1) is determined by:

$$I = CD \times 10^{-4} \qquad (2\text{-}4)$$

Where: C = the average percent of impervious cover of the developed area (in percent)

D = % of the subarea that is developed

Determination of TC and R is carried out by the solution of Equations 2-1, 2-2 and 2-3.

These parameters may then be input into the HEC-HMS program to model the runoff process. Input of the time-area curve is handled internally by HEC-HMS unless the engineer specifies a particular time-area relationship. An example of the step-by-step procedure for the development of a design runoff hydrograph is presented in Section 2.2.4.

For a detailed discussion of unit hydrograph theory and application, the engineer is referred to the Handbook of Applied Hydrology, by Ven Te Chow, 1964.

2.2.3.2 Adjustment for Ponding

The presence of significant areas of ponding in a drainage subarea will have a pronounced effect on the nature of the runoff hydrograph from that subarea. Storage in ponding areas tends to cause peak flow rates to be decreased and the time at which the peak flow occurs to be delayed. To account for this effect, an adjustment can be made in the R parameter, which reflects the storage-routing characteristics of the subarea. Figure 2-2 provides sets of equations and curves that relate the percent of the subarea affected by ponding to an adjustment coefficient for R. Determination of the adjustment coefficient is a two-step process. First, an adjustment is determined based on the areal extent of the ponding in the subarea. Second, the fact that only a portion of the entire subarea will drain through the ponded area, and thus be affected by it, is accounted for.

For example, if a subarea of ten square miles has two square miles of ponded area, the percent ponding would be 20%. From Figure 2-2, it is seen that the appropriate adjustment factor to R (for the 100-year event) is 1.80.

This adjustment factor (RM) must then be modified if not all of the ten square mile subarea is affected by ponding. If, for instance, an additional one square mile of the subarea drains through the two square mile ponding area, only 30% of the entire drainage subarea is affected by ponding. The adjustment factor would thus be reduced by 70% (i.e. [(1.8-1.0) x .30] + 1.0 = 1.24).

If a ponding area (such as a gravel pit) does not allow runoff to pass through it for a particular design storm event, then that portion of the area drainage into the pond plus the pond surface area itself should be eliminated from the drainage area as being non-contributing. The remaining portions of the drainage area would not require any adjustment to its R value for this particular ponding area.

2.2.4 Procedure for Developing a Design Runoff Hydrograph

The following general procedure (and example) should be followed in developing design runoff hydrographs in Fort Bend County.

1. Determine the required frequency and duration of the design storm from the applicable County criteria. (This usually will be the 100-year, 24-hour storm event.)

   Example: For this example assume that a peak discharge is needed to hydraulically design a major channel for the 100-year, 24-hour storm event.

2. Develop the design storm hyetograph. This process can be carried out internally by HEC-HMS as discussed in Section 2.2.1. It is required that depth-duration data, as presented in Table 2-1, be input into HEC-HMS. If necessary, the engineer may input a different rainfall pattern. If the drainage area upstream of the design point is greater than approximately 10-15 square miles, depth-area relationships should be considered.

   Example: Table 2-1 was used to assign the appropriate depth of rainfall for each of the various durations as follows:

<u>100-Year Frequency Design Storm</u>

Duration	Depth in Inches
5 min.	0.90
15 min.	2.01
60 min.	4.55
2 hours	6.05
3 hours	6.85
6 hours	8.40
12 hours	10.45
24 hours	12.50

These values are input in the HEC-HMS frequency storm table.

3. Determine losses. This procedure is carried out internally by HEC-HMS, but it is required that the values of the variables presented in Section 2.2.2 of this manual be input in HEC-HMS.

   <u>Example</u>: For all watershed calculations in Fort Bend County, the following values of the loss function variables are recommended.

Variable	Value
Initial Range (or DLKTR in HEC-1)	0.0
Initial coefficient (or STRKR in HEC-1)	0.5
Coef. Ratio (or RTIOL in HEC-1)	3.0
Exponent (or ERAIN in HEC-1)	0.6
Impervious % (or RTIMP in HEC-1)	100 x I (to be calculated)

4. Determine physical characteristics of the watershed including channel length, channel slope, Manning's weighted "n" value, effective percent imperviousness and average basin slope. These parameters (as illustrated in Figure 2-3) are calculated as follows:

Length (L)	The length of the longest watercourse within the subarea to the watershed divide in miles.
Channel Slope (S)	The average slope of the middle 75% of the longest watercourse in the subarea, in feet per mile.
Manning's Weighted "n" (N)	The Manning's roughness coefficient as a weighted average value representative of flow roughness in the subarea's main watercourse. It should account for portions of the design flow contained in the overbanks as well as the main channel. A recommended simplified procedure is to divide the basin into upstream and downstream halves, determine the representative composite "n" value for a typical section in each half, then weight the upstream value 25% and the downstream value 75%.
Average Basin Slope (S_o)	The average slope of the land draining overland into the longest watercourse, in feet per mile.
Effective Impervious Ratio (I)	The average percent of the impervious cover of the developed area, in percent, times the percent of the total subarea considered to be developed for design purposes times 10^{-4}.

<u>Example</u>: (from Figure 2-3)

L	=	2.84 miles
S	=	55-foot drop over 75% of 2.84 miles = 25.8 feet/mile
N	=	Upstream composite "n" value = .061
		Downstream composite "n" value = .049
		N = (.25).061 + (.75)(.049) = .052
S_o	=	36 feet/mile
I	=	$C \times D \times 10^{-4} = (35 \times 57) \times 10^{-4} = 0.1995$

5. Determine TC and R and input in HEC-HMS. The Clark parameters are determined by solution of Equations 2-1, 2-2 and 2-3.

   Example:

$$TC + R = 128\,\frac{(2.84/\sqrt{25.8})^{.57}(.052)^{.8}}{(36.0)^{.11}(10)^{.1995}} = 3.68$$

   And

   TC = (3.68) x 0.38 (log 36) = 2.18
   R = 3.68 – 2.18 = 1.50

6. Adjust Clark's R-coefficient to account for ponding area.

   Example: From Figure 2-3 it is seen that 0.107 square miles of the total 2.13-square mile watershed is affected by ponding. Thus, the percent ponded area is (.107/2.13) = 5.02%. From Figure 2-2, the appropriate adjustment factor for R for the 100-year storm event is 1.50. Of the total 2.13-square mile watershed, it is seen in Figure 2-3 that .198 square miles (9.30% of the watershed) are affected by ponding. Thus, the adjustment factor of 1.50 must be reduced as follows:

   (1.5-1.0)(.0930) + 1.0 = 1.05

   The value of R calculated in Step 5 above is then multiplied by the modified adjustment factor.

   1.50 (1.05) = 1.58

This adjusted value of R is input into HEC-HMS instead of the R value calculated in Step 5.

2.2.5 Flood Routing

As a flood wave passes downstream through a channel or detention facility, its shape is altered due to the effects of storage. The procedure for determining how the shape of the flood hydrograph changes is termed flood routing. Flood routing can be used to determine the effects of this storage on a flood's runoff pattern (i.e. its hydrograph).

Flood routing can be classified into two broad but related categories: open channel routing and reservoir routing. Reservoir routing is generally used to determine the effectiveness of storm water detention generally used in reducing downstream peak flood flow rates. Open channel routing is a refinement of the description of an area's rainfall-runoff process. It modifies the time rate of runoff due to storage within the channel and its overbanks. Analysis of areas with very flat overbanks and wide flood plains should consider channel routing to determine possible peak discharge attenuation.

The recommended technique for both channel and reservoir routing is the Modified Puls method. The Modified Puls method is based on the assumption of an invariable discharge-storage relationship and a constantly level pool in the storage reach of interest. The HEC-HMS program provides a routine for this flood routing technique. The required storage-discharge relationships for this routing technique can be obtained by use of the HEC-RAS backwater program for a variety of flow conditions. Care must be taken in developing these storage-discharge relationships with HEC-RAS. Cross-sections need to be provided that adequately define all of the flood plain storage available at various water levels. However, only the effective area of the cross-section should be used to establish the proper discharge-water level relationship. For a discussion of the Modified Puls routing technique and other methodologies, the engineer is referred to the Handbook of Applied Hydrology, by Ven Te Chow, 1964.

2.3 DRAINAGE AREA – DISCHARGE CURVES

Drainage area-discharge curves represent a simplified method for the determination of the peak discharge in a relatively small watershed. Usage of this type analysis requires that the watershed and its physical characteristics be relatively uniform and not contain complex hydrologic features such as ponding areas, storage basins or watershed overflows. The curves developed for this manual for the 25 and 100-year rainfall events, respectively, are shown in Figures 2-4 and 2-5, and are applicable to drainage areas between 200 and 2,000 acres. Since there is such a great variation in the physical characteristics of partially developed watersheds along with a wide range of conveyance capacity (i.e.

flood plain storage), these curves were developed for a typical watershed assuming adequate conveyance capacity and uniformly-spaced development. Applicable flow rates for existing conditions in the design of detention facilities should be determined on a case-by-case basis working closely with the Drainage District Engineer (See Section 6.0).

Whenever the situation requires the determination of a complete flood hydrograph, and not just a peak discharge, Malcom's small watershed method, as described in Section 2-5, should be used.

2.4 RATIONAL METHOD

The Rational Method represents an accepted method for determining peak storm runoff rates for small watersheds that have a drainage system unaffected by complex hydrologic situations such as ponding areas, storage basins and watershed transfers (overflows) of storm runoff. This widely used method provides satisfactory results if understood and applied correctly. It is generally recommended that in Fort Bend County the Rational Method be used only for areas less than 200 acres.

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

$$Q = CiA \qquad (2\text{-}5)$$

Where:

Q is defined as the peak rate of runoff in cubic feet per second. Actually, Q is in units of inches per hour per acre. Since this rate of in/hr/ac differs from cubic feet/second by less than one percent, the more common cfs is used.

C is the dimensionless coefficient of runoff representing the ratio of peak discharge per acre to rainfall intensity (i).

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

A is the area in acres contributing runoff to the point of interest during the critical time of concentration.

Basic assumptions associated with the Rational Method are:

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

2. The frequency or recurrence interval of the peak discharge is equal to the frequency of the average (uniform) rainfall intensity associated with the critical time of concentration (duration).

3. The time of concentration is the critical time of concentration and is discussed under paragraph 2.4.2 of this manual.

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

5. Rainfall intensity is uniform during the storm duration.

6. The contributing area is that area that drains to the point of interest within the critical time of concentration.

2.4.1 Runoff Coefficient (C)

In relating peak rainfall rates to peak discharges, the runoff coefficient “C” in the Rational Formula is dependent on the character of the drainage area’s surface. The rate and volume of a storm’s rainfall that reaches an area’s storm sewer system depends on the relative porosity (imperviousness), ponding character, slope and conveyance properties of the surface. Soil types, vegetation condition and impervious surfaces, such as asphalt pavements and roofs of buildings, are the major determining factors in selecting an area’s “C” factor. The type and condition of the surface determines its ability to absorb precipitation and transport runoff. The rate at which a soil absorbs precipitation generally decreases as and if the rainfall continues for an extended period of time. The soil absorption or infiltration rate is also influenced by the presence of soil moisture before a rain (antecedent precipitation), the rainfall intensity, the proximity of the ground water table, the degree of soil compaction, the porosity of the subsoil, vegetation, ground slopes, depressions, and storage. On-site inspections and aerial photographs may prove valuable in estimating the nature of the surface within the drainage area.

It should be noted that the runoff coefficient "C" is the variable of the Rational Method which is least susceptible to precise determination. Proper use requires judgment and experience on the part of the engineer, and its use in the formula implies a fixed ratio for any given drainage area, which in reality is not the case. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff.

Coefficients for specific surface types can be used to develop a composite runoff coefficient based in part on the percentage of different types of surfaces in the drainage area. This procedure is often applied to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area.

Table 2-3 presents recommended values for the runoff coefficient "C" for various residential districts and specific surface types for 5-10 year frequency storms. These values were derived from numerous sources (see References 9, 20, 31, and 32). Adjustment of the "C" value for use with larger (less frequent) storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f, which is used to account for antecedent precipitation conditions. The Rational Formula now becomes:

$$Q = CiAC_f \tag{2-6}$$

Table 2-4 presents recommended values of C_f. The product of C times C_f should not exceed 1.0.

2.4.2 Rainfall Intensity (i)

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

The time of concentration used in the rational equation is the critical time of concentration for the point of interest. The critical time of concentration is the time associated with the peak runoff from all or part of the upstream drainage area to the point of interest. Runoff from a watershed usually reaches a peak at the time when the entire drainage area is contributing; in which case, the time of concentration is the time for water to flow from the most remote point in the watershed to the point of interest. However, the runoff rate may reach a peak prior to the time the entire upstream drainage area is contributing. In this instance, only the portions of the drainage area able to contribute flow to the point of interest during the critical time of concentration should be used in determining the peak discharge. A trial and error procedure can be used to determine the critical time of concentration.

The time of concentration to any point in a storm drainage system is a combination of the "inlet time" and the "time of flow in the conduit".

The inlet time is the time for water to flow over the surface to the storm sewer inlet. Inlet time decreases as the slope and the imperviousness of the surface increases, and it increases as the distance over which the water has to travel increases and as retention by the contact surfaces increases. Average velocities for estimating travel time for overland flow can be calculated using Figure 2-6.

The inlet time shall be determined by direct computation using the following formula:

$$T = \frac{D_F}{60V} \tag{2-7}$$

where

T	=	overland flow time (minutes).
D_F	=	flow distance (feet).
V	=	average velocity of runoff flow (ft/sec).

If the overland flow time is calculated to be in excess of 20 minutes, the designer should verify that the time is reasonable considering the projected ultimate development of the area.

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

The statistical relationship between the rainfall intensity and duration for the 25-year and 100-year frequency storms are shown in Figure 2-7. These two curves are presented for the 25-year and 100-year design frequencies for durations from 5 minutes to 24 hours since the two frequencies are used in channel design. Table 2-1 presents rainfall amounts for a variety of durations and frequencies.

2.4.3 Drainage Area (A)

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

As mentioned previously, the drainage area used in determining peak discharges if the portion of the area that contributes flow to the point of interest within the critical time of concentration.

2.5 HYDROGRAPH DEVELOPMENT FOR SMALL WATERSHEDS

A technique for hydrograph development which is useful in the design of detention facilities serving relatively small watersheds has been presented by H.R. Malcom. This method can be used for watersheds up to approximately 2 square miles (1280 acres), but is recommended to be used for watersheds of 1 sq. mile (640 acres) or less.

This procedure can be used in conjunction with the drainage area-discharge curves or the Rational Method. The methodology utilizes a pattern hydrograph to obtain a curvilinear design hydrograph which peaks at the design flow rate and which contains a runoff volume consistent with the design rainfall. The pattern hydrograph is a step function approximation to the dimensionless hydrograph proposed by the Bureau of Reclamation and the Natural Resources Conservation Service.

. Malcom's Method consists of the following equations:

(1) $$T_p = \frac{V}{1.39Q_p}$$

(2) $$q_i = \frac{Q_p}{2}\left[1 - \cos\left(\frac{\pi t_i}{T_p}\right)\right]$$ for $t_i \leq 1.25\ T_p$

(3) $$q_i = 4.34\,Q_p e^{\left(-1.30\frac{t_i}{T_p}\right)}$$ for $t_i > 1.25\ T_p$

*Calculator must be in radian mode.

Where:

Q_p = peak design flow rate in cfs

T_p = time to Q_p in seconds

V = total volume of runoff for the design storm in cubic feet

ti and qi = the respective time and flow rates which determine the shape of the hydrograph.

A plot of a hydrograph illustrating these parameters is included as Figure 2-8.

The peak design flow rate can be calculated directly either from the drainage area – discharge curves or the Rational Method depending upon the size of the area considered. The total volume of runoff is dependent on the level of development of the area (i.e. percent of impervious cover). Typical loss rate totals for the 25- and 100-year, 24-hour rainfall events are included in Table 2-5.

TABLE 2-1

POINT RAINFALL AMOUNTS (INCHES) FOR

VARYING DURATIONS AND FREQUENCIES

IN FORT BEND COUNTY, TEXAS

	Rainfall Frequency						
Duration	1-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
5-minute	0.55	0.56	0.63	0.68	0.77	0.83	0.90
15-minute	1.10	1.20	1.37	1.50	1.70	1.85	2.01
30-minute	1.55	1.90	2.30	2.65	3.05	3.35	3.70
60-minute	1.95	2.30	2.82	3.18	3.72	4.14	4.55
2-hour	2.30	2.78	3.65	4.20	4.85	5.45	6.05
3-hour	2.50	3.15	4.00	4.70	5.45	6.10	6.85
6-hour	2.90	3.60	4.83	5.35	6.70	7.55	8.40
12-hour	3.35	4.20	5.75	6.80	8.20	9.25	10.45
24-hour	3.80	4.90	6.75	8.20	9.60	11.00	12.50
2-day	-	5.70	7.50	9.20	11.00	12.50	14.00
4-day	-	6.60	8.80	10.50	12.50	14.20	16.00
7-day	-	7.60	10.00	12.00	14.20	16.20	17.80
10-day	-	8.40	10.80	13.00	15.50	17.50	20.00

Source: Hydro-35 (5-60 minutes), TP-40 (2-24 hour), and TP-49 (2 to 10 day). “1-yr” values produced by extrapolation and are typically used for interior drainage analysis.

TABLE 2-2

TYPICAL AVERAGE VALUES

FOR

IMPERVIOUS COVER

Type of Development	Percentage of Impervious Cover
Commercial and Business Areas	85
Industrial	72
Residential	
Average lot size	
1/8 Acre or less	65
1/4 Acre	38
1/3 Acre	30
1/2 Acre	25
1 Acre	20

Source: NRCS TR55, Urban Hydrology for Small Watersheds (Table 2.2).

TABLE 2-3
RATIONAL METHOD RUNOFF COEFFICIENTS
FOR 5-10 YEAR FREQUENCY STORMS

Description of Area	Runoff Coefficients For Basin Slopes		
	Less than 1%	1% - 3.5%	3.5%-5.5%
Residential Districts			
Single Family Areas			
(Lots greater than ½ acre)	0.30	0.35	0.40
Single Family Areas			
(Lots ¼ - ½ acre)	0.40	0.45	0.50
Single Family Areas			
(Lots less than ¼ acre)	0.50	0.55	0.60
Multi-Family Areas	0.60	0.65	0.70
Apartment Dwelling Areas	0.75	0.80	0.85
Business Districts			
Downtown Areas	0.85	0.87	0.90
Neighborhood Areas	0.75	0.80	0.85
Industrial Districts			
Light Areas	0.50	0.65	0.80
Heavy Areas	0.60	0.75	0.90
Railroad Yard Areas	0.20	0.30	0.40
Parks, Cemeteries	0.10	0.18	0.25
Playgrounds	0.20	0.28	0.35
Streets			
Asphalt	0.80	0.80	0.80
Concrete	0.85	0.85	0.85
Drives and Walks			
(Concrete)	0.85	0.85	0.85
Roofs	0.85	0.85	0.85
Lawn Areas			
Sandy Soil	0.05	0.08	0.12
Clay Soil	0.15	0.18	0.22
Undeveloped Areas			
Sandy Soil			
Woodlands	0.15	0.18	0.25
Pasture	0.25	0.35	0.40
Cultivated	0.30	0.55	0.70
Clay Soil			
Woodlands	0.18	0.20	0.30
Pasture	0.30	0.40	0.50
Cultivated	0.35	0.60	0.80

TABLE 2-4

FREQUENCY FACTOR ADJUSTMENT

Frequency Of Storm (years)	Frequency Factor (C_f)
5	1.00
25	1.10
50	1.20
100	1.25

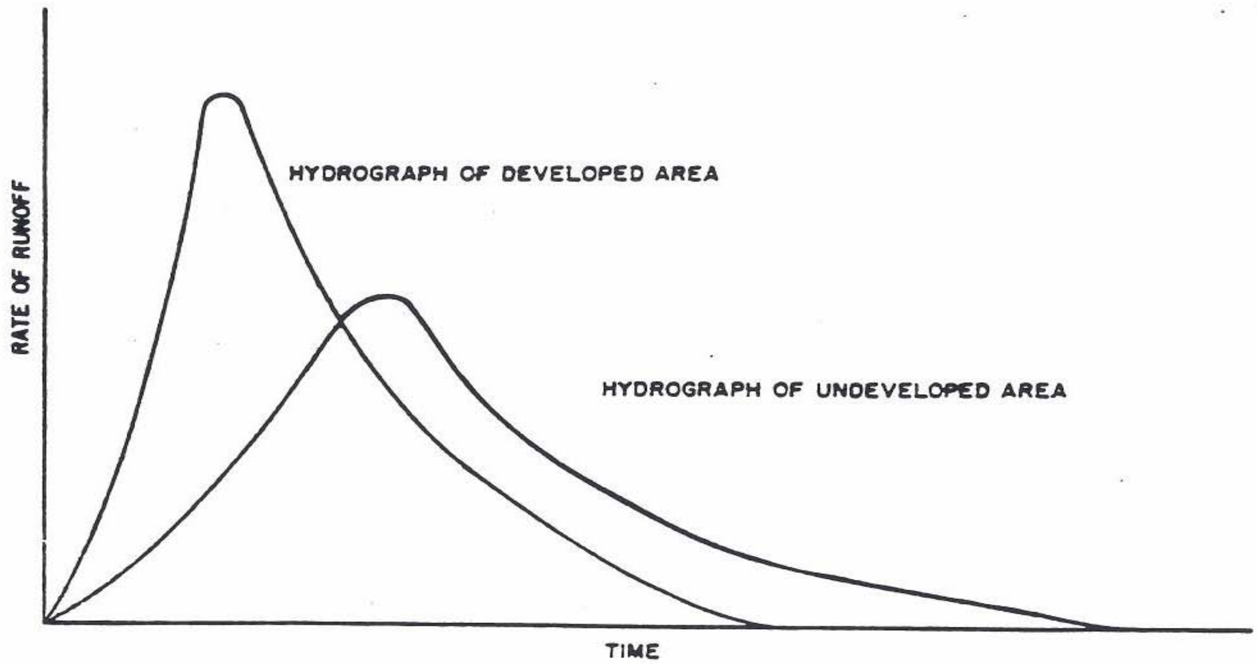
Note: The product of C times C_f should not be greater than 1.0

Source: Urban Storm Drainage Criteria Manual, 1969 (Reference #31)

TABLE 2-5

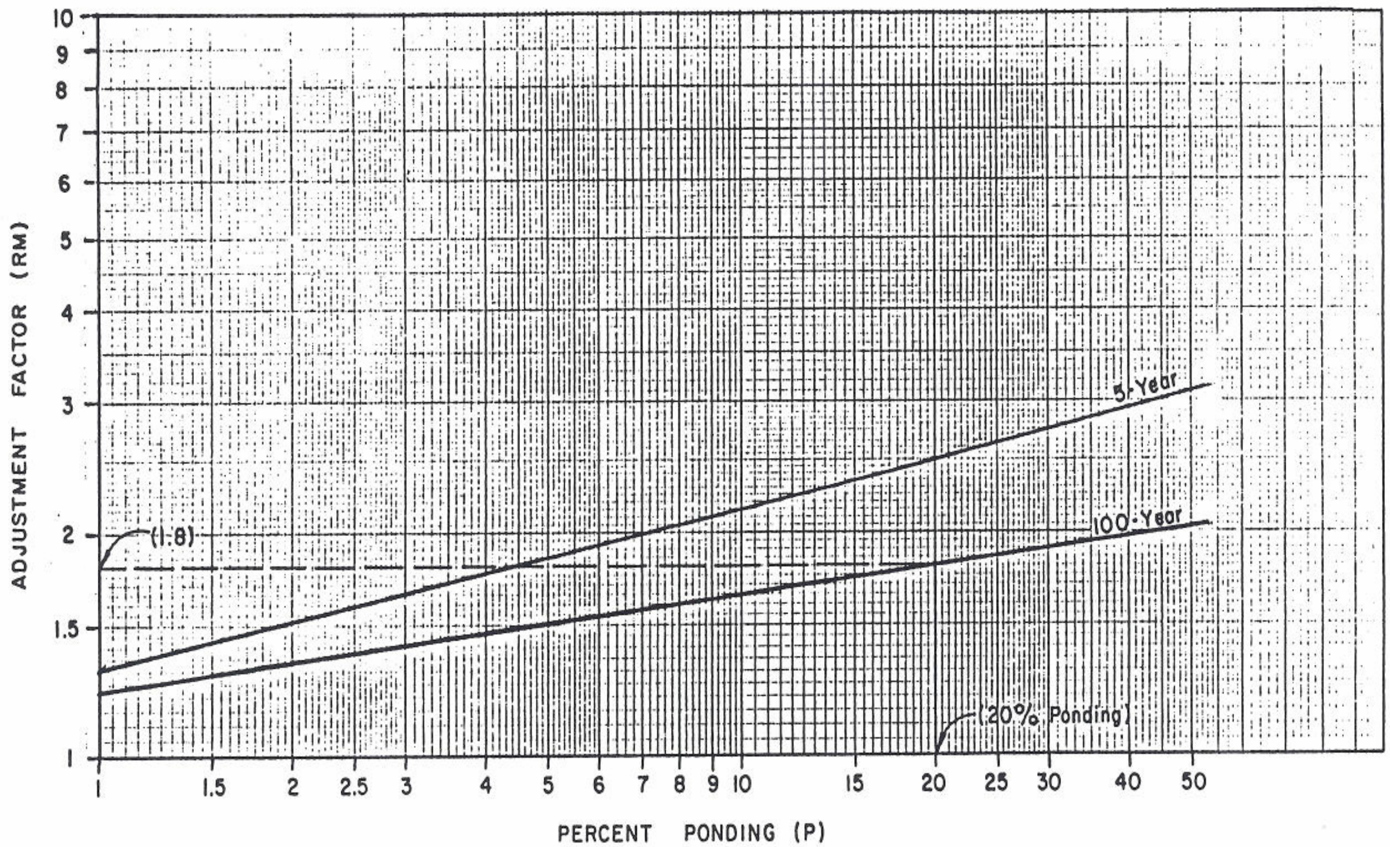
EXCESS RAINFALL FOR COMPUTING RUNOFF VOLUMES

Percent Impervious	10-Yr, 24-Hr Rainfall (in)	Losses (in)	Excess Rainfall (in)	25-Yr, 24-Hr Rainfall (in)	Losses (in)	Excess Rainfall (in)	100 -Yr, 24-Hr Rainfall (in)	Losses (in)	Excess Rainfall (in)
0	8.2	4.13	4.07	9.6	4.43	5.17	12.5	5.14	7.36
20	8.2	3.31	4.89	9.6	3.55	6.05	12.5	4.12	8.38
40	8.2	2.48	5.72	9.6	2.66	6.94	12.5	3.09	9.41
60	8.2	1.65	6.55	9.6	1.77	7.83	12.5	2.06	10.44
80	8.2	0.83	7.37	9.6	0.89	8.71	12.5	1.03	11.47



EFFECT OF WATERSHED DEVELOPMENT
ON STORM HYDROGRAPH

August 1986	FIGURE 2-1

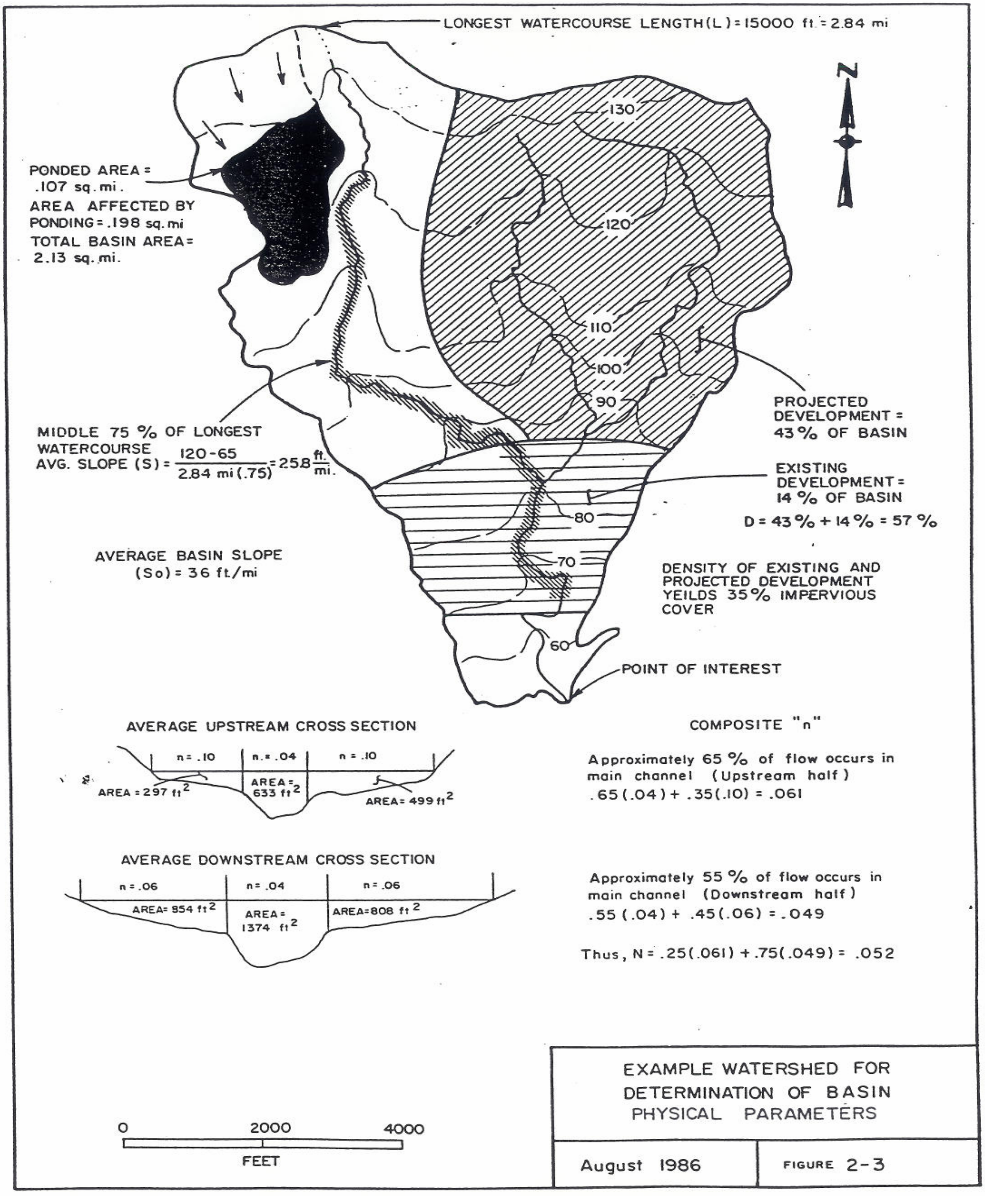


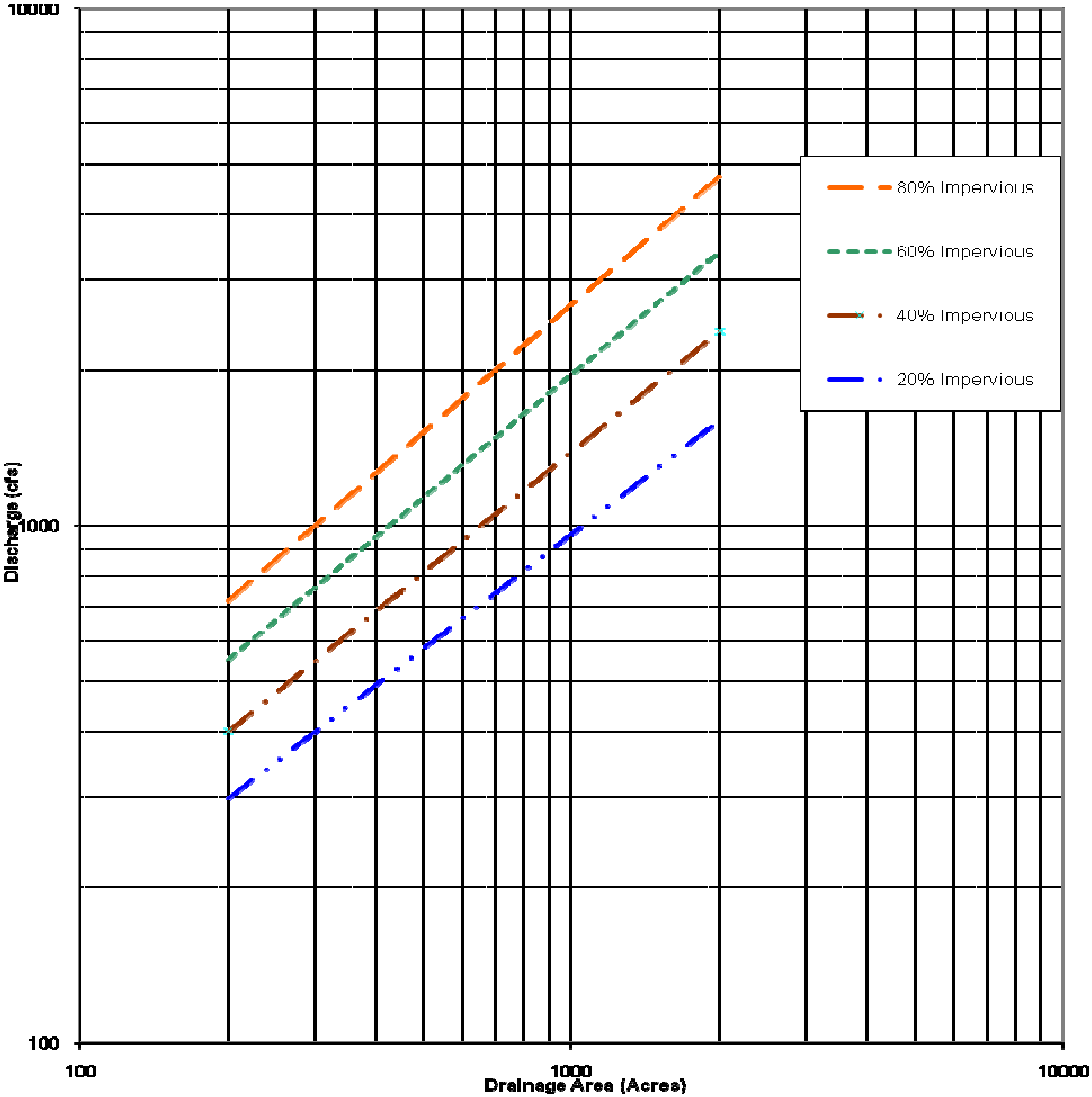
PONDING ADJUSTMENT FACTOR FOR CLARKS STORAGE COEFFICIENT (R) FOR FORT BEND COUNTY, TEXAS

STORM EVENT	ADJUSTMENT FACTOR (RM) EQUATION
5 YEAR	$RM = 1.31\ P^{0.214}$
10 YEAR	$RM = 1.28\ P^{0.199}$
25 YEAR	$RM = 1.25\ P^{0.171}$
50 YEAR	$RM = 1.23\ P^{0.153}$
100 YEAR	$RM = 1.21\ P^{0.132}$
500 YEAR	$RM = 1.17\ P^{0.086}$

Source:
Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas, February, 1984.

August 1986	FIGURE 2-2

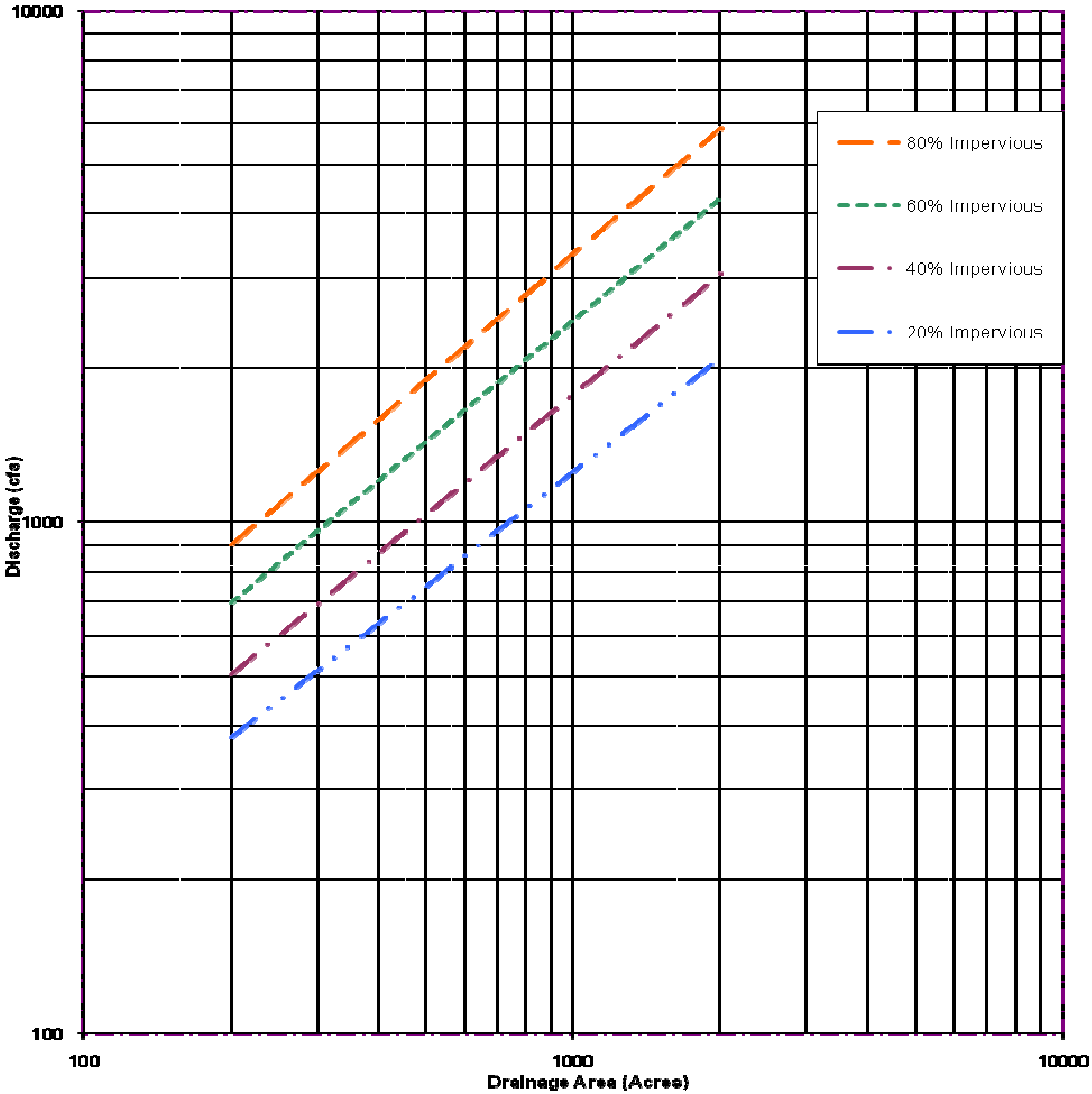

LONGEST WATERCOURSE LENGTH (L) = 15000 ft = 2.84 mi
N
130
120
110
100
90
80
70
60
PONDED AREA = .107 sq. mi.
AREA AFFECTED BY PONDING = .198 sq. mi
TOTAL BASIN AREA = 2.13 sq. mi.
MIDDLE 75 % OF LONGEST WATERCOURSE
AVG. SLOPE (S) = 120-65 / 2.84 mi (.75) = 25.8 ft./mi.
PROJECTED DEVELOPMENT = 43 % OF BASIN
EXISTING DEVELOPMENT = 14 % OF BASIN
D = 43 % + 14 % = 57 %
AVERAGE BASIN SLOPE (So) = 36 ft./mi
DENSITY OF EXISTING AND PROJECTED DEVELOPMENT YEILDS 35% IMPERVIOUS COVER
POINT OF INTEREST
AVERAGE UPSTREAM CROSS SECTION
n = .10
n = .04
n = .10
AREA = 297 ft2
AREA = 633 ft2
AREA = 499 ft2
AVERAGE DOWNSTREAM CROSS SECTION
n = .06
n = .04
n = .06
AREA = 954 ft2
AREA = 1374 ft2
AREA = 808 ft2
COMPOSITE "n"
Approximately 65 % of flow occurs in main channel (Upstream half)
.65 (.04) + .35 (.10) = .061
Approximately 55 % of flow occurs in main channel (Downstream half)
.55 (.04) + .45 (.06) = .049
Thus, N = .25 (.061) + .75 (.049) = .052
0
2000
4000
FEET
EXAMPLE WATERSHED FOR DETERMINATION OF BASIN PHYSICAL PARAMETERS
August 1986
FIGURE 2-3



25-Year Drainage Area-Discharge Curves

for Fort Bend County Texas

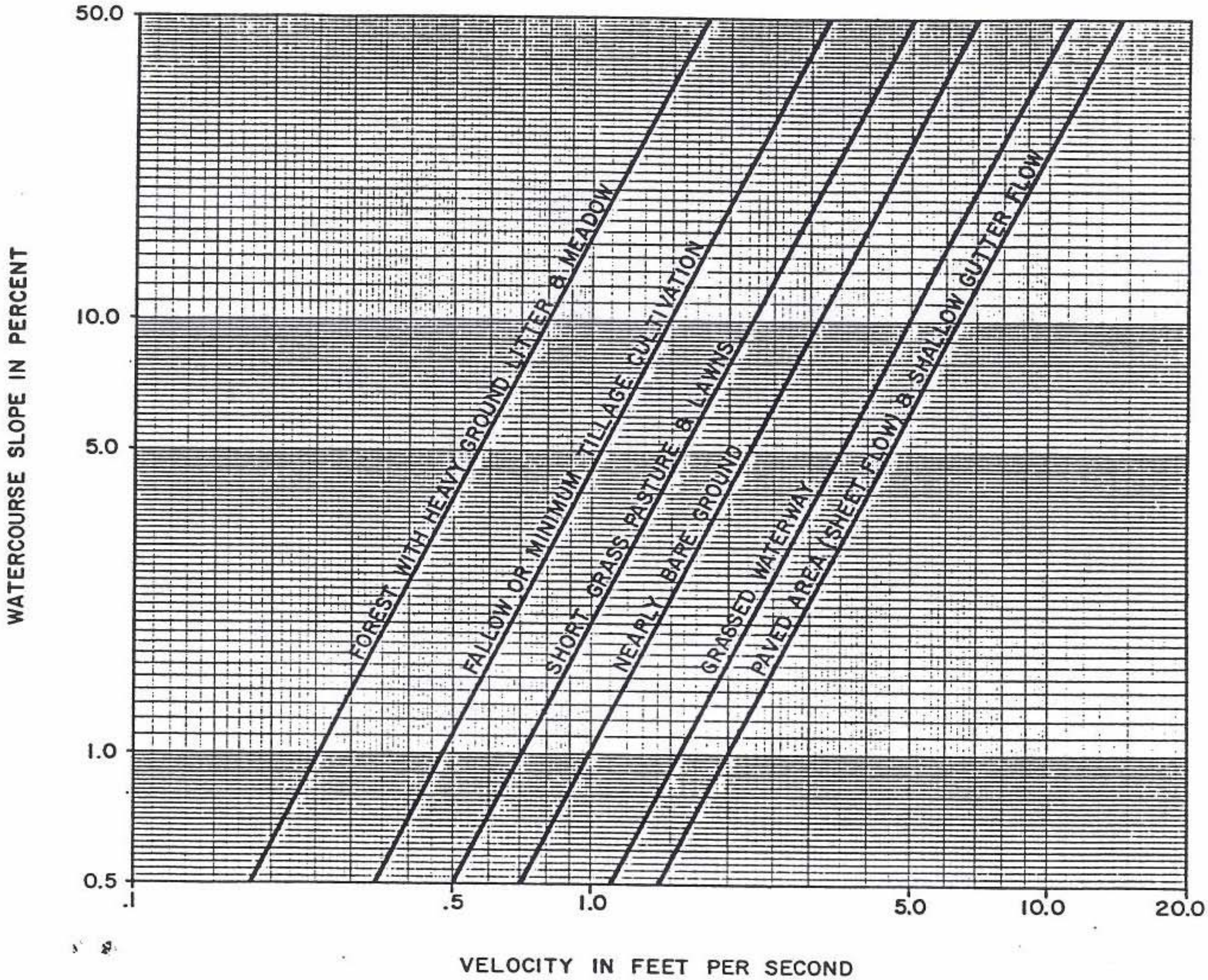
FIGURE 2-4



100-Year Drainage Area-Discharge Curves

for Fort Bend County Texas

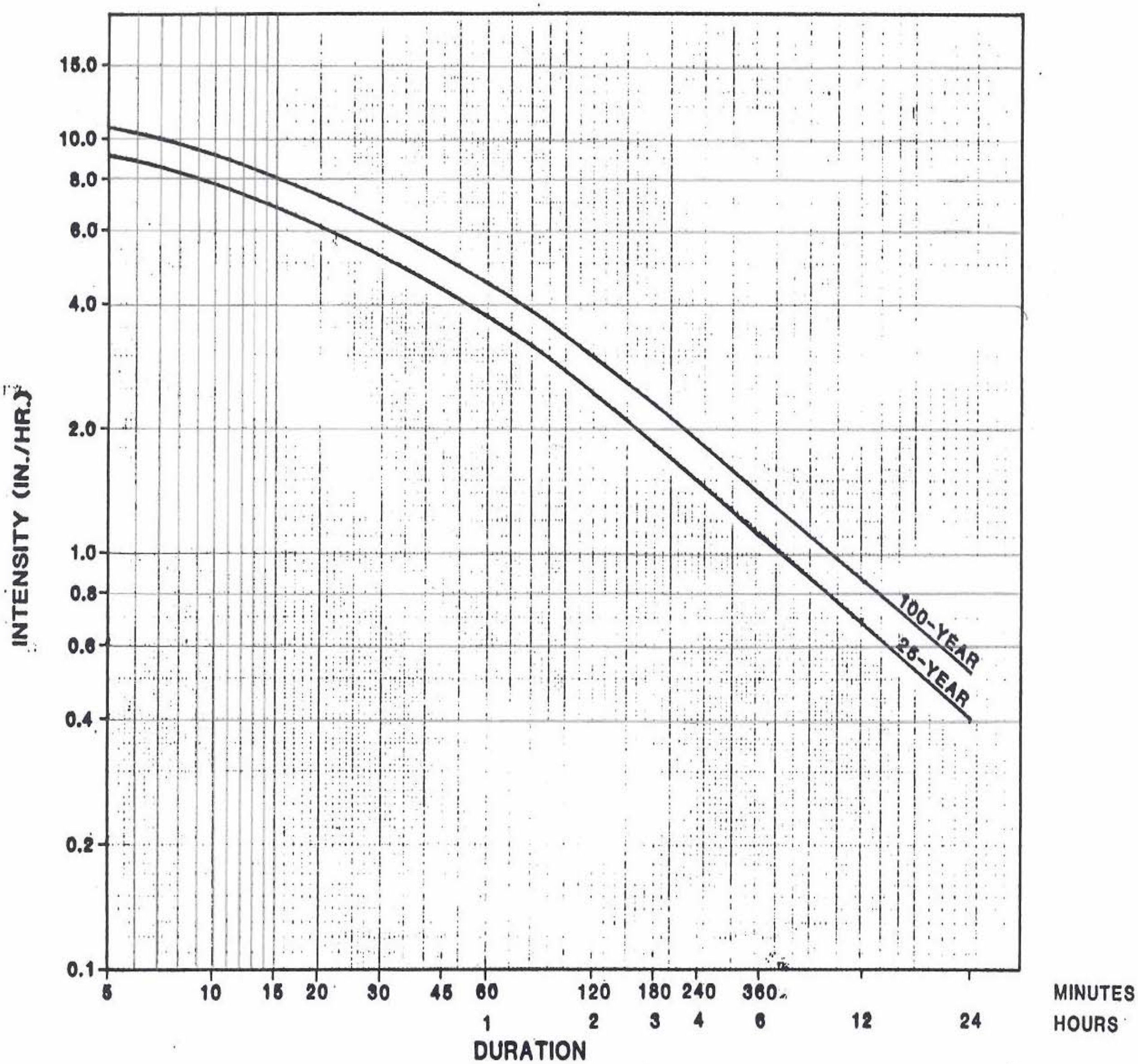
FIGURE 2-5



AVERAGE VELOCITIES FOR ESTIMATING TRAVEL TIME
FOR OVERLAND FLOW
FOR FORT BEND COUNTY, TEXAS

Source: Urban Hydrology for Small Watersheds, Technical Release No. 55, Soil Conservation Service, January, 1975.

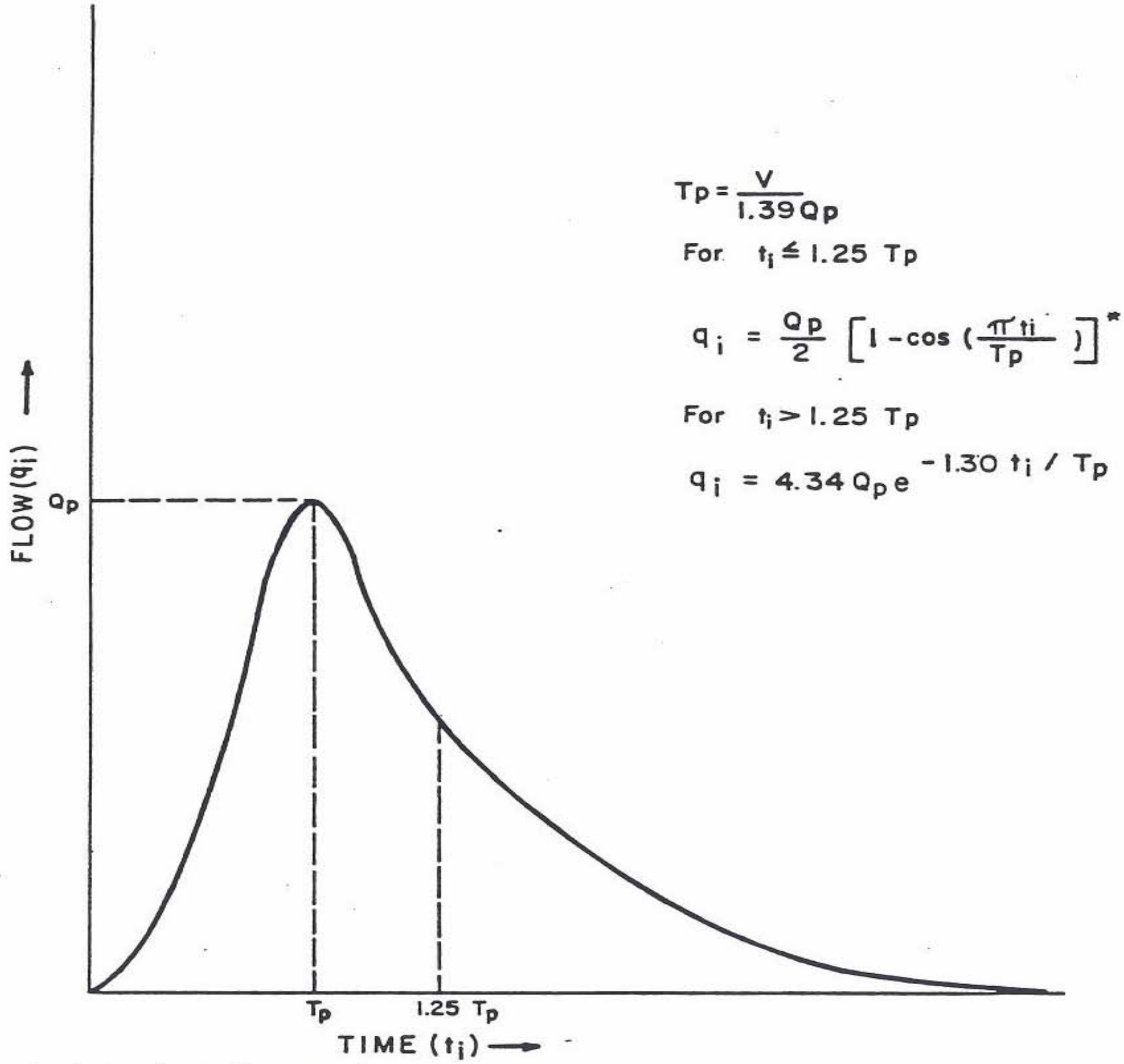
FIGURE 2-6



RAINFALL INTENSITY/DURATION CURVES FOR FORT BEND COUNTY , TEXAS

DEVELOPED FROM TP-40 AND HYDRO-35

JULY 1987 FIGURE 2-7



* With calculator in radian mode.

MALCOM'S METHOD OF HYDROGRAPH DEVELOPMENT

SOURCE: Criteria Manual For Design of Flood Control and Drainage Facilities in Harris County, Texas February, 1984

JULY 1987 FIGURE 2-8

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3.0 OPEN CHANNEL FLOW

3.1 GENERAL

This section summarizes the practical considerations, technical principles, and criteria necessary for proper design of open channels. The analysis of open channel flow also aids in determining other flow-related concerns, such as, culvert tailwater depths, time of concentration calculations (travel times), and flood elevations.

In a major drainage system, open channels offer significant advantages over closed conduits in regard to cost, flow capacity, flood storage, recreation, and aesthetics. However, open channels require considerable right-of-way and maintenance. However, open channels require considerable right-of-way and maintenance. Careful consideration must be given in the design process to insure that disadvantages are minimized and the benefits maximized. When a design approach not covered in the manual is to be used, it should be reviewed and discussed with the Fort Bend County Drainage District Engineer prior to commencing significant portions of the design effort.

All open channel hydraulic computations are to be computed in HEC-RAS version 3.1.3 (or newer) for newly developed models. Versions of HEC-RAS must be consistent throughout each project. Additional models (other than HEC-RAS) may be used, such as SWMM (or SWMM variants like XP-SWMM), FLO-2D, MIKE 11/MIKE FLOOD, Quick 2.1.0, WSPRO, or others.

Prior approval from the Fort Bend County Drainage District Engineer is required to use any hydraulic model other than HEC-RAS. Modeling which will require a FEMA submittal must use a FEMA approved model.

3.2 OPEN CHANNEL HYDRAULICS – AN OVERVIEW

Flow conditions in an open channel are characterized as steady or unsteady, uniform or varied, subcritical or supercritical.

3.2.1 Steady or Unsteady Flow

Steady flow occurs when the velocity of successive fluid particles at a particular location is the same for successive periods of time. Therefore, the velocity is constant with respect to time ($\frac{dv}{dt}=0$) although it may vary at different locations in the channel. This statement implies that the flow rate Q must also be constant with respect to time. For unsteady flow, the velocity at a particular point is not constant with respect to time. The unsteady option in HEC-RAS can be used when the following situations are present:

1. Rapid changes in discharge and elevation
2. Channel network has slopes less than 5 feet/mile
3. Varying tailwater or backwater effects dominate
4. Flood forecasting for major rivers
5. Large and complex river systems

Prior to using unsteady flow modeling, a coordination meeting should be held with FBCDD staff.

3.2.2 Uniform Flow

Uniform flow occurs when the magnitude and direction of the velocity are not changing ($\frac{dv}{dx}=0$) from location to location in the channel. This statement implies that the depth of flow is also not changing with respect to distance along the channel.

A true state of uniform flow is difficult to obtain under most conditions. Nevertheless, when a channel is sufficiently long and sufficiently unchanging such that the flow depth is not changing (i.e. the channel resistance and gravity forces can be considered to be balanced), the flow may be assumed to be uniform for design purposes.

3.2.3 Varied Flow

When the physical configuration, slope, or surface roughness of a channel changes, or when a disturbance such as a weir or bridge embankment is introduced in the channel, the depth and velocity of the flow will vary along the channel in the vicinity of the disturbance. If the degree of change is small enough that a hydrostatic pressure distribution can be assumed in the flow, then the flow is considered to be gradually varied. If the degree of change is so large that the pressure distribution is no longer hydrostatic at the point of change, then the flow profile is rapidly varied and must be analyzed on a site- specific basis.

3.2.4 Subcritical or Supercritical Flow

The celerity of small gravity waves in a shallow channel is given by the term $(gy)^{1/2}$ where g is the acceleration due to gravity and y is the depth. When the velocity of flow in a channel exceeds this value, the flow is supercritical. When it is less than this value, the flow is subcritical. Hence, the ratio of velocity of flow to celerity $(v/(gy)^{1/2})$, known as Froude Number, is less than 1 for subcritical flow and more than 1 for supercritical flow. Supercritical flow is generally characterized by high velocities and shallow depths, while subcritical flow is characterized by slower velocities and greater depths. The most important distinction between these two states of flow is that the effect of a disturbance in the channel, such as a bridge constriction, cannot be propagated upstream in supercritical flow as it can in subcritical flow. Therefore, subcritical flow is controlled by downstream channel conditions while supercritical flow is controlled by upstream channel conditions.

3.2.5 Critical Depth

When the velocity of flow in a channel is equivalent to the velocity of a gravity wave $(gy)^{1/2}$, critical flow at critical depth exists. Hence, for critical flow, the value of the Froude Number is 1. Flow at or around critical is characterized by instability and should be avoided in channel design except at specific flow transition points such as weirs and sluice gates. Near critical flow, small changes in hydraulic conditions will cause exaggerated changes in depth and velocity.

The critical depth for a given channel configuration and flow rate can be determined using the following procedure:

From open channel hydraulics theory it is given that specific energy ($E=y + v^2/2g$) is at a minimum when the depth is critical. By differentiating the expression for specific energy and further manipulating the resulting equation, the depth (y) becomes critical depth (y_c) and the following expression is obtained for application to a trapezoidal channel:

$$Q/ (g)^{1/2} = \frac{(b(y_c)+z(y_c)^2)^{3/2}}{(b+2zy_c)^{1/2}} \tag{3-1}$$

where b = channel bottom width (ft)

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

y_c = critical depth (ft)

Q = discharge (cfs)

z = channel side slope where z equals the horizontal displacement for one unit of vertical displacement.

Thus if Q, z, and b are known, the critical depth can be determined by solving Equation 3-1 to find y_c by trial.

3.2.6 Manning's Equation

Manning's equation is an empirical equation which related friction slope, flow depth, channel roughness, and channel cross-sectional shape to flow rate. The friction slope is a measure of the rate at which energy is being lost in the flow to channel resistance. When the channel slope and the friction slope are equal ($S_f = S_o$) the flow is uniform and Manning's equation may be used to determine the depth for uniform flow (normal depth).

Manning's equation is as follows:

$$V = \frac{1.49}{n} R^{2/3} S_f^{1/2} \tag{3-2}$$

Or

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

where Q = total discharge (cfs)

V = velocity of flow (ft/sec)

n = Manning's coefficient of roughness

A = cross-sectional area of the flow (ft^2)

R = hydraulic radius of the channel (ft) (flow area/wetted perimeter)

S_f = friction slope, the rate at which energy is lost due to channel resistance

Figure 3-1 provides a nomograph for the solution of Equation 3-2.

Normal depth may be determined by using Equation 3-3. The area (A) and the hydraulic radius (R) are written in terms of the depth (y_o). Knowing the discharge (Q), Manning's "n" value, and the channel slope (S_o), Equation 3-3 can be solved by trial to find normal depth (y_o). Figure 3-2 provides a nomograph for the solution of Equation 3-3 for trapezoidal channels.

3.2.6.1 Manning's "n" Value

Manning's "n" value is an experimentally derived constant which represents the effect of channel roughness in the Manning's equation. Considerable care must be given to the selection of an appropriate "n" value for a given channel due to its significant effect on the character of the flow. Table 3-1 provides a listing of "n" values for various channel conditions. Table 3-2 presents a method to compute a roughness coefficient based on various channel characteristics.

3.3 CHANNEL DESIGN

The proper hydraulic design of a channel is of primary importance to insure that nuisance drainage conditions, flooding, sedimentation and erosion problems to not occur. The following general criteria should be utilized in the design of open channels.

3.3.1 Design Frequency

Open channels shall be designed to contain the runoff from the 100-year frequency 24-hour duration storm within the channel banks while providing one foot of freeboard. In those cases where channel modifications are necessary to control increased flows from proposed development, there should be no increase in water surface elevations in the hydraulic model upstream or downstream of the proposed project for the design frequencies noted in Section 3.3.2. In addition, the channel must be designed to have sufficient freeboard to provide for adequate drainage of lateral storm sewers during the 25-year storm. If the capacity of the existing channel downstream of the project is less than the 100-year design discharge, consideration shall be given for more frequent events to ensure that the frequency of downstream flooding is not increased.

3.3.2 Required Analyses

In order to ensure that the design is adequate, analyses must be performed for the 10-year, 25-year, and 100-year storm events. Lesser events may be required by the Drainage District, depending on local conditions.

The following information must be submitted to the Fort Bend County Drainage District Engineer for the design of open channels.

1. A vicinity map of the site and subject reach. The subject reach is defined as the stretch of channel necessary for any altered flow profile to match the upstream and downstream existing profiles.

2. A detailed map of the area and subject reach with all pertinent physiographic information.

3. A watershed map showing the existing and proposed drainage area boundary along with all subarea delineations and all areas of existing or proposed development.

4. Discharge calculations specifying methodology and key assumptions used including discharges at key locations.

5. Hydraulic calculations specifying methodology used. All assumptions and values of the design parameters must be clearly stated.

6. A profile of the subject reach which includes the following:

   a. All pertinent water surface profiles. This will minimally include the 10-, 25- and 100-year frequency floods for both existing and proposed channel conditions.
   b. All existing and proposed bridge, culvert and pipeline crossings.
   c. The location of all tributary and drainage confluences.
   d. The location of all hydraulic structures (e.g. dams, weirs, drop structures, etc.)

7. A map delineating existing and proposed rights-of-way.

8. Benchmark, elevation, datum and year of adjustment.

9. Typical existing and proposed cross-sections.

10. A soils report which addresses erosion and slope stability.

In zones specified by FEMA as Special Flood Hazard Areas (SFHAs), including Zones A, AE, AO, and AH, additional hydraulic analyses may be required. A floodway analysis is required when changes are proposed to regulatory floodways. No development or other encroachment including fill is allowed in a regulatory floodway which will result in an increase in Base Flood Elevations (BFEs). Any fill in the SFHA that reduces the conveyance capacity of the flooding source must be offset with a hydraulically equivalent mitigation volume to maintain or improve the existing conveyance capacity. Floodplain storage mitigation shall be required for all creeks and streams except the Brazos River. In areas of the SFHA noted as approximate study (Zone A), a 1:1 ratio of cut to fill is required for any fill within that portion of the SFHA noted as Zone A.

3.3.3 Design Considerations

The path taken by an existing, naturally-carved channel often represents the most logical general pathway of flow. For runoff rates associated with undeveloped conditions, the natural channel is largely stable against erosion and is topographically efficient in draining adjacent land. In light of this, it is logical that the engineer should consider taking advantage of naturally carved drainageways when locating and designing open channels.

Although there are numerous channel designs available to the engineer, a judicious design must conform to certain hydraulic, aesthetic, and safety-related standards. In situations where the use of a natural drainage course is infeasible, the engineer must choose between an earthen channel and a lined channel. Grassed channels generally produce lower flow velocities and greater channel storage. They are, in most cases, aesthetically and economically superior to concrete lined waterways. However, grass-lined channels require more right-of-way, are vulnerable to erosion, and must be continually maintained. They can also have problems with side slope stability and/or sediment deposition.

In areas where land values are extremely high, or right-of-way is limited, concrete lined channels may be the design of choice. However, concrete channels can be significantly more expensive. In addition, they tend to move water faster and store less water possibly resulting in higher peak discharges downstream.

3.3.3.1 Optimal Design Flow Characteristics

When designing a channel, the following flow considerations should be addressed:

Velocity – Excessive velocities can cause erosion and may pose a threat to safety. Velocities which are too low may allow sediment deposition and subsequent channel clogging. Table 3-3 provides average and maximum allowable velocities based on the

25-year flow. Minimum velocities are those produced by a channel invert slope of 0.05 percent.

Flow Depth – Deep channels are generally difficult to maintain and can be hazardous. Therefore, design depths should be as shallow as practical while allowing enough depth to accommodate future storm sewer systems.

Freeboard – Since there is no universally accepted rule governing the amount of freeboard required for a channel, selection of a safe amount should be based on confidence in the design discharge estimates, stability of the flow profile and the expected damage from water overflowing channel banks. A minimum value of one foot is required to provide the needed safety. The necessity for additional freeboard shall be considered on the outside channel edge along curves.

3.3.3.2 Optimal Channel Configuration Characteristics

When designing a channel, the following guidelines for the physical configuration of the channel should be observed:

Invert Slope – Slope of the channel invert is generally governed by topography and the energy head required for flow. Since invert slope directly affects channel velocities, channels should have sufficient grade to prevent significant siltation but grades should not be so large as to create erosion problems. In Fort Bend County, the minimum recommended channel invert slope shall be 0.05 percent. Topographic conditions may necessitate a flatter slope in certain areas and prior discussion with the Fort Bend County Drainage District Engineer is suggested. The maximum channel invert shall be limited by maximum flow velocities as given in Table 3-3. Appropriate channel drop structures may be used to limit channel invert slope in steep areas.

Side Slope – In grass-lined channels, normal maximum slope is 4 (horizontal):1 (vertical), which is also the practical limit for mowing equipment. In some areas, side slopes flatter than 4:1 may be necessary due to local soil conditions.

Bottom Width – In grass-lined channels the minimum channel bottom width should be six feet. In concrete-lined channels the minimum bottom width should be eight feet.

Curvature – In general, centerline curves should be as gradual as possible and not have a radius of less than three times the design flow top width unless erosion protection is provided and not less than 100 feet. The maximum curvature for any man-made channel should be 90º.

Manning's "n" Value – The following values of the Manning's roughness coefficient should be used in man-made channels. Alternative values should be discussed with the Fort Bend County Drainage District Engineer.

Channel Cover	"n" Value
Grass-lined	0.04
Concrete-lined	0.015

Confluences – The angle of intersection between the tributary and main channels should be between 15º and 45º. Angles in excess of 45º are permissible but are discouraged. Angles in excess of 90º are not permitted. If the ditch or channel is enlarged, deepened, or new, the Fort Bend County Drainage District Engineer will require the addition of an adequately sized and designed access path across the ditch or channel to allow for access of maintenance equipment. This may include the requirement for pipe(s), stabilized access location or concrete lining. Coordinate with the Fort Bend County Drainage District Engineer for each location.

Transitions – Expansions and contractions should be designed to create minimal flow disturbance and thus minimal energy loss. Transition angles should be less than 12 degrees. When connecting rectangular to trapezoidal channels, a warped or wedge-type transition is recommended.

Location – Channels should be located a sufficient distance away from existing and proposed roads, buildings, and other infrastructure to protect the stability of those items. Where channels cross roadways, adequate slope stabilization and erosion control measures shall be provided.

3.3.4 Minimum Requirements for Channel Design

The minimum requirements for the design of various type channels applicable to Fort Bend County are listed below. Requirements for grass-lined and concrete-lined channels are listed in the following sections.

3.3.4.1 Grass-Lined Channels

The following are minimum requirements to be used in the design of all grass-lined channels:

1. Maximum side slopes shall be 4:1. Slopes flatter than 4:1 may be necessary in some areas due to local soil conditions.

2. Minimum bottom width is six (6) feet.

3. A minimum maintenance berm is required on both sides of the channel of between 15 and 30 feet, depending upon channel size. For top widths of 30 feet or less, 15-foot berms are acceptable, for top widths between 30 and 60 feet, 20-foot berms are required, and for top widths of 60 feet or greater, 30-foot berms are required along both sides of the channel. The elevation of the top of the berm should be at natural ground along the channel reach. See Table 3-4.

4. Backslope interceptor structures are necessary at a maximum of 800 foot intervals to prevent sheet flow over the ditch side slopes.

5. Channel slopes must be revegetated with a perennial grass cover (typically Common Bermuda) immediately after construction to minimize erosion.

6. Flow from roadside ditches must be conveyed to the channel through a roadside ditch interceptor structure and pipe. See ditch interceptor structure and pipe detail, Figure 5-5.

7. Unless waived by the Fort Bend County Drainage District Engineer, a geotechnical investigation and report must be provided.

3.3.4.2 Concrete-Lined Trapezoidal Channels

All partially or fully concrete-lined trapezoidal channels must meet or exceed the following minimum design requirements:

1. All concrete shall be Class A concrete unless noted otherwise.

2. Fully lined cross-sections shall have a minimum bottom width of eight (8) feet.

3. Concrete slope protection placed on 3:1 side slopes shall have a minimum thickness of 5 inches and minimum 6 x 6 x W2.9 x W2.9 welded wire fabric or equivalent reinforcing.

4. Concrete slope protection placed on 2:1 side slopes shall have a minimum thickness of 5-inches and minimum 6 x 6 x W4.0 x W4.0 welded wire fabric or equivalent reinforcing.

5. The minimum side slopes for any concrete lined areas shall be 2:1 and ensure that the escape stairways are included as per Sec. 3.3.4.3 (6).

6. All slope paving shall include a minimum 18-inch toe wall at the top and sides and a 24-inch toe wall across or along the channel bottom for clay soils. In sandy soils, a 36-inch toe wall is recommended across the channel bottom.

7. In instances where the channel is fully lined, backslope drainage structures may not be required. Partially lined channels will require backslope drainage structures.

8. Weep holes shall be used to relieve hydrostatic head behind lined channel sections. The specific type, spacing and construction method for the weep holes will be based on the recommendations of the geotechnical report.

9. Where construction is to take place under conditions of mud and/or standing water, a seal slab of Class C concrete shall be placed in channel bottom prior to placement of concrete slope paving.

10. Control joints shall be provided at approximately twenty-five feet on center. The use of a sealing agent shall be utilized to prevent moisture infiltration.

3.3.4.3 Rectangular Concrete Pilot Channels

In areas where it is necessary to use a vertical-walled rectangular section, the following minimum requirements are to be addressed:

1. All concrete shall be Class A concrete unless noted otherwise.

2. The structural steel design should be based on ASTM A 615, Grade 60 steel.

3. Minimum bottom width shall be eight (8) feet.

4. For bottom widths twelve (12) feet or greater, the channel bottom shall be graded at 1% toward the channel center line.

5. Minimum height of vertical walls shall be four (4) feet. Heights above this shall be in two (2) foot increments. Exceptions will be considered on a case-by-case basis.

6. Escape stairways shall be located at the upstream side of all street crossings, but not to exceed 1,400 feet intervals.

7. For rectangular concrete pilot channels with grass side slopes the top of the vertical wall should be constructed to allow for future placement of concrete slope paving.

8. Weep holes should be used to relieve hydrostatic pressures. The specific type, spacing and construction method for the weep holes will be based on the recommendations of the geotechnical report.

9. Where construction is to take place under conditions of mud and/or standing water, a seal slab of Class C concrete should be placed in channel bottom prior to placement of concrete slope paving.

10. Concrete pilot channels may be used in combination with slope paving or a maintenance shelf. Horizontal paving sections should be analyzed as one way paving capable of supporting maintenance equipment having a concentrated wheel load of up to 1,350 lbs.

11. Control joints shall be provided at approximately twenty-five feet on center. The use of a sealing agent shall be utilized to prevent moisture infiltration.

3.4 EROSION

Erosion protection is necessary to insure that channels maintain their capacity and stability and to avoid excessive transport and deposition of eroded material. The three main parameters which affect erosion are vegetation, soil type and the magnitude of flow velocities and turbulence. In general, silty and sandy soils are the most vulnerable to erosion.

The necessity for erosion protection should be anticipated in the following settings:

1. Areas of channel curvature, especially where the radius of the curve is less than three times the design flow top width.

2. Around bridges where channel transitions create increased flow velocities.

3. When the channel invert is steep enough to cause excessive flow velocities.

4. Along grassed channel side slopes where significant sheet flow enters the channel laterally.
5. At channel confluences.

6. In areas where the soil is particularly prone to erosion.

Sound engineering judgment and experience should be used in locating areas requiring erosion protection. It is often prudent to analyze potential erosion sites following a significant flow event to pinpoint areas of concern.

3.4.1 Minimum Erosion Protection Requirements

Minimum requirements for Fort Bend County are as follows:

Confluences – Figure 3-3 presents the minimum requirements for determining when erosion protection or channel lining are necessary given the angle of the confluence. A healthy cover of grass must also be established above the top edge of the lining extending to the top of the bank. The top edge of the lining shall extend to the 25-year water surface elevation.

Bends – When required, erosion protection must extend along the outside bank of the bend and at least 20 feet downstream. Additional protection on the channel bottom and inside bank, or beyond 20 feet downstream, will be required if maximum allowable velocities are exceeded. See Table 3-3.

Culverts – In areas where outlet velocities exceed five feet per second on to a grass-lined channel, channel lining or an energy dissipation structure will be required.

Outfalls – Erosion protection will be necessary in areas of high turbulence or velocity as typically found at the outfall of backslope drains, roadside ditches, and storm sewers into the main channel. See Figures 3-4, 3-5, 3-6 and 3-7 for typical pipe and storm sewer outfall details. Toe walls at edges of slope paving shall be a minimum of 18” deep.

3.4.2 Structural Erosion Controls

When flow velocities exceed those allowed in Table 3-3 or when soils are deemed excessively erosive by a geotechnical engineer, acceptable structural erosion control shall be provided. The slope protection must extend up the channel bank at least to the elevation of the 25-year flood level.

3.4.2.1 Riprap

The use of riprap is an allowable erosion control measure only in those locations where concrete slope paving is not feasible. Riprap is defined as broken concrete rubble or well-rounded stone. A discussion of riprap design can be found in Hydraulic Design of Flood Control Channels, EM 1110-2-1601, U.S. Department of the Army, Corps of Engineers, July 1970 (or latest version).

3.4.2.2 Concrete Slope Paving

Minimum requirements for partially or fully concrete-lined channels are presented in Section 3.3.

3.4.2.3 Backslope Drainage Systems

The use of backslope drains and swales is required in Fort Bend County. These systems collect overland flow from channel overbanks and other areas not draining to the storm sewer collection system. Their purpose is to prevent excessive overland flow from eroding grass-lined channel side slopes as it enters the channel. Subject to County approval, back-slope drains may not be required in undeveloped or sparsely developed areas.

The design engineer should carefully consider the drainage area to be intercepted by such systems, particularly when the channel passes through large areas of undeveloped acreage where large quantities of naturally occurring sheet flow could overload the backslope swale and drainage system. In these areas, drain spacing and backslope drainage pipe requirements may have to be modified to account for the conditions. Refer to Figure 3-4 for backslope drainage design.

Documentation of drainage area for each backslope drain system, as well as hydraulic pipe and swale sizing calculations, must be provided by the engineer.

General requirements for backslope drains and swales are as follows:

1. Minimum backslope drain pipe shall be 24” in diameter.

2. Maximum spacing is 800 feet (or 400 feet to the swale high point).

3. The drain structure and swale centerline should be six feet inside the channel right-of-way line.

4. Minimum design depth in swale is 0.5 feet.

5. Maximum design depth in swale is 2.0 feet.

6. Minimum gradient for swale invert is 0.2%.

7. Swale should have a maximum side slope of 3:1.

3.4.2.4 Sloped Drops

Sloped drop structures are recommended when the required drop elevation is small, generally 1-4 feet. They tend to be the most economical and topographically versatile means to accomplish a drop. Slope drops should be no steeper than 3:1 and no flatter than 4:1.

Slopes drops shall be constructed of concrete slope paving or of cellular concrete articulated mats. Riprap or an appropriate alternate erosion protection shall be provided upstream and downstream of the drop.

When subcritical flow approaches a drop, depth decreases and velocity increases as the flow nears critical. Accordingly, appropriate erosion protection must be provided sufficiently upstream such that flow velocities are not excessive in any unprotected reach of channel. The minimum recommended distance is 20 feet.

Downstream of the drop, the required length for protection is dependent on the length of the hydraulic jump. As a rough estimate the jump length may be assumed equal to q/2, one-half of the design flow per unit width of channel. The use of riprap or a combination of riprap and concrete slope paving is recommended downstream of the drop to force the jump closer to the drop. A minimum of 20 feet of riprap is required downstream of any slope paving used at a drop structure to help reduce velocities and protect the concrete toe. The minimum recommended apron length is 40 feet.

3.4.2.5 Baffled Chutes

Baffled chutes are used in drainageways when a relatively large change in elevation is necessary. The baffle blocks prevent undue acceleration of the flow as it passes down the chute. Baffled chutes are generally laid out on a 2:1 slope (no steeper) and can be designed to discharge up to 60 cfs per foot of channel width. The lower end of the chute is constructed to below streambed level and backfilled as necessary thereby minimizing degradation or scour of the streambed. No tailwater or stilling basin is required as velocities will remain moderate.

The following simplified step-by-step procedure taken from the U.S. Department of the Interior Bureau of Reclamation publication, Progress Report V – Research Study on Stilling Basins, Energy Dissipators, and Associated Apurtenances, Section 9, Hydraulic Laboratory Report No. Hyd-445, April 28, 1961 (or latest version) and is recommended for the design of baffled chutes. For a more detailed discussion, the engineer is referred to U.S. Department of the Interior Bureau of Reclamation publication, Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25.

Step-By-Step Design Procedure:

1. The baffled apron should be designed for the 100-year discharge, Q.

2. The unit discharge q = Q/W may be as high as 60 cubic feet per second per foot of chute width, W. Less severe flow conditions at the base of the chute exist for 35 cubic feet per second and a relatively mild condition occurs for unit discharges of 20 cubic feet per second and less.

3. Entrance velocity, V_1, should be as low as practical. Ideal conditions exist when $V_1 = (gq)^{1/3} - 5$ (See Curve D, Figure 3-8). Flow conditions are not acceptable when $V_1 = (gq)^{1/3}$ (See Curve C, Figure 3-8).

4. The vertical offset between the approach channel floor and the chute is used to create a stilling pool or desirable V_1 and will vary in individual installations; Figure 3-9 shows a typical approach pool. Use a short radius curve to provide a crest on the 2:1 sloping chute. Place the first row of baffle piers close to the top of the chute no more than 12 inches in elevation below the crest.

5. The baffle pier height, H, should be about $0.8D_c$ (see Curve B, Figure 3-8). The critical depth on the rectangular chute is $D_c = (q^2/g)^{1/3}$ (see Curve A, Figure 3-8). Baffle pier height is not a critical dimension but should not be less than recommended. The height may be increased to $0.9\ D_c$.

6. Baffle pier widths and spaces should be equal, preferably about 3/2 H, but not less than H. Other baffle pier dimensions are not critical; suggested cross section is shown in Figure 3-9. Partial blocks, width 1/3 H to 2/3 H, should be placed against the training walls in Rows 1, 3, 5, 7, etc., alternating with spaces of the same width in Rows 2,4, 6, etc.

7. The slope distance between rows of baffle piers should be 2 H, twice the baffle height H. When the baffle height is less than 3 feet, the row spacing may be greater than 2 H but should not exceed 6 feet.

8. The baffle piers are usually constructed with their upstream faces normal to the chute surface; however, piers with vertical faces may be used. Vertical face piers tend to produce more splash and less bed scour, but differences are not significant.

9. Four rows of baffle piers are required to establish full control of the flow, although fewer rows have operated successfully. Additional rows beyond the fourth maintain the control established above, and as many rows may be constructed as is necessary.

The chute should be extended to below the normal downstream channel elevation. At least one row of baffles should be buried in the backfill.

10. The chute training walls should be three times as high as the baffle piers (measured normal to the chute floor) to contain the main flow of water and splash. It is impractical to increase the wall heights to contain all the splash.

11. Erosion protection measures should be placed at the downstream ends of the training walls to prevent eddies from undermining the walls.

3.5 WATER SURFACE PROFILES

The state of flow in a channel is at all times either uniform, gradually varied, or rapidly varied. A different method for determining water surface profiles is applicable to each of these conditions of flow.

3.5.1 Uniform Flow

When a section of channel is sufficiently long and unchanging such that the flow depth is not changing (i.e. the forces of gravity and channel resistance can be considered balanced), then the flow profile can be analyzed, assuming uniform flow. Under these circumstances, the depth, which is constant, can be determined with Manning's equation (see Section 3.2.6).

3.5.2 Gradually Varied Flow

In the majority of channel flow situations, the state of flow is gradually varied. In other words, the depth is gradually changing with longitudinal distance along the channel due to an imbalance between the forces of gravity and channel resistance.

The recommended means for determining flow profiles under these conditions is with the standard step method. The standard step method is an iterative process in which the one-dimensional energy equation is solved to find the water surface elevation at a cross-section. Manning's equation is utilized to determine channel losses due to friction. Losses due to channel non-uniformities are usually calculated with empirical coefficients.

A widely accepted computer model for calculating gradually varied flow profiles is the U.S. Army Corps of Engineers' program HEC-RAS, River Analysis System. The use of the older HEC-2 software program could be considered on a case by case basis. Discuss your hydraulic modeling approach with the Fort Bend County Drainage Engineer prior to beginning the hydraulic analysis.

The HEC-RAS model can readily accommodate modifications in channel design and losses at bridges, culverts, drop structures, and transitions. The program begins computation at a cross-section of known or estimated water surface elevation and proceeds upstream for subcritical flow, and downstream for supercritical flow.

The following general guidelines should be followed with the use of the HEC-RAS program:

1. Cross-sections should be spaced such that the channel configuration between them is largely uniform. In areas where channel properties are rapidly changing, the distance between cross-sections should be appropriately less.

2. The accuracy of the flow profile is largely dependent on a correct determination of the starting water surface elevation, especially in the vicinity of the first cross-section. The best method of determining starting water surface elevation is with a known rating curve or from past backwater studies. The least favorable is the slope-area method, which determines normal depth given the friction slope and discharge. It is important to begin water surface profile analyses a significant distance downstream of the point(s) of interest for subcritical flow and upstream of the point(s) of interest for supercritical flow.

3. Errors can occur with the improper handling of energy losses, thus loss coefficients should be chosen carefully. The engineer should carefully select a particular bridge routing and understand its operation. If the independent hand calculation of a head loss can be accomplished more accurately, it should be input to the program. Proper care should be taken to ascertain that computed losses are reasonable.

3.5.3 Rapidly Varied Flow

When depth changes abruptly over a short distance the flow profile is rapidly varied. Rapidly varied flow is a local phenomenon which occurs in such areas as the contraction beneath a sluice gate, where the channel slope changes from mild to steep, where the flow passes over a weir, and in a hydraulic jump. Determination of the change of the flow profile at such locations must be carried out on a site-specific basis by the engineer.

3.5.4 Energy Losses

Analysis of flow profiles in open channels must include proper consideration of energy losses due to local disturbances such as bridges, drop structures, transitions and confluences. In many cases, such head losses are adequately handled with empirical coefficients. When specific site conditions warrant a more careful analysis, or when a particular program cannot handle local losses, hand calculated losses may be utilized in the flow profile. The following guidelines should be followed for typical sources of non-frictional energy loss.

3.5.4.1 Expansions and Contractions

Losses at transitions are generally expressed in terms of the absolute change in velocity head between downstream and upstream of the transition. The head loss is given by:

$$h_1 = C\frac{(V_2^2 - V_1^2)}{2g} \qquad (3\text{-}4)$$

where	h_1	=	head loss across the transition (ft)
	C	=	empirical expansion or contraction coefficient
	V_2, V_1	=	average channel velocity (fps) of the downstream and upstream sections, respectively
	g	=	acceleration of gravity (32.2 ft/sec^2)

Typical transition loss coefficients for subcritical flow are as follows:

Transition	Coefficient	
	Contraction	Expansion
Gradual or warped	0.1	0.3
Bridge Sections, wedge, Straight-lined	0.3	0.5
Abrupt or square-edged	0.6	0.8

Source: HEC-RAS User Manual.

The above transition loss coefficients are also adequate for general design with supercritical flow; however, the effects of standing waves and other considerations make exact determination of losses in supercritical flow difficult. Therefore, with important transitions, a more detailed analysis may be necessary (see Section 3.6).

3.5.4.2 Bends

The HEC-RAS program does not make allowances for energy losses due to significant bends in the channel. In most cases, losses in channel bends are negligible. However, when the radius of a bend is less than three times the design top width of flow, energy losses due to the bend should be specifically included in the backwater analysis. Any bend loss analysis should be clearly documented in the submitted analysis. Such losses are expressed in terms of the velocity head multiplied by a loss coefficient and may be input to a computer run and can be expressed as:

$$h_L = C_F \frac{V^2}{2g}$$

where	h_L	=	head loss (ft)
	C_F	=	coefficient of resistance
	V	=	average channel velocity (feet per second)
	g	=	acceleration of gravity (32.2 ft/sec^2)

3.5.4.3 Bridges

There are various methods available to compute losses associated with flow through a bridge. Sources of energy loss in bridges include flow resistance, channel transitions, and direct obstructions to the flow such as piers. Each bridge should be examined individually to determine the best approach. The bridge routines found in HEC-RAS are recommended for their versatility and flexibility. Additional information on HEC-RAS analysis of bridges and culverts can be found in the HEC-RAS manuals which are available on-line.

The use of alternative means for computing bridge-related losses is encouraged when the engineer is properly aware of how and why such a strategy is appropriate and its results are reasonable.

3.6 SUPERCRITICAL TRANSITIONS

The design engineer should be aware that if flow through a transition is supercritical, standing waves will be generated and additional freeboard may be necessary to safely contain the flow. For a discussion of the analysis of supercritical flow in transitions, the engineer is referred to the U.S. Army Corps of Engineer's publication Hydraulic Design of Flood Control Channels, EM 1110-2-1601, July, 1970 (or latest version).

3.7 RIGHT-OF-WAY

All new drainage facilities must take into consideration the existing drainage upstream. In addition, new development must provide the ultimate planned right-of-way width based on fully developed watershed conditions. Fully developed conditions mean undetained flows from 100 percent development of the watershed with future impervious conditions being typical to existing development patterns within the area.

The amount of right-of-way required for open channels shall be based on full development of the watershed and is dependent on channel top width and channel type (earthen or lined) as required to accommodate the discharge resulting from the 100-year, 24-hour rainfall event. Adequate area must be set aside for both the channel itself and the adjacent berm required for channel maintenance. Minimum right-of-way requirements for Fort Bend County include the channel from bank to bank plus the maintenance berm areas on both sides and shall be dedicated at the time of platting of the adjacent property. However, if additional right-of-way is required to serve upstream development prior to downstream platting, sufficient right-of-way must be dedicated to accommodate the improved channel and provide adequate maintenance berms. See Table 3-4.

3.8 UTILITY LINE CROSSINGS

Prior to design, the Fort Bend County Drainage District Engineer should be contacted for information pertaining to the ultimate channel cross-section and right-of-way. In addition, County approval must be obtained for all future utility lines crossing Fort Bend County flood control facilities. All manholes required for the utility conduit shall be located outside of the ultimate Fort Bend County right-of-way.

TABLE 3-1

VALUES OF THE MANNING ROUGHNESS COEFFICIENT – "n"

Type of Channel and Description	Minimum	Normal	Maximum
A. Lined or Built-up Channels			
A1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
A2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	--
8. On irregular excavated rock	0.022	0.027	--
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017

TABLE 3-1 (Cont'd)

Type of Channel and Description	Minimum	Normal	Maximum
i. Asphalt			
1. Smooth	0.013	0.013	--
2. Rough	0.016	0.016	--
j. Vegetal lining	0.030	--	0.050
B. Excavated or Dredged			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stone bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline – excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush or banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
C. Natural Streams			
C1. Minor streams (top width at flood stage <100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			

TABLE 3-1(Concluded)

Type of Channel and Description	Minimum	Normal	Maximum
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
C2. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
C3. Major streams (top width at flood stage >100 ft). The "n" value is less than that for minor streams of similar description because banks offer less effective resistance			
a. Regular section with no boulders or brush	0.025	--	0.060
b. Irregular and rough section	0.035	--	0.100

Source: Open-Channel Hydraulics by Ven Te Chow (1959).

TABLE 3-2

COMPUTATION OF COMPOSITE ROUGHNESS COEFFICIENT

FOR EXCAVATED AND NATURAL CHANNELS

$N = (n_0 + n_1 + n_2 + n_3 + n_4)\ m$

	Channel Conditions	Value
Material Involved	Earth	0.020
n_0	Rockcut	0.025
	Fine Gravel	0.024
	Coarse Gravel	0.028
Degree of	Smooth	0.000
Irregularity	Minor	0.005
n_1	Moderate	0.010
	Severe	0.020
Variation of Channel	Gradual	0.000
Cross-Section	Alternating Occasionally	0.005
n_2	Alternating Frequently	0.010-0.015
Relative Effect of	Negligible	0.000
Obstructions	Minor	0.010-0.015
n_3	Appreciable	0.020-0.030
	Severe	0.040-0.060
Vegetation	Low	0.005-0.010
n_4	Medium	0.010-0.025
	High	0.025-0.050
	Very High	0.050-0.100
Degree of	Minor	1.000
Meandering	Appreciable	1.150
m	Severe	1.300

Source: Open-Channel Hydraulics by Ven Te Chow (1959).

TABLE 3-3

ALLOWABLE 25-YEAR VELOCITIES FOR CHANNEL DESIGN

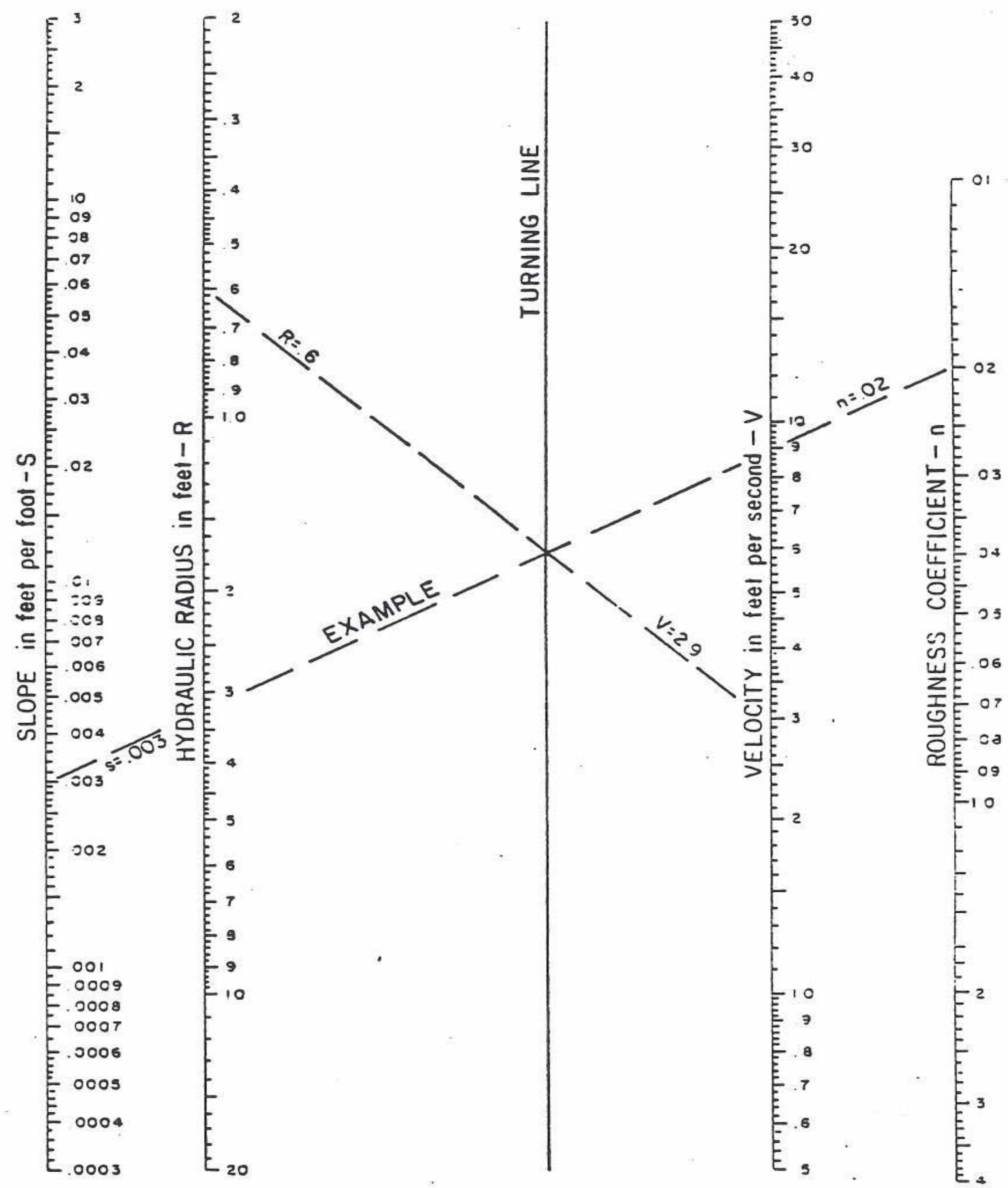
Channel Description	Average Velocity (Feet Per Second)	Maximum Velocity (Feet Per Second)
Grass Lined:		
Predominantly Clay Soil	3.0	5.0
Predominantly Sand Soil	2.0	4.0
Concrete Lined	6.0	10.0

Derived from the Criteria Manual for the Design of Flood Control and Drainage Facilities in Harris County, Texas, February 1984.

TABLE 3-4

RIGHT-OF-WAY REQUIREMENTS FOR FORT BEND COUNTY, TEXAS

Channel Type	Top Width	Maintenance Berm Width Necessary on Both Sides of Channel
All	TW ≤ 30 feet	15 feet
	30 feet < TW < 60 feet	20 feet
	TW ≥ 60 feet	30 feet

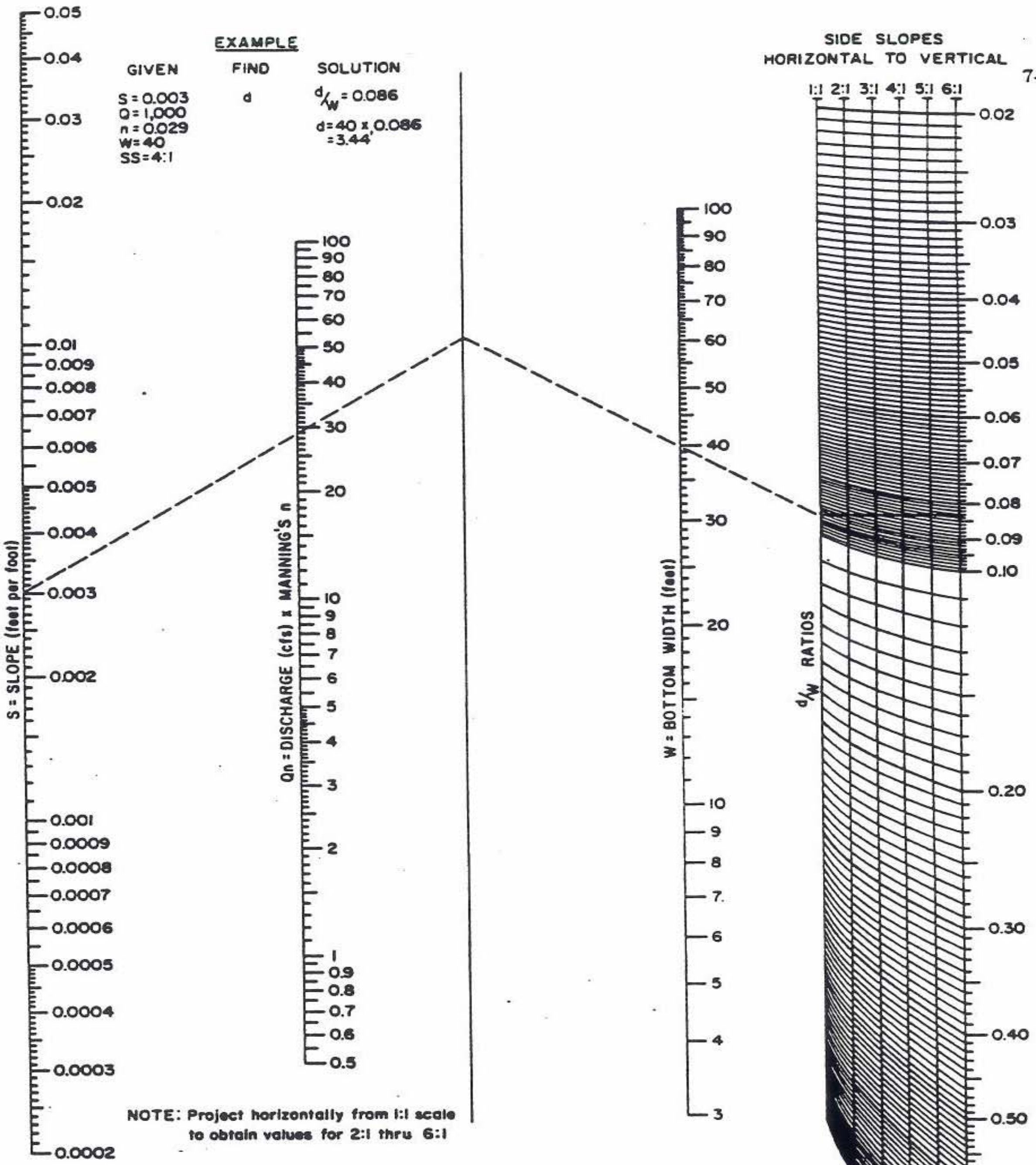


NOMOGRAPH FOR SOLUTION OF MANNING'S EQUATION

Source: Illinois Department of Transportation

August 1986

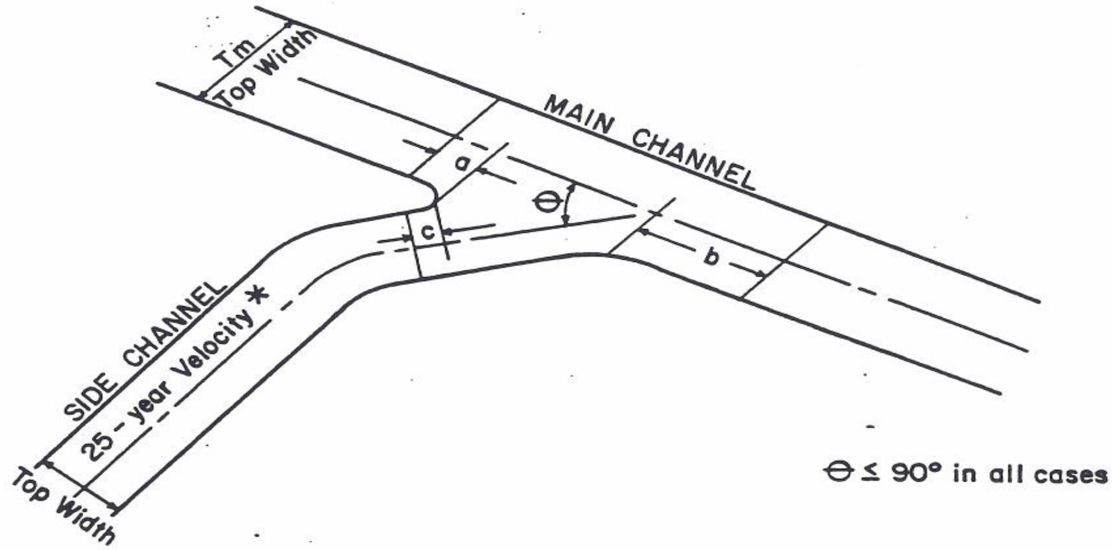
FIGURE 3-1



UNIFORM FLOW FOR TRAPEZOIDAL CHANNELS

TEXAS HIGHWAY DEPARTMENT

August 1986	FIGURE 3- 2



MINIMUM EXTENT OF EROSION PROTECTION

Location	Distance (ft.)
a	20
b	longer of 50' or 0.75 x Tm ÷ Tan θ
c	20'

25-year Velocity * in Side Channel (feet per second)	ANGLE OF INTERSECTION θ	
	15°—45°	45°—90°
4 or more	Protection	Protection
2 – 4	No Protection	Protection
2 or less	No Protection	No Protection

* Note: 25-year velocity in side channel assuming no backwater from main channel

Note: Erosion protection must be provided to the level of the 25-year water surface elevation

SOURCE : Criteria Manual For Design of Flood Control and Drainage Facilities in Harris County, TX Feb., 1984.

REQUIRED EROSION PROTECTION AT CHANNEL CONFLUENCE

August 1986 | 3-3

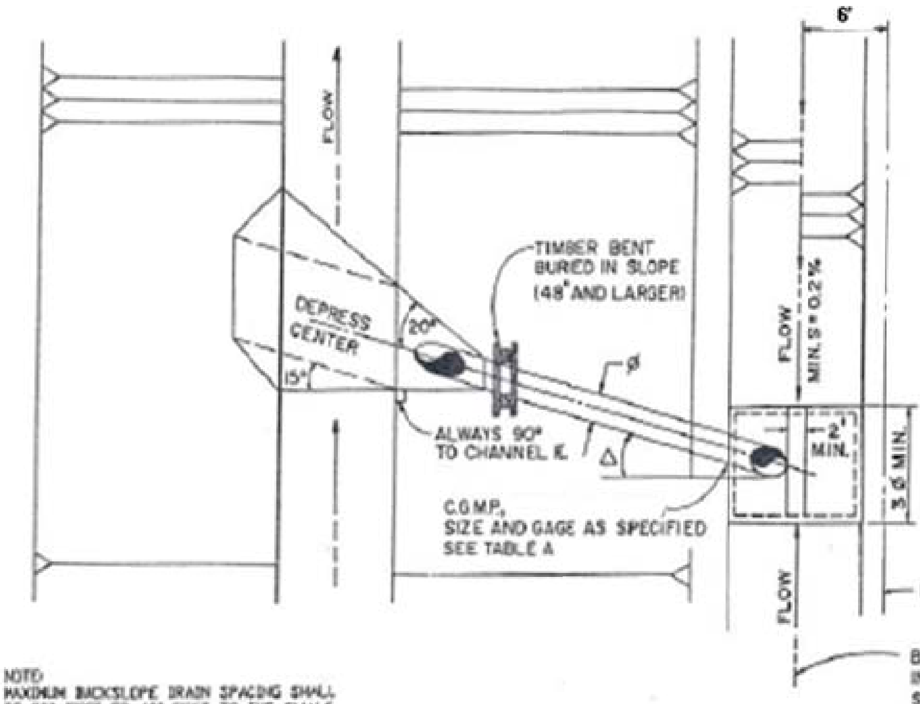


TABLE A

SIZE 2 2/3" X 1/2" CORRUGATION	PIPE GAUGE	BAND COUPLER GAUGE	SIZE 3"X1" & 5"x1" CORRUGATION	PIPE GAUGE	BAND COUPLER GAUGE
24"	16	16			
30"	16	16			
36"	16	16			
42"	14	16			
48"	14	16	48"	16	18
54"	12	14	54"	16	18
60"	12	14	60"	16	18
66"	10	12	66"	16	18
72"	10	12	72"	16	18
78"	8	10	78"	14	16
84"	8	10	84"	14	16

TABLE B

PIPE DIA.	SLOPE	VELOCITY
24"	0.6%	3.25 f.p.s.
36"	0.3%	3.00 f.p.s.
42"	0.2%	2.75 f.p.s.
48"	0.2%	3.00 f.p.s.
54"	0.2%	3.25 f.p.s.

NOTE:
MAXIMUM BACKSLOPE DRAIN SPACING SHALL BE 800 FEET OR 400 FEET TO THE SWALE HIGH POINT.

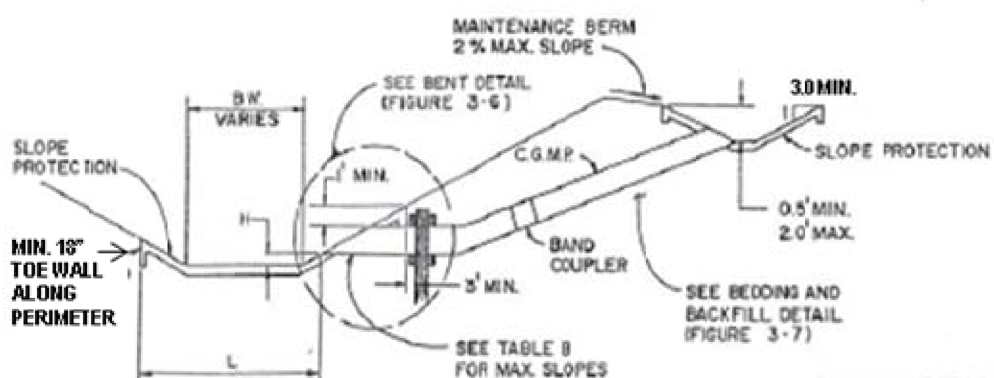
Δ PROP. 24" TO 42" Δ = 15°
PROP. 48" AND LARGER Δ = 30°

H : FOR PIPE SIZES 24" TO 42" H=3' MAX. AND 1' MIN.

FOR PIPE SIZES 48" AND LARGER H=1' MAX. AND MIN.

L : $\frac{B.W.}{PIPE\ DIA.} \leq$ 7' - 6" ⇒ L WILL EXTEND ONE PIPE DIA. ABOVE FLOWLINE ON OPPOSITE BANK (ODD)

$\frac{B.W.}{PIPE\ DIA.} >$ 7' - 6" ⇒ L = 6 DIA OR MIN. 1'-6" INTO B.W. WHICHEVER IS GREATER



NOTE: CONCRETE SLOPE PAVING SHALL HAVE A MINIMUM THICKNESS OF 4". MINIMUM REINFORCING STEEL SHALL BE #3 REBAR AT 18" O.C. OR 6 x 6 x W4.0 x W4.0 WELDED WIRE FABRIC

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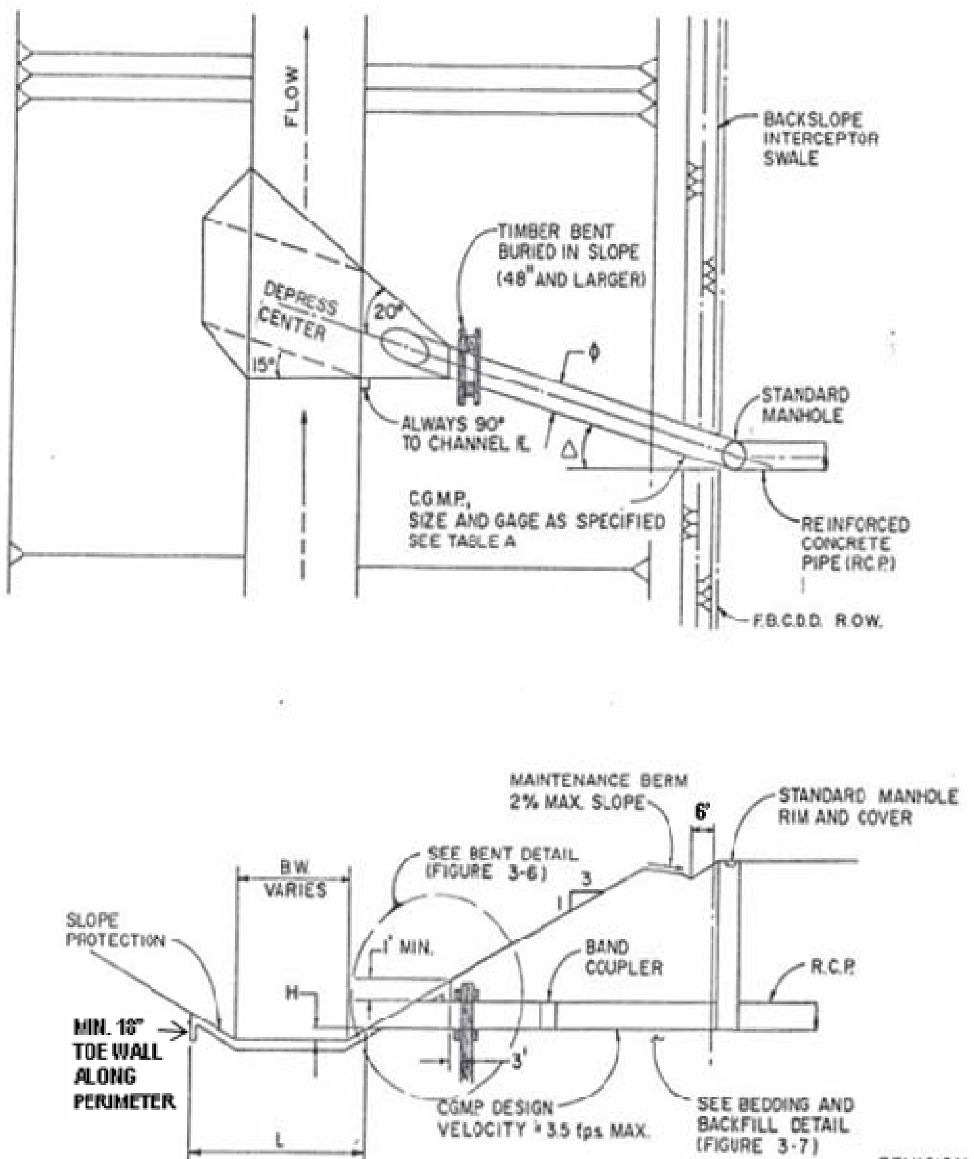
2-5-20-97
3-12-01-10

TYPICAL BACKSLOPE DRAIN DETAIL FOR FORT BEND COUNTY, TEXAS	
August 1986	FIGURE 3-4



NOTE: CONCRETE SLOPE PAVING SHALL HAVE A MINIMUM THICKNESS OF 4". MINIMUM REINFORCING STEEL SHALL BE #3 REBAR AT 18" O.C. OR 6 x 6 x W4.0 x W4.0 WELDED WIRE FABRIC

REVISION 1-7-13-88
2-5-20-97
3-12-01-10

TABLE A

SIZE 2 2/3" X 1/2" CORRUGATION	PIPE GAUGE	BAND COUPLER GAUGE	SIZE 3"X1" & 5"X1" CORRUGATION	PIPE GAUGE	BAND COUPLER GAUGE
24"	16	16			
30"	16	16			
36"	16	16			
42"	14	16			
48"	14	16	48"	16	18
54"	12	14	54"	16	18
60"	12	14	60"	16	18
66"	10	12	66"	16	18
72"	10	12	72"	16	18
78"	8	10	78"	14	16
84"	8	10	84"	14	16

H: FOR PIPE SIZES 24" TO 42"
H=3' MAX. AND 1' MIN.

FOR PIPE SIZES 48" AND LARGER
H=1' MAX. AND MIN.

L: $\frac{B.W.}{PIPE\ \emptyset} \leq 7'\text{-}6" \Rightarrow$ L WILL EXTEND ONE PIPE Ø ABOVE ℄ ON OPPOSITE BANK (MIN)

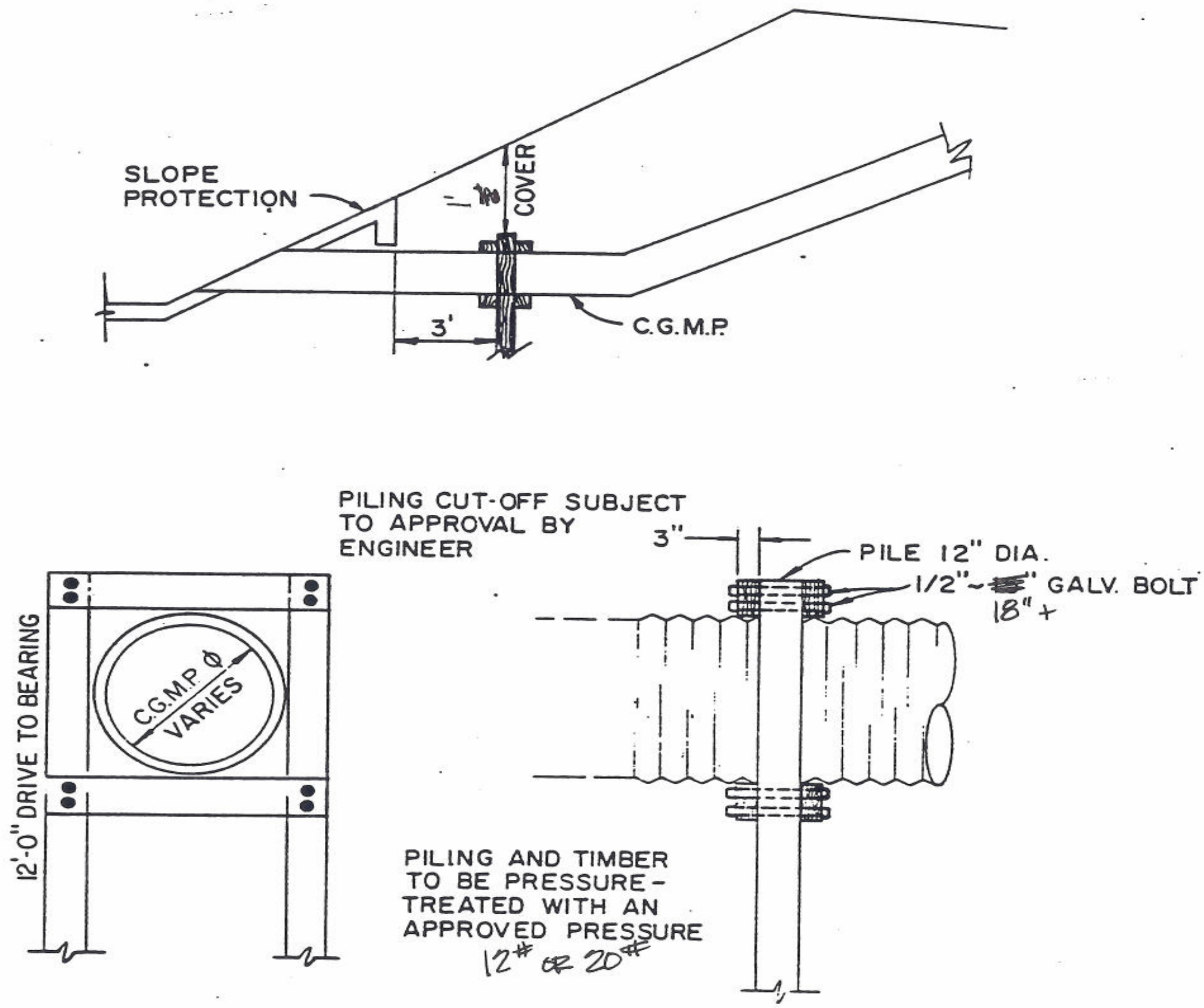
$\frac{B.W.}{PIPE\ \emptyset} > 7'\text{-}6" \Rightarrow$ L = 6 Ø OR MIN. 1'-6" INTO B.W. WHICHEVER IS GREATER

Δ: PROP. 24" TO 42" Δ = 15°
PROP. 48" AND LARGER Δ = 30°

TYPICAL STORM SEWER OUTFALL DETAIL FOR
FORT BEND COUNTY, TEXAS

August 1986

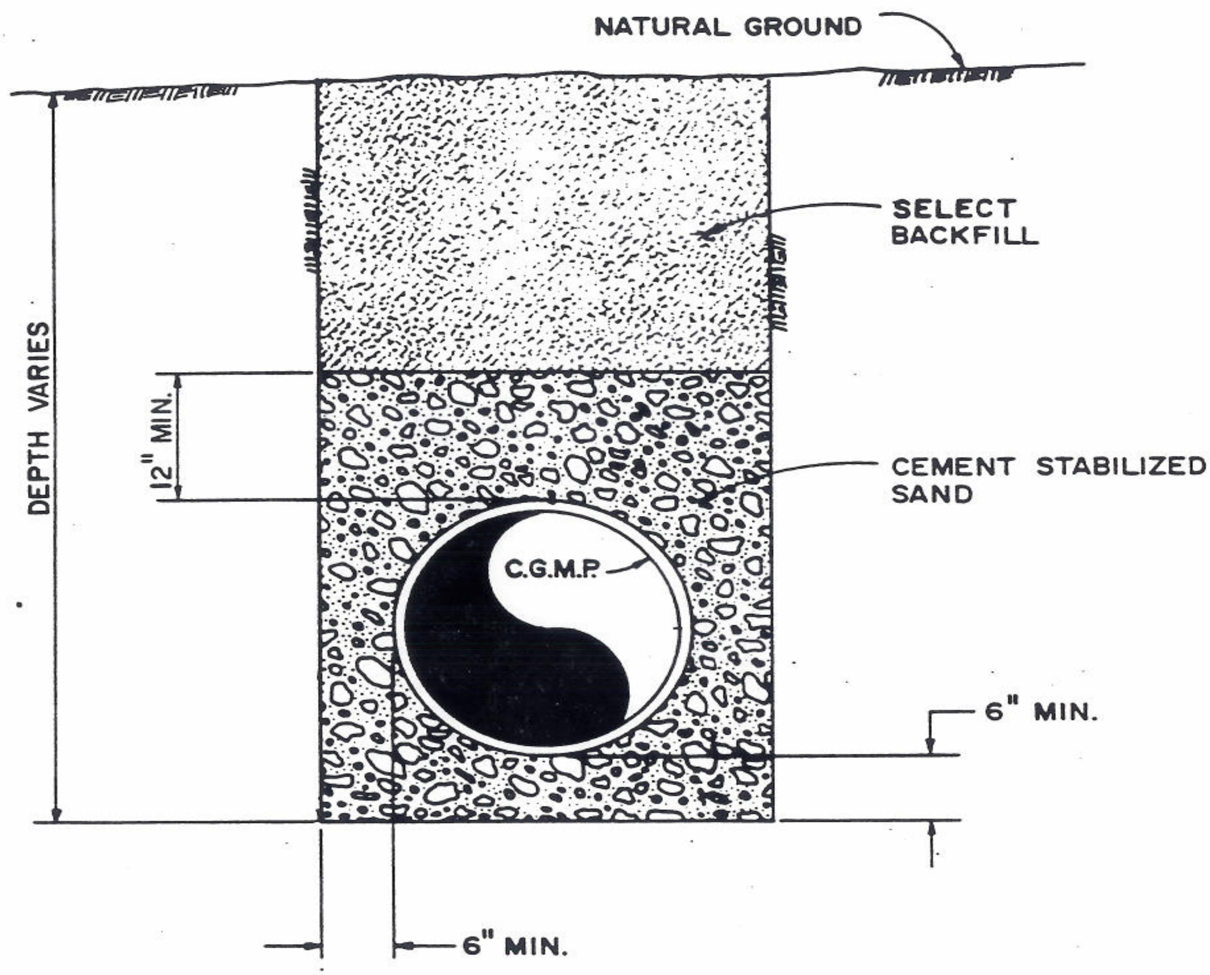
FIGURE 3-5



BENT FOR CORRUGATED METAL PIPE OUTFALL

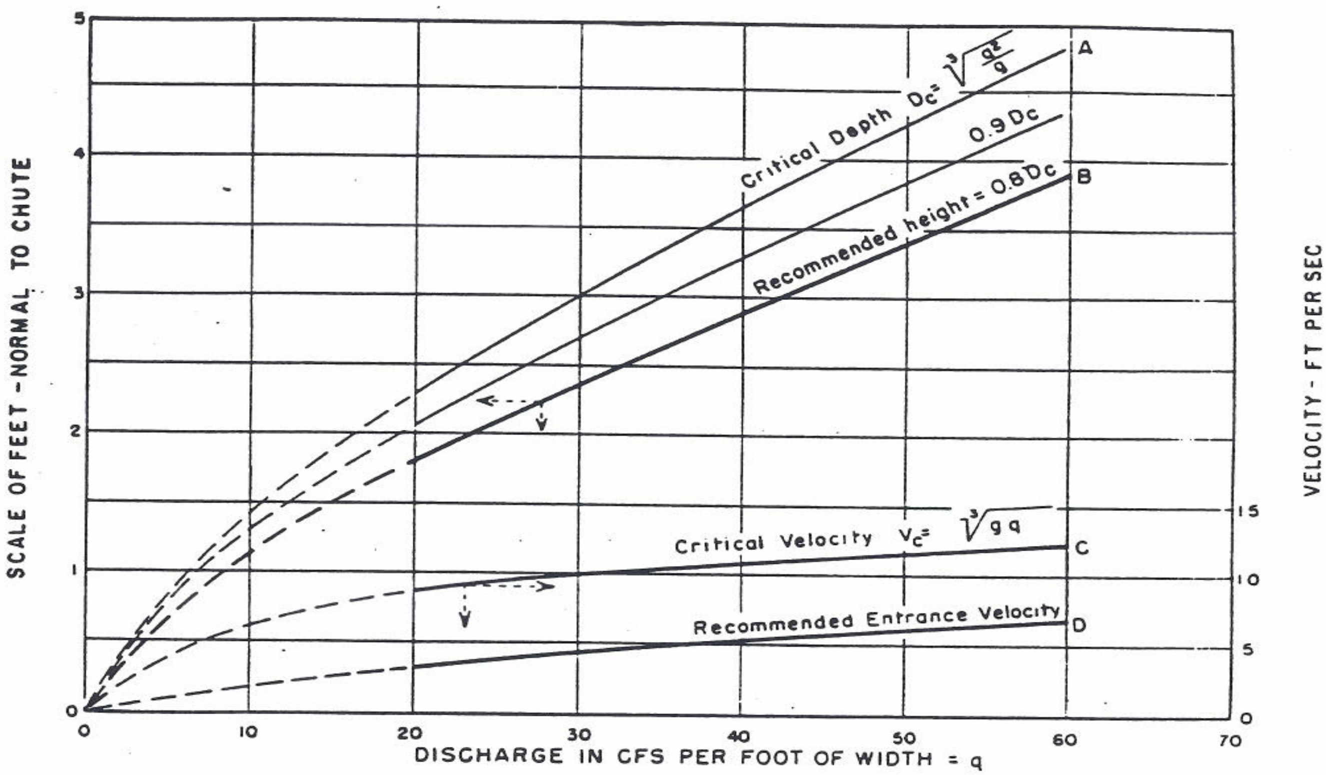
48-INCH AND LARGER

TYPICAL BENT DETAIL FOR C.G.M.P. OUTFALL FOR FORT BEND COUNTY, TEXAS	
August 1986	FIGURE 3-6



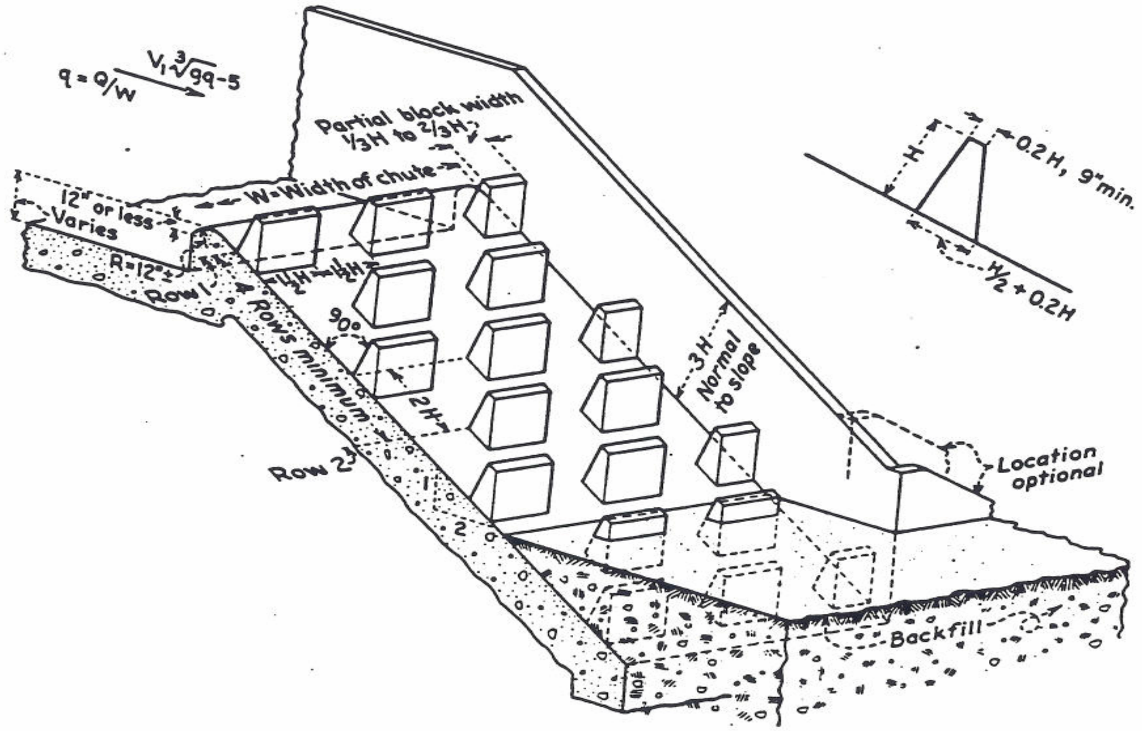
BEDDING AND BACKFILL DETAIL

TYPICAL BEDDING AND BACKFILL DETAIL FOR C.G.M.P. OUTFALL FOR FORT BEND COUNTY, TEXAS	
August 1986	FIGURE 3-7



SOURCE: Progress Report V- Research Study on Stilling Basins, Energy Dissipators and Associated Appurtenances, Hyd-445, Bureau of Reclamation, April, 1961.

RECOMMENDED BAFFLE PIER HEIGHTS AND ALLOWABLE VELOCITIES	
August 1986	FIGURE 3-8



SOURCE: Progress Report V- Research Study on Stilling Basins, Energy Dissipators and Associated Appurtenances, Hyd-445, Bureau of Reclamation, April, 1961.

BAFFLED CHUTE TYPICAL DETAIL	
August 1986	FIGURE 3-9

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4.0 CULVERTS AND BRIDGES

4.1 GENERAL

For small drainage areas the most economical means of moving open channel flow beneath a road or railroad is generally with culverts. Discussion in this section will address procedures for determining the most cost effective culvert size and shape given a design discharge and allowable headwater elevation. The design procedures for the culverts referenced in this section pertain only to those in the main channels and not those in roadside ditches which are covered in Section 5 - Storm Sewers and Overland Flow. In addition, this section will include a brief discussion of the hydraulic and hydrologic considerations pertinent to bridge design. This section considers all design to be completed for ultimate development. Where appropriate, the actual construction of a crossing may be phased as development occurs. In this case, both the ultimate and the interim phase must be shown on the construction plans. Calculations for each must be submitted for approval. The ultimate right-of-way is required even for an interim phase of construction.

4.2 CULVERTS

4.2.1 Design Frequency

All culverts in Fort Bend County shall be designed to handle the 100-year flood flow for fully developed conditions without causing upstream or downstream water surface profiles to exceed maximum levels as defined in Section 3.3.1.

4.2.2 Culvert Alignment

Culverts shall be aligned parallel to the longitudinal axis of the channel to insure maximum hydraulic efficiency and minimum erosion. In areas where a change in alignment is necessary, the turn shall be made upstream in the natural channel and appropriate erosion protection shall be provided.

4.2.3 Culvert Length

Culverts shall be designed to span the road or railroad right-of-way.

4.2.4 Headwalls

Headwalls and endwalls shall be utilized to control erosion and scour, to anchor the culvert against lateral pressures, and to insure bank stability. All headwalls shall be constructed of reinforced concrete and may be straight and parallel to the channel, flared or warped, with or without aprons, as required by site and hydraulic conditions. Protective guardrails should be included along culvert headwalls. Table 4-1 provides some general guidelines for choosing a headwall.

4.2.5 Minimum Culvert Sizes

The minimum pipe culvert diameter shall be 24 inches and the minimum box culvert dimensions shall be 2 feet by 2 feet. These restrictions are made to guard against flow obstruction. Sizes less than these shall be considered on a case-by-case basis.

4.2.6 Manning's "n" Values

The minimum Manning's "n" value to be used in concrete culverts shall be 0.013. For corrugated metal, the "n" value shall be as follows:

Corrugation (Span x Depth)	"n"
2-2/3" x 1/2"	0.024
3" x 1"	0.027
5" x 1"	0.027
6" x 2"	0.030

4.2.7 Erosion

Culverts, because of their hydraulic characteristics, generally increase the velocity of flow over that found in the natural channel. For this reason, the tendency for erosion, especially at the outlet, must be addressed. In general, culvert discharge velocities in unprotected channels should not exceed allowable channel velocities as defined in Table 3-3.

4.2.8 Structural Requirements

The following minimum structural requirements shall also be met for culvert design in Fort Bend County:

1. All precast reinforced concrete pipe should be ASTM C-76 (minimum).

2. All precast reinforced concrete box culverts with more than two feet of earth cover shall be ASTM C789-79.

3. All precast reinforced concrete box culverts with less than two feet of cover shall be ASTM 850-79.

4. All corrugated metal pipes shall be ASMT A-760.

5. ASSHTO HS20-44 loading should be used for all culverts.

6. Guardrails are suggested at all roadway culvert crossings. The approach ends of the guardrail shall be flared away from the roadway and properly anchored. Where guardrails encroach on access easements or maintenance berms, an additional easement shall be provided that ensures a minimum of 15 feet of clear access to the channel for maintenance equipment.

7. Joint sealing material for precast concrete culverts shall comply with "AASHTO Designation M-198 74 I, Type B, Flexible Plastic Gasket (Bitumen)", specifications.

8. Two sack per ton cement stabilized sand shall be used for backfill around culverts.

9. A 6-inch bedding of two sacks per ton cement stabilized sand required for all precast concrete box culverts.

4.3 CULVERT HYDRAULIC DESIGN

The fundamental objective of hydraulic design of culverts is to determine the most economical diameter at which the design discharge is passed without exceeding the allowable headwater elevation or causing erosion problems. However, there are numerous hydraulic considerations in culvert design which can render the decision-making process somewhat complex.

4.3.1 Culvert Design Procedure

The culvert design procedures presented here are based on information provided in the U.S. Department of Transportation (USDOT) publication Hydraulic Charts for the Selection of Highway Culverts, Hydraulic Engineering Circular No. 5, December 1965.

The nomographs presented herein cover the range of pipe and box culverts commonly used in drainage design.

The inlet control nomographs are scaled to represent the headwater-discharge relationships developed by the National Bureau of Standards in their report No. 4444: Hydraulic Characteristics of Commonly Used Pipe Sizes, by John L. French, and Hydraulics of Conventional Highway Culverts, by H.G. Bossy. Charts 1 through 7 present the inlet control nomographs including examples of their use.

The outlet control nomographs (Charts 8-14) were developed by USDOT from iterative solutions of Equation 4-3 for various flow conditions combined with a range of culvert lengths, shapes and sizes. It should be noted that for flow depths less than 0.75D the nomograph solutions are not reliable and the reader is referred to USDOT HEC No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts, for an alternative solution method other than hand calculation. However, a long-hand solution of Equation 4-1 provides the best analysis when HW is less than 0.75 D and/or the barrel length is less than 50 feet.

Alternatively, HEC-RAS can be used to design and analyze culverts.

4.3.2 Culvert Flow Types

The hydraulic capacity of a culvert is said to be either inlet-controlled or outlet-controlled. Inlet control means that the discharge in the culvert is limited by the hydraulic and physical characteristics of the inlet alone. These include headwater depth, barrel shape, barrel cross-sectional area, and the type of inlet edge. For inlet control, the barrel roughness, length, and slope are not factors in determining culvert capacity.

Under outlet control, the discharge capacity of the culvert is dependent on all of the hydraulic variables of the structure. These include headwater depth, tailwater depth as well as barrel shape, cross-sectional area, barrel roughness, slope, and length.

4.3.3 Headwater Depth

In all culvert design, headwater, or depth of ponding at the entrance to the culvert, is an important factor in culvert capacity. The headwater depth (HW) is the vertical distance from the culvert entrance invert to the energy line of the approaching flow. Due to low velocities in most entrance pools and the difficulty in determining velocity head in any flow, the energy line can often be assumed coincident with the water surface.

4.3.4 Tailwater Depth

For culverts under outlet control, tailwater depth is an important factor in computing both headwater depth and the hydraulic capacity of the culvert. If flow in the channel downstream of the culvert is subcritical, a computer-aided backwater analysis or calculation of normal depth is warranted to determine the tailwater elevation. If the downstream flow is supercritical, tailwater is inconsequential to the culvert's hydraulic capacity.

4.3.5 Inlet-Controlled Flow

Under inlet control, the culvert entrance may or may not be submerged. However, in all cases inlet-controlled flow through the culvert barrel is free surface flow. When the culvert inlet is submerged, the most reliable means for determining discharge is with standard empirical relationships. Nomographs (Charts 1 through 7), which plot headwater vs. discharge for various culvert sizes and shapes under inlet control, are based on laboratory research with models and full scale prototypes.

4.3.6 Outlet-Controlled Flow

Due to the flat terrain, a majority of the culverts in Fort Bend County are outlet-controlled.

Culverts, with outlet control, flow with the culvert barrel full or partially full for part or all of the barrel length. Both the headwater and tailwater may or may not submerge the culvert.

If the culvert is flowing, the energy required to pass a given quantity of water is stored in the head (H). From energy considerations it can be shown that H is the difference between the hydraulic grade line at the outlet and the energy grade line at the inlet (expressed in feet).

When a given discharge passes through a culvert, stored energy, represented by the total head (H) is dissipated in three ways. A portion is lost to turbulence at the entrance (H_e); a portion is lost to frictional resistance in the culvert barrel (H_f); and a portion is lost as the kinetic energy of flow through the culvert is dissipated in the tailwater (H_v). From this, the following relationship is evident:

$$H = H_e + H_f + H_v \quad (4\text{-}1)$$

The velocity head (H_v) is equal to $V^2/2g$ where V is the mean velocity of flow (in fps) in the culvert barrel.

The entrance loss (H_e) is expressed in terms of the velocity head multiplied by an entrance loss coefficient k_e.

An expression for the friction loss (H_f) is derived from Manning's equation:

$$H_f = \left(\frac{29n^2L}{R^{1.33}}\right)\frac{V^2}{2g} \tag{4-2}$$

Where n = Manning's roughness coefficient

L = culvert barrel length (ft)

R = the hydraulic radius (ft)

G = the gravitational constant (32.2 ft/sec^2)

V = mean velocity of flow in the culvert (ft/sec)

Rearranging Equation 4-1 it is seen that for full flow

$$H = \left(1 + k_e + \frac{29n^2L}{R^{1.33}}\right)\frac{V^2}{2g} \tag{4-3}$$

Equation 4-3 may be solved for H using the full flow nomographs (Charts 8-14) located at the conclusion of this section of the manual. Each nomograph is drawn for a particular barrel shape and material and a single value of Manning's "n" as noted on the respective charts. These nomographs may be used for other values of "n" by modifying the culvert length as directed in the instructions for use of the full-flow nomographs.

Figure 4-1 represents the various hydraulic elements of flow through a culvert and reveals graphically that the head (H) is equivalent to the vertical distance between the energy grade line at the inlet and the hydraulic grade line at the outlet.

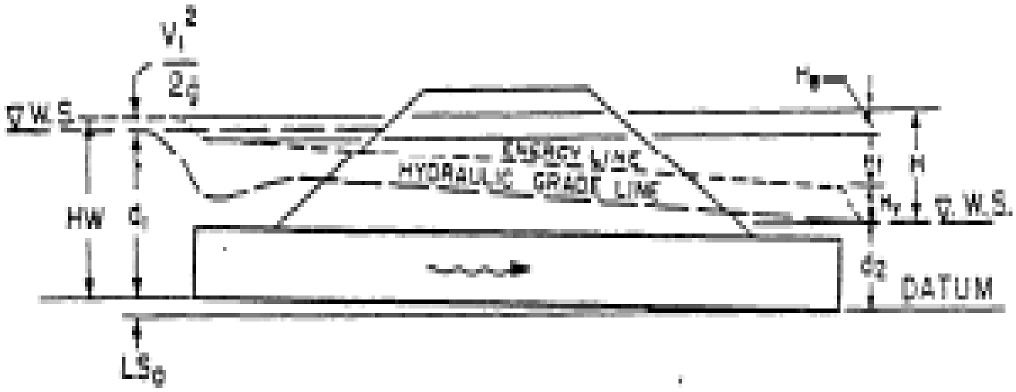


Figure 4-1 Hydraulic Elements of Flow through Culvert
(Source: Hydraulic Charts for the Selection of Highway Culverts, Federal Highway Administration, December 1965)

It also reveals the following relationship for full flow conditions:

$$H = H_v + H_e + H_f = d_1 + \frac{V_1^2}{2g} + LS_o - d_2 \qquad (4\text{-}4)$$

Where d_1, d_2 = flow depths as shown in Figure 4-1 (ft)

S_o = culvert barrel slope (ft/ft)

In culvert design it is generally required that the depth of the headwater (HW) be determined. The headwater depth is defined as the distance from the elevation of the culvert entrance invert to the elevation of the energy grade line in the headwater pool. From Figure 4-1, it is seen that $HW = D_1 + V_1^2/2g$. Since the velocity head in the entrance pool is usually small under ponded conditions, the headwater pool elevation can be assumed in most situations to be coincident with the energy grade line.

Rearranging Equation 4-4, the following expression for HW is derived:

$$HW = H + d_2 - LS \qquad (4\text{-}5)$$

When the culvert outlet is submerged by the tailwater, the above equation can be solved directly to determine HW. However, when the tailwater is below the crown of the culvert, it becomes necessary to redefine $d_{2,}$ which is taken as the greater of the following two values:

(1) TW

(2) $\frac{d_c + D}{2}$

where d_c = critical depth in the culvert as read from Charts 15 through 20 (ft)

TW = tailwater depth above the invert of the culvert outlet (ft)

D = height of the culvert (ft)

4.3.7 <u>Conditions at Entrance</u>

Culvert performance is significantly affected by inlet efficiency, especially for conditions of inlet-controlled flow. Changes in the culvert edge geometry can significantly change discharge capacity. Selection of a particular inlet type is contingent on the relative weightings the engineer assigns to considerations of the effect on peak flows, cost, and topography. In other words, the ideal inlet geometry is not necessarily the most efficient.

The entrance head losses may be determined by the following equation:

$$H_e = K_e \left(\frac{V_2^{2} - V_1^{1}}{2g} \right) \qquad (4\text{-}6)$$

Where h_e = entrance head loss (ft)

V_2 = velocity of flow in culvert (fps)

V_1 = velocity of flow approaching culvert (fps)

K_e = entrance loss coefficient.

For calculation of headwater with inlet-controlled culverts, the design nomographs presented in this manual account for various typical kinds of inlet geometry.

For calculation of headwater with outlet-controlled culverts, typical values of the entrance coefficient (K_e) for a wide range of inlet types are provided in Table 4-2.

4.3.8 Step-by-Step Design Procedures

It is possible by involved hydraulic computations to determine the probably type of flow under which a culvert will operate for a given set of conditions. However, such computations can be avoided by determining the headwater necessary for a given discharge under both inlet and outlet flow conditions. The larger of the two will define the type of control and the corresponding headwater depth. The following is the recommended procedure for selection of culvert size:

Step 1: List design data.

a. Design discharge (Q), in cfs, with return period.

b. Approximate length (L) of culvert, in feet.

c. Slope of culvert. If grade is given in percent, convert to slope in feet per feet.

d. Allowable headwater depth, in feet, which is the vertical distance from the culvert invert (flowline) at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert.

e. Flow velocities in the channel upstream and downstream of the proposed culvert location.

f. Type of culvert for first trial selection, including barrel material, barrel cross-sectional shape and entrance type.

Step 2: Determine the first trial culvert size.

Since the procedure given is one of trial and error, the initial trial size can be determined in several ways:

a. Past experience and engineering judgment.

b. By using an approximating equation such as $\frac{Q}{6} = A$ from which the trial culvert dimensions are determined. A is the culvert barrel cross-sectional area and 6 is an estimate of barrel velocity in feet per second.

c. Initially, utilize the inlet control nomographs (Charts 1-7) for the culvert type selected. An $\frac{HW}{D}$ must be assumed, say $\frac{HW}{D} = 1.5$, along with the given Q to determine a trial size.

If any trial size is too large in dimension because of limited height of embankment or availability of size, multiple culverts may be used by dividing the discharge appropriately among the number of barrels used. Raising the embankment height or the use of pipe arch and box culverts with width greater than height should also be considered. Final selection should be based on applicability and costs.

Step 3: Find headwater depth for trial size culvert.

a. Assuming Inlet Control –

(1) Using the trial size from Step 2, find the headwater depth (HW) by use of the appropriate inlet control nomograph (Charts 1-7). Tailwater (TW) conditions are to be neglected in this determination. HW in this case is found by multiplying $\frac{HW}{D}$ obtained from the nomographs by the height of culvert (D).

(2) If HW is greater or less than allowable, try another trial size until HW is acceptable for inlet control before computing HW for outlet control.

b. Assuming Outlet Control –

(1) Approximate the depth of tailwater (TW), in feet, above the invert at the outlet for the design flood condition in the outlet channel. (See general discussion on tailwater, Section 4.3.3.)

(2) For tailwater (TW) elevation equal to or greater than the top of the culvert at the outlet, set d_2 equal to TW and find HW by the following equation:

$$HW = H + d_2 - LS_o \qquad (4\text{-}5)$$

Where HW = vertical distance in feet from culvert invert (flowline) at entrance to the pool surface

H = head loss in feet as determined from the appropriate nomograph (Charts 8-14)

d_2 = vertical distance in feet from culvert invert at outlet to the hydraulic grade line

S_o = slope of barrel (feet/feet)

L = culvert length (feet)

(3) For tailwater (TW) elevations less than the top of the culvert at the outlet, find headwater HW by Equation 4-5 as in Step b(2) above except that

$d_2 = \frac{d_c + D}{2}$ or TW (whichever is greater)

Where d_c = critical depth in feet (Charts 15 through 20)

Note: d_c cannot exceed D

D = height of culvert opening (feet)

Note: Headwater depth determined in Step b(3) becomes increasingly less accurate as the headwater computed by this method falls below the value:

$$D + (1 = k_e)\frac{V^2}{2g}$$

c. Compare the headwaters found in Step 3a and Step 3b (Inlet Control and Outlet Control). The higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected.

d. If outlet control governs and the HW is higher than is acceptable, select a larger trial size and find HW as instructed under Step 3B. (Inlet control need not be checked, since the smaller size was satisfactory for this control as determined under Step 3a.)

Step 4: Try additional culvert types or shapes worthy of consideration and determine their size and HW by the above procedure.

Step 5: Compute outlet velocities for size and types to be considered in selection and determine need for channel protection.

a. If outlet control governs in Step 3c above, outlet velocity equals $\frac{Q}{A_o}$, where A_o is the cross-sectional area of flow in the culvert barrel at the outlet. If d_c or TW is less than the height of the culvert barrel, use A_o corresponding to d_c or TW depth, depending on whichever gives the greater area of flow. A_o should not exceed the total cross-sectional area A of the culvert barrel.

b. If inlet control governs in Step 3c, outlet velocity can be assumed to equal mean velocity in open-channel type flow in the barrel as computed by Manning's equation for the rate of flow, barrel size, roughness and slope of culvert selected.

Step 6: Record final selection of culvert with size, type, required and computed headwater, outlet velocity and economic justification.

4.4 BRIDGES

4.4.1 Bridge Design Considerations

Bridges must be designed to pass the 100-year design flow without causing adverse impacts or erosion problems in the channel or detention basin.

For new bridges, the low chord (at the center of the bridge) must be 1.5 feet or more above the existing or fully developed 100-year water surface elevation, whichever is higher. At no point shall the low chord of the new bridge be less than 1’ above the 100-year water surface elevation.

Newly constructed bridges must be designed to completely span the existing or proposed channel such that the channel will pass under the bridge without modifications. Energy losses due to flow transitions shall be minimized. In addition, provision must be made for future channel enlargements should they become necessary.

When a bridge is proposed to be replaced with a new structure, the low chord elevation and the cross-sectional area of the bridge opening should be equaled or exceeded. If this is not feasible, the bridge design must be coordinated with the Fort Bend County Drainage District Engineer.

When guardrails or bridge rails are proposed, and the rails and/or the structures will restrict access to drainage easements or maintenance berms, an additional easement shall be provided that ensures a minimum of 15 feet of clear access to the channel for maintenance equipment.

4.4.1.1 Bents and Abutments

Bents and abutments must be aligned parallel to the longitudinal axis of the channel so as to minimize obstruction of the flow. Bents shall be placed as far away from the channel centerline as possible and if possible should be eliminated entirely from the channel bottom.

4.4.1.2 Interim Channels

Bridges and bents constructed on existing or interim channels shall be designed to accommodate the ultimate channel section with a minimum of structural modifications.

4.4.1.3 Erosion Protection

Increased turbulence and velocities associated with flow in the vicinity of bridges requires the use of erosion protection in affected areas.

4.5 HEC-RAS

All hydraulic computations are to be computed in HEC-RAS version 3.1.3 (or newer) with differentiation between pressure flow and open channel flow for bridges and culverts. Versions of HEC-RAS must be consistent throughout each project.

Models other than HEC-RAS may be used for bridge and culvert computations. However, prior approval from the Drainage District is required to use hydraulic models other than HEC-RAS. Modeling that will require a FEMA submittal must use a FEMA approved model.

TABLE 4-1

HEADWALL GUIDELINES

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

Parallel Headwall and Endwall
1. Approach velocities are less than 6 fps.
2. Backwater pools may be permitted.
3. Approach channel is undefined.
4. Ample right-of-way or easement is available.
5. Downstream channel protection is not required.

Flared Headwall and Endwall
1. Channel is well defined.
2. Approach velocities are greater than 6 fps.
3. Medium amounts of debris exist.

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

Warped Headwall and Endwall
1. Channel is well defined and concrete lined.
2. Approach velocities are greater than 8 fps.
3. Medium amounts of debris exist.

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

Source: Drainage Criteria Manual, City of Austin, Texas.

TABLE 4-2

INLET LOSS COEFFICIENTS USED FOR

CULVERTS FLOWING WITH OUTLET CONTROL

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End section conforming to fill slope	0.5
Beveled edges (33.7º or 45º bevels)	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls (square-edge)	0.5
Mitered to conform to fill slope (paved or unpaved slope)	0.2
*End section conforming to fill slope	0.5
Beveled edges (33.7º or 45º bevels)	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimensions or beveled edges on 3 sides	0.2
Wingwalls at 30º to 75º to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension or beveled top edge	0.2
Wingwalls at 10º to 25º to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or sloped-tapered inlet	0.2

Source: U.S. Department of Transportation (1965).

*Note: "End section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design, have a superior hydraulic performance.

INLET-CONTROL NOMOGRAPHS
Charts 1 through 7
Instructions for Use

1. To determine headwater (HW), given Q, and size and type of culvert.

   a. Connect with a straightedge the given culvert diameter or height (D) and the discharge Q, or $\frac{Q}{B}$ for box culverts; mark intersection of straightedge on $\frac{HW}{D}$ scale marked (1).

   b. If $\frac{HW}{D}$ scale marked (1) represents entrance type used, read $\frac{HW}{D}$ on scale (1). If another of the three entrance types listed on the nomograph is used, extend the point of intersection in (1) horizontally to scale (2) or (3) and read $\frac{HW}{D}$.

   c. Compute HW by multiplying $\frac{HW}{D}$ by D.

2. To determine discharge (Q) per barrel, given HW, and size and type of culvert.

   a. Compute $\frac{HW}{D}$ for given conditions.

   b. Locate $\frac{HW}{D}$ on scale for appropriate entrance type. If scale (2) or (3) is used, extend $\frac{HW}{D}$ point horizontally to scale (1).

   c. Connect point $\frac{HW}{D}$ scale (1) as found in (b) above and the size of culvert on the left scale. Read Q or $\frac{Q}{B}$ on the discharge scale.

   d. If $\frac{Q}{B}$ is read in (c) multiply by B (span of box culvert) to find Q.

3. To determine culvert size, given Q, allowable HW and type of culvert.

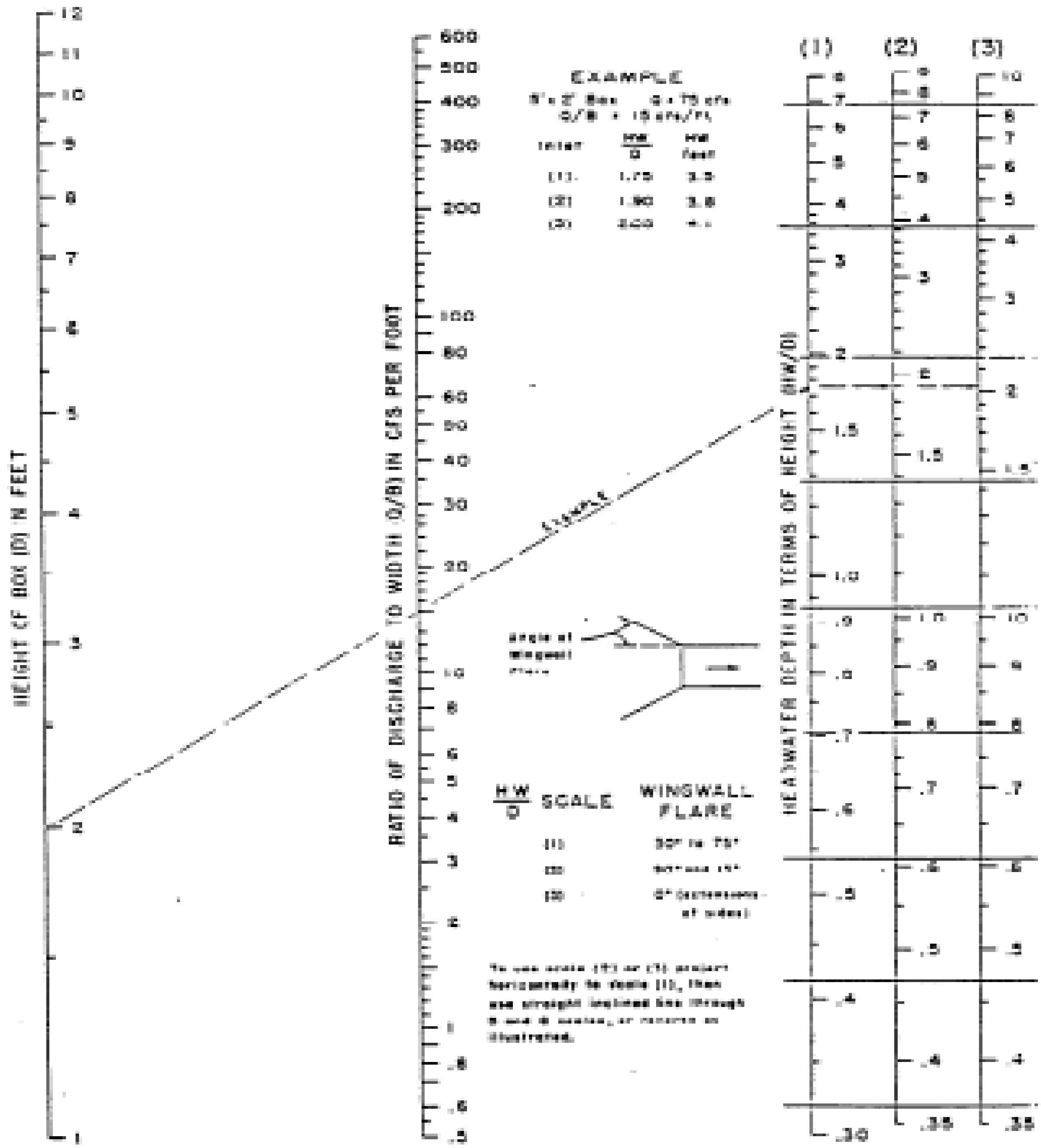
   a. Using a trial size, compute $\frac{HW}{D}$.

   b. Locate $\frac{HW}{D}$ on scale for appropriate entrance type. If scale (2) or (3) is used, extend $\frac{HW}{D}$ point horizontally to scale (1).

   c. Connect point on $\frac{HW}{D}$ on scale (1) as found in (b) above to given discharge and read diameter, height or size of culvert required for $\frac{HW}{D}$ value.

   d. If D is not that originally assumed, repeat procedure with a new D.

CHART 1



HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

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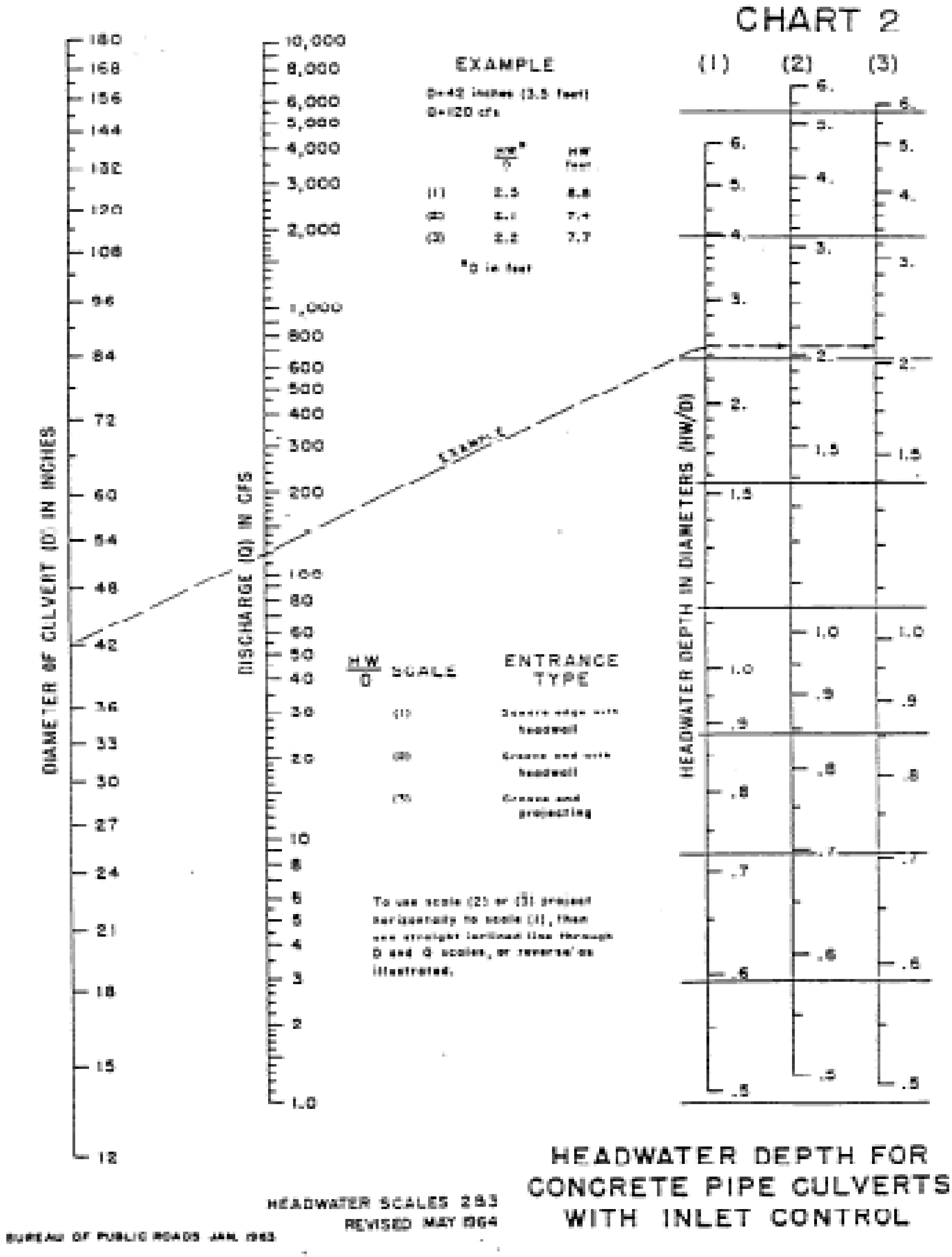
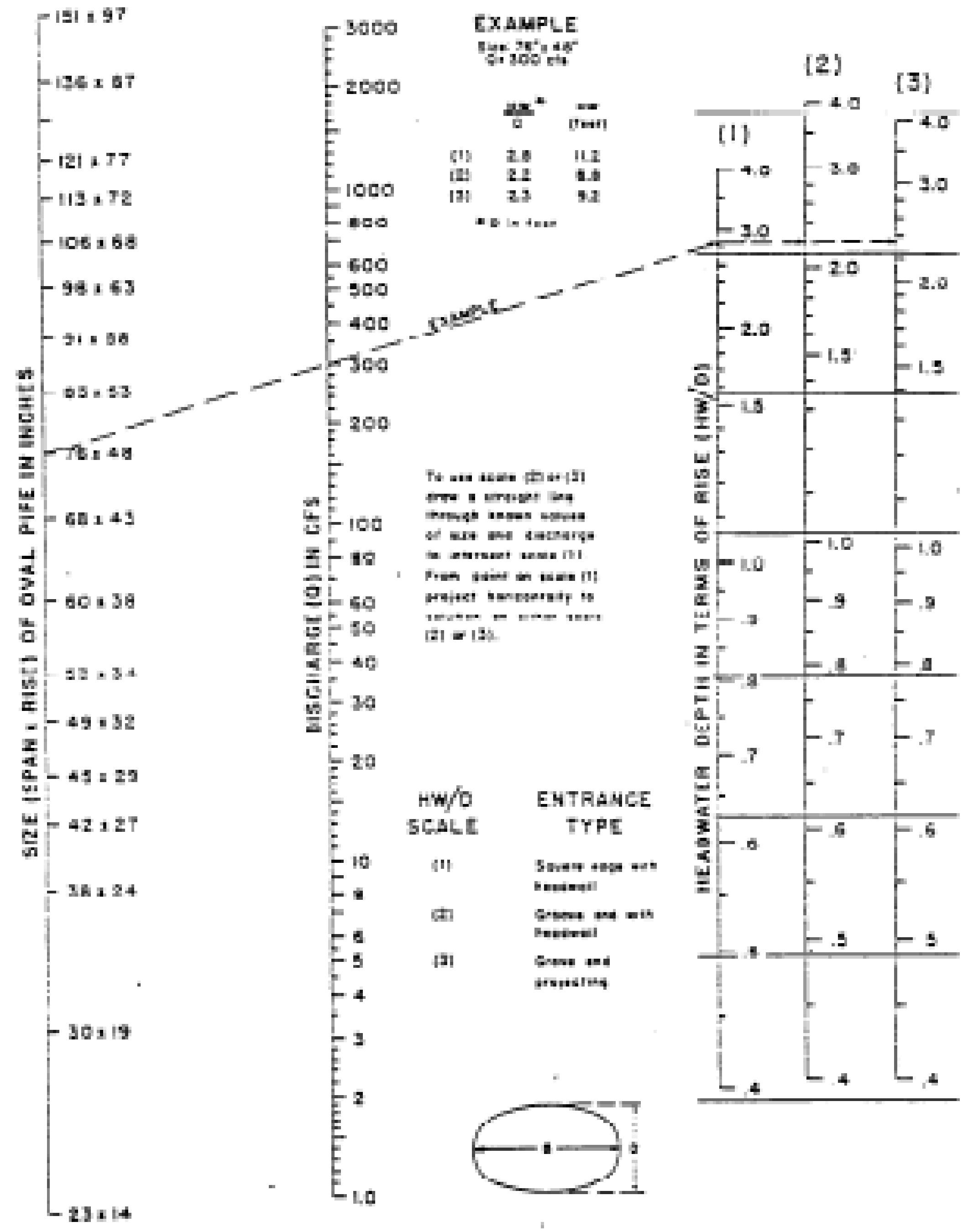


CHART 2
EXAMPLE
D=42 inches (3.5 feet)
Q=120 cfs
HW*/D HW feet
(1) 2.5 8.8
(2) 2.1 7.4
(3) 2.2 7.7
*D in feet
DIAMETER OF CULVERT (D) IN INCHES
DISCHARGE (Q) IN CFS
HEADWATER DEPTH IN DIAMETERS (HW/D)
EXAMPLE
HW/D SCALE
ENTRANCE TYPE
(1) Square edge with headwall
(2) Groove end with headwall
(3) Groove end projecting
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
HEADWATER SCALES 2&3
REVISED MAY 1964
BUREAU OF PUBLIC ROADS JAN. 1963
HEADWATER DEPTH FOR
CONCRETE PIPE CULVERTS
WITH INLET CONTROL



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

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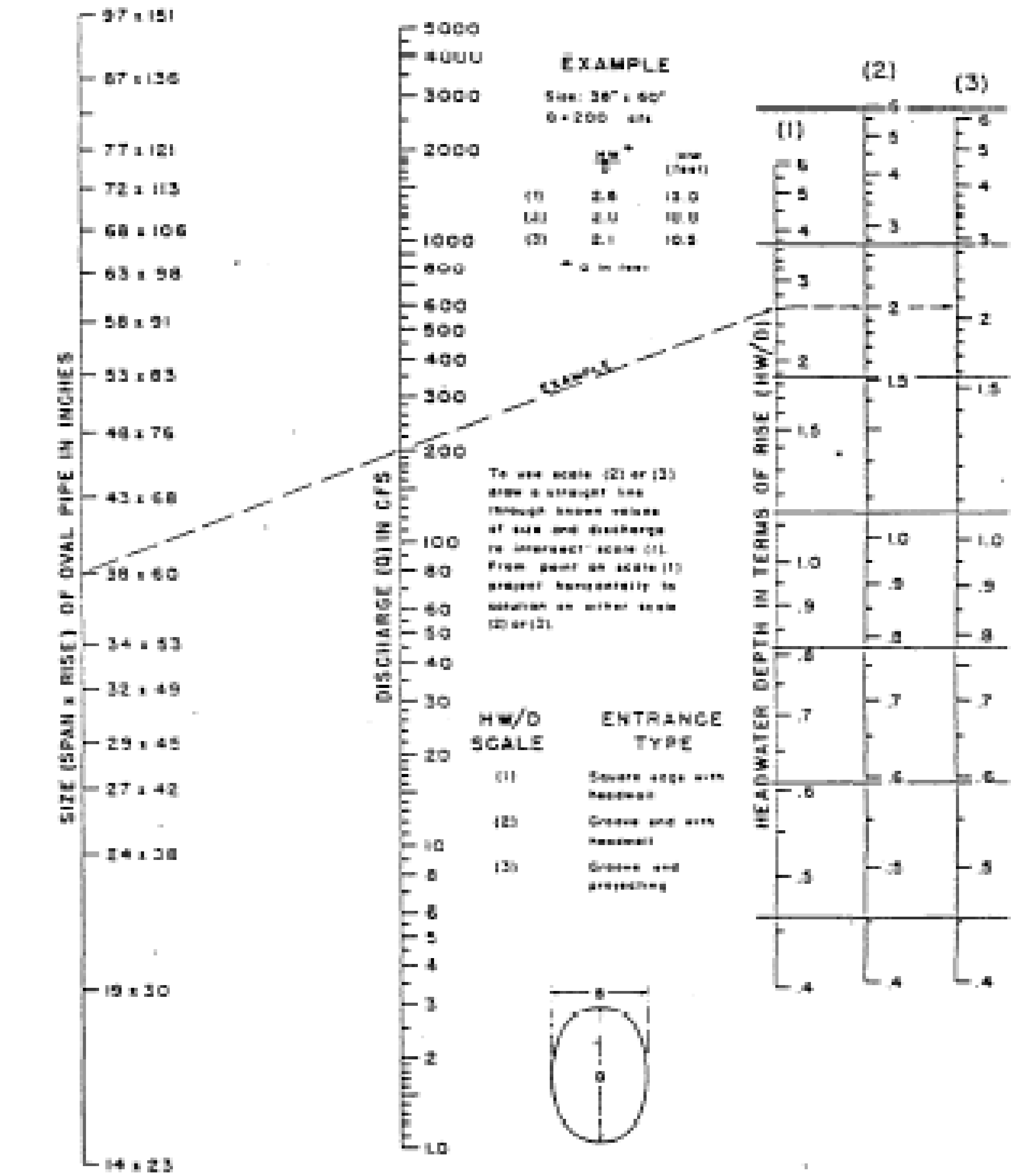
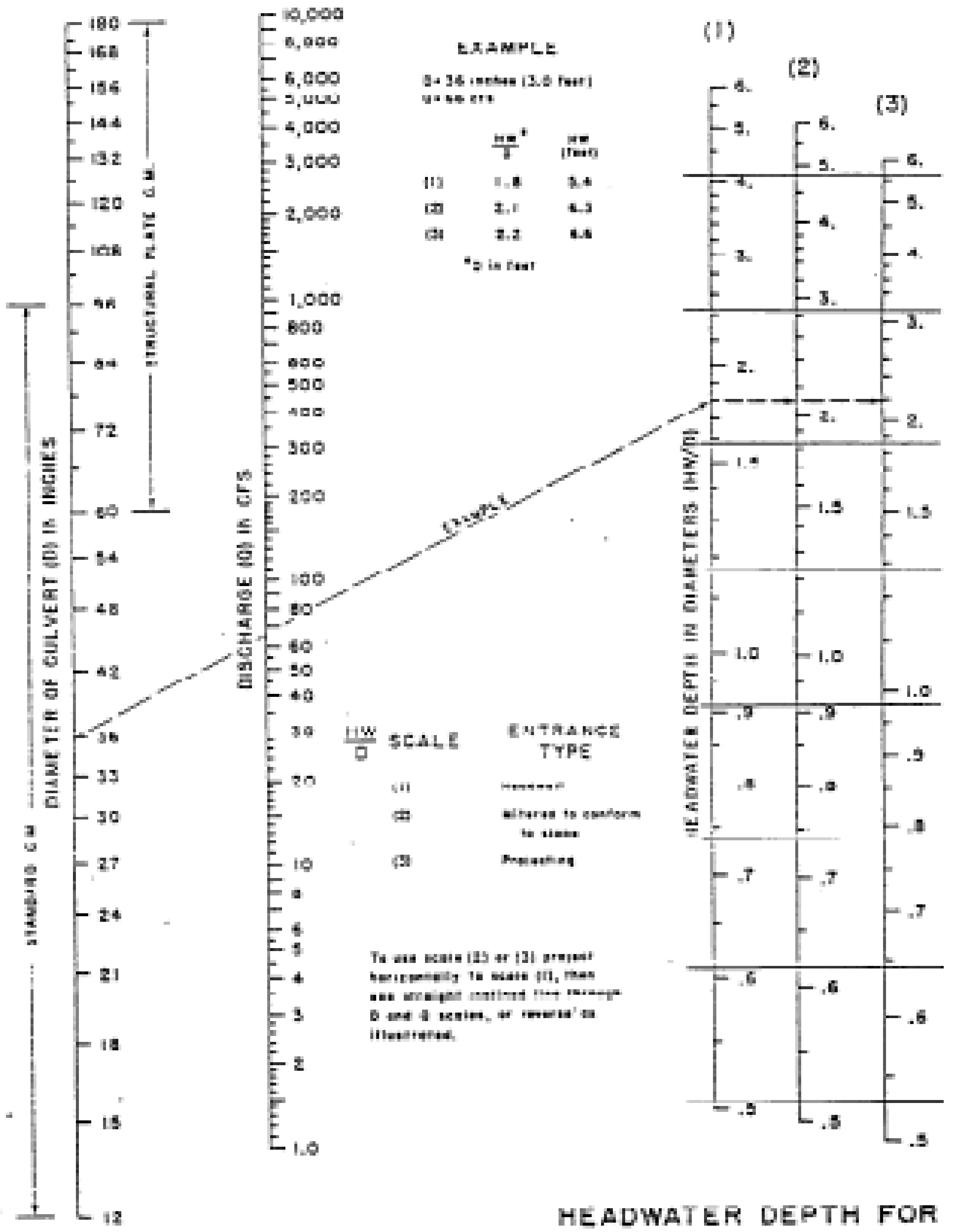
CHART 4



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

CHART 5



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

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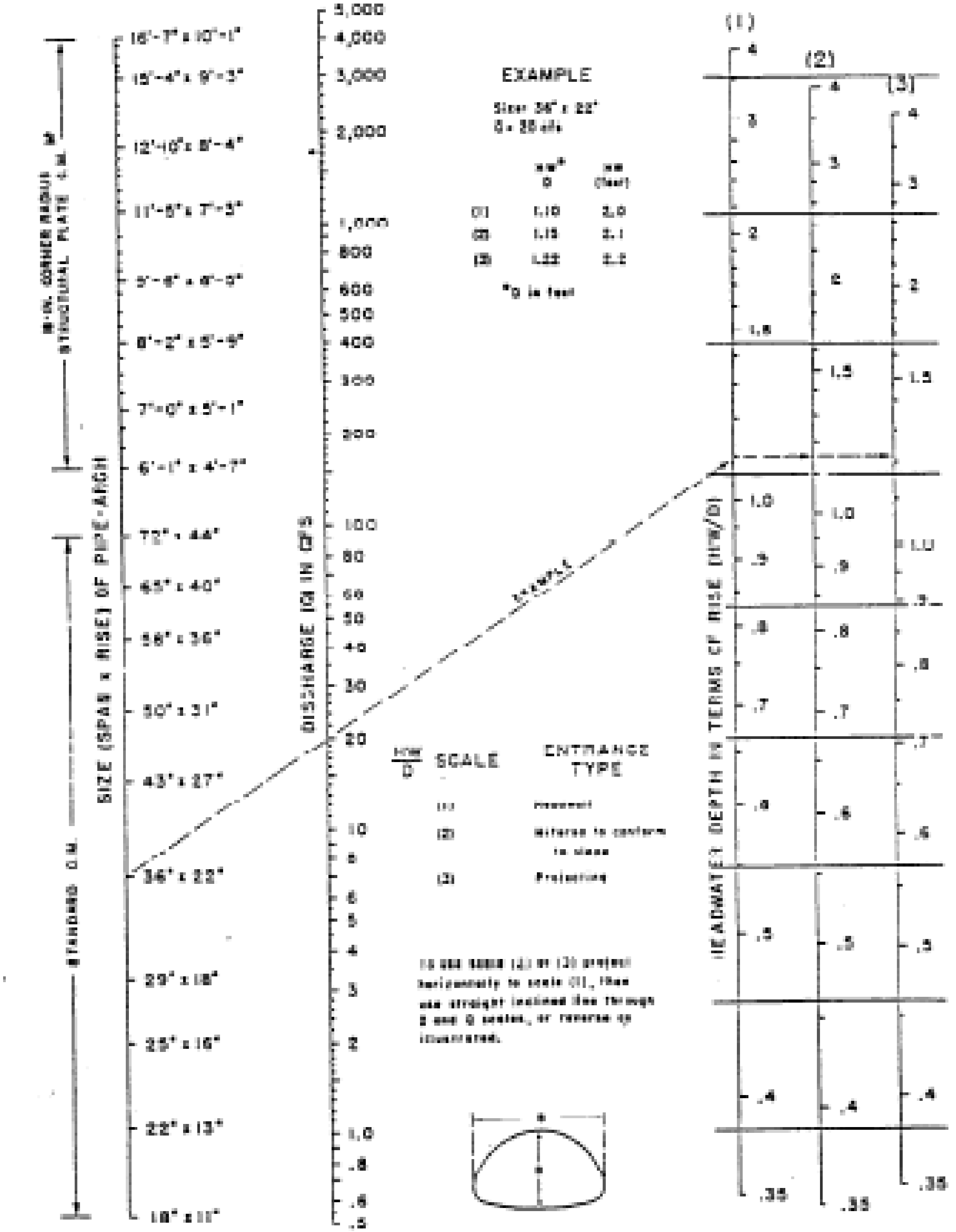
CHART 6



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

*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

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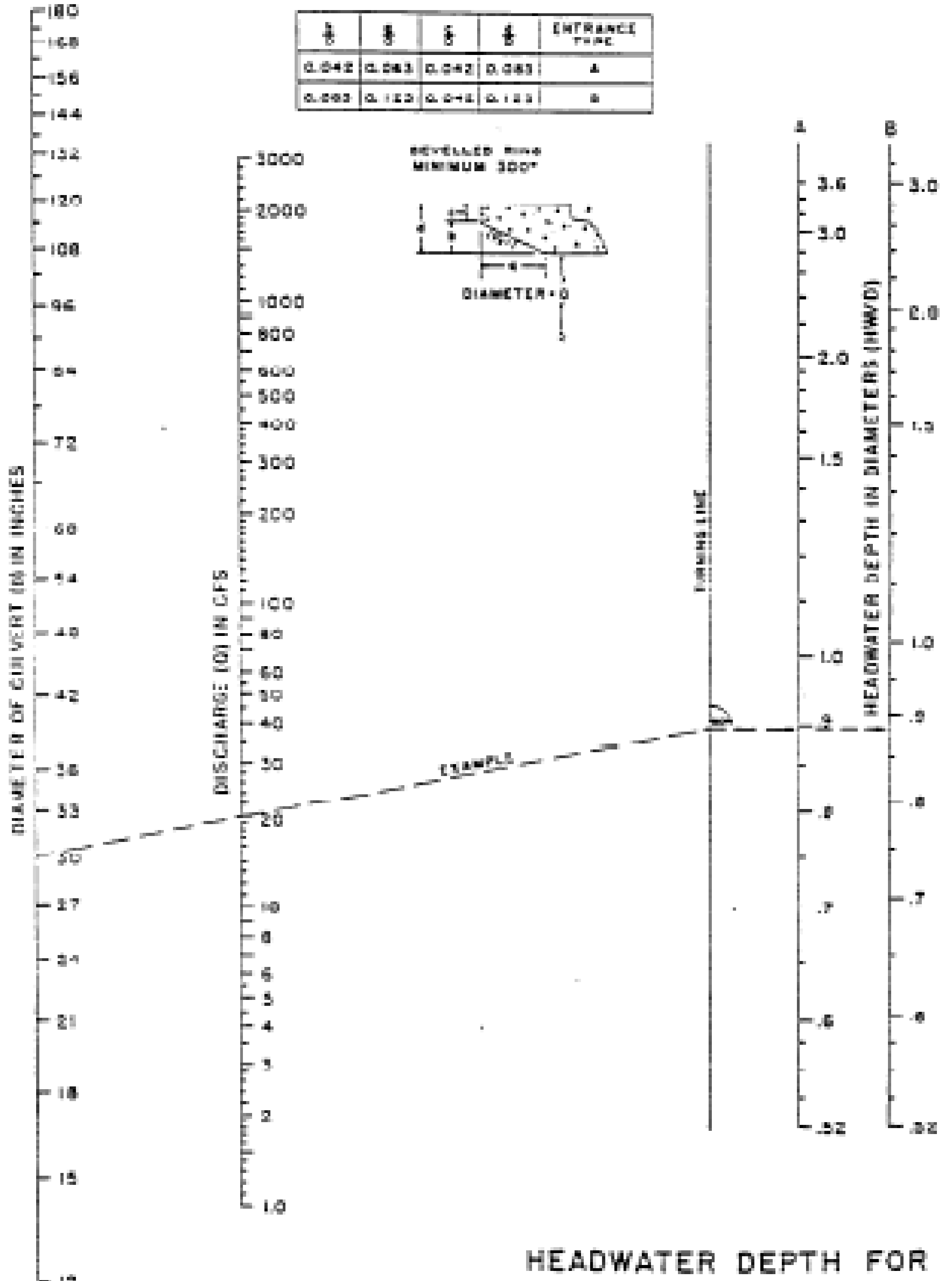
CHART 7



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HEADWATER DEPTH FOR
CIRCULAR PIPE CULVERTS
WITH BEVELLED RING
INLET CONTROL

OUTLET-CONTROL NOMOGRAPHS

Charts 8 through 14

Instructions for Use

Outlet control nomographs solve Equation 4-3, for head H when the head H for some part-full flow conditions with outlet control. These nomographs do not give a complete solution for finding headwater HW, since they only give H in Equation 4-5, $HW = H + d_2 - LS_0$.

1. To determine head H for a given culvert and discharge Q.
   - a. Locate appropriate nomograph for type of culvert selected. Find k_e for entrance type in Table 4-2.
   - b. Begin nomograph solution by locating starting point on length scale. To locate the proper starting point on the length scales, follow instructions below:
     - (1) If the n value of the nomograph corresponds to that of the culvert being used, select the length curve for the proper k_e and locate the starting point at the given culvert length. If a k_e curve is not shown for the selected k_e, see (2) below. If the n value for the culvert selected differs from that of the nomograph, see (3) below.
     - (2) For the n value of the nomograph and a k_e intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two chart scales in proportion to the k_e values.
     - (3) For a different roughness coefficient n_1 than that of the chart n, use the length scales shown with an adjusted length L_1, calculated by the formula:

$$L_1 = L \frac{n_1^{\,2}}{n}$$ See instruction 2 for n values.

c. Using a straightedge, connect point on length scale to size of culvert barrel and mark the point of crossing on the "turning line". See instruction 3 below for size considerations for rectangular box culvert.

d. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in feet on the head (H) scale. For values beyond the limit of the chart scales, find H by solving Equation 4-3.

2. For appropriate values of n, section 4.2.6.

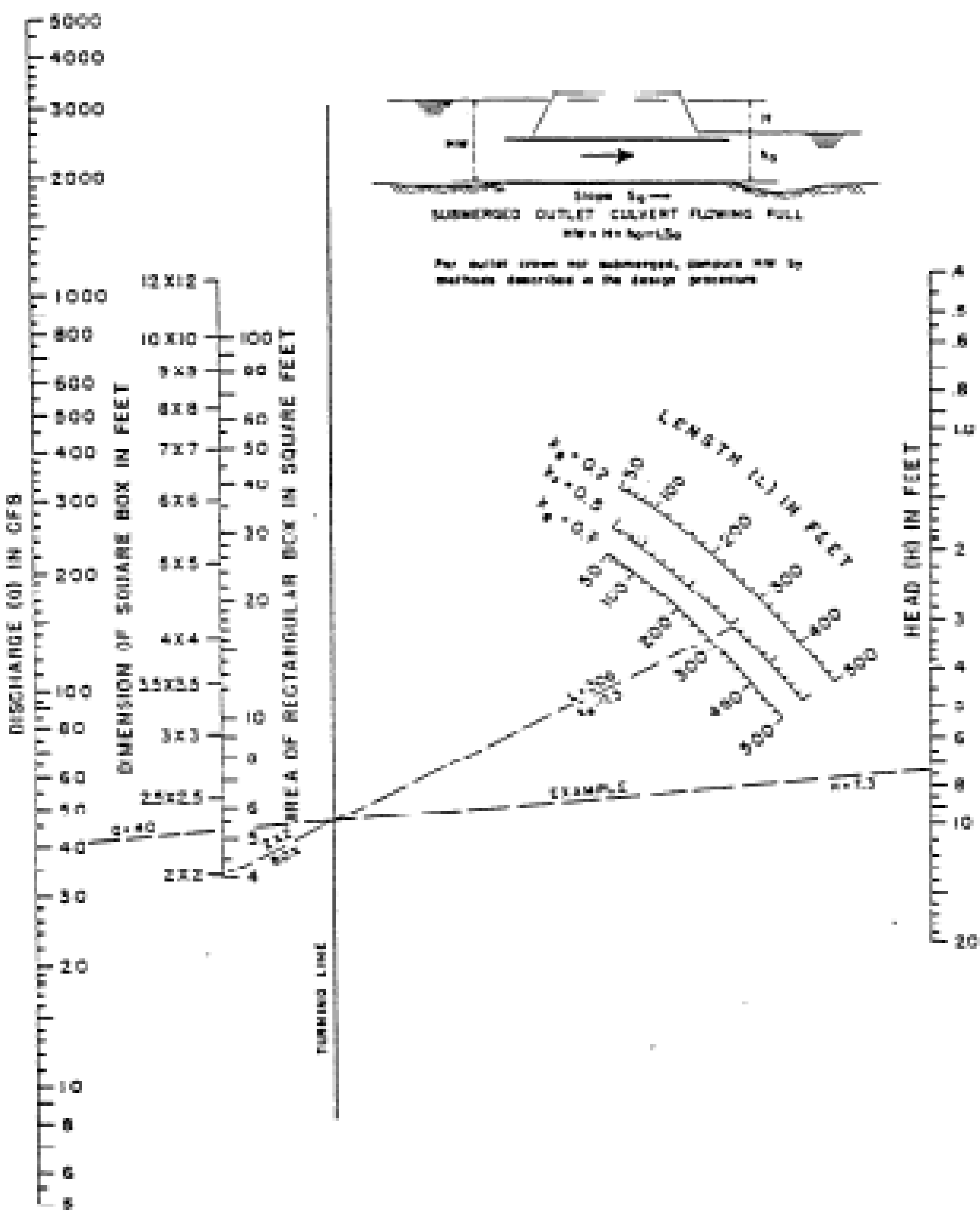
3. To use the box culvert nomograph, chart 8, for full-flow for other than square boxes.

a. Compute cross-sectional area of the rectangular box.

b. Connect proper point (see instructions 1) on length scale to barrel area[1] and mark point on turning line.

c. Pivot the straightedge on this point on the turning line of connect given discharge rate. Read head in feet on the head (H) scale.

[1] The area scale on the nomograph is calculated for barrel cross-sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 1/2D. For other box proportions use equation 4-3 for more accurate results.



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

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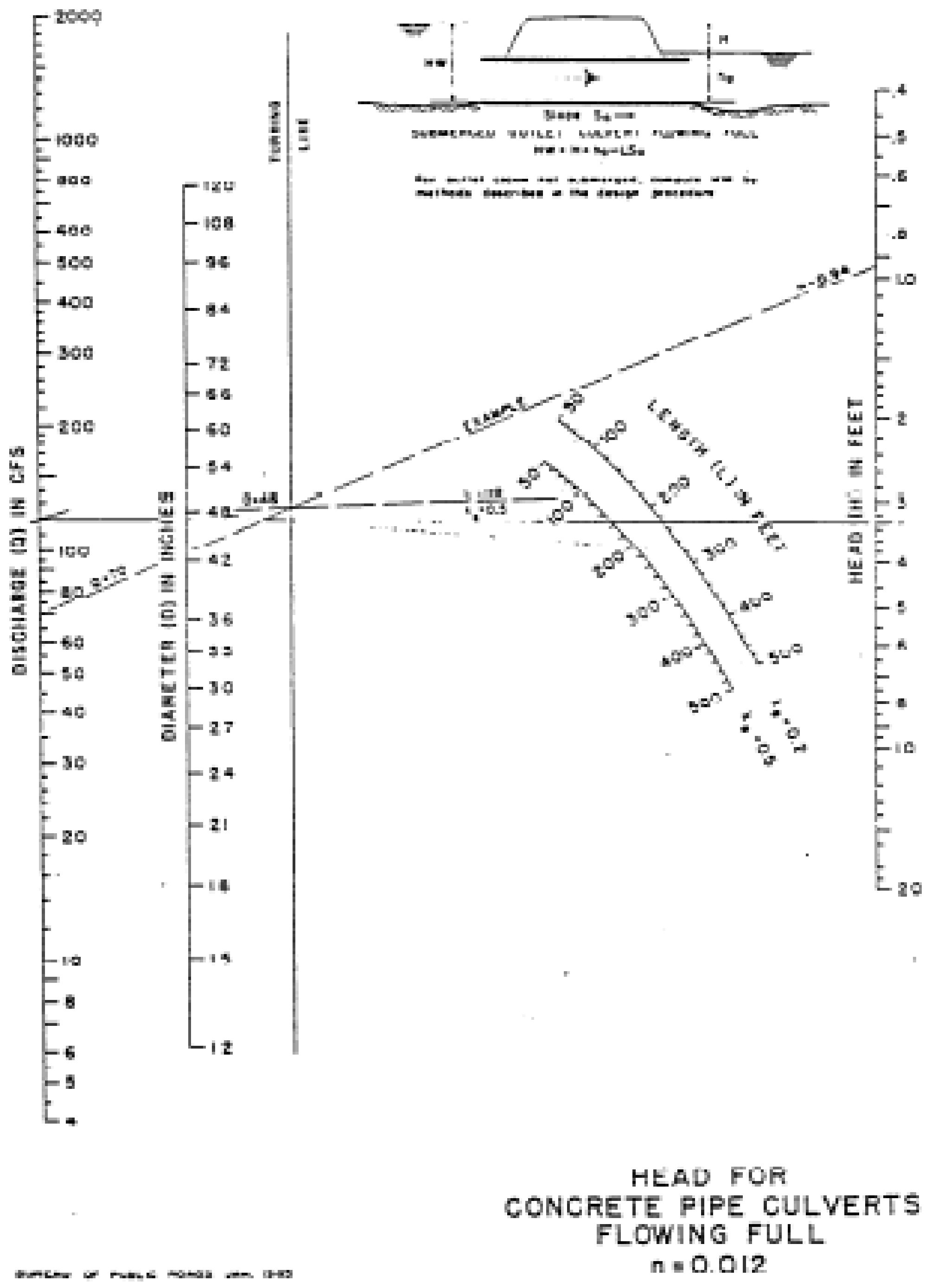
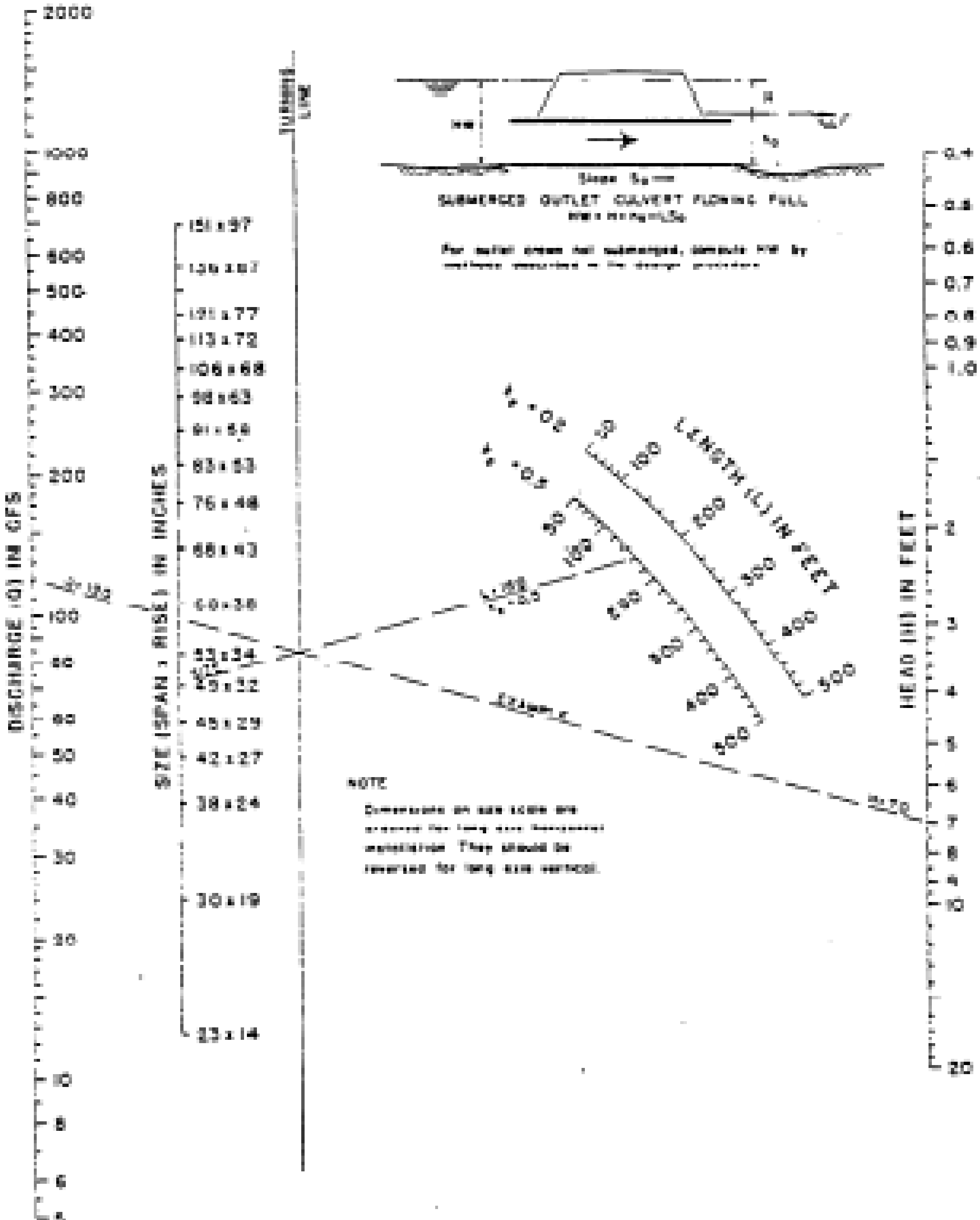


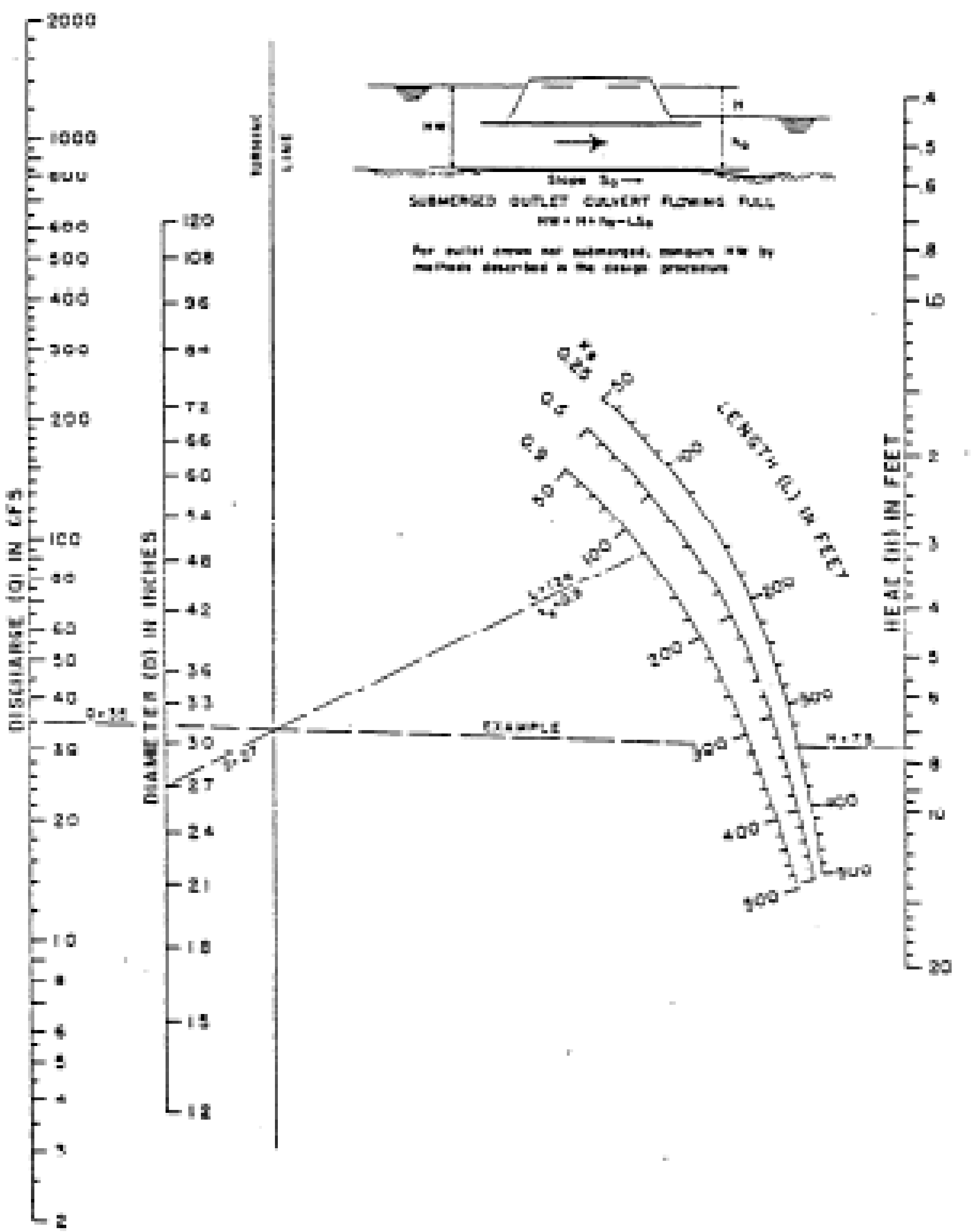
HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012



HEAD FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL OR VERTICAL
FLOWING FULL
$n = 0.012$

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 11



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

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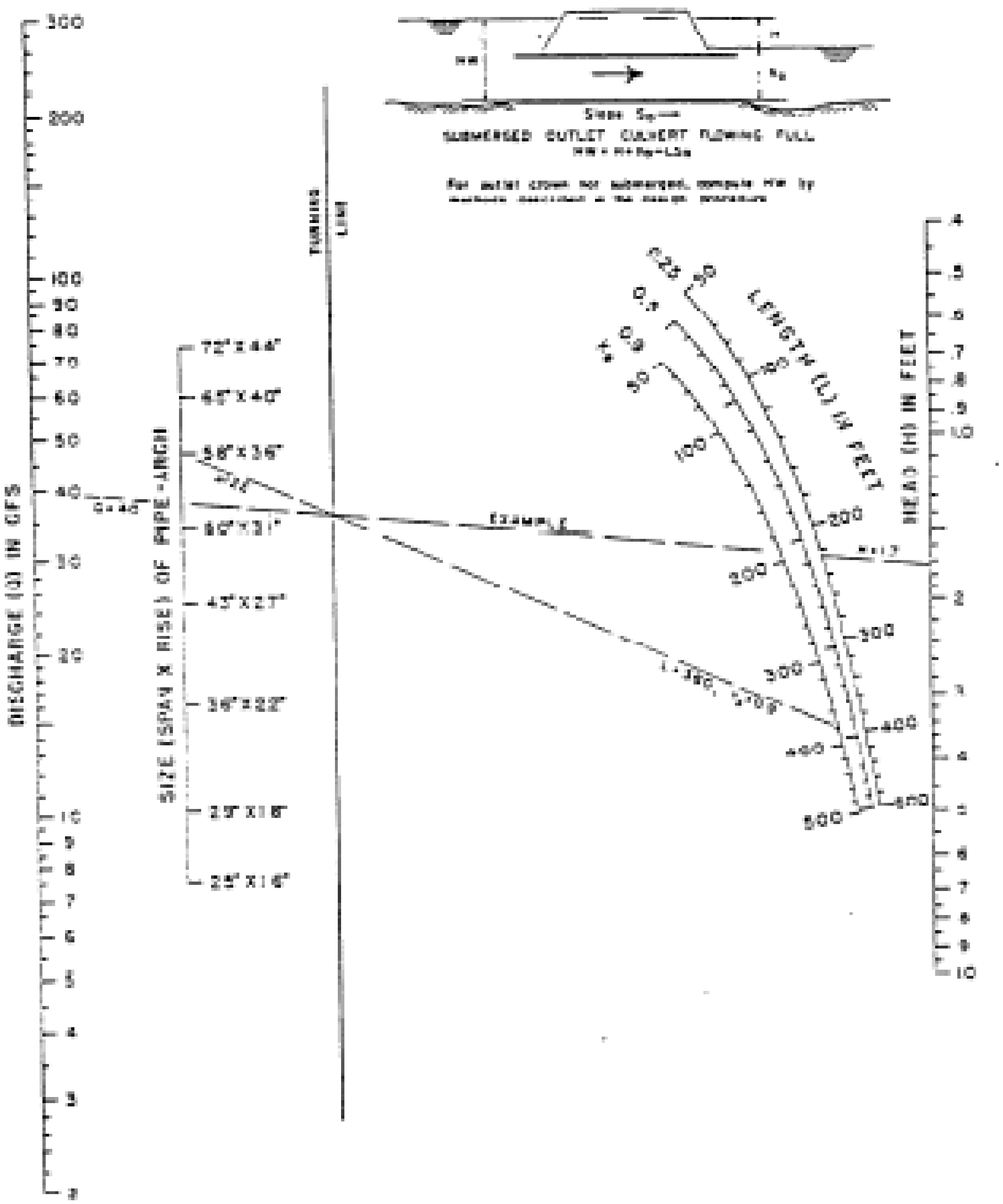
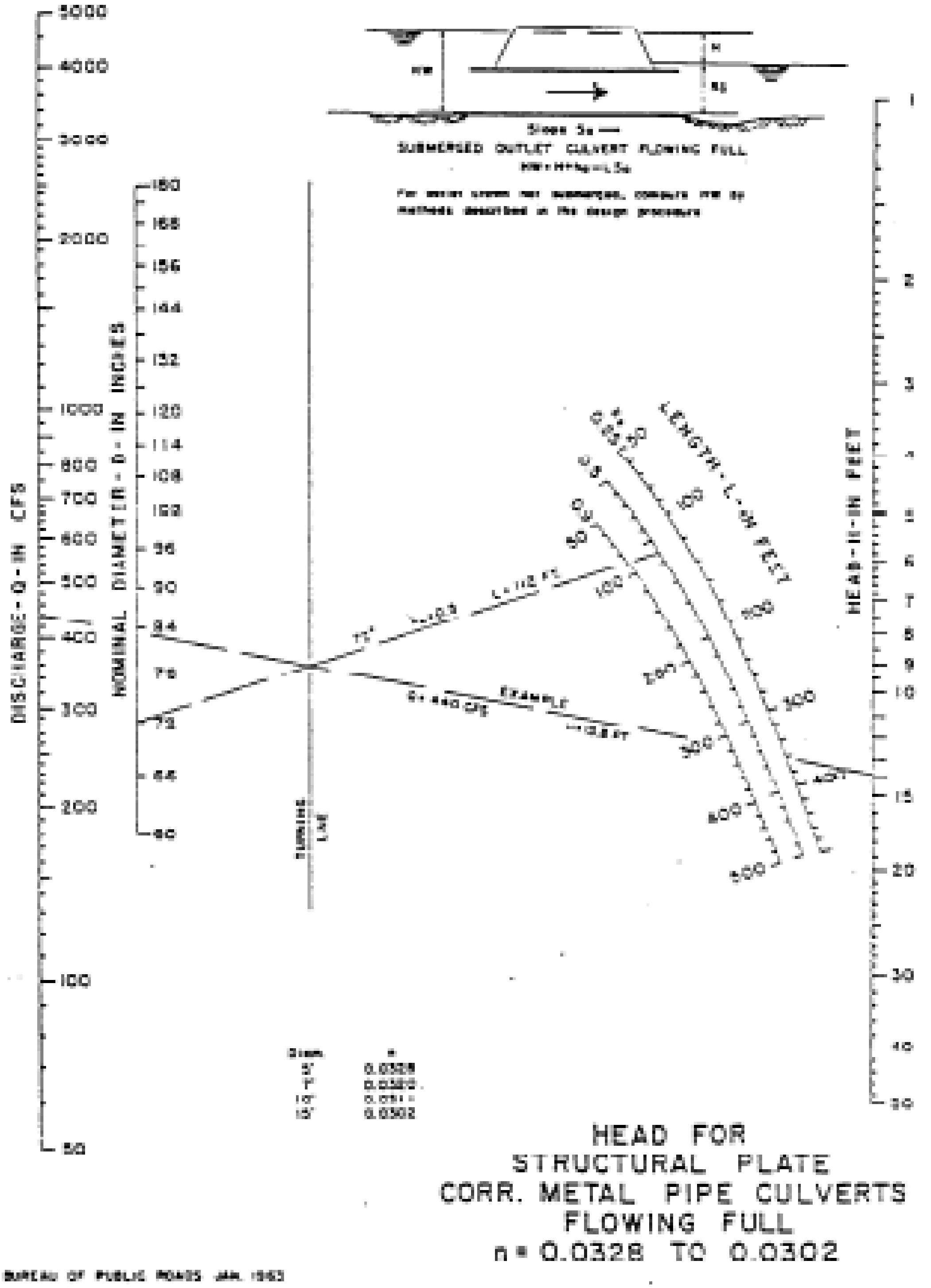
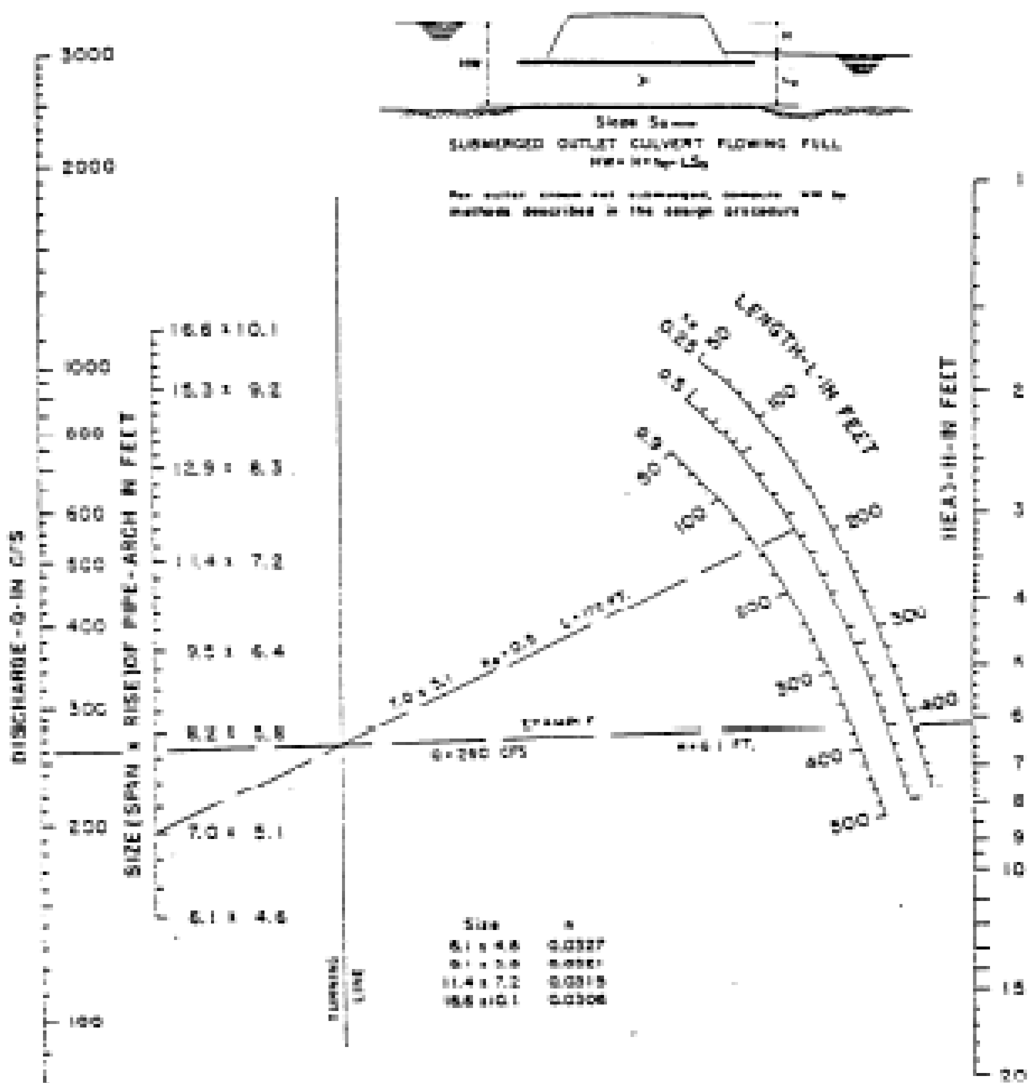


HEAD FOR
STANDARD C. M. PIPE-ARCH CULVERTS
FLOWING FULL
n = 0.024



HEAD FOR
STRUCTURAL PLATE
CORR. METAL PIPE CULVERTS
FLOWING FULL
n = 0.0328 TO 0.0302

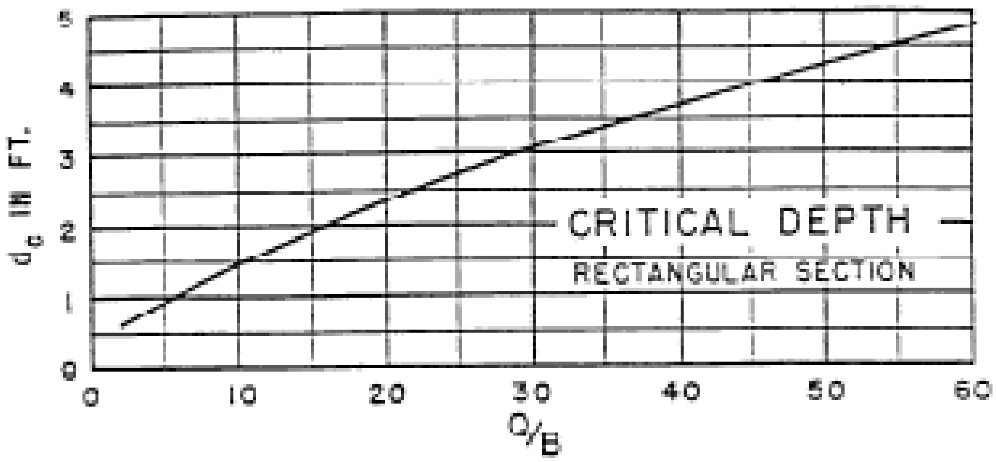
CHART 14

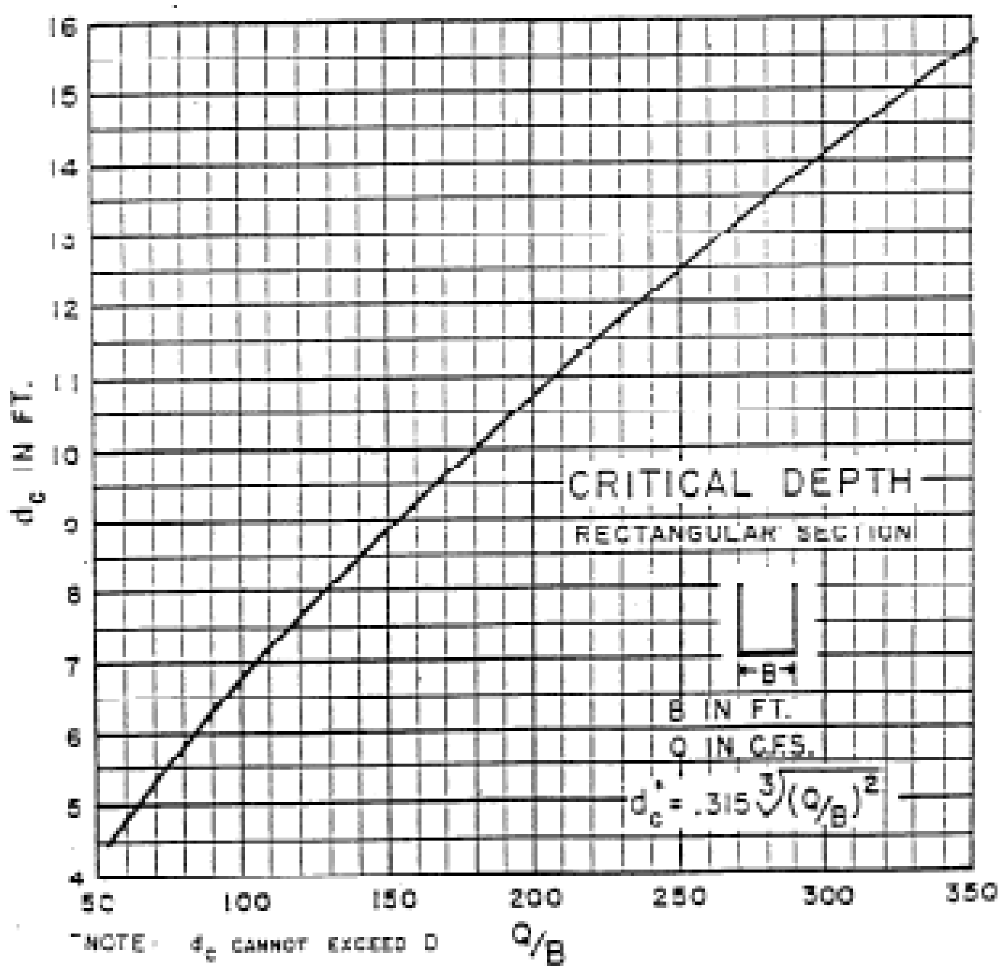


HEAD FOR
STRUCTURAL PLATE
CORRUGATED METAL
PIPE ARCH CULVERTS
18 IN. CORNER RADIUS
FLOWING FULL
n=0.0327 TO 0.0306

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 15





NOTE: d_c CANNOT EXCEED D

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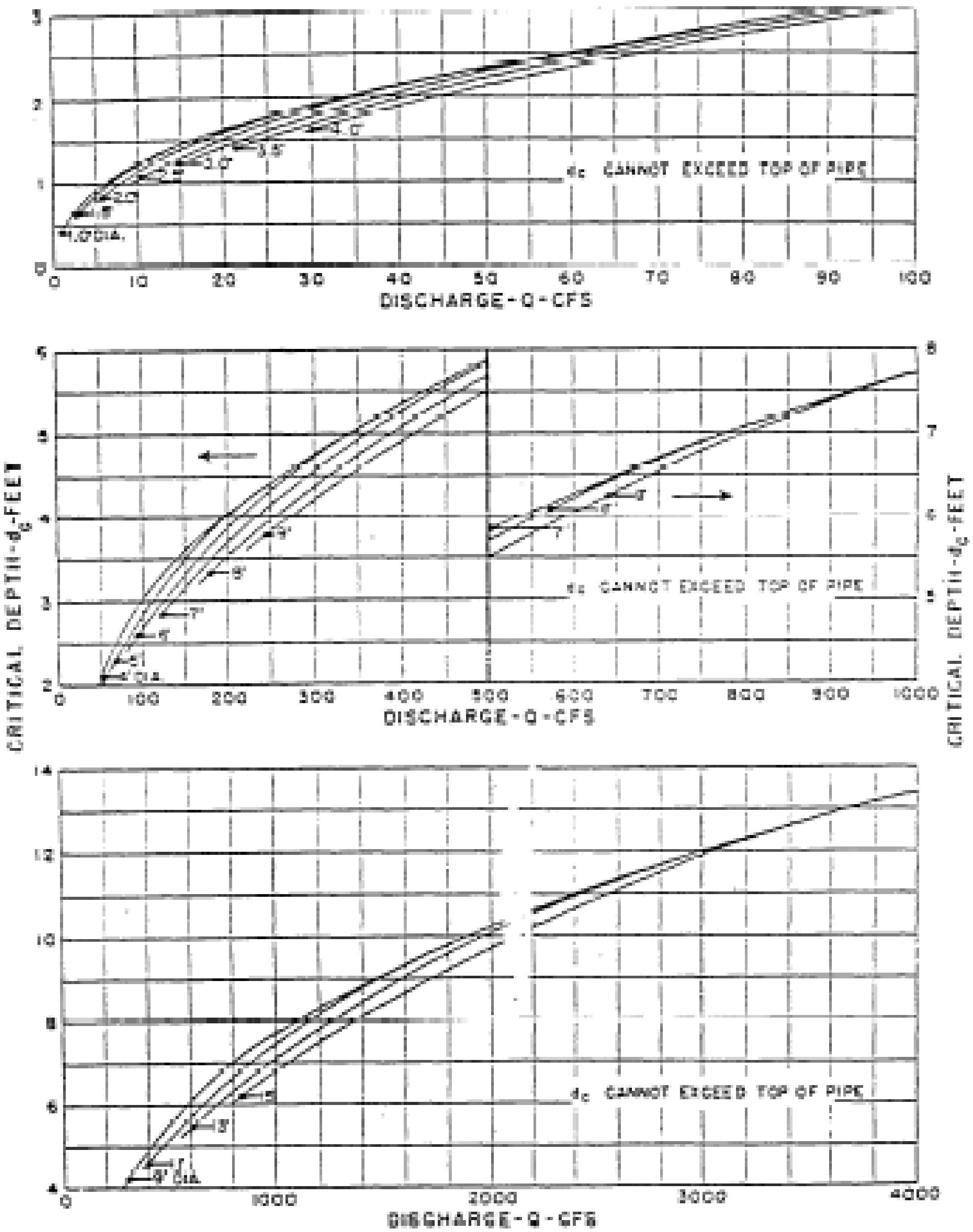
CHART 16



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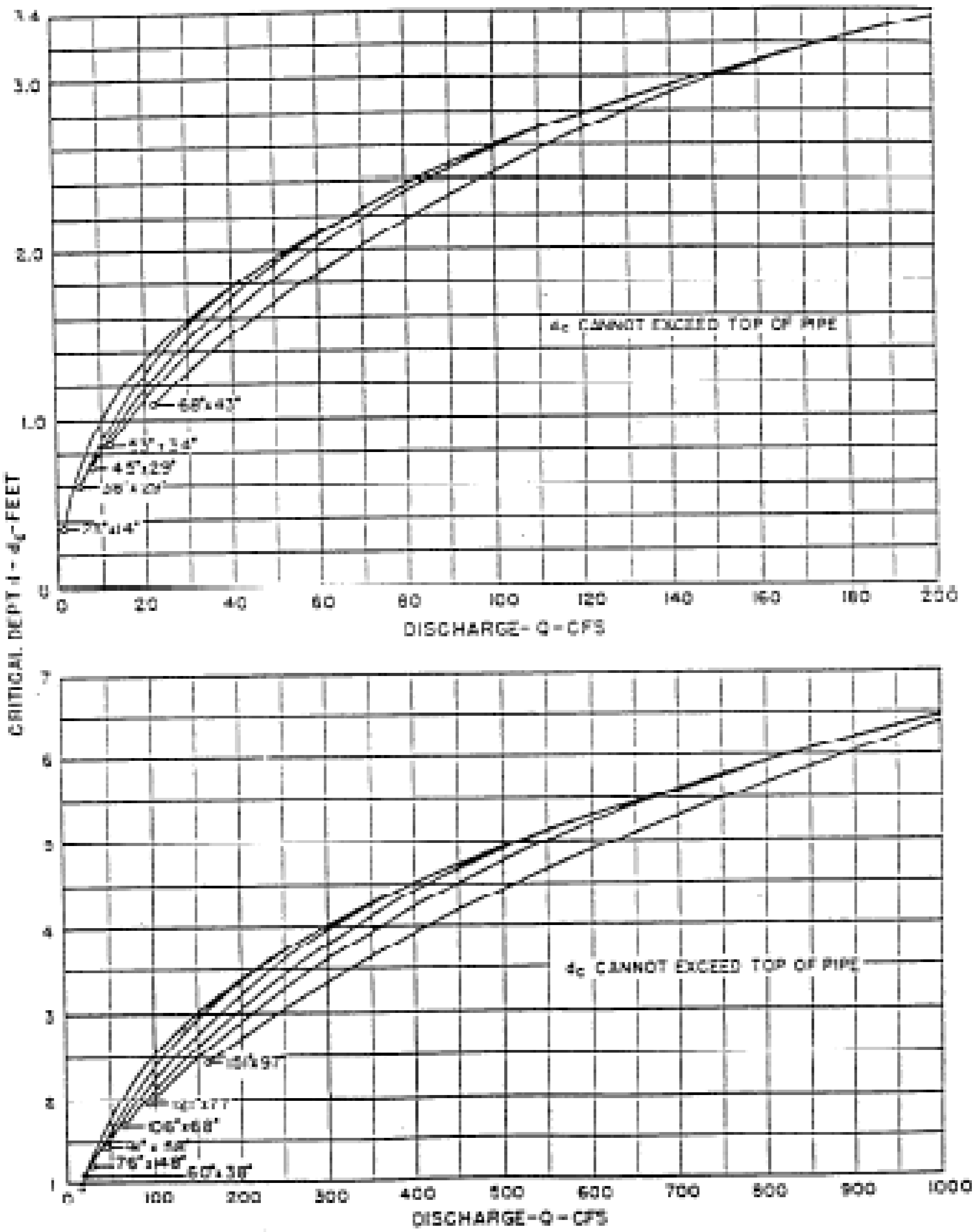
JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

CHART 17

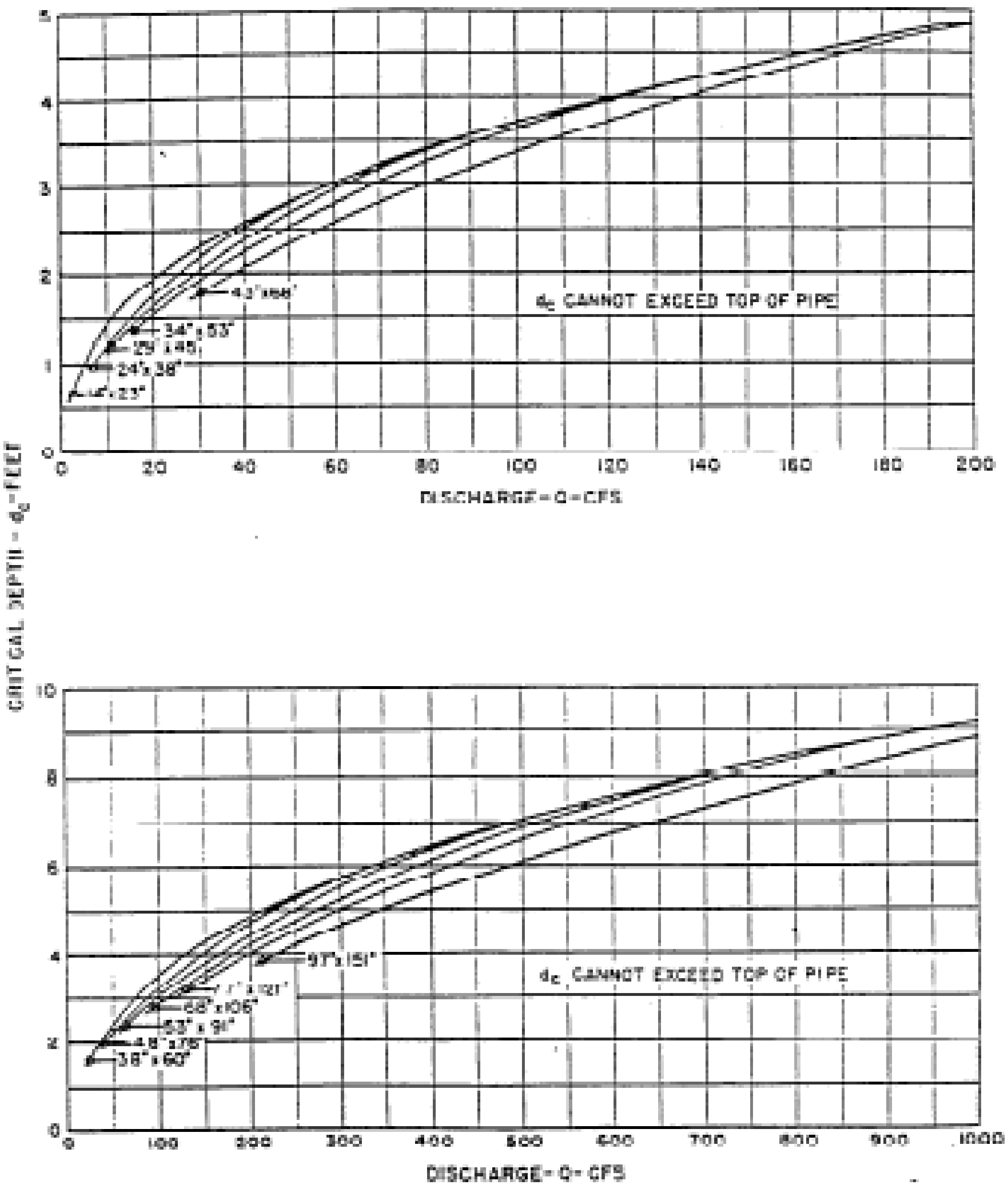


BUREAU OF PUBLIC ROADS

JAN. 1964

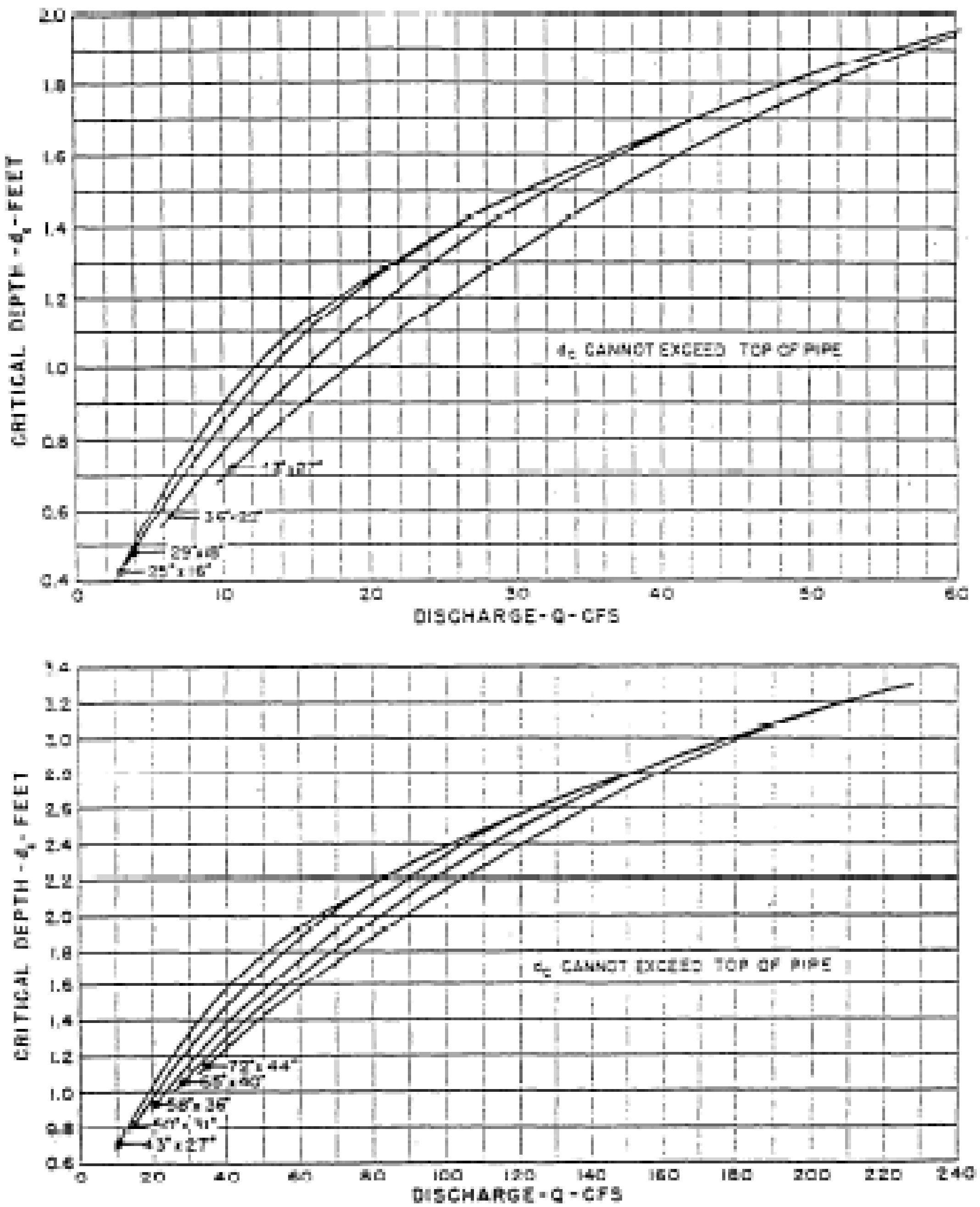
CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL

CHART 18



BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS VERTICAL



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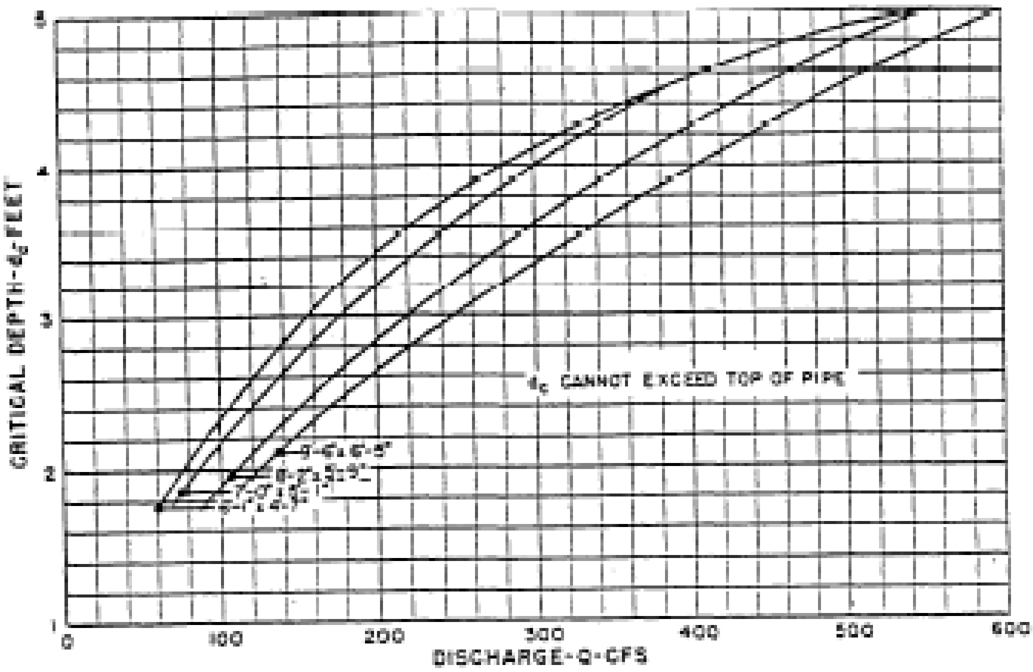
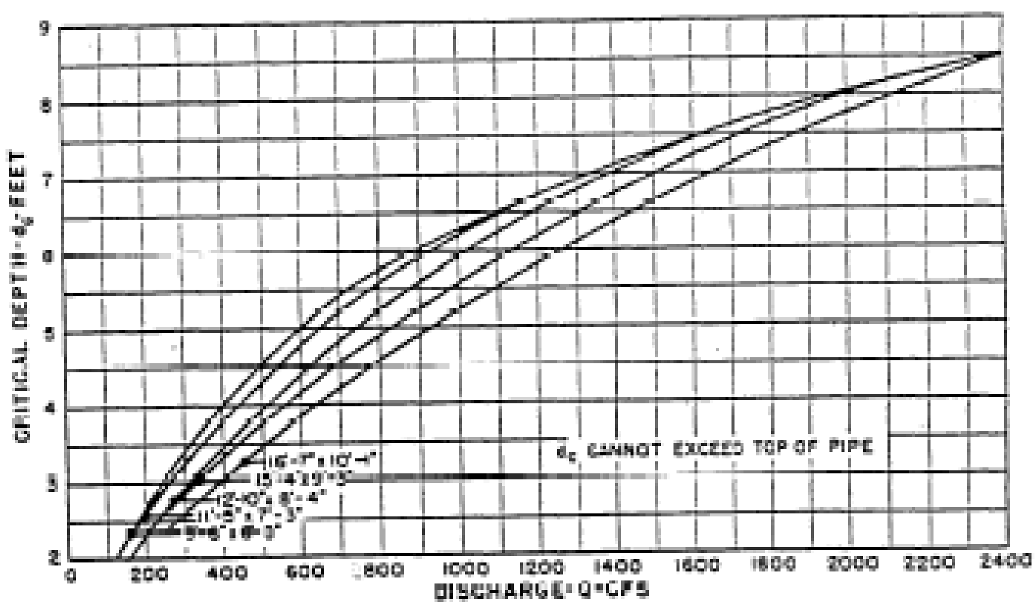
JAN. 1964

CRITICAL DEPTH
STANDARD C.M. PIPE-ARCH

CHART 20





BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
STRUCTURAL PLATE
C. M. PIPE-ARCH
18 INCH CORNER RADIUS

TABLE OF CONTENTS

5.0 STORM SEWERS AND OVERLAND FLOW

5.1 GENERAL

Due to the flat terrain in Fort Bend County, it is infeasible in certain areas to convey the runoff from extreme rainfall events entirely via an underground storm sewer system. Local flooding will occur in areas away from the primary drainage channels because it is simply uneconomical to provide a storm sewer pipe large enough to totally carry the infrequent, severe storm events. For this reason, a sheet flow analysis is required so that street design and alignment assure that excess runoff from extreme storm events will be conveyed to primary drainage channels safely. The development or project is not allowed to increase flows into the receiving channel, ditch or drainage system without sufficient mitigation and supporting calculations. Sheet flow corridors and extreme event swales shall be designated and all required right-of-way shall be established. Special consideration must also be given for off-site sheet flows and their impacts on the planned subdivision.

The discussion presented in this section will be directed primarily at curb-and-gutter streets with underground storm sewers. Roadside ditch systems are acceptable in certain instances, but are not preferred.

5.2 RUNOFF ANALYSES

5.2.1 Frequency Considerations

Flooding in Fort Bend County is generally associated with one of two types of severe rainfall events. The first type is a localized high intensity rainfall of short duration which floods a small localized area causing ponding of water and interruption of traffic flow. The second type is a more generalized rainfall of longer duration, which can cause more widespread flooding and can result in severe damage and loss of life. This second of storm event is generally used to design channels for drainage large areas.

In designing storm sewers for draining small developments, it is the localized high intensity, short duration rainfall event which is used. However, since these storm sewers usually drain into open channels, which are used to convey the runoff from larger areas, the design must take into consideration the interaction of these two systems.

Figure 5-1 illustrates the effect on the hydraulic grade line of a storm sewer for three outlet conditions. Assuming the outlet channel is at its 25-year water level, it can be seen from Part A of Figure 5-1 that the hydraulic grade line for the standard design condition remains at or below the gutter level at the furthest inlet. For this condition, there is no street ponding and the storm sewers are functioning at or below their design capacity.

Parts B and C of the Figure show the case where the tailwater condition is above the design level. Street ponding begins to occur throughout the storm sewer drainage system, as the storm sewers are unable to operate at their design capacity. This local flooding situation could also occur when the tailwater is below design conditions if local rainfall is in excess of that used in the design of the storm sewer system. As this widespread street ponding starts to occur, provisions must be made to limit the depth of ponding to a level below that which will cause significant property damage. In general, flood elevations shall be considered unacceptable when they exceed the lowest of the following: 1) one foot over natural ground; 2) one foot over top of curb; or 3) one foot below the lowest slab elevation.

5.2.2 General Design Guidelines

Storm sewers shall be designed to carry the design storm peak flow (See Section 5.2.3 for design frequency). To obtain the design storm peak flow, the Rational Method can be used for drainage areas less than 200 acres. For areas from 200 to 2000 acres, the discharge curves can be used to obtain the peak flows. To obtain peak flow for larger drainage areas, hydrologic modeling with HEC-HMS should be used. A detailed description of these techniques is contained in Section 2.0 of this manual.

The grading of the development or lots shall conform to the project plans. The grading of the site or lots shall be from the back of the lots or development to the front of the lot or to an applicable designated drainage system that has been designed to convey the project flows.

When filling lots adjacent to a channel, a transition of the back of the lot to natural ground at the channel right-of-way must occur so as to not hinder maintenance operations within the channel right-of-way. The design should also minimize the amount of the back of lot draining directly to the channel right-of-way. Review of the preliminary design by the Fort Bend County Drainage District Engineer should be obtained before any detailed engineering is performed.

For all storm sewer systems or for enclosing an existing open channel, the hydraulic calculations and hydraulic profiles along with the construction plans of the closed-conduit system must be submitted to the Fort Bend County Drainage District Engineer for review.

A preliminary design should first be performed utilizing the design storm and the Rational Method and partially summarized in Table 5-1. Then, if necessary, adjust the sizes of the pipes or boxes to meet the required hydraulic grade line criteria outlined in Section 5.2.3 which follows.

Generally, no more than one storm sewer outfall per 1000 feet of channel or one outfall per smaller tract will be allowed on each side of the receiving channel, detention basin or waterway.

5.2.3 Specific Design Flow Frequency Criteria

The recommended design flow frequency criteria to be used for continuous closed-conduit systems are given below:

1. For all drainage areas, the design flows shall be determined utilizing the Rational Method and storm sewer curves (See Figure 5-2) shown in Table 5-1 as a minimum. The conduit shall be designed in accordance with methodology as outlined in Section 5.3.2.

2. For portions of the system serving areas between 100 acres and 200 acres, it is additionally required that the 25-year hydraulic grade line be at or below the gutter line for the portion of the system which drains 100 or more acres. For this computation, the 25-year discharge for fully developed conditions based on the Rational Method (See Section 2.4) should be used. A 25-year design water surface should be assumed in the outfall channel.

3. For portions of the system serving an area larger than 200 acres, the 100-year flow for fully developed conditions should be used (based on the hydrological modeling using HEC-HMS) to insure that the 100-year hydraulic grade line will be below the natural ground elevation at all points along this portion of the closed system. A 25-year design water surface should be assumed in the outfall channel.

4. For systems designed in accordance with (2) or (3), sufficient additional inlet capacity shall be provided to allow for entry into the closed-conduit system of runoff in excess of the runoff conveyed through the storm sewer system up to the design capacity of the closed-conduit system.

5. For all areas, overland flow shall be considered as discussed in Section 5.4.

6. Closed systems adjoined to an upstream open channel shall be designed for the 100-year ultimate discharge.

5.3 STORM SEWERS

5.3.1 Design Criteria

The following specific criteria and requirements shall apply to the design and construction of storm sewers in Fort Bend County. The following criteria were taken primarily from General Design Requirements for Sanitary Sewers, Storm Sewers, Water Lines, and Paving, City of Houston (1983 or latest version).

1. Calculation of the hydraulic grade line for design conditions in a specific branch of storm sewer shall proceed upstream from the level of the 25-year water surface elevation in the outfall channel.

2. The minimum diameter of a pipe in a sewer line shall be 24”.

3. The Manning's "n" value to be used in a reinforced concrete pipe storm sewer shall be 0.013. For corrugated metal pipe, the "n" value shall be as shown in Table 5-2.

4. The minimum velocity of flow to be allowed in a section of storm sewer flowing full shall be 3 fps. The maximum velocity shall be 10 fps.

5. Provisions must be made for all adjacent undeveloped areas with natural drainage patters directing overland flow into and across planned development.

6. Before a particular storm sewer design will be reviewed, the following items must be presented:

   a. A contour and drainage area map showing all pertinent subareas, including contribution off-site areas.

   b. A listing of all relevant hydrologic design flow calculations, which shall include all contributing off-site flows.

   c. Calculations for determining the hydraulic gradient, along with a profile of its location.

   d. A plan showing the location of all manholes and inlets, and the alignment of all storm sewers in the right-of-way.

   e. A profile showing the placement of storm sewers and the location of all pipe size changes, grade changes, and pipe intersections.

7. All storm sewers and appurtenant construction shall conform to the City of Houston Department of Public Works publication <u>Specifications for Sewer Construction</u>, Form E-14-62, City of Houston Drawing Nos. 529-S-1, 530-S-1, 530-S-2, and all subsequent revisions, or approved equal. All outfalls into ditches, channels, streams or detention ponds shall include the use of erosion protection in accordance with Figures 3-4 and 3-5.

8. All storm sewers shall be constructed with reinforced concrete pipe, or approved equal. Corrugated galvanized metal pipe, or other approved equal, may be used only at the storm sewer outfall into unlined channels. The use of polymer or other approved coatings for outfall pipes is required. Submit applicable coating information to the FBCDD Engineer for consideration.

9. All cast-in-place concrete storm sewers shall follow the alignment of the right-of-way or easement.

10. All pre-cast concrete pipe storm sewers shall be typically designed in a straight line or shall conform to the Texas Department of Transportation Specifications and all subsequent revisions or approved equal.

11. All storm sewer inlet leads shall be designed in a straight line alignment.

12. Storm sewers shall be located in public street rights-of-way or in easements that will not prohibit future maintenance access.

13. In most cases where easements are restricted to storm sewers, the pipe should be centered within the limits of the easement.

14. For all storm sewers having a cross-sectional area equivalent to a forty-two inch (42”) inside diameter pipe or larger, soil borings with logs shall be made along the alignment of the storm sewer at intervals not to exceed five hundred feet (500’) and to a depth not less than three feet (3’) below the flowline of the sewer. The required bedding of the storm sewer as determined from these soil borings shall be shown in the profile of each respective storm sewer. The design engineer shall inspect the open trench and may authorize changes in the bedding indicated on the plans. Such changes shall be shown on the record drawings and, along with soil boring logs, submitted to the County Drainage District Office. All bedding shall be constructed as specified in the Texas Department of Transportation Specifications and all subsequent revisions, or approved equal.

15. All storm sewer outfalls shall conform to the requirements and specifications defined in Section 3.0, Open Channel Flow, and Figure 3-5.

5.3.2 General Design Methodology

It is recommended that design of a storm sewer system proceed as follows:

1. Determine the 25-year water surface elevation in the channel at the storm sewer outfall using appropriate backwater calculations.

2. Determine the design flow rates for all sections of storm sewer based on drainage area size.

3. Assuming storm sewer pipes are full at design flows, determine the appropriate sizes for all sections of storm sewer using Manning's equation and assuming uniform flow conditions.

4. Begin calculation at the 25-year water surface elevation in the outfall channel and plot the hydraulic gradient for the design storm. Include all relevant energy losses. The hydraulic gradient must not exceed the roadway gutter flowline elevation.

5.3.3 Head Losses

Head losses at structures shall be determined for inlets and manholes in the design of closed conduits. The design engineer should determine the relative significance of the minor losses and their applicability to the design. If they are insignificant, they may be omitted.

5.3.3.1 Head Losses at Structures

The equation for the head loss (feet) at an inlet or manhole is as follows:

$$\text{Head loss} = \frac{V_2^{\,2} - KV_1^{\,2}}{2g} \qquad (5\text{-}1)$$

where

V_1 = velocity in the upstream pipe (fps).

V_2 = velocity in the downstream pipe (fps).

K = junction or structure coefficient of loss. (See Table 5-3)

5.3.3.2 Entrance Losses

A special case of sudden contraction is the entrance loss for pipes. The equation for head loss at the entrance to a pipe is given as follows:

$$\text{head loss} = K\frac{V^2}{2g} \qquad (5\text{-}2)$$

where

K = entrance loss coefficient. (See Table 5-4.)

V = flow velocity in pipe (fps).

5.3.4 Manholes

Manholes shall be placed at the location of all pipe size or cross section changes, pipe sewer intersections or P.I.'s, pipe sewer grade changes, street intersections, at maximum intervals of 500 feet measured along the centerline of the pipe sewer; and at all inlet lead intersections with the pipe sewer where precise concrete pipe sewers are designed.

5.3.5 Inlets

Three types of inlets are recommended for use in Fort Bend County, the Type "BB" Inlet, Type "C-1" Inlet and the Type "H-2" Inlet (as identified by the City of Houston). All inlets shall be constructed as specified in the Fort Bend County Design Standards and Details, or approved equal.

5.3.5.1 Inlet Capacity

The capacity of inlets shall be determined as shown in Step 7 of Section 5.4.3 of this manual. All inlets shall be designed to carry at least the design storm frequency runoff.

5.3.5.2 Inlet Spacing

Curb inlets must be spaced to handle the design storm discharge so that the hydraulic gradient does not exceed the roadway gutter elevation. Inlets shall be spaced so that the maximum travel distance of water in the gutter will not exceed six hundred feet (600’) one way for residential streets and three hundred feet (300’) one way on major thoroughfares and streets within commercial developments. Curb inlets shall be located on intersection side streets to major thoroughfares for all original designs or developments. Special conditions warranting other locations of inlets shall be determined on a case-by-case basis.

5.4 STREET DRAINAGE OF STORM SEWER OVERFLOW

When the capacity of the underground system is exceeded and street ponding begins to occur, careful planning can reduce or eliminate the flood hazard for adjacent properties. Street layout and pavement grades along with extreme event swales are the key components in developing a successful system which can convey the storm sewer overflows to the outfall channel designed to carry the 100-year storm runoff. The following design methodology and example is derived from typical criteria in Harris County.

5.4.1 Land Plan and Street Layout

Designing an effective internal system must begin with the land plan and street layout. Awareness of overland flow problems in this early phase of the development process can reduce costly revisions and delays later on in the project. When designing drainage systems, attention needs to be given to special problems created by the topography. Excessive street cuts which can create ponding levels that hamper vehicle access and/or present a flood hazard must be avoided.

Some examples of undesirable sheet flow patterns are depicted in Figure 5-3 and include:

a) Cul-de-sac streets sloping downhill designed so that sheet flow can only escape through building lots.

b) The placing of a curve or turn in a roadway in a low area so that sheet flow into that curve or turn can escape only through existing building lots.

c) Many streets “T”ing into one street which is lower than the intercepting streets so that sheet flow down the streets can escape only through existing building lots.

Proper engineering foresight in the design of items such as emergency relief swales or underground systems can solve these potential problems. Some examples of acceptable sheet flow patterns are shown in Figure 5-4.

The maximum allowable ponding level for a new street is the lowest of the following: (1) one foot above natural ground; (2) one foot above top of curb; or (3) one foot below the lowest slab elevation. The design engineer must check to see if the storm drainage system can convey flows from a 100-year storm event without ponding water in the street at levels that exceed the maximum allowable level. The 100-year discharge can be obtained by following the procedures outlined in Section 2.4. A 25-year tailwater condition should be assumed in the outlet channel. If the maximum level would be exceeded, the engineer must analyze the route along the street system that will convey the overflows to the major drainage channel and verify that the overflows will not exceed the maximum allowable ponding level. In making this analysis, the engineer can account for the portion of flows that would be carried by the sewer system in addition to the street system, assuming a 25-year tail-water condition. The engineer must also verify that there are no increases in flows into the receiving channel. These increases are not allowed without appropriate mitigation and approval by the Fort Bend County Drainage District.

5.4.2 Conveyance of Surface flow to Primary Channels

Once it has been determined that ponding levels are excessive and where the collective sheet flow is going to go, provisions must be made to get the overflows into the appropriate drainage channel. This may be done through the use of additional pipe capacity and inlets or by using a surface swale.

The surface flow conveyance system shall be contained within an easement dedicated to the Drainage District. The easement shall be of sufficient width to operate and maintain the system.

Since a surface swale system would act only under emergency conditions and would not function under normal circumstances, all precautions must be taken to insure that the relief system will function when needed. The recommended design procedure for sizing storm sewers for sheet flow conveyance is presented in Section 5.4.3. The design procedure recommended for sizing of the surface swale is similar to the procedure for the pipe outfall as described in Section 5.4.3. First, the appropriate values from steps one and two are computed, then the required extreme event swale cross-section is determined by normal depth calculations, sizing the swale such that an acceptable water surface is achieved.

5.4.3 Design Procedure for Pipe Outlet

This section outlines the procedure recommended for designing an underground pipe system to convey overflows to a primary drainage channel. Because the majority of subdivisions in Fort Bend County are designed with curb-and-gutter streets, modification of the last storm sewer reach is generally all that is necessary to handle the overflow.

The recommended procedure is given below along with an example based on the drainage system presented in Figure 5-3 (c).

1. Determine the 100-year peak flow at the point of concentration from all existing and future contributing drainage areas for 100% development conditions. In the example, the contributing drainage area is 40 acres and the 100-year discharge is 147 cfs.

2. Determine the 25-year frequency water-surface elevation in the drainage channel at the pipe outfall point. Based on a 25-year backwater profile, the water surface elevation in the channel for the example is 97.0 feet.

3. Determine the maximum energy head, H, available between the outfall point and ponding area by subtracting the maximum allowable ponding elevation in the ponding area from the channel's 25-year water surface elevation.

   With a slab elevation of 101.5 feet and a top of curb and natural ground elevation of 100.0 feet in this example, the maximum allowable ponding elevation is the lowest of the follows: 1) one foot over natural ground; 2) one foot over the top of curb; or 3) one foot below the lowest floor elevation. In this case, the maximum elevation is controlled by the lowest floor elevation and is 100.5 feet. There is 3.5 feet of head available (H).

4. Establish a size of the storm sewer pipe and compute the head loss using the following equation:

$$h_p = \frac{4.66 Q^2 n^2 L}{D^{16/3}} \qquad (5\text{-}3)$$

where

h_p = Head loss in feet

Q = 100-year discharge in cubic feet per second

n = Manning's "n" value

D = Diameter of pipe in feet

L = Length of pipe in feet

For this example, 65 linear feet of 60-inch corrugated metal pipe (cmp) with a Manning's "N" value of 0.024 and 120 linear feet of 60-inch reinforced concrete pipe (rcp) with a Manning's "n" value of 0.013 was selected. The head loss is as follows:

$$h_p = \frac{4.66\,(Q^2)}{D^{16/3}} \left(n^2_{\ cmp}\, L_{\ cmp} + n^2_{\ rcp}\, L_{\ rcp}\right)$$

$$h_p = \frac{4.66\,(147)^2}{5^{16/3}} \left((0.024)^2\,(65) + (0.013)^2\,(120) \right)$$

$$= 1.09 \text{ feet}$$

5. Compute the head loss through the leads, h_L, using Equation 5-3. Experience has shown that the 24-inch diameter leads generally cause excessive head loss. The 30-inch diameter leads are satisfactory in most cases, while the 36-inch leads are too large for the most common street inlets type "B-B" and "C-1". Therefore, the 30-inch diameter was selected.

Estimate the percentage of 100-year runoff flowing through each lead.

Assume the 147 cfs to be divided between three leads as follows:

Lead 1	20-foot lead with a flow of 56 cfs.
Lead 2	20-foot lead with a flow of 56 cfs.
Lead 3	45-foot lead with a flow of 37 cfs.

$$h_{L_1} = h_{L_2} = \frac{4.66\, n^2\, Q^2\, L}{D^{16/3}} = \frac{4.66\,(0.013)^2\,(56)^2\,(20)}{2.5^{16/3}}$$

$$= 0.37 \text{ foot}$$

$$h_{L_3} = \frac{4.66\,(0.013)^2\,(37)^2\,(45)}{2.5^{16/3}}$$

$$= 0.37 \text{ foot}$$

6. Determine the energy head available at each inlet using the equation:

$$h_i = H = h_p - h_L \qquad (5\text{-}4)$$

If h_i is negative, the hydraulic grade line is above the maximum ponding elevation. Increase the capacity of the system and repeat steps 4, 5, and 6.

If h_i is positive, check the elevation of the hydraulic grade line relative to the maximum ponding elevation. For grade lines above the gutter line, use h_i as the energy head on the inlet; otherwise, make the value of h_i equal to the maximum ponding elevation minus the gutter elevation.

For this example, assume the hydraulic grade line is above the gutter elevation. Since the head loss through the three leads in the example are similar, the available head at each inlet is:

$$h_i = 3.5\text{-}1.1\text{-}0.37 = 2.03$$

7. Determine the type of inlets required to handle the portion of the 100-year flow reaching the ponding area. The flow through the inlet(s) must be equal to or

greater than the flows estimated in Step 5 for each lead. Use the following orifice equation to compute the flow into each inlet.

$$Q = CA \quad (2gh_i)^{1/2} \tag{5-5}$$

Where

Q = discharge in cubic feet per second

C = orifice coefficient (0.8 for inlets)

A = area of inlet opening. (Type “B-B” = 2.14 square feet and Type “C-1” = 6.50 square feet.)

G = acceleration of gravity (32.2 ft/sec^3)

h_i = as defined in Step 6

Type “C-1” inlets are selected for Inlet 1 and Inlet 2 and
Type “B-B” inlets are selected for Inlet 3 across the street.

$$Q_{C-1} = 0.8(6.50) \quad (64.4\ (2.0)\ )^{1/2} = 59 \text{ cfs}$$

$$2Q_{B-B} = 2\ (0.8(2.14) \quad (64.4\ (2.0)^{1/2}) = 38 \text{ cfs}$$

Thus, it is shown that a Type “C-1” inlet at Inlet 1, a Type “C-1” inlet at Inlet 2, and two Type “B-B” inlets at Inlet 3 will convey the 100-year sheet flow to the channel with the energy head available. If this inlet choice is adequate, the design is complete.

8. Repeat Steps 4 through 7 until the combination of storm sewer pipe, leads, and inlets adequately conveys the 100-year sheet flow to the channel with the energy head available, and is the most economical.

5.4.4 Design Procedure for Extreme Event Swale

Design calculations shall be provided that substantiate the design elevation of the extreme event swale and the needed design requirements of the extreme event swale. The design and construction of the extreme event swale shall be consistent with the needed capacity at each location.

The extreme event swale within the right-of-way of detention ponds and outfall channels shall have a minimum 6-foot bottom width and 6:1 side slopes. The swale will be designed to use interlocking concrete blocks or concrete slope paving to protect from erosion.

5.4.5 Roadside Ditch Drainage

Under certain conditions, roadside ditch drainage is acceptable as an alternative to curb-and-gutter systems. However, a similar potential for flooding exists when flow in roadside ditches exceeds capacity. Provisions must be made to assure that the amount of water ponded behind an elevated roadway does not reach damaging levels. Projects or developments that drain to roadside ditches are only allocated a pro rata share of the existing ditch capacity. (See Figure 5-5 for typical roadside ditch drain detail.)

5.4.5.1 Preliminary Approval

Preliminary approval for the use of roadside ditch systems must be obtained from the Fort Bend County Drainage District Engineer prior to the submittal of contour and drainage area maps, and hydrologic and hydraulic calculations.

5.4.5.2 Design Criteria

The following requirements taken from the General Design Requirements for Sanitary Sewers, Storm Sewers, Water Lines and Paving, City of Houston (1983 or latest version) must also be met in the design of roadside ditch systems in Fort Bend County:

1. The design flow shall be determined based on the projected land use and the rainfall runoff curves from Figure 5-2.

2. Minimum acceptable ditch section shall have a side slope no steeper than 4 horizontal to 1 vertical.

3. The minimum bottom width for roadside ditches shall be two feet.

4. The “n” coefficient for the ditch calculations shall be a minimum of 0.04. All values must be justified.

5. The minimum grade or slope of the ditches shall be 0.10%.

6. Hydraulic design computations must be submitted for each drainage ditch system. Computations shall include the effect of future driveway culverts, which shall be sized taking into account design flow and ditch depth.

7. The computed water surface of the ditches shall be a minimum of 0.5 feet below finished grade elevations along the street edge of pavement.

8. The entire ditch must be revegetated immediately after construction to minimize erosion.

9. Erosion control methods shall be utilized in the ditch designs where velocities of flow are calculated to be greater than five feet per second or where soil conditions dictate their need.

10. The minimum depth of the ditches shall be 18 inches and the maximum depth shall be 4 feet.

TABLE 5-1

RAINFALL RUNOFF CURVES FOR
FORT BEND COUNTY

Time of Concentration (minutes)	Ci Values for Curve No.					
	1	2	3	4	5	6
10	2.06	1.72	1.38	1.15	.92	.58
11	2.03	1.70	1.36	1.13	.91	.57
12	2.00	1.67	1.34	1.11	.90	.56
13	1.97	1.65	1.32	1.10	.88	.55
14	1.95	1.63	1.31	1.09	.87	.55
15	1.92	1.61	1.29	1.07	.86	.54
16	1.91	1.59	1.28	1.06	.85	.54
17	1.89	1.57	1.27	1.05	.84	.53
18	1.87	1.56	1.25	1.04	.83	.52
19	1.85	1.55	1.24	1.03	.83	.52
20	1.84	1.54	1.23	1.02	.82	.51
21	1.82	1.53	1.22	1.02	.82	.51
22	1.80	1.51	1.21	1.01	.81	.51
23	1.79	1.50	1.20	1.00	.80	.50
24	1.78	1.49	1.19	1.00	.80	.50
25	1.77	1.48	1.18	1.00	.80	.50
26	1.75	1.47	1.17	1.00	.80	.50
27	1.75	1.46	1.17	1.00	.80	.50
28	1.74	1.45	1.16	1.00	.80	.50
29	1.73	1.44	1.16	1.00	.80	.50
30	1.72	1.43	1.15	1.00	.80	.50
31	1.71	1.43	1.15	1.00	.80	.50
32	1.70	1.42	1.14	1.00	.80	.50
33	1.69	1.41	1.13	1.00	.80	.50
34	1.68	1.40	1.13	1.00	.80	.50
35	1.67	1.40	1.12	1.00	.80	.50
36	1.66	1.39	1.12	1.00	.80	.50
37	1.65	1.38	1.11	1.00	.80	.50
38	1.65	1.38	1.11	1.00	.80	.50
39	1.64	1.37	1.10	1.00	.80	.50
40	1.64	1.37	1.10	1.00	.80	.50

TABLE 5-2
VALUES OF MANNING'S ROUGHNESS COEFFICIENT (n)
FOR CORRUGATED METAL PIPE

Corrugation (Span x Depth)	"n"
2-2/3" x 1/2"	0.024
3" x 1"	0.027
5" x 1"	0.027
6" x 2"	0.030

Source: Criteria Manual for the Design of Flood Control and Drainage Facilities in Harris County, Texas, February, 1984.

TABLE 5-3

COEFFICIENTS AT STRUCTURES

Type of Structure	Coefficient (K)
Inlet on main line	0.50
Inlet on main line with branch lateral	0.25
Manhole on main line with 22-1/2º lateral	0.75
Manhole on main line with 45º lateral	0.50
Manhole on main line with 60º lateral	0.35
Manhole on main line with 90º lateral	0.25

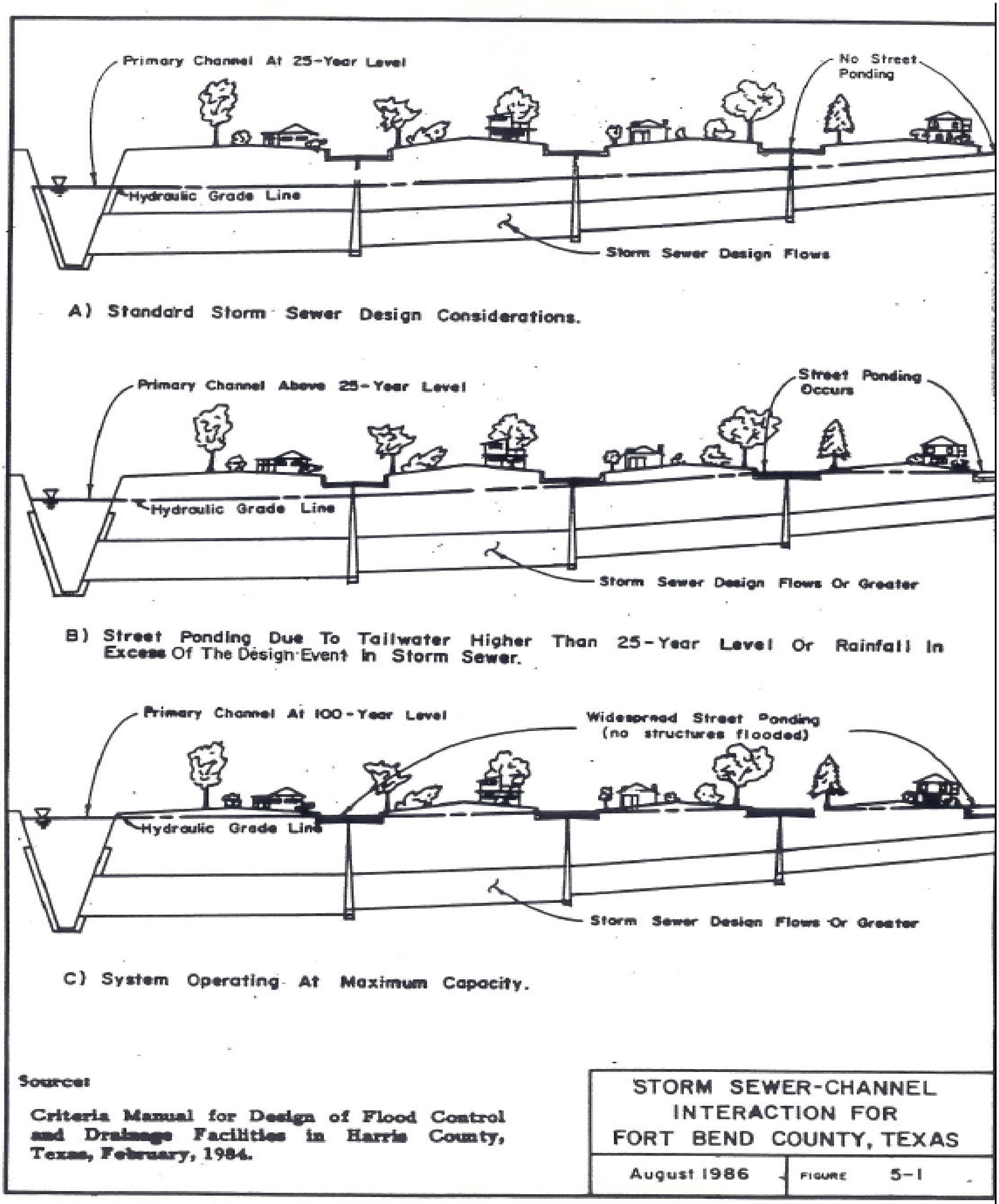
Source: City of Waco, Texas, Storm Drainage Design Manual

TABLE 5-4

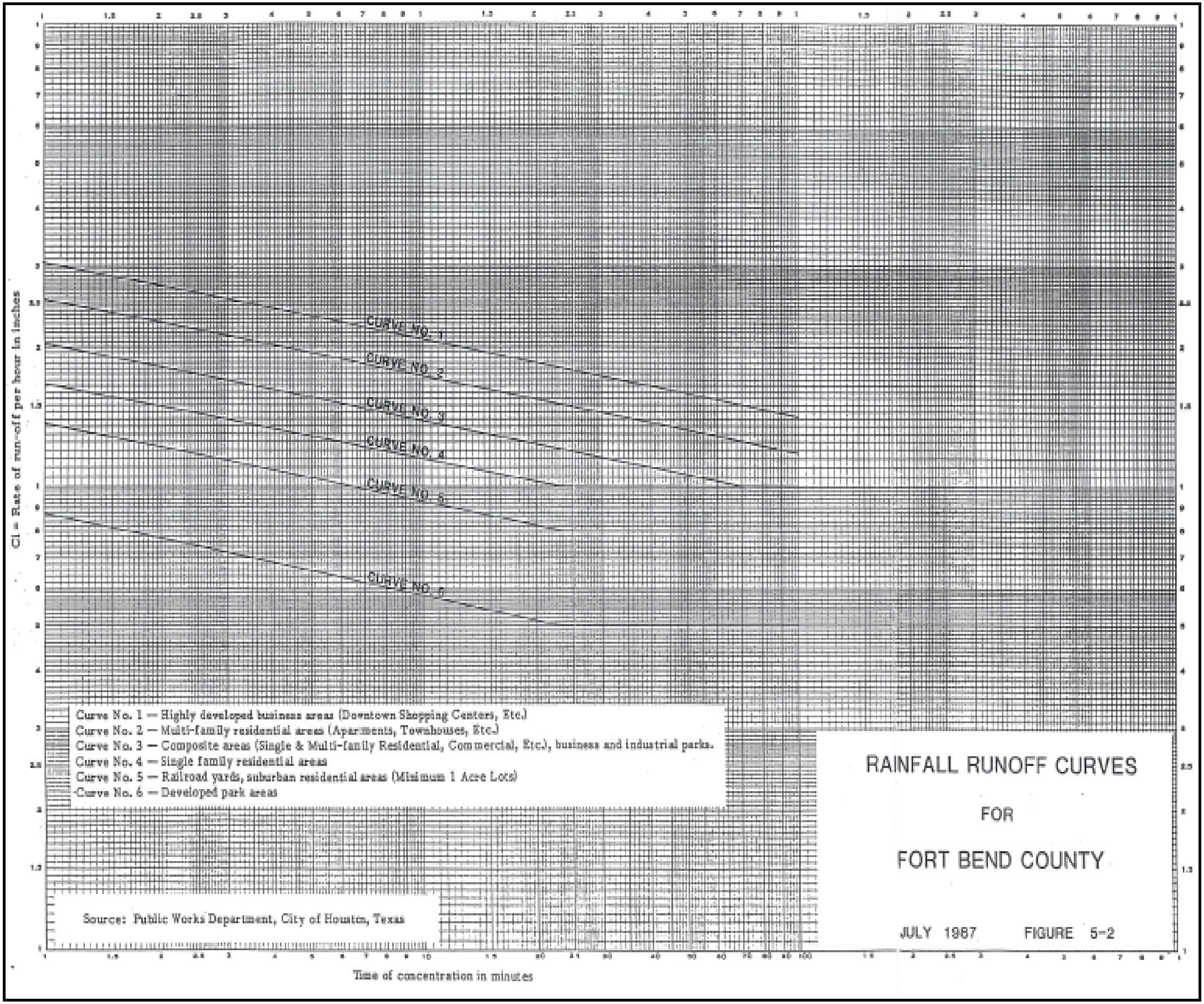
COEFFICIENTS FOR ENTRANCE LOSSES

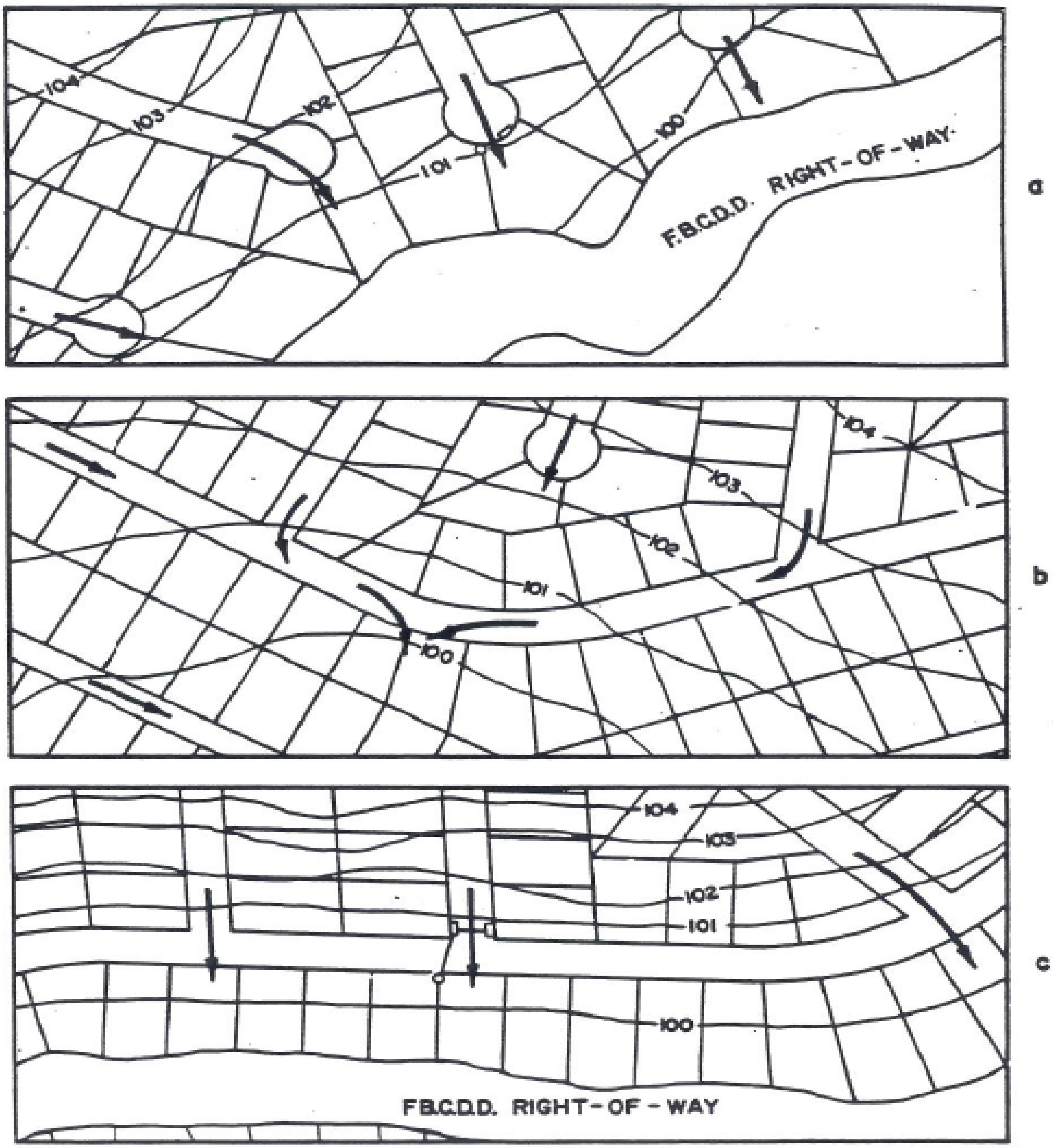
Type of Entrance	Coefficient (K)
Pipe, Concrete[1]	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
Inlet or Manhole at beginning of line[2]	1.25

Source: (1) Hydraulic Charts for the Selection of Highway Culverts, U.S. Department of Commerce, December, 1965.
(2) City of Waco, Texas, Storm Drainage Design Manual.



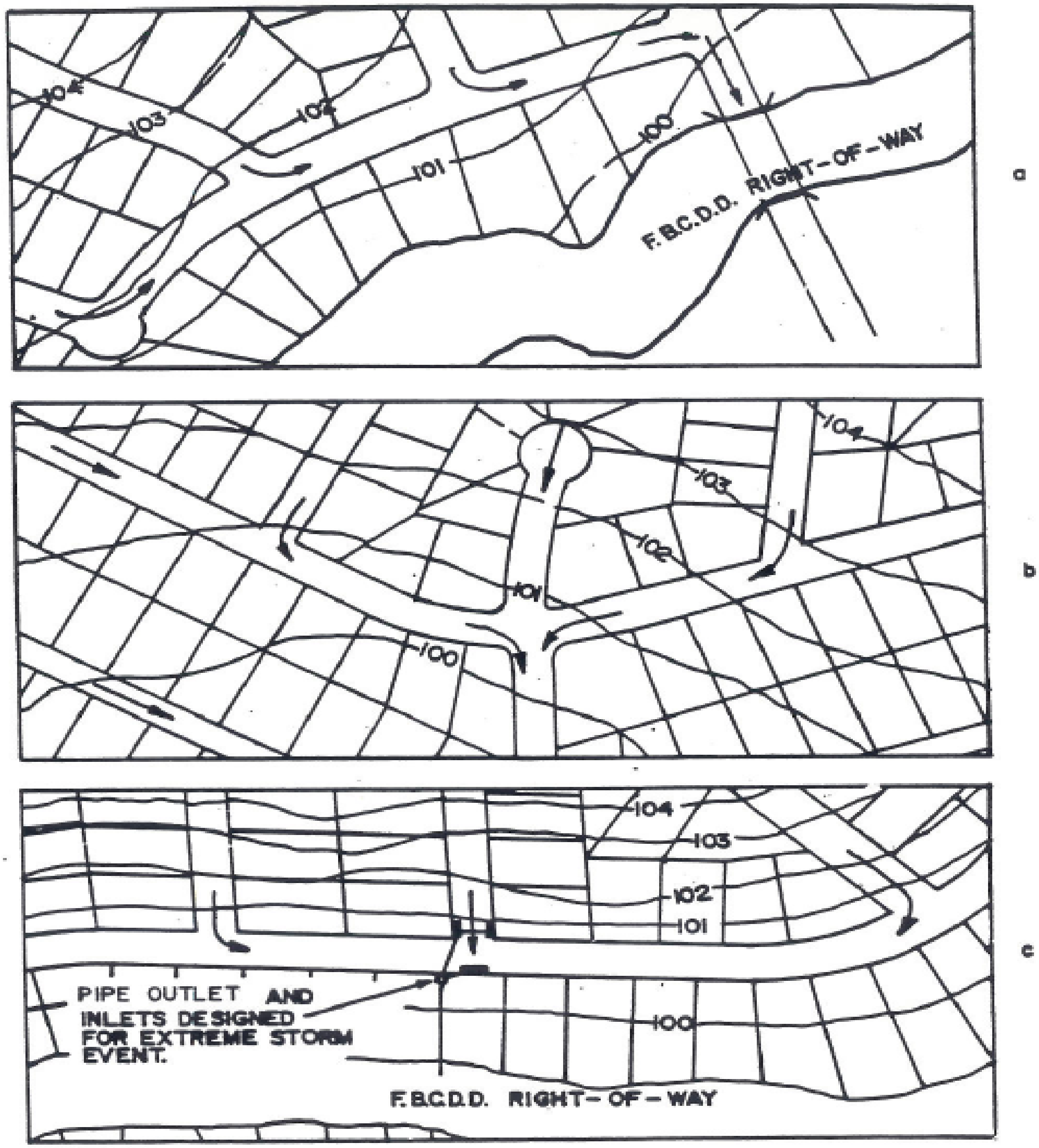
Primary Channel At 25-Year Level
No Street Ponding
Hydraulic Grade Line
Storm Sewer Design Flows
A) Standard Storm Sewer Design Considerations.
Primary Channel Above 25-Year Level
Street Ponding Occurs
Hydraulic Grade Line
Storm Sewer Design Flows Or Greater
B) Street Ponding Due To Tailwater Higher Than 25-Year Level Or Rainfall In Excess Of The Design Event In Storm Sewer.
Primary Channel At 100-Year Level
Widespread Street Ponding (no structures flooded)
Hydraulic Grade Line
Storm Sewer Design Flows Or Greater
C) System Operating At Maximum Capacity.
Source:
Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas, February, 1984.
STORM SEWER-CHANNEL INTERACTION FOR FORT BEND COUNTY, TEXAS
August 1986
FIGURE 5-1


CURVE NO. 1
CURVE NO. 2
CURVE NO. 3
CURVE NO. 4
CURVE NO. 5
CURVE NO. 6
CI = Rate of run-off per hour in inches
Time of concentration in minutes
Curve No. 1 — Highly developed business areas (Downtown Shopping Centers, Etc.)
Curve No. 2 — Multi-family residential areas (Apartments, Townhouses, Etc.)
Curve No. 3 — Composite areas (Single & Multi-family Residential, Commercial, Etc.), business and industrial parks.
Curve No. 4 — Single family residential areas
Curve No. 5 — Railroad yards, suburban residential areas (Minimum 1 Acre Lots)
Curve No. 6 — Developed park areas
Source: Public Works Department, City of Houston, Texas
RAINFALL RUNOFF CURVES
FOR
FORT BEND COUNTY
JULY 1987
FIGURE 5-2



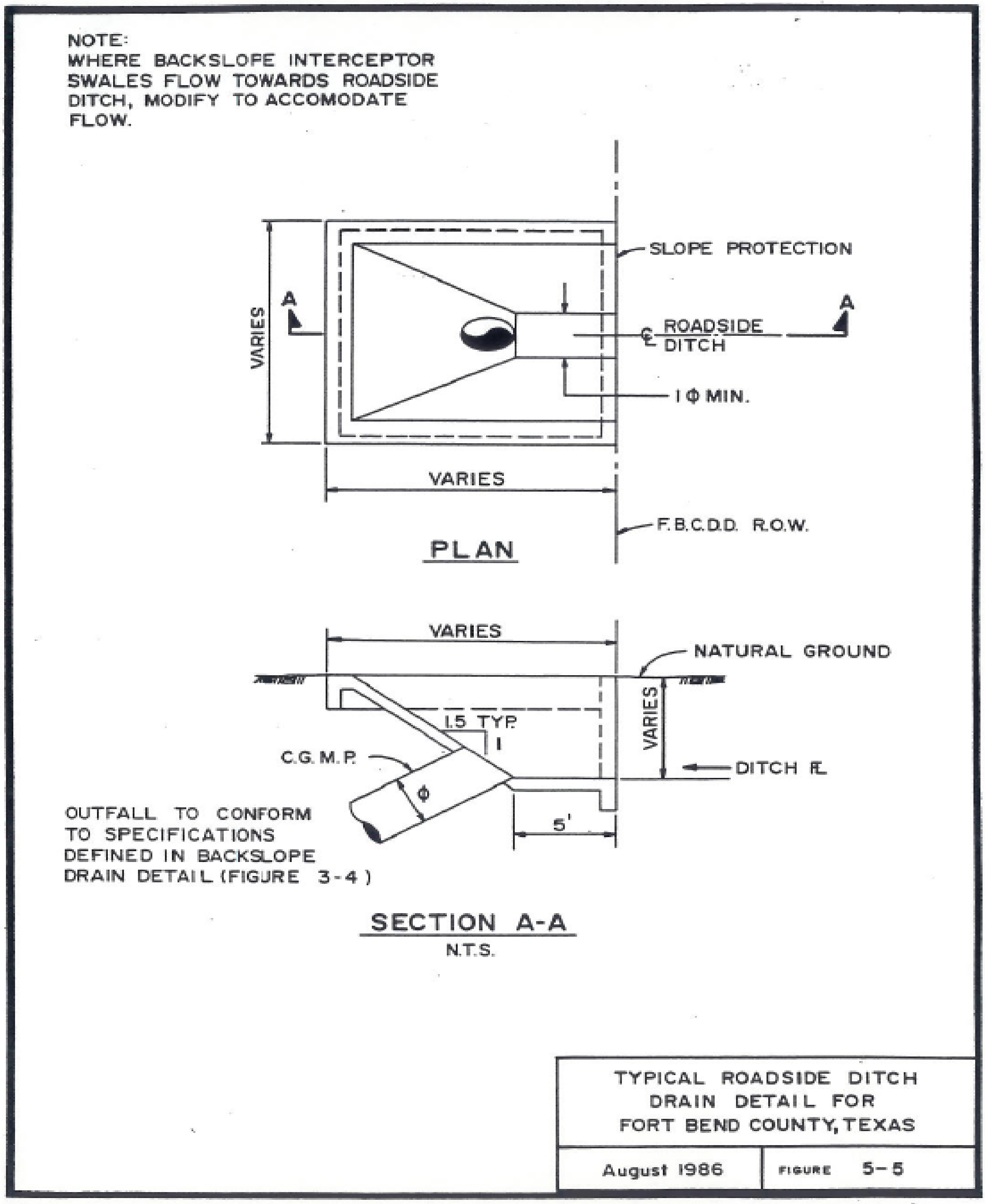
Source: Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas, Feb. 1984.

UNDESIRABLE SHEET FLOW PATTERNS FOR FORT BEND COUNTY, TEXAS	
August 1986	FIGURE 5-3



Source: Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas, Feb. 1984.

ACCEPTABLE SHEET FLOW PATTERNS FOR FORT BEND COUNTY, TEXAS	
August, 1986	FIGURE 5-4


NOTE:
WHERE BACKSLOPE INTERCEPTOR SWALES FLOW TOWARDS ROADSIDE DITCH, MODIFY TO ACCOMODATE FLOW.
SLOPE PROTECTION
A
A
VARIES
℄ ROADSIDE DITCH
1 Φ MIN.
VARIES
F.B.C.D.D. R.O.W.
PLAN
VARIES
NATURAL GROUND
1.5 TYP.
1
VARIES
C.G.M.P.
DITCH FL
Φ
5'
OUTFALL TO CONFORM TO SPECIFICATIONS DEFINED IN BACKSLOPE DRAIN DETAIL (FIGURE 3-4)
SECTION A-A
N.T.S.
TYPICAL ROADSIDE DITCH DRAIN DETAIL FOR FORT BEND COUNTY, TEXAS
August 1986
FIGURE 5-5

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6.0 STORM RUNOFF STORAGE

6.1 GENERAL

In an area such as Fort Bend County, which is generally characterized by flat terrain, the introduction of impervious cover and improved runoff conveyance serves in many cases to increase flood peaks quite dramatically over those for existing conditions. Increases in flows over the existing condition flows off of the site or development to receiving waterways, channels or roadside drainage systems are not allowed unless appropriate mitigation is supplied nearby and that applicable supporting analysis is supplied and agreed upon by the FBCDD Engineer. When physical, topographic, and economic conditions allow it, channel improvements downstream of the development are often used to prevent increased flooding. When this is not feasible, a widely used practice is runoff detention or retention storage, wherein the storm volume is held back in the watershed and released at an acceptable rate. This section of the manual presents information on storage techniques, including guidance for the design of appropriate storm runoff storage facilities. See also Section 8- Drainage Design Criteria for Rural Subdivisions.

6.2 MASTER PLANS

Development in a watershed can have complex and far-reaching consequences on the overall hydrologic regime. For this reason, careful plans for anticipating and meeting the long-term flood control and drainage needs of Fort Bend County have been drawn up on a watershed by watershed basis, although not all watersheds have master plans. Each watershed "master plan" has been formulated to provide the most practical and efficient basin-wide approach to the hydrologic consequences of ongoing or future development, including proper coordination of storm detention facilities and channel improvements. Accordingly, the Fort Bend County Drainage District Engineer must be consulted concerning the status of a particular master plan and the preferred watershed flood control strategies and alternatives. In addition, the models used to develop a master plan must be used in the design or analysis of new projects within the watershed. The models can be obtained from the Fort Bend County Drainage District.

If a master plan is not available for the watershed in which you are trying to develop or design a project you may be able to develop localized master planning information related to your location within the watershed. That planning should include close coordination with the Fort

Bend County Drainage District Engineer. The master plan shall provide for ultimate development upstream including full conveyance flow, to facilitate potential future channel improvements along the channel. The plan shall provide for adequate channel sizing and maintenance berms for ultimate development assuming storm sewer outfall depth requirements. This planning is needed to ensure that adequate right of way, channel sizing and channel depth is provided for each project.

6.3 STORAGE CLASSIFICATION

Storage systems may be classified as either on-line or off-line facilities. They may be designed for either detention or retention of storm water.

6.3.1 Retention Storage

In a retention storage facility, runoff is captured and released only after the storm event is over and the downstream water surface has subsided. A retention storage system is seldom used and when it is special outlet devices or pumps are usually required.

6.3.2 Detention Storage

The vast majority of flood control storage is handled by detention facilities. The purpose of detention storage is to hold storm runoff back but release it continuously at an acceptable rate through a flow-limiting outlet structure, thus controlling downstream peak flows.

6.3.3 On-line Storage

An on-line storage facility is one in which the total storm runoff volume passes through the retention or detention facility's outflow structure.

6.3.4 Off-line Storage

An off-line storage design is one in which storm runoff does not begin to flow into the storage facility until the discharge in the channel reaches some critical value above which unacceptable downstream flooding will occur. An off-line facility serves to store only the runoff volume associated with the high flow rate portions of the flood event.

6.4 DESIGN PROCEDURES

The following design procedures are intended to insure that new development with detention will not cause any adverse impacts on existing flooding conditions downstream. (Note: The design engineers should contact the Fort Bend County Drainage District Engineer for any specific requirements for the watershed in which the proposed facility is to be located.)

Development drainage reports shall include summary charts that detail the characteristics of the storage facility and show no increase in peak flow rates and/or water surface elevations

6.4.1 For Drainage Areas <50 Acres

The maximum allowable release rate from the detention facility during the 100-year storm event is 0.125 cfs/acre.

The acre-feet of flood control storage, S, to be provided by the facility for the 100-year storm event is shown in Figure 6-1 and Table 6-1 below. The percentage of impervious area used for the storage calculation shall include all areas that are paved or where gravel or crushed stone is used, all rooftops and other covered areas, and all other impervious surfaces, including the portion of the detention pond below the 100-year design water surface elevation.

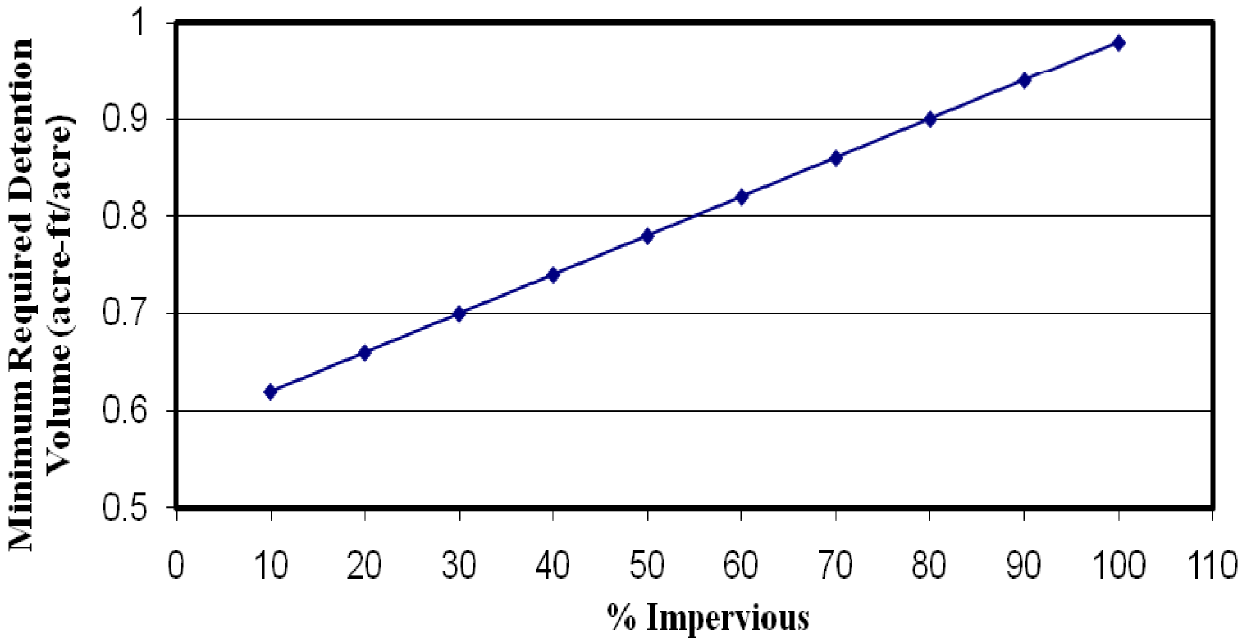

Figure 6-1 Minimum Required Detention Volume for Drainage Areas Less Than 50 Acres

TABLE 6-1

MINIMUM DETENTION RATES FOR

DRAINAGE AREAS LESS THAN 50 ACRES

% Impervious	Minimum Detention Rate (Acre-feet/Acre)
10 %	0.62
20%	0.66
30%	0.70
40%	0.74
50%	0.78
60%	0.82
70%	0.86
80%	0.90
90%	0.94
100%	0.98

The size of the outlet pipe that is required to pass the maximum allowable release rate during the 100-year storm is to be computed assuming outlet control (See Section 4.3.6), by establishing a maximum ponding level in the detention facility during the 100-year storm and assuming a tailwater at the top of the downstream end of the outlet pipe or at a depth in the outlet channel associated with the maximum release flowrate, whichever is higher.

6.4.2 For Drainage Areas ≥50 Acres and <640 Acres

The design engineer has the option to follow the simplified procedure previously described for areas smaller than 50 acres, or the more detailed analysis outlined below for areas larger than 640 acres.

6.4.3 For Drainage Areas ≥640 Acres

The HEC-HMS computer model will be used to size the facility and the outlet structure so that downstream flooding conditions will not be increased. Inflow and outflow hydrographs for detention analysis may be established using other methods approved by the Fort Bend County Drainage District. The existing conditions HEC-HMS model should be established in conjunction with the Fort Bend County Drainage District Engineer. Once existing conditions are established, the new development with the detention facility will be analyzed for the 10-, 25-, and 100-year storm events (and smaller events if the downstream channel has less than 10-year capacity).

The detention facility should be sized such that there is no increase in flow rate and water surface elevation at any point along the channel using HEC-HMS and HEC-RAS. Additional models may be considered, however they should be presented to the District for approval prior to starting.

The maximum allowable outflow rate should be determined from the procedure explained under release rates and maximum allowable discharge.

6.4.4 Design Tailwater Depth

In order to route the inflow hydrograph through the detention facility in the hydrologic model, a relationship must be established between the volume of storage in the pond and the corresponding amount of discharge through the outflow structure. In most cases in Fort Bend County this relationship is directly dependent on the elevation of the tailwater at the outlet of the outflow structure.

For the purpose of establishing an outflow rating curve, detention facilities that are evaluated using computer models shall use a variable tail-water condition based on the frequency storm being analyzed. The variable tail-water stage hydrograph can be developed using the rating curve and the flow hydrograph at the tail-water location. In certain situations where this assumption may be shown not to be reasonable, an alternative tail-water condition can be presented for approval to the Fort Bend County Drainage District.

6.4.5 Release Rates and Maximum Allowable Discharge

For drainage area less than 50 acres, the maximum release rate shall be 0.125 cfs and should be designed assuming a tailwater elevation at the top of the downstream end of the outfall pipe. When outfalling into a roadside ditch, the release rate shall be limited to the proposed developments pro rata share of the bank full capacity of the receiving ditch, considering the ditch at bank-full for the design tailwater condition. Supporting documentation should be submitted that demonstrates the calculations used to determine this share.

If using computer modeling, use variable tail-water conditions and existing conditions flows as the allowable release rate.

6.4.6 Downstream Impact Analysis Requirements

Analyze using HEC-HMS and HEC-RAS through the entire downstream channel section for the 10-, 25- and 100-year events and show no increase in flow rates and/or water surface elevations. If the outfall channel has less than 10-year storm capacity, the analysis must also include the 2-year event.

6.4.7 Final Sizing of Pond Storage and Outflow Structure

Detention or retention facilities shall be sized such that at least one foot of freeboard shall be maintained during the 100-year storm event, as measured from the minimum elevation of the top of the detention or retention facility berm to the maximum 100-year storm water surface elevation.

Detention basins and storm sewer outfalls shall be placed one foot above the flow-line of the receiving channels, creeks and detention pond. The minimum recommended outflow pipe for a detention facility is 24 inches. An 18-inch outflow pipe can be used when outfalling into a roadside ditch. The roadside ditch outfall must have the end of pipe cut to match the roadside ditch side slope and one foot of stabilized sand around the pipe. When further flow restriction is necessary, the restriction should be located at a manhole outside of the Fort Bend County channel right-of-way.

All detention facilities shall be adequately maintained in accordance with the original design so that the basin storage and outfall operate properly. The owner of the basin is responsible for maintaining the basin to the satisfaction of the Fort Bend County Drainage District Engineer.

6.4.8 Storm Sewer Hydraulic Gradients

The hydraulic gradients in storm sewers shall be determined using procedures outlined in Section 5 of this manual. The starting water surface elevation for these calculations shall be the 25-year maximum pond elevation.

If the simplified procedure was used to design the detention facility, the 25-year ponding level can be estimated as being 80% of the depth of the 100-year ponding level.

6.4.9 Allowances for Extreme Storm Events

Design consideration must be given to storm events in excess of the 100-year flood. An emergency spillway, overflow structure, or swale must be provided as necessary to effectively handle the extreme storm event. See Section 5 of this manual for additional criteria for extreme event swale design and sizing.

In places where a control structure is to be utilized to provide detention directly in the channel, due consideration must be given to the consequences of a failure, and if a significant hazard exists, the control structure must be adequately designed to prevent such hazards.

In addition, detention facilities which measure greater than six feet in height are subject to Title 31 Texas Administrative Code (TAC) Chapter 299 (Subchapters A through E), effective May 13, 1986, and all subsequent changes. The height of a control structure, detention facility or dam is defined as the distance from the lowest point on the crest of the dam (or embankment), excluding spillways, to the lowest elevation on the centerline or downstream toe of the dam (or embankment) including the natural stream channel. Subchapters A through E of Chapter 299 classifies dam sizes and hazard potential and specify required failure analyses and spillway design flood criteria. Appendix B includes a copy of these sections of the TAC.

6.4.10 Erosion Controls

The erosional tendencies associated with a detention pond are similar to those found in an open channel. For this reason the same type of erosion protection are necessary, including the use of backslope swales and drainage systems (as outlined in Section 3), proper re-vegetation, and pond surface lining where necessary. Proper protection must especially be provided at pipe outfalls or junctions into the facility, pond outlet structures and overflow spillways where excessive turbulence and velocities will cause erosion.

The erosion protection could include concrete slope paving, adequately designed erosion control blocks or paving sections. Should erosion be observed, it will be the requirement of the owner of the facility to make appropriate repairs and or corrections to the design or construction to fix any erosion problems.

6.5 MULTIPURPOSE LAND USE

The amount of land required for a storm water detention facility is generally quite substantial. For this reason, it is logical that storage facilities could serve a secondary role as parks or recreational areas whenever possible. Such dual use areas will be allowed only after proper review of the design scenario and approval of the specific project by the Fort Bend County Drainage District Engineer.

A parking lot may be used as part of the detention system, provided that the maximum depth of water over the inlet does not exceed nine (9”) inches and the maximum depth in the parking stall does not exceed six (6”) inches.

When a dual use facility is proposed, a joint use agreement is required between the entity using the facility for detention, and the entity sponsoring the secondary use. This agreement must specify the maintenance responsibilities of each party.

Highly urbanized areas which do not have the option of conventional detention ponds due to available land may store storm water underground on the site, pending Fort Bend County Drainage District approval.

If wet bottom features are planned for a detention facility adequate design considerations shall be provided and included in the design and construction to make the facility:

1. Safe to the public

2. Accessible for maintenance

3. Easy to maintain

4. Provide a minimum depth of 6 feet.

6.5.1 Approval of Private, Dual-Use or Multi-Use Facilities

For privately maintained, dual-use or multi-use each storm water detention facility will be reviewed and approved only if:

1. The facility has been designed to meet or exceed the requirements contained within this manual; and

2. Provisions are made for the facility to be adequately maintained.

3. If walking paths, jogging trails or other amenity is anticipated sufficient details of the paths, jogging trails, amenity and designs for each of these shall be provided to Fort Bend County Drainage District Engineer for review and comment. The trail or path geometry and location may require special requirements, thicker base or top surface to provide access for maintenance vehicles to cross the facility. Any impact or damage to the trail or path from Fort Bend County Drainage District vehicles will not be the responsibility of the Fort Bend County Drainage District.

6.5.2 Maintenance

Each development which provides detention shall make provisions to ensure future maintenance of the detention facility. Typically, a property owners association, LID, WCID or MUD will be established and given the responsibility to maintain the drainage facility. The entity responsible for the maintenance of the facility shall be noted on the plat or plans.

A 30-foot wide access and maintenance easement shall be provided from street, road or adequate access way to and around any drainage ditch, channel or the entire detention pond. This is in addition to the dedication required for the pond itself. Figure 6-2 below shows the minimum criteria for maintenance berms in different development scenarios.

If guard rails or other impediments will block access to drainage ditches or detention facilities, adequate provisions shall be provided to allow reasonable access to the channel or drainage facility as approved by FBCDD staff.

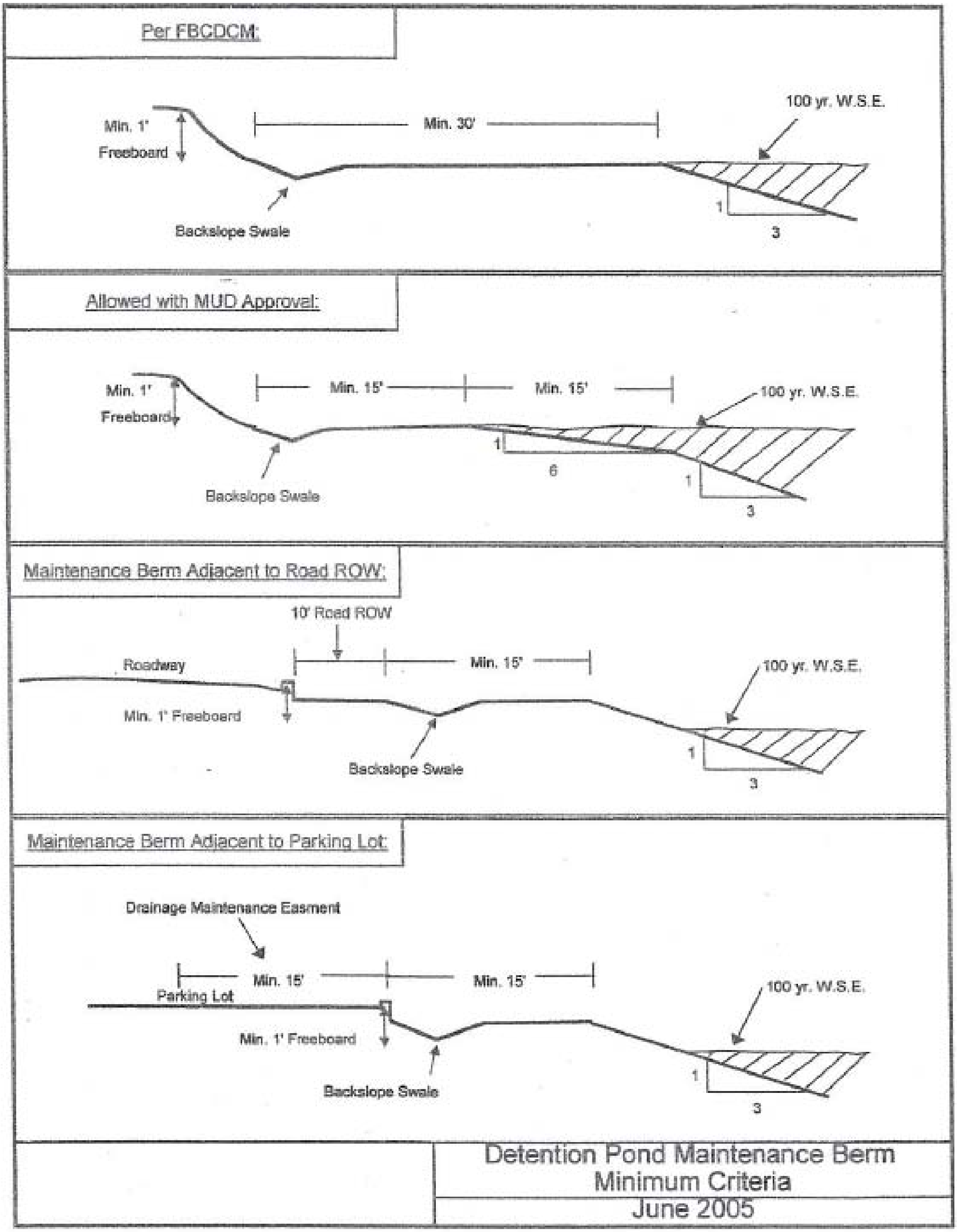

Figure 6-2 – Detention Basin Maintenance Berm Minimum Criteria

6.6 PUMP DETENTION

Pumped detention systems will not be maintained by Fort Bend County under any circumstances and will be approved for use only under the following minimum conditions:

1. A gravity system is not feasible from an engineering and economic standpoint;

2. At least two pumps are provided, each of which is sized to pump the design flow rate; if a triplex system is used, any two of the three pumps must be capable of pumping the design flow rate;

3. The selected design outflow rate must not aggravate downstream flooding. (Example: A pump system designed to discharge at the existing 100-year flow rate each time the system comes on-line could aggravate flooding for more frequent storm events.).

4. Fencing of the control panel is provided to prevent unauthorized operation and vandalism;

5. Adequate assurance is provided that the system will be operated and maintained on a continuous basis;

6. Emergency source of power is provided.

It is recommended that if a pump system is desired, review of the preliminary conceptual design by the Fort Bend County Drainage District Engineer be obtained before any detailed engineering is performed.

6.7 GEOTECHNICAL INVESTIGATION

Before initiating final design of a detention pond, a detailed soils investigation by a geotechnical engineer should be undertaken. The following minimum requirements shall be addressed:

1. The ground water conditions at the proposed site;

2. The type of material to be excavated from the pond site and its suitability for additional use;

3. If a dam is to be constructed, adequate investigation of potential seepage problems through the dam and attendant control requirements, the availability of suitable embankment material and the stability requirements for the dam itself;

4. Potential for structural movement or areas adjacent to the pond due to the induced loads from existing or proposed structures and methods of control that may be required;

5. Stability of the pond side slopes for short term and long term conditions.

6.8 GENERAL REQUIREMENTS FOR DETENTION POND CONSTRUCTION

The structural design of detention facilities is very similar to the design of open channels. For this reason, all requirements from Section 3.0 pertaining to the design of lined or unlined channels shall also apply to lined or unlined detention facilities.

In addition, the following guidelines are applicable:

1. Pond Bottom Design – A pilot channel shall be provided in detention facilities to insure that proper and complete drainage of the storage facility will occur. Concrete pilot channels shall have a minimum depth of two inches and a minimum flowline slope of .0005 ft/ft. Unlined pilot channels shall have a

minimum depth of two feet, a minimum flowline slope of .001 ft/ft, and maximum sideslopes of 3:1.

The bottom slopes of the detention basin should be graded toward the pilot channel at a minimum slope of 0.005 ft/ft, and a recommended slope of 0.0075 ft/ft.

Detention basins which make use of a channel section for detention storage may not be required to have a pilot channel, but should be built in accordance with the requirements for open channels as outlined in Section 3.0.

2. Outlet Structure – The outlet structure for a detention pond is subject to higher than normal head water conditions and erosive velocities for prolonged periods of time. For this reason the erosion protective measures are very important.

   Reinforced concrete pipe used in the outlet structure should conform to ASTM C-76 Class III with compression type rubber gasket joints conforming to ASTM C-443. Pipes, culverts and conduits used in the outlet structures should be carefully constructed with sufficient compaction of the backfill material around the pipe structure as recommended in the geotechnical analysis. Generally, compaction density should be the same as the rest of the structure. The use of pressure grouting around the outlet conduit should be considered where soil types or conditions may prevent satisfactory backfill compaction. Pressure grouting should also be used where headwater depths could cause backfill to wash out around the pipe.

6.9 STORM WATER QUALITY BMPs AND PHASE II NPDES PERMIT

Fort Bend County encourages the use of storm water quality (SWQ) best management practices (BMPs) such as floatable collection screens, wet bottom features in detention basins and other practices. Water quality features must not interfere with the function, operation, maintenance, or rehabilitation of the detention basin and must comply with all applicable criteria.

6.10 LOW IMPACT DEVELOPMENT

LID is the site design strategy with a goal of maintaining or replicating the pre-development hydrologic regime through the use of design techniques to create a functionally equivalent hydrologic landscape. LIDs are based on controlling storm water at the source by the use of micro-scale controls that are distributed throughout the site. These multifunctional site designs incorporate alternative storm water management practices such as functional landscape that act as storm water facilities, depression storage and open drainage swales. Fort Bend County encourages using the LID features in the watershed. These features should be discussed with the Fort Bend County Drainage District Engineer prior to the design process to ensure that the proposed features are acceptable to the Fort Bend County Drainage District.

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6.0 STORM RUNOFF STORAGE

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6.2 MASTER PLANS

Development in a watershed can have complex and far-reaching consequences on the overall hydrologic regime. For this reason, careful plans for anticipating and meeting the long-term flood control and drainage needs of Fort Bend County have been drawn up on a watershed by watershed basis, although not all watersheds have master plans. Each watershed "master plan" has been formulated to provide the most practical and efficient basin-wide approach to the hydrologic consequences of ongoing or future development, including proper coordination of storm detention facilities and channel improvements. Accordingly, the Fort Bend County Drainage District Engineer must be consulted concerning the status of a particular master plan and the preferred watershed flood control strategies and alternatives. In addition, the models used to develop a master plan must be used in the design or analysis of new projects within the watershed. The models can be obtained from the Fort Bend County Drainage District.

If a master plan is not available for the watershed in which you are trying to develop or design a project you may be able to develop localized master planning information related to your location within the watershed. That planning should include close coordination with the Fort

Bend County Drainage District Engineer. The master plan shall provide for ultimate development upstream including full conveyance flow, to facilitate potential future channel improvements along the channel. The plan shall provide for adequate channel sizing and maintenance berms for ultimate development assuming storm sewer outfall depth requirements. This planning is needed to ensure that adequate right of way, channel sizing and channel depth is provided for each project.

6.3 STORAGE CLASSIFICATION

Storage systems may be classified as either on-line or off-line facilities. They may be designed for either detention or retention of storm water.

6.3.1 Retention Storage

In a retention storage facility, runoff is captured and released only after the storm event is over and the downstream water surface has subsided. A retention storage system is seldom used and when it is special outlet devices or pumps are usually required.

6.3.2 Detention Storage

The vast majority of flood control storage is handled by detention facilities. The purpose of detention storage is to hold storm runoff back but release it continuously at an acceptable rate through a flow-limiting outlet structure, thus controlling downstream peak flows.

6.3.3 On-line Storage

An on-line storage facility is one in which the total storm runoff volume passes through the retention or detention facility's outflow structure.

6.3.4 Off-line Storage

An off-line storage design is one in which storm runoff does not begin to flow into the storage facility until the discharge in the channel reaches some critical value above which unacceptable downstream flooding will occur. An off-line facility serves to store only the runoff volume associated with the high flow rate portions of the flood event.

6.4 DESIGN PROCEDURES

The following design procedures are intended to insure that new development with detention will not cause any adverse impacts on existing flooding conditions downstream. (Note: The design engineers should contact the Fort Bend County Drainage District Engineer for any specific requirements for the watershed in which the proposed facility is to be located.)

Development drainage reports shall include summary charts that detail the characteristics of the storage facility and show no increase in peak flow rates and/or water surface elevations

6.4.1 For Drainage Areas <50 Acres

The maximum allowable release rate from the detention facility during the 100-year storm event is 0.125 cfs/acre.

The acre-feet of flood control storage, S, to be provided by the facility for the 100-year storm event is shown in Figure 6-1 and Table 6-1 below. The percentage of impervious area used for the storage calculation shall include all areas that are paved or where gravel or crushed stone is used, all rooftops and other covered areas, and all other impervious surfaces, including the portion of the detention pond below the 100-year design water surface elevation.

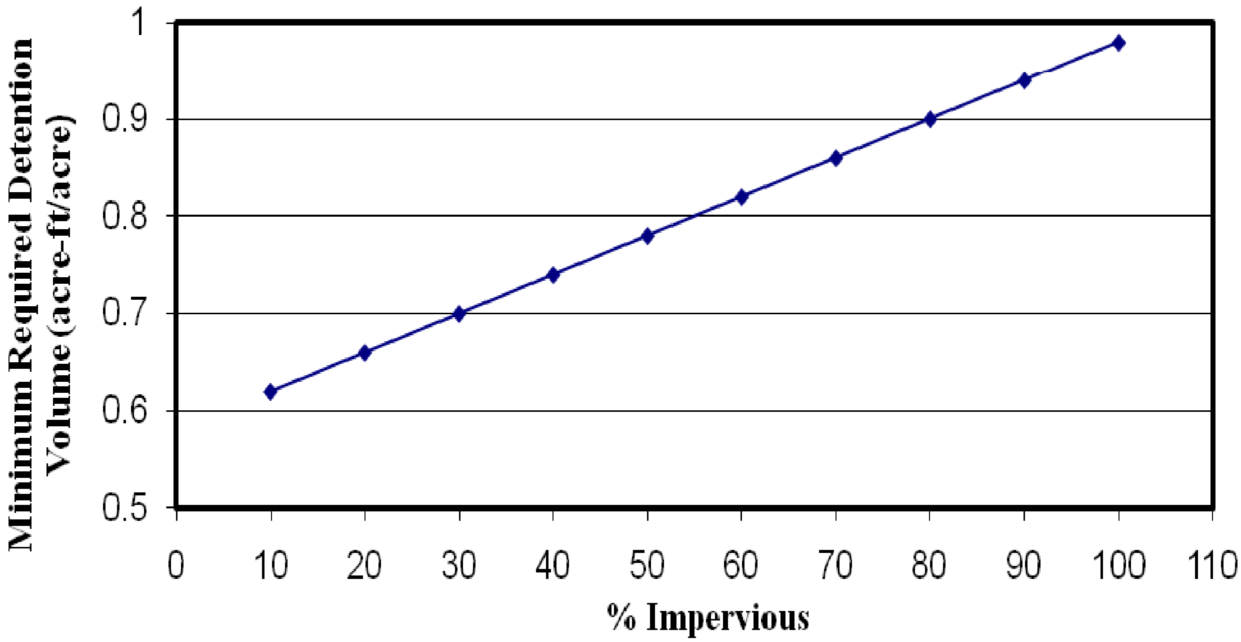

Figure 6-1 Minimum Required Detention Volume for Drainage Areas Less Than 50 Acres

TABLE 6-1

MINIMUM DETENTION RATES FOR

DRAINAGE AREAS LESS THAN 50 ACRES

% Impervious	Minimum Detention Rate (Acre-feet/Acre)
10 %	0.62
20%	0.66
30%	0.70
40%	0.74
50%	0.78
60%	0.82
70%	0.86
80%	0.90
90%	0.94
100%	0.98

The size of the outlet pipe that is required to pass the maximum allowable release rate during the 100-year storm is to be computed assuming outlet control (See Section 4.3.6), by establishing a maximum ponding level in the detention facility during the 100-year storm and assuming a tailwater at the top of the downstream end of the outlet pipe or at a depth in the outlet channel associated with the maximum release flowrate, whichever is higher.

6.4.2 For Drainage Areas ≥50 Acres and <640 Acres

The design engineer has the option to follow the simplified procedure previously described for areas smaller than 50 acres, or the more detailed analysis outlined below for areas larger than 640 acres.

6.4.3 For Drainage Areas ≥640 Acres

The HEC-HMS computer model will be used to size the facility and the outlet structure so that downstream flooding conditions will not be increased. Inflow and outflow hydrographs for detention analysis may be established using other methods approved by the Fort Bend County Drainage District. The existing conditions HEC-HMS model should be established in conjunction with the Fort Bend County Drainage District Engineer. Once existing conditions are established, the new development with the detention facility will be analyzed for the 10-, 25-, and 100-year storm events (and smaller events if the downstream channel has less than 10-year capacity).

The detention facility should be sized such that there is no increase in flow rate and water surface elevation at any point along the channel using HEC-HMS and HEC-RAS. Additional models may be considered, however they should be presented to the District for approval prior to starting.

The maximum allowable outflow rate should be determined from the procedure explained under release rates and maximum allowable discharge.

6.4.4 Design Tailwater Depth

In order to route the inflow hydrograph through the detention facility in the hydrologic model, a relationship must be established between the volume of storage in the pond and the corresponding amount of discharge through the outflow structure. In most cases in Fort Bend County this relationship is directly dependent on the elevation of the tailwater at the outlet of the outflow structure.

For the purpose of establishing an outflow rating curve, detention facilities that are evaluated using computer models shall use a variable tail-water condition based on the frequency storm being analyzed. The variable tail-water stage hydrograph can be developed using the rating curve and the flow hydrograph at the tail-water location. In certain situations where this assumption may be shown not to be reasonable, an alternative tail-water condition can be presented for approval to the Fort Bend County Drainage District.

6.4.5 Release Rates and Maximum Allowable Discharge

For drainage area less than 50 acres, the maximum release rate shall be 0.125 cfs and should be designed assuming a tailwater elevation at the top of the downstream end of the outfall pipe. When outfalling into a roadside ditch, the release rate shall be limited to the proposed developments pro rata share of the bank full capacity of the receiving ditch, considering the ditch at bank-full for the design tailwater condition. Supporting documentation should be submitted that demonstrates the calculations used to determine this share.

If using computer modeling, use variable tail-water conditions and existing conditions flows as the allowable release rate.

6.4.6 Downstream Impact Analysis Requirements

Analyze using HEC-HMS and HEC-RAS through the entire downstream channel section for the 10-, 25- and 100-year events and show no increase in flow rates and/or water surface elevations. If the outfall channel has less than 10-year storm capacity, the analysis must also include the 2-year event.

6.4.7 Final Sizing of Pond Storage and Outflow Structure

Detention or retention facilities shall be sized such that at least one foot of freeboard shall be maintained during the 100-year storm event, as measured from the minimum elevation of the top of the detention or retention facility berm to the maximum 100-year storm water surface elevation.

Detention basins and storm sewer outfalls shall be placed one foot above the flow-line of the receiving channels, creeks and detention pond. The minimum recommended outflow pipe for a detention facility is 24 inches. An 18-inch outflow pipe can be used when outfalling into a roadside ditch. The roadside ditch outfall must have the end of pipe cut to match the roadside ditch side slope and one foot of stabilized sand around the pipe. When further flow restriction is necessary, the restriction should be located at a manhole outside of the Fort Bend County channel right-of-way.

All detention facilities shall be adequately maintained in accordance with the original design so that the basin storage and outfall operate properly. The owner of the basin is responsible for maintaining the basin to the satisfaction of the Fort Bend County Drainage District Engineer.

6.4.8 Storm Sewer Hydraulic Gradients

The hydraulic gradients in storm sewers shall be determined using procedures outlined in Section 5 of this manual. The starting water surface elevation for these calculations shall be the 25-year maximum pond elevation.

If the simplified procedure was used to design the detention facility, the 25-year ponding level can be estimated as being 80% of the depth of the 100-year ponding level.

6.4.9 Allowances for Extreme Storm Events

Design consideration must be given to storm events in excess of the 100-year flood. An emergency spillway, overflow structure, or swale must be provided as necessary to effectively handle the extreme storm event. See Section 5 of this manual for additional criteria for extreme event swale design and sizing.

In places where a control structure is to be utilized to provide detention directly in the channel, due consideration must be given to the consequences of a failure, and if a significant hazard exists, the control structure must be adequately designed to prevent such hazards.

In addition, detention facilities which measure greater than six feet in height are subject to Title 31 Texas Administrative Code (TAC) Chapter 299 (Subchapters A through E), effective May 13, 1986, and all subsequent changes. The height of a control structure, detention facility or dam is defined as the distance from the lowest point on the crest of the dam (or embankment), excluding spillways, to the lowest elevation on the centerline or downstream toe of the dam (or embankment) including the natural stream channel. Subchapters A through E of Chapter 299 classifies dam sizes and hazard potential and specify required failure analyses and spillway design flood criteria. Appendix B includes a copy of these sections of the TAC.

6.4.10 Erosion Controls

The erosional tendencies associated with a detention pond are similar to those found in an open channel. For this reason the same type of erosion protection are necessary, including the use of backslope swales and drainage systems (as outlined in Section 3), proper re-vegetation, and pond surface lining where necessary. Proper protection must especially be provided at pipe outfalls or junctions into the facility, pond outlet structures and overflow spillways where excessive turbulence and velocities will cause erosion.

The erosion protection could include concrete slope paving, adequately designed erosion control blocks or paving sections. Should erosion be observed, it will be the requirement of the owner of the facility to make appropriate repairs and or corrections to the design or construction to fix any erosion problems.

6.5 MULTIPURPOSE LAND USE

The amount of land required for a storm water detention facility is generally quite substantial. For this reason, it is logical that storage facilities could serve a secondary role as parks or recreational areas whenever possible. Such dual use areas will be allowed only after proper review of the design scenario and approval of the specific project by the Fort Bend County Drainage District Engineer.

A parking lot may be used as part of the detention system, provided that the maximum depth of water over the inlet does not exceed nine (9”) inches and the maximum depth in the parking stall does not exceed six (6”) inches.

When a dual use facility is proposed, a joint use agreement is required between the entity using the facility for detention, and the entity sponsoring the secondary use. This agreement must specify the maintenance responsibilities of each party.

Highly urbanized areas which do not have the option of conventional detention ponds due to available land may store storm water underground on the site, pending Fort Bend County Drainage District approval.

If wet bottom features are planned for a detention facility adequate design considerations shall be provided and included in the design and construction to make the facility:

1. Safe to the public
2. Accessible for maintenance
3. Easy to maintain
4. Provide a minimum depth of 6 feet.

6.5.1 Approval of Private, Dual-Use or Multi-Use Facilities

For privately maintained, dual-use or multi-use each storm water detention facility will be reviewed and approved only if:

1. The facility has been designed to meet or exceed the requirements contained within this manual; and

2. Provisions are made for the facility to be adequately maintained.

3. If walking paths, jogging trails or other amenity is anticipated sufficient details of the paths, jogging trails, amenity and designs for each of these shall be provided to Fort Bend County Drainage District Engineer for review and comment. The trail or path geometry and location may require special requirements, thicker base or top surface to provide access for maintenance vehicles to cross the facility. Any impact or damage to the trail or path from Fort Bend County Drainage District vehicles will not be the responsibility of the Fort Bend County Drainage District.

6.5.2 Maintenance

Each development which provides detention shall make provisions to ensure future maintenance of the detention facility. Typically, a property owners association, LID, WCID or MUD will be established and given the responsibility to maintain the drainage facility. The entity responsible for the maintenance of the facility shall be noted on the plat or plans.

A 30-foot wide access and maintenance easement shall be provided from street, road or adequate access way to and around any drainage ditch, channel or the entire detention pond. This is in addition to the dedication required for the pond itself. Figure 6-2 below shows the minimum criteria for maintenance berms in different development scenarios.

If guard rails or other impediments will block access to drainage ditches or detention facilities, adequate provisions shall be provided to allow reasonable access to the channel or drainage facility as approved by FBCDD staff.

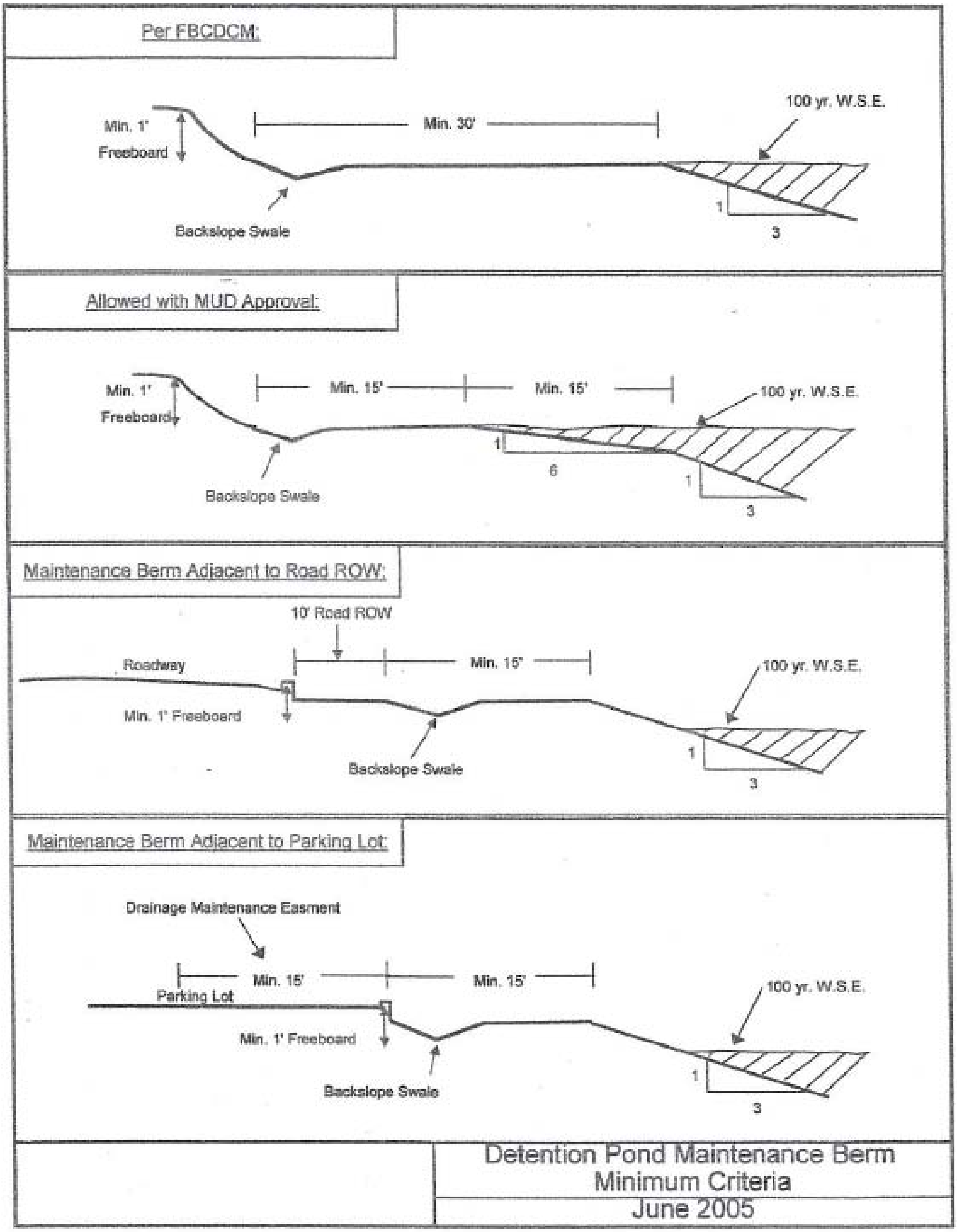

Figure 6-2 – Detention Basin Maintenance Berm Minimum Criteria

6.6 PUMP DETENTION

Pumped detention systems will not be maintained by Fort Bend County under any circumstances and will be approved for use only under the following minimum conditions:

1. A gravity system is not feasible from an engineering and economic standpoint;

2. At least two pumps are provided, each of which is sized to pump the design flow rate; if a triplex system is used, any two of the three pumps must be capable of pumping the design flow rate;

3. The selected design outflow rate must not aggravate downstream flooding. (Example: A pump system designed to discharge at the existing 100-year flow rate each time the system comes on-line could aggravate flooding for more frequent storm events.).

4. Fencing of the control panel is provided to prevent unauthorized operation and vandalism;

5. Adequate assurance is provided that the system will be operated and maintained on a continuous basis;

6. Emergency source of power is provided.

It is recommended that if a pump system is desired, review of the preliminary conceptual design by the Fort Bend County Drainage District Engineer be obtained before any detailed engineering is performed.

6.7 GEOTECHNICAL INVESTIGATION

Before initiating final design of a detention pond, a detailed soils investigation by a geotechnical engineer should be undertaken. The following minimum requirements shall be addressed:

1. The ground water conditions at the proposed site;

2. The type of material to be excavated from the pond site and its suitability for additional use;

3. If a dam is to be constructed, adequate investigation of potential seepage problems through the dam and attendant control requirements, the availability of suitable embankment material and the stability requirements for the dam itself;

4. Potential for structural movement or areas adjacent to the pond due to the induced loads from existing or proposed structures and methods of control that may be required;

5. Stability of the pond side slopes for short term and long term conditions.

6.8 GENERAL REQUIREMENTS FOR DETENTION POND CONSTRUCTION

The structural design of detention facilities is very similar to the design of open channels. For this reason, all requirements from Section 3.0 pertaining to the design of lined or unlined channels shall also apply to lined or unlined detention facilities.

In addition, the following guidelines are applicable:

1. Pond Bottom Design – A pilot channel shall be provided in detention facilities to insure that proper and complete drainage of the storage facility will occur. Concrete pilot channels shall have a minimum depth of two inches and a minimum flowline slope of .0005 ft/ft. Unlined pilot channels shall have a

minimum depth of two feet, a minimum flowline slope of .001 ft/ft, and maximum sideslopes of 3:1.

The bottom slopes of the detention basin should be graded toward the pilot channel at a minimum slope of 0.005 ft/ft, and a recommended slope of 0.0075 ft/ft.

Detention basins which make use of a channel section for detention storage may not be required to have a pilot channel, but should be built in accordance with the requirements for open channels as outlined in Section 3.0.

2. Outlet Structure – The outlet structure for a detention pond is subject to higher than normal head water conditions and erosive velocities for prolonged periods of time. For this reason the erosion protective measures are very important.

   Reinforced concrete pipe used in the outlet structure should conform to ASTM C-76 Class III with compression type rubber gasket joints conforming to ASTM C-443. Pipes, culverts and conduits used in the outlet structures should be carefully constructed with sufficient compaction of the backfill material around the pipe structure as recommended in the geotechnical analysis. Generally, compaction density should be the same as the rest of the structure. The use of pressure grouting around the outlet conduit should be considered where soil types or conditions may prevent satisfactory backfill compaction. Pressure grouting should also be used where headwater depths could cause backfill to wash out around the pipe.

6.9 STORM WATER QUALITY BMPs AND PHASE II NPDES PERMIT

Fort Bend County encourages the use of storm water quality (SWQ) best management practices (BMPs) such as floatable collection screens, wet bottom features in detention basins and other practices. Water quality features must not interfere with the function, operation, maintenance, or rehabilitation of the detention basin and must comply with all applicable criteria.

6.10 LOW IMPACT DEVELOPMENT

LID is the site design strategy with a goal of maintaining or replicating the pre-development hydrologic regime through the use of design techniques to create a functionally equivalent hydrologic landscape. LIDs are based on controlling storm water at the source by the use of micro-scale controls that are distributed throughout the site. These multifunctional site designs incorporate alternative storm water management practices such as functional landscape that act as storm water facilities, depression storage and open drainage swales. Fort Bend County encourages using the LID features in the watershed. These features should be discussed with the Fort Bend County Drainage District Engineer prior to the design process to ensure that the proposed features are acceptable to the Fort Bend County Drainage District.

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7.0 LEVEED AREAS

Flood plains cover a significant area within Fort Bend County, Texas. This area may be developed to the limits of the floodway if a levee system is constructed to protect the area from high water levels on the adjacent watercourse (usually the Brazos River). The components of the levee system shall include an internal drainage system, a levee, a pump station or adequate storage capacity, and a gravity outlet with an outfall channel to the river. The Fort Bend County design criteria for each component are defined in the following sections.

The county's minimum design standards shall be governed by the rules and regulations as established by the Federal Emergency Management Agency (FEMA) including any updates as they occur. In general, FEMA is not responsible for building, maintaining, operating, or certifying levee systems. FEMA does, however, develop and enforce the regulatory and procedural requirements that are used to determine whether a completed levee system should be credited with providing 100-year (1-percent-annual-chance) flood protection. These requirements are documented in Section 65.10 of the National Flood Insurance Program (NFIP) regulations. The engineer is advised to check the current FEMA rules and regulations. Maintenance of these facilities generally will not be the responsibility of Fort Bend County.

7.1 INTERNAL DRAINAGE SYSTEM

The internal drainage system for the leveed area shall include the network of channels, lakes, and storm sewers which drain the leveed area to the outfall structure. Refer to Section 3.0 Open Channel Flow, Section 5.0 Storm Sewers and Overland Flow and Section 6.0 Storm Runoff Storage for Fort Bend County construction requirements and design criteria.

7.2 LEVEE SYSTEM

7.2.1 Frequency Criteria

The levee system shall include a levee embankment that will protect the development from the 100-year frequency flood event on the adjacent watercourse. Protection from the 100-year frequency event shall include protection from the 100-year water surface elevation on the watercourse, as well as protection from any associated wind and wave action.

7.2.2 Design Criteria

General design criteria for levees in Fort Bend County are shown below. However, all levees should be designed in accordance with the U.S. Army Corps of Engineers (COE) Engineer Manual EM 1110-2-1913 (30 April 2000, or most current edition). If conflicts exist between the COE manual and the criteria shown below, the Fort Bend County Drainage District Engineer should be consulted for direction.

1. A geotechnical investigation shall be required on the levee foundation (the existing natural ground). Soil borings shall be required with a maximum spacing of 1,000 feet and a minimum depth equal to twice the height of the levee embankment.

2. The foundation area shall be stripped for the full width of the levee. Stripping shall include removal of all grass, trees, and surface root systems.

3. Embankment material shall be CH or CL as classified under the Unified Soil Classification System and shall have the following properties:

    a. Liquid Limit greater than or equal to 30.
    b. Plasticity Index greater than or equal to 15.
    c. Percent Passing No. 200 Sieve greater than or equal to 50.

    A geotechnical investigation shall be required on the embankment material to determine the levee side slopes and methods employed to control subsurface seepage.

4. The embankment material shall be compacted to a minimum density of 95 percent using the standard proctor compaction test at approximately plus or minus three percent optimum moisture content. The embankment material shall be placed in lifts of not more than 12 inches thick.

5. The levee top and side slopes shall be adequately protected by grass cover or other suitable material.

6. The minimum levee top width shall be ten feet.

7. The levee side slope shall be one vertical to a minimum of three horizontal.

8. Both levees and floodwalls should provide at least 1 foot freeboard above FEMA minimum requirement. The FEMA minimum for riverine levees is as shown below:

   a. In accordance with Section 65.10 of the NFIP, a minimum freeboard of 3 feet above the water-surface level of the base flood must be provided for riverine levees.
   b. An additional 1 foot above the minimum is required within 100 feet on either side of structure (e.g., bridges) riverward of the levee or wherever the flow is constricted.
   c. An additional 0.5 foot above the minimum at the upstream end of the levee tapering to not less than the minimum at the downstream end of the levee, is also required.
   d. Occasionally, exceptions to minimum riverine freeboard requirements above may be approved if the following criteria are met:
      1) Appropriate engineering analyses demonstrating adequate protection with a lesser freeboard must be submitted.
      2) The material presented must evaluate the uncertainty in the estimated base flood elevation profile and include, but not necessarily be limited to a) an assessment of statistical confidence limits of the 1 % AEP discharge b) Changes in stage-discharge relationships and c) Sources, potential, and magnitude of debris, sediment, and ice accumulation.
      3) It must be shown that the levee will remain structurally stable during the base flood when such additional loading considerations are imposed.
   e. Under no circumstances will freeboard of less than 2 feet above BFE be accepted.

9. The levee shall be continuous and shall either completely encompass the development or tie into natural ground located outside of the limits of the adjacent watercourse's 100-year floodplain.

10. All pipes and conduits passing through the levee shall have anti-seepage devices, flap gates, and slope protection.

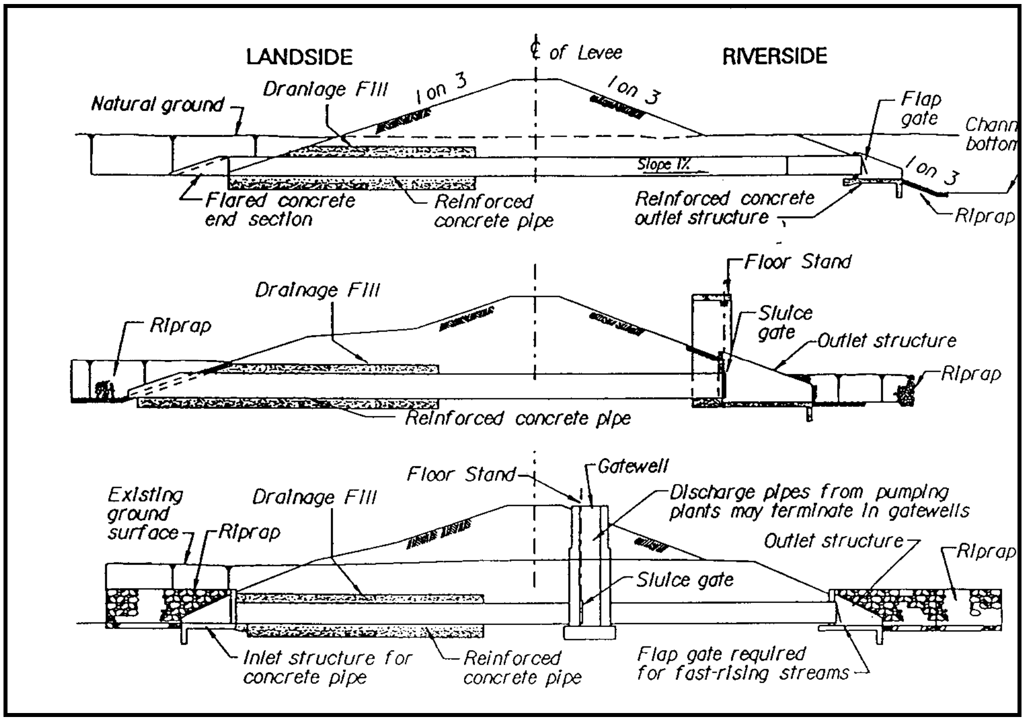
    a. Antiseepage devices have been employed in the past to prevent piping or erosion along the outside wall of the pipe. The term "antiseepage devices" usually referred to metal diaphragms (seepage fins) or concrete collars that extended from the pipe into the backfill material. The diaphragms and collars were often referred to as "seepage rings." However, many piping failures have occurred in the past where seepage rings were used. Assessment of these failures indicated that the presence of seepage rings often results in poorly compacted backfill at its contact with the structure.

    b. Where pipes or conduits are to be constructed through new or existing levees:

        1) Seepage rings or collars should not be provided for the purpose of increasing seepage resistance. Except as provided herein, such features should only be included as necessary for coupling of pipe sections or to accommodate differential movement on yielding foundations. When needed for these purposes, collars with a minimum projection from the pipe surface should be used.

        2) A 0.45-m (18-in) annular thickness of drainage fill should be provided around the landside third of the pipe, regardless of the size and type of pipe to be used, where landside levee zoning does not provide for such drainage fill. For pipe installations within the levee foundation, the 0.45-m (18-in) annular thickness of drainage fill shall also be provided, to include a landside outlet through a blind drain to ground surface at the levee toe, connection with previous underseepage features, or through an annular drainage fill outlet to ground surface around a manhole

structure. The figure below shows typical sections of drainage structures through levees.



Typical Sections, Drainage Structures through Levees
(From EM 1110-2-1913)

11. The minimum right-of-way for the levee shall be from toe to toe. In addition, the establishment of an easement for maintenance and access, which may be located within the right-of-way, shall be required. Access shall be provided with either a minimum 10-foot easement adjacent to the levee, a minimum 10-foot levee top width or a minimum 10-foot horizontal berm on either side of the levee. A minimum 20-foot wide easement should be established in at least two locations to provide access to the levee right-of-way from a nearby public road.

7.3 PUMP STATIONS

7.3.1 Frequency Criteria

To prevent flooding within leveed areas, pumps are recommended (instead of only storage) to remove interior drainage when the exterior river stage reaches a level that prevents gravity outflow. In order to determine the required pump capacity so that the maximum ponding level within the leveed area will not be exceeded on the average more than about once in 100 years, the following design criteria have been developed.

The two sets of criteria provided below differ depending on whether the storm that occurs over the leveed area during high exterior river stages is an independent or dependent event as compared to the storm that produced the high river stages. For a detailed discussion of the development of this criteria, see Appendix C. In Fort Bend County, the levees along the Brazos River should be analyzed independently (using coincidental events, criteria 7.3.1.1) and all other levees should be analyzed dependently (using same events, criteria 7.3.1.2).

7.3.1.1 Design Criteria Assuming Coincidental Events

This criterion presumes that the storm event causing a high flood stage outside of the leveed area is independent of the storm event occurring over the leveed area (e.g. a leveed area draining into the Brazos River in Fort Bend County). The following steps should be taken for determining the required pumping capacity.

1. Select the maximum ponding level within the leveed area that should not be exceeded more than once in 100 years on the average. Normally, this level will be equal to the maximum water surface elevations associated with the 100-year flood event computed in designing the internal drainage system (channels) of the leveed area, including the required minimum freeboard of one foot. This will be the level which, when equaled or exceeded by exterior flood stages, will prevent gravity outflow and require total pumping to remove any runoff that might occur within the leveed area.

2. From a rating or backwater curve applicable to the location on the watercourse where the gravity outflow point of the leveed area exists, determine the discharge corresponding to the maximum ponding level. See Figures 7-1-1 through 7-1-18 for multiple flood profiles from which a discharge can be derived. These profiles are based on the hydraulic model from the (Preliminary) Flood Insurance Study of Fort Bend County, Texas, 2009.)

3. Determine the percentage of time that the discharge (obtained from Step 2 above) is equaled or exceeded. Given this percentage of time, determine the frequency of the rainfall event corresponding to the coincidental probability of these two events. (For the Brazos River, Figure 7-2 shall be used to determine directly the frequency of rainfall from the discharge corresponding to the maximum ponding elevation.)

4. Use TP-40 (see Figure 7-3) or other appropriate rainfall frequency curve to obtain the rainfall amounts associated with the return period (obtained from Step 3 above) to be used for determining the required pumping capacity.

7.3.1.2 Design Criteria Assuming Same Event

This criteria presumes the storm event causing high flood stages outside of the leveed area is the same (dependent) storm event occurring over the leveed area. The design rainfall amounts to be used for sizing the required pump capacity will be associated with the 100-year rainfall event. (See Table 2-1 for rainfall amounts derived from TP-40 and Hydro-35).

7.3.2 <u>Design Criteria</u>

All leveed areas within Fort Bend County that are equipped with a pump station shall be capable of maintaining the design pumping capacity with its largest single pump inoperative. The capacity of a pump station designed under Section 7.3.1.1 shall be adequate to remove a minimum volume of water from the leveed area within 24 hours without exceeding the maximum ponding elevation within the leveed area. If a pump station is not provided, adequate storage volume below the maximum ponding level must be provided to contain the entire design storm. The volume of runoff to be pumped shall be the greater of either:

1. The runoff resulting from the appropriate rainfall amount as determined in Step 4 of Section 7.3.1.1.

2. A minimum of 1½ inches of runoff from fully developed areas and 1 inch of runoff from undeveloped areas over the contributing watershed.

A pump station designed under Section 7.3.1.2 shall have a combination of storage volume/pumping capacity adequate to maintain the runoff resulting from the 100-year frequency event below the maximum ponding level. The minimum pumping capacity shall be the same as number two above. All pump stations in Fort Bend County shall be equipped with auxiliary power for emergency usage.

7.4 GRAVITY OUTLET AND OUTFALL CHANNEL

An outlet shall be required to release by gravity from the leveed area through the outfall channel to the adjacent watercourse during low flow conditions on the receiving channel. The outlet shall be equipped with an automatically functioning gate to prevent any external flow from entering the leveed area.

The outlet and outfall channel shall be designed in accordance with Section 3 - Open Channel Flow. The velocities within the outfall channel at the adjacent river shall not exceed 5.0 feet per second.

7.5 REVIEW PROCESS

When a levee system is required for development, the following information shall be submitted to the Fort Bend County Drainage District for review:

1. Preliminary Submittal

    a. A vicinity map showing the proposed levee location in relation to the 100-year flood plain and floodway of the adjacent river.
    b. The preliminary design of the levee cross-section based upon the geotechnical investigation.
    c. The preliminary design of the pump station capacity

2. Final Submittal

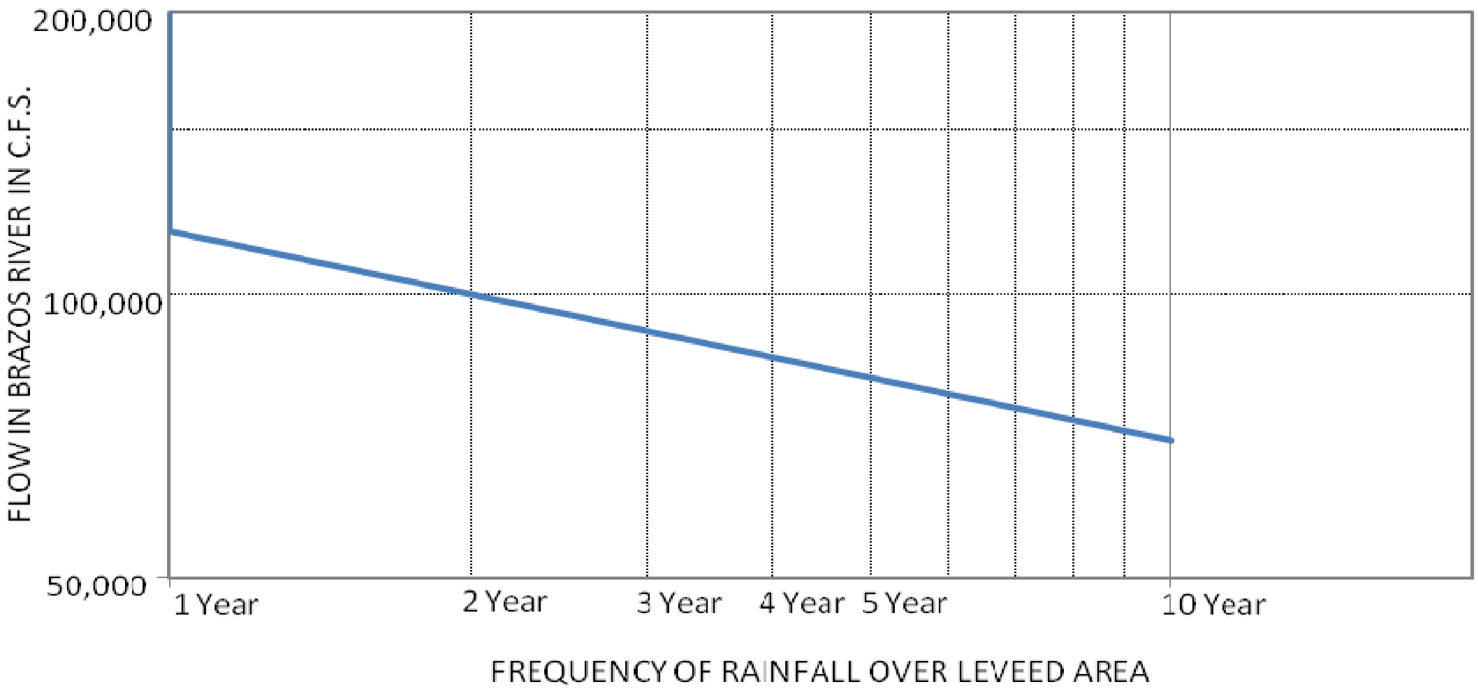
    a. The final design of the levee cross-section and location.
    b. The final design of the pump station capacity.
    c. The hydraulic calculations showing that the maximum ponding elevation is not exceeded within the leveed area more than once in 100 years on the average.
    d. The construction drawings and technical specifications for the levee and pump station along with final design computations for the levee, pump station and channels.

In accordance with the current Texas Water Code, Texas Commission on Environmental Quality (TCEQ) approval shall be required on the following.

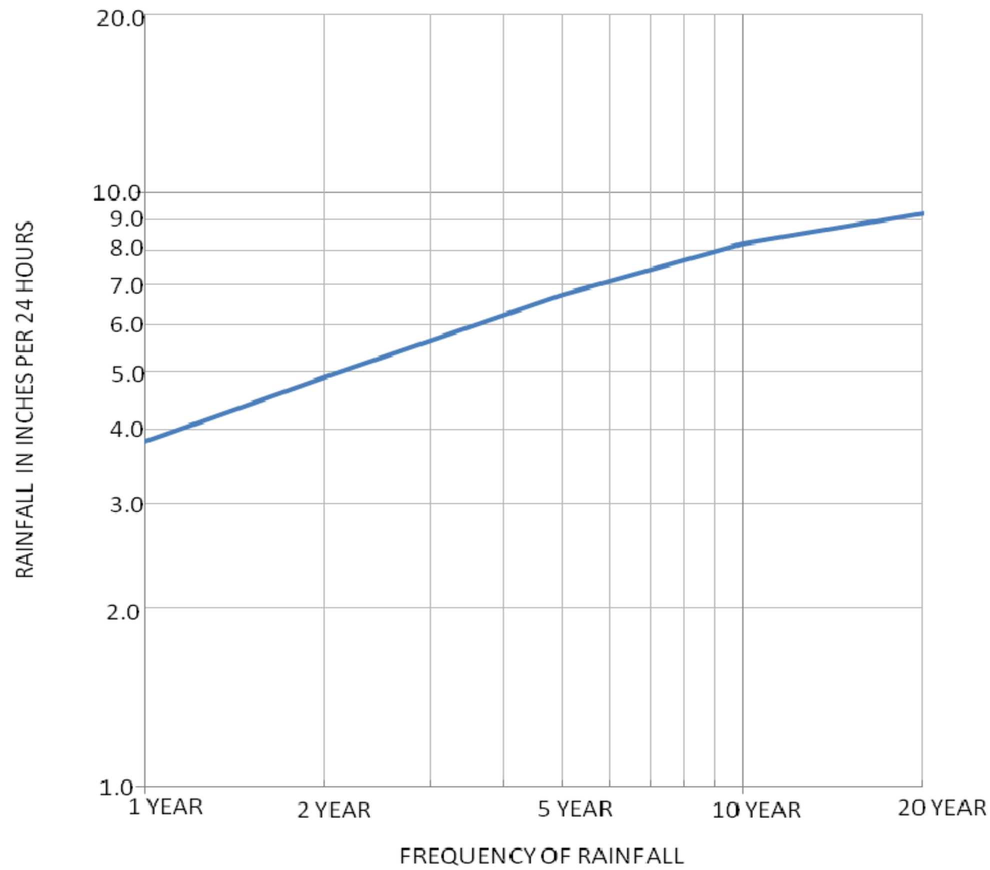
1. Levee improvement district proposed plans of reclamation.

2. Preliminary plans for construction of levees or other improvements.

3. Final plans for levees and other improvements.



Flow/Frequency For Fort Bend County, Texas	
August 1986	Figure 7-2



Rainfall / Frequency For Fort Bend County, Texas	
August 1986	Figure 7-3

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8.0 DRAINAGE DESIGN CRITERIA FOR RURAL SUBDIVISIONS

8.1 PURPOSE

The Fort Bend County Drainage Criteria Manual ("DCM"), adopted in 1987, was intended to address design procedures for 100-year design channels and for storm sewer systems in response to the expanding urban development taking place in the county. However, this DCM did not specifically address certain drainage issued related to large lot subdivisions which typically are built in the rural areas of the county.

The purpose of this design criteria is to make available an alternative drainage procedure that can be used in the design of detention facilities for such rural-type subdivisions.

Typically, such developments consist of large-acre lots with minimal drainage improvements. Little change to the natural storm runoff occurs as a result of such rural subdivisions being developed. In recognition of this, this criteria has been developed such that the effect is to reduce the amount of on-site detention otherwise required by the DCM. However, this is minimal criteria for acceptance by the Fort Bend County Drainage District. Individual circumstances may warrant an enhanced drainage and/or detention system.

8.2 QUALIFICATIONS

The following qualifications are established and must be met in order to be considered a rural subdivision for purposes of utilizing this alternative design criterion:

1. Lot size of 1 acre or greater;

2. Maximum percent impervious cover based upon lot size (see Figure 8-1);

3. Roadside ditch drainage system (vs. curb and gutter); and

4. No major drainage improvements that would significantly alter the natural drainage patterns in the area for large flood events.

8.3 DESIGN CRITERIA

The following design criteria shall be utilized for rural subdivisions:

1. Minimum slab elevations – two (2) feet above natural ground, or 18” above the 100-year floodplain, or one (1) foot above the crown of any downgradient roadway, whichever is higher.

2. Roadways

   a. R.O.W. – Seventy (70) feet wide.

   b. Crown – Maximum of one (1) foot above natural ground.

   c. Roadside drainage system – Open ditch with 3:1 side slopes; equalizer pipes under roadway at least every 1,000 feet (minimum 24-inch diameter RCP) if roadway blocks natural drainage path.

3. Lot drainage – Swales may be constructed along lot lines to provide for minimal drainage of lots. Other than lot line swales and building pads, lots shall not be significantly graded.

4. Detention Requirements – See Figure 8-1 for amount of on-site detention required. Discharge pipe to be maximum 18-inch diameter RCP, or equivalent.

8.4 SUBMITTALS

1. Drainage area map showing existing drainage ways on or adjacent to property.

2. Map(s)/drawing(s) showing existing drainage patterns before development and proposed drainage patterns after development, for both small storm events and large storm events.

3. Preliminary (and eventually final) plat with the following plat notes:

a. The latest floodplain information, including Base Flood Elevation, and Flood Insurance Rate Map Panel Number and Date.

b. Land use within the subdivision is limited to an average imperviousness of no more than ____ percent. (Obtain maximum percent imperviousness from Figure 8-1 for the corresponding average lot size shown on the plat.) The drainage and/or detention system has been designed with the assumption that this average percent imperviousness will not be exceeded. If this percentage is to be exceeded a replat and/or redesign of the system may be necessary.

c. The minimum slab elevation shall be 18" above 100-year floodplain elevation, or at least 2 feet above natural ground, or 1ft above the crown of any down-gradient roadway, whichever is higher. Floodplain information note should be included.

d. This rural subdivision employs a natural drainage system which is intended to provide drainage for the subdivision that is similar to that which existed under pre-development conditions. Thus, during large storm events, ponding of water should be expected to occur in the subdivision to the extent it may have prior to development, but such ponding should not remain for an extended period of time. Street ponding information notes should be included.

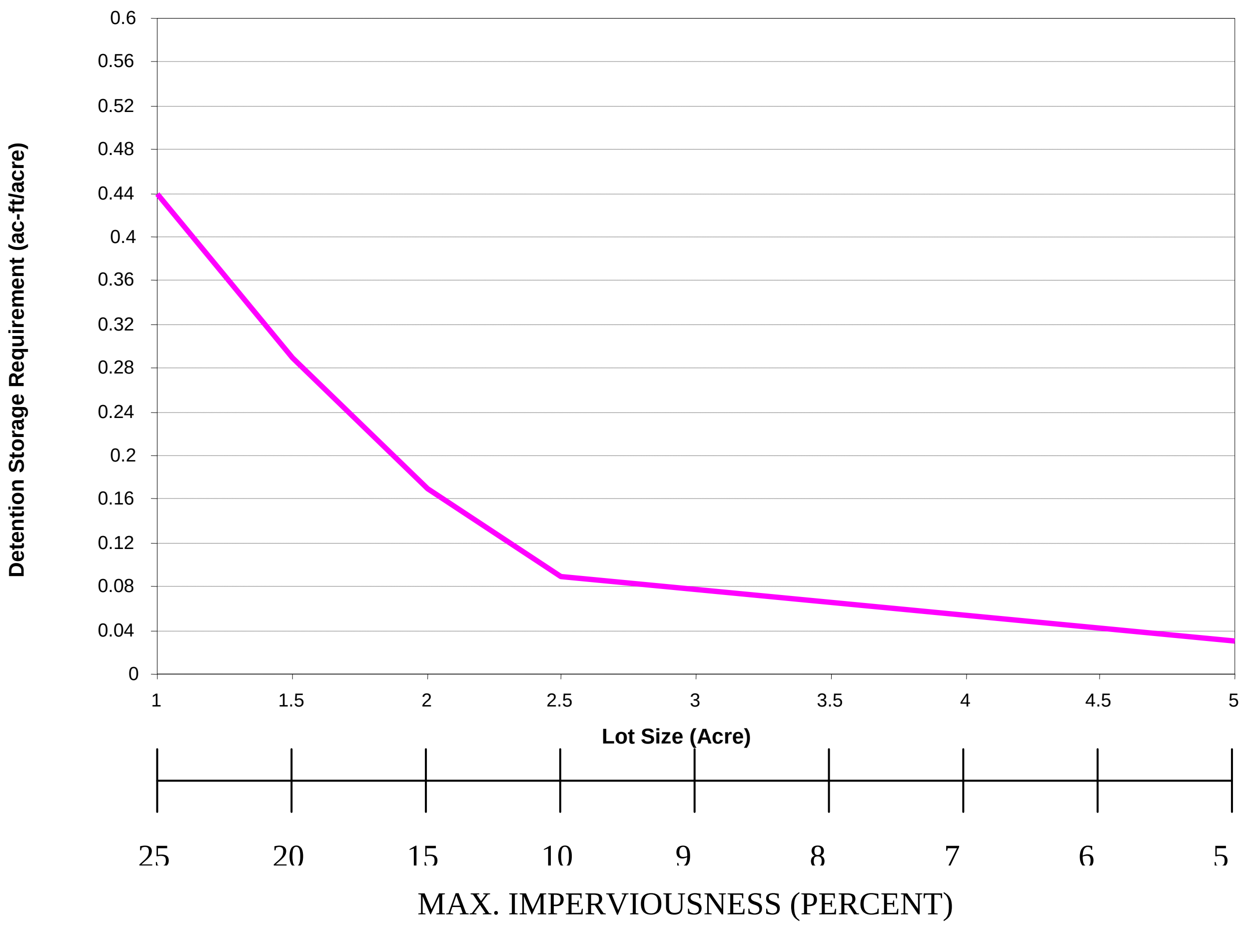

Figure 8-1 Detention Storage Requirements for Rural Subdivisions

8.5 TECHNICAL ANALYSIS OF DETENTION REQUIREMENTS FOR RURAL SUBDIVISIONS

The purpose for requiring detention for developing a subdivision is to minimize the adverse impact the development has on downstream flooding. This adverse impact is caused by a combination of additional runoff, due to the reduction of infiltration caused by the increase in imperviousness associated with development, and a higher rate of runoff, due to the reduced time of concentration cause by the more efficient drainage system associated with development. The detention requirement was developed so as to minimize these adverse impacts typically with urban development (involving less than 1-acre lots).

Rural subdivisions, however, generally involve lot sizes greater than 1 acre. These subdivisions also generally have less impervious cover per acre and a less effective drainage system than do urban developments. Therefore a technical analysis was performed in order to determine the appropriate detention storage that should be required for rural subdivisions in lieu of the standard detention storage that should be required for rural subdivisions in lieu of the standard detention required for urban development under the Fort Bend County Drainage Criteria Manual.

8.6 ANALYSIS OF RUNOFF VOLUME

As the percent of imperviousness associated with a development project increases, the availability of ground surface for infiltration is reduced; and therefore, the amount of rainfall that becomes runoff is increased. An evaluation was made as to how much of an increase in runoff volume (i.e. rainfall excess) occurs as the percent imperviousness increases.

The 100-year, 24-hour excess for various percentages of imperviousness is as follows:

% IMPERV	RAINFALL (INCHES)	RAINFALL EXCESS (INCHES)	INCREASE IN RAINFALL EXCESS	
			Inches	Ac-ft/Ac
0	12.5	7.34	--	--
5	12.5	7.59	0.25	0.02
10	12.5	7.85	0.51	0.04
15	12.5	8.11	0.77	0.06
20	12.5	8.37	1.03	0.09
25	12.5	8.63	1.29	0.11

The above increases in rainfall excess show the additional runoff volume attributable to the various increases in imperviousness, and presumably the amount of detention storage in acre-ft. per acre that would be needed to offset such additional runoff so as to minimize its adverse impact downstream.

8.7 ANALYSIS OF RUNOFF RATE

Usually as development occurs, the corresponding drainage system is improved, as compared to the undeveloped condition, so as to more effectively remove storm water runoff away from the property and reduce the amount and duration of standing and/or high water near residences or commercial buildings. Such an improved drainage system tends to reduce the time it takes storm water to be transported off-site, thereby causing an increase in the peak runoff rate associated with the development as compared to its undeveloped condition.

However, many rural subdivisions tend to provide minimal improvements to the natural drainage system, especially as to large storms events. Therefore, an analysis was made as to what effect rural subdivisions might have on the peak rate of runoff in order to determine an appropriate detention requirement to offset any adverse impact to downstream flooding.

The Rational Equation ($Q = ciA$) is the preferred method for computing the peak runoff for an area of less than 100 acres, which applies to most rural subdivisions. The runoff coefficient, *c*, represents the type of land used and its slope, as well as the soil type and its rate of infiltration. Values of *c* were obtained from Table 2-3 of the criteria manual.

The rainfall intensity, *i*, depends upon the storm frequency and the time of concentration for the area. The drainage area, *A*, is computed in acres.

An evaluation was made of three parameters used to compute the peak runoff for an undeveloped area as compared to the same area being developed with a rural subdivision. This comparison would assist in determining the amount of detention that might be needed to offset any increase in the peak runoff from an area when it is developed into a rural subdivision.

Assuming there is no significant change in the overall drainage pattern of an area during a large storm event as a result of developing a rural subdivision, the size of the drainage area, *A*, used to compute the peak runoff should not change between undeveloped conditions versus a rural development.

The runoff coefficient *c*, is an estimated value; for undeveloped pastureland and cultivated land with clay soil, it is 0.30 and 0.35 respectively, per Table 2-3 of the criteria manual.

For a residential subdivision with lot sizes greater than ½ acre, the *c* value is also 0.30. Thus, with a rural subdivision with lot sizes of 1 acre or larger, the runoff coefficient for the developed condition would be essentially equal to the undeveloped condition.

However, the remaining parameter in the Rational Equation is the rainfall intensity, *i*, which is a function of the time of concentration. The extent to which the time of concentration changes due to the development of a rural subdivision depends largely upon the improvement that is made to the natural drainage system, something that is highly site-specific. Yet it is reasonable to assume that as the lot sizes get smaller and the percent of imperviousness increases, there will be a tendency for the time of concentration to be reduced. This would result in an increase in the peak rate of runoff and require some amount of detention storage to offset this component of the adverse impact due to the development of a rural subdivision.

8.8 DETERMINATION OF REQUIRED DETENTION

Based on the above analysis, the runoff volume is increased as a result of development and imperviousness increasing. On-site detention is required to reduce the impact that this increased runoff volume might have on flooding downstream. The amount of on-site detention required is equal to the increase in rainfall excess. In addition, as the percent imperviousness increases, the time of concentration tends to decrease thereby raising the possibility that the peak runoff may increase, necessitating additional detention to be required.

The amount of detention required to offset this impact is difficult to quantify, since the possible increase in peak runoff is highly site-specific. However, it is assumed that this component of the adverse impact from development will be virtually non-existent for very large acre lots (i.e. low percent imperviousness), but will become more important as the lot sizes decrease.

Therefore, a comparison was made between the detention storage required under the Fort Bend County Criteria Manual and that required solely due to the increase in runoff volume associated with a rural subdivision, as show in Figure 8-2. The criteria manual curve was based upon the equation S/A = $\sqrt{I}$, referenced in the criteria manual, in which the percent imperviousness, I, that was used to develop this curve, was the maximum percent imperviousness allowable for each lot size. This curve presumably reflected the detention required to offset both the impact due to additional runoff volume and the impact due to the increase in the peak rate of runoff generally attributable to the drainage systems associated with urban-type subdivisions. The Volume Only curve shown on Figure 8-2 was based solely upon the detention requirement to offset the increase in runoff volume determined in the Section above.

Based upon these two curves, the curve to be selected for this rural subdivision criteria should be expected to closely follow the runoff volume curve for the larger lot sizes and then diverge towards the criteria manual curve as the lot sizes decrease.

The resulting detention storage to be required for rural subdivisions to minimize any increase in flooding downstream as a result of such a development was selected to be as follows:

% Impervious	Detention Storage Required (Ac-ft/Ac)		
	Due to Vol. Increase	Assumed for Peak Q Inc	TOTAL
0	0	0	0
5	0.02	0.01	0.03
10	0.04	0.05	0.09
15	0.06	0.11	0.17
20	0.09	0.20	0.29
25	0.11	0.33	0.44

This tabulated information has been transferred onto Figure 8-2 on the following page, along with the incorporation of lot sizes associated with maximum percent imperviousness.

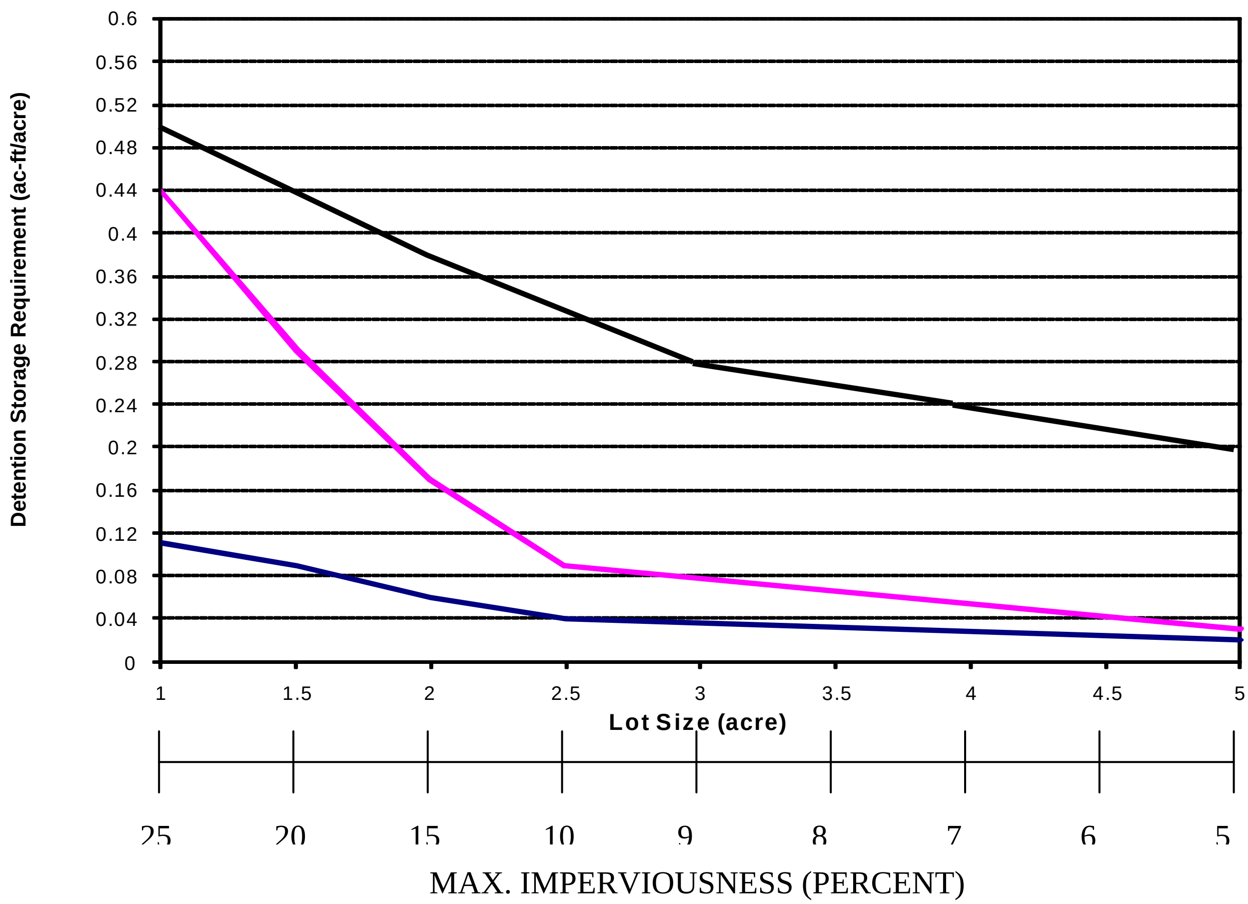

Figure 8-2 Comparison of Different Storage Requirements

Legend:

Black Line – Criteria Manual Curve S/A = $\sqrt{I}$

Magenta Line – New Detention Storage Requirement

Blue Line – Storage Due to Volume Increase Only

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APPENDIX A

DEVELOPMENT OF THE HYDROLOGIC METHODOLOGY

INTRODUCTION

In order to conduct the master drainage study for Fort Bend County, a methodology has been established for computing flows and water levels for all drainage analyses to be performed. It was initially determined that the established methodology should satisfy the following objectives:

(1) Be technically sound;
(2) Be easy to apply;
(3) Be capable of showing the effects of development on the flow regime of a watershed; and
(4) Be a useful tool for evaluating drainage regulation strategies for the County.

After reviewing a number of methodologies that had been utilized in Fort Bend County, it was concluded that use of a computer model employing the unit hydrograph theory would be the best approach for providing the necessary capabilities; and, therefore, this was adopted as the basic hydrologic methodology for the subsequent drainage studies.

Harris County had recently adopted its hydrologic methodology that was accepted by the Federal Emergency Management Agency (FEMA) and was used to revise the Flood Insurance Study that had been conducted for that county. This methodology has also been used to evaluate and design drainage improvements throughout Harris County. This method involved the use of a computer model (HEC-HMS) that includes the unit hydrograph approach (using Clark's unitgraph coefficients). As a result, we reviewed Harris County's methodology with the intent of adopting their approach with some changes, such as simplifying the procedure and making it more suitable for the type of watersheds typical of Fort Bend County. The following is a detailed explanation of the derivation of the hydrologic methodology developed and used for the Fort Bend County Master Drainage Study.

DERIVATION OF METHODOLOGY

The general hydrologic method adopted for conducting storm water computations in Fort Bend County is very similar to that used in Harris County. It includes use of the Clark unit hydrograph approach and a rainfall-loss exponential function contained in the HEC-HMS computer program that accounts for variation of loss with intensity of basin-average rainfall as well as with increasing ground wetness during the storm.

For unit hydrograph computations, a standard time-area function contained in HEC-HMS is used, along with Clark's unitgraph parameters TC (time of concentration) and R (Storage coefficient).

For the exponential loss rate function contained in HEC-HMS, the loss parameters used are initial coefficient (STRKR in HEC-1), coefficient ratio (RTIOL in HEC-1) and exponent (ERAIN in HEC-1) in the equations:

$$L = K \times P^{ERAIN} \qquad \text{(Eq. 1)}$$

and

$$K = (\text{initial coefficient or STRKR}) / (\text{coefficient ratio or RTIOL})^{(0.1 \times CUML)} \qquad \text{(Eq. 20)}$$

where:

L = loss rate in inches per hour

K = loss rate coefficient

P = rainfall intensity in inches per hour

ERAIN = exponent between 0.0 (constant loss) and 1.0 (loss proportional to rainfall)

Initial coefficient or STRKR = loss coefficient at start of storm

coefficient ratio or RTIOL = loss recession coefficient

CUML = accumulated loss since start of storm in inches

These unit hydrograph (TC and R) and loss (initial coefficient or STRKR, coefficient ration or RTIOL and exponent or ERAIN) parameters, required as input into the HEC-HMS program, have been derived from observed flood data and, insofar as is feasible, related to various basin characteristics (such as length, slope, percent development) so that information can be generated on the rainfall-runoff relationship for a given watershed where no runoff data are available. Derivation of these parameters is based on optimization studies using primarily rainfall and runoff data for those gages in the Houston metropolitan area (the best source of data for watersheds located close to Fort Bend County) considered most representative of streams in Fort Bend County.

BASIC DATA

Rainfall and runoff data for large storms published in open file reports of the U.S. Geological Survey were used for derivation of Unit hydrograph and loss parameters. Table 1 lists the stations and storms used.

Basin characteristics for the watershed area above these stations were obtained in part from previous reports and in part from topographic maps and aerial photographs. A summary of the pertinent basin characteristics is contained in Table 2.

The lst 12 stations listed in Tables 1 and 2 are supplementary stations selected from earlier studies made by the U.S. Army Corps of Engineers and turner, Collie and Braden, Inc. in order to provide data on TC and R for areas with steep slopes and other ranges of basin characteristics.

HYDROGRAPH ANALYSIS

The unit-hydrograph and loss-rate optimization routine in HEC-1 was used to derive values of the 2 unit-hydrograph and 3 loss-rate parameters for each of the 33 storms analyzed. HEC-HMS is the developed version of HEC-1, therefore the parametric values (such as loss parameters) of HEC-1 can be used for HEC-HMS. This manual has been revised with recommendation to replace HEC-1 by HEC-HMS for future use. Since TC and R have similar impacts on a unit hydrograph, the HEC-1 program uses transformed parameters of TC+R and R/(TC+R) for optimization computations.

The results of the reconstitution of these storm hydrographs were considered to be generally of high quality. Average error of the first run showed computed peak versus observed peak flow to be 6 percent, with about half of the computed peaks higher and half lower than observed peaks. Results of the first run are given in Table 3.

Parameter values of the loss function have no individual meaning. In order to compare values of initial coefficient (STRKR-the primary loss parameter) it is necessary to use the same values of exponent (ERAIN) and the same values of coefficient ratio (RTIOL) for every storm. A second computer run was made using a constant exponent (ERAIN - 0.6) and constant coefficient ratio (RTIOL - 3.0) approximately equal to the average values obtained in the first run. (Substantial rounding of these averages was permitted, since their standard error is large). This increased the average error to 7 percent, which is very minor compared to the substantial simplifications of the model thus obtained. Results of this second run are also given in Table 3.

A third run was then made using the relationship of TC/(TC+R) shown in Figure 1 (obtained from the second run results) as well as constant values of exponent (ERAIN -0.6) and coefficient ratio (RTIOL - 3.0). Thus, only the 2 parameters, TC+R and initial coefficient or STRKR, were derived in this third run. Errors in peak flows increased to 10 percent on the average, but the reconstitutions still are generally very good and unbiased. Results of this run are given in the last 2 columns of Table 3.

CORRELATION WITH BASIN CHARACTERISTICS

Since variables exponent or ERAIN and coefficient ratio or RTIOL in equations 1 and 2 are assigned values about equal to the averages obtained in the unit hydrograph derivations of the first 16 stations of Table 1 (areas hydrologically similar to those in Fort Bend County), the only variables remaining to be related to basin characteristics are initial coefficient or STRKR, TC+R and R/(TC+R).

The loss index initial coefficient (STRKR), does not correlate significantly with soil characteristics within Harris County where the loss data were derived. Table 4 shows an analysis of variance, which indicates that the variance of initial coefficient (STRKR) between storms at the same station is even greater than between station averages. This simply means that the data are inadequate to distinguish loss indexes at different locations. It is also considered that losses in this region are similar to those in Fort Bend County. Consequently an average coefficient of 0.5 for initial coefficient (STRKR) is adopted for Fort Bend County areas.

Values of TC/(TC+R) or R/(TC+R) do not correlate appreciably with any basin characteristics within the Harris County area represented by the first 17 stations of Table 2. However, when data for other stations (18 thru 29 of Table 3A) are considered, there is a good correlation with basin slope, as shown in Figure 1. The relationship shown was adopted for Fort Bend County and is considered to reflect adequately the logical relationship between basin slope and basin storage. Upper and lower limits on the ratio were set arbitrarily to prevent unreasonably small values of TC or R in future applications.

The log of the variable TC+R (from Table 3 and 3A) was correlated with several variables in an attempt to find the best correlation with certain basin characteristics with results as follows:

Variables	Correlation Coefficient
log L	.840
log $L/\sqrt{S}$	.821
log $L/\sqrt{S}$, log N	.913
log $L/\sqrt{S}$, log N, D	.931
log $L/\sqrt{S}$, log N, D, log S_0	9.31

On the basis of these results and the fact that including the last variable, S_0, provides a logical addition to the resulting relationship, the following regression equation was adopted:

$$TC+R = 128 \frac{(L\sqrt{S})^{.57} \; N^{.8}}{S_0^{.11} \times 10^{I}} \qquad \text{(Eq. 3)}$$

Where: TC = Clark's time of concentration

R = Clark's storage coefficient

L = length of the longest watercourse within a subarea (in miles)

S = average slope of the longest watercourse in its middle 75 percent (in feet/mile)

N = Manning's roughness coefficient for the longest watercourse weighted in proportion to distance from upstream end

S_0 = average basin slope of land draining into the longest watercourse (in feet/mile)

I = effective imperviousness ratio (.0035D for the regression analysis)

D = percent urban development

This function is plotted on Figure 2 along with the basic data used.

PONDING

Certain subareas, for which a flood hydrograph is to be computed, have ponding areas that will have an effect on the runoff being generated from the subarea. As the flood hydrograph passes through these ponding areas, the peak flow is reduced, and the time at which that peak flow occurs is delayed. An appropriate means to account for this effect in computing the flood hydrograph for such a subarea, using the hydrology methodology previously discussed, is to adjust upward the Clark's R coefficient, since this coefficient represents the storage-routing characteristics of the subareas.

The Soil Conservation Service, in their Technical Report No. 55, presents three tables of adjustment factors to the peak discharges of various frequency flood events in relation to the percent ponding in the subarea. The difference among the three tables is in the amount of the subarea's runoff that is affected by the ponding area (i.e. whether it is located either in the upper middle or lower portion of the subarea). Figure 3 provides a set of equations and curves that relate the percent of ponding (i.e. the percent ratio of the pond's surface area to the total drainage area of the subarea) to an adjustment factor for Clark's R coefficient. These equations correspond to the SCS table that presumes virtually all of the runoff from the drainage area passes through the ponding areas. Therefore, once the appropriate adjustment factor, RM, is derived from these equations, this factor needs to be prorated downward as the percent of the drainage are that is affected by the ponding area(s) goes from 100% down towards 0%. For example, if a subarea of 5 square miles has two lakes with a total combined surface area of ½ square mile, the percent ponding would be 10 and the RM factor for a 100-year event would be 164%. This would be the adjustment factor to be applied to the R coefficient previously computed from TC+R only if 100% of the subarea drains into or through these two lakes. If only 50% of the subarea drains into these two lakes, then the RM factor of 164% would be reduced to 132% as the appropriate adjustment factor to be applied to the R coefficient. If R had previously been determined to be 19.7, the new R that reflects the effect of ponding would be 19.7 x 1.32 = 26.0.

If a ponding area does not allow runoff to pass through it (e.g. a gravel pit), then that portion of the area that drains into the pond, plus the pond surface area itself, should be eliminated from the drainage area of the subarea as being non-contributing area.

COMPARISON OF METHODOLOGIES

A comparison was made between the newly developed hydrology methodology and other previously used methodologies for information purposes. Table 5 shows a comparison of 100-yer computed discharge values for a number of the watersheds used in developing the new methodology. The variability in the results is inherent in the use of different methodologies and may also reflect differences in drainage area size and percent imperviousness. Table 6 shows a comparison of 100-year computed discharge values for some of the watersheds studied in Fort Bend County during the Master Drainage Study. Here, the variability in the values as shown in the table is directly related to the differences in the methodologies used.

CONCLUSION

The hydrologic methodology developed for use in the Fort Bend County watershed studies is very similar to that used in Harris County, and will produce similar results. It is designed to be easier, more direct and more definitive in application. The ponding adjustment procedure is also very similar to that used in the Harris County methodology; however, the differences in the two procedures are a result of the different approaches taken in development of the hydrologic methodologies and the way that ponding is defined and accounted for in computing the unit graph parameters.

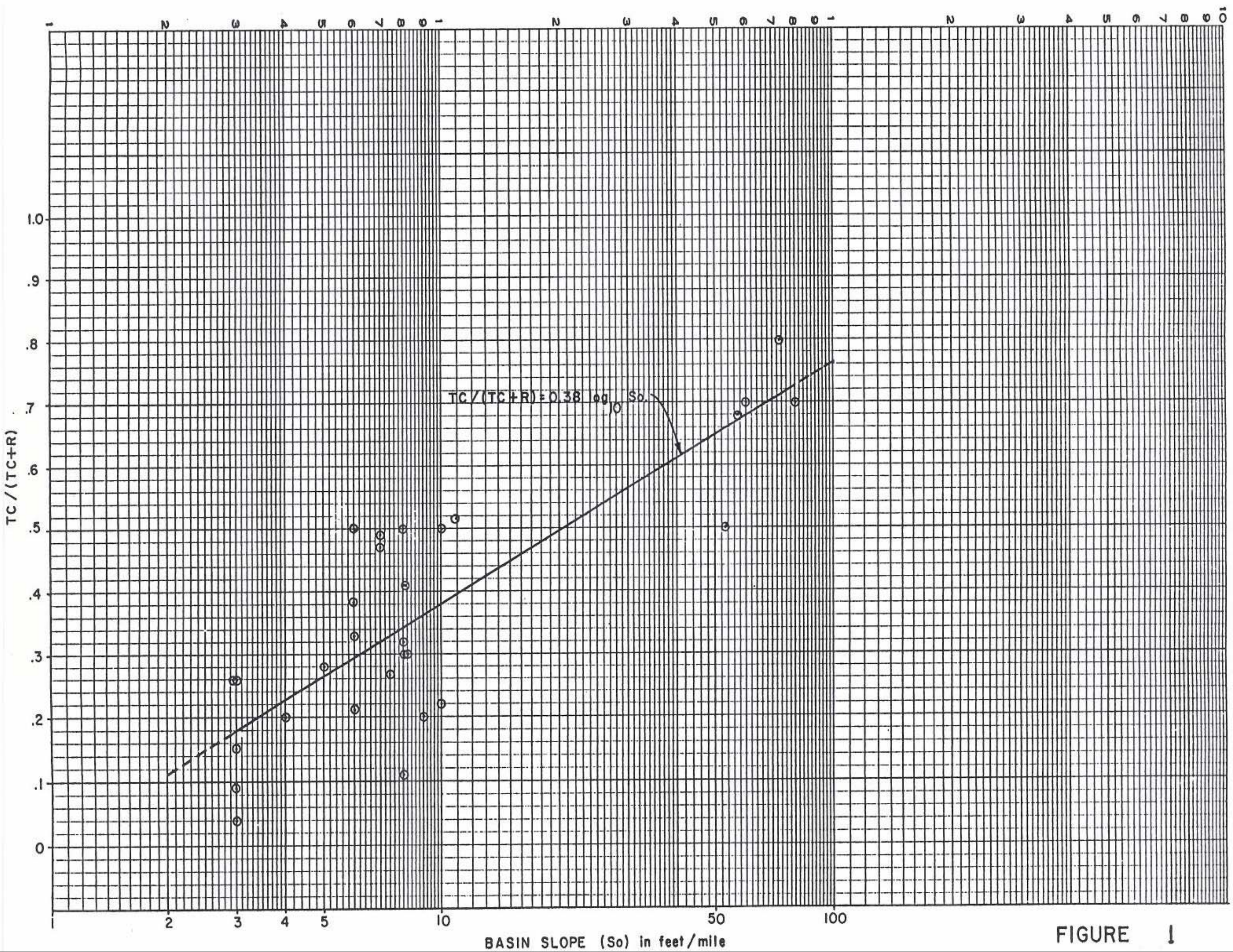


FIGURE 1

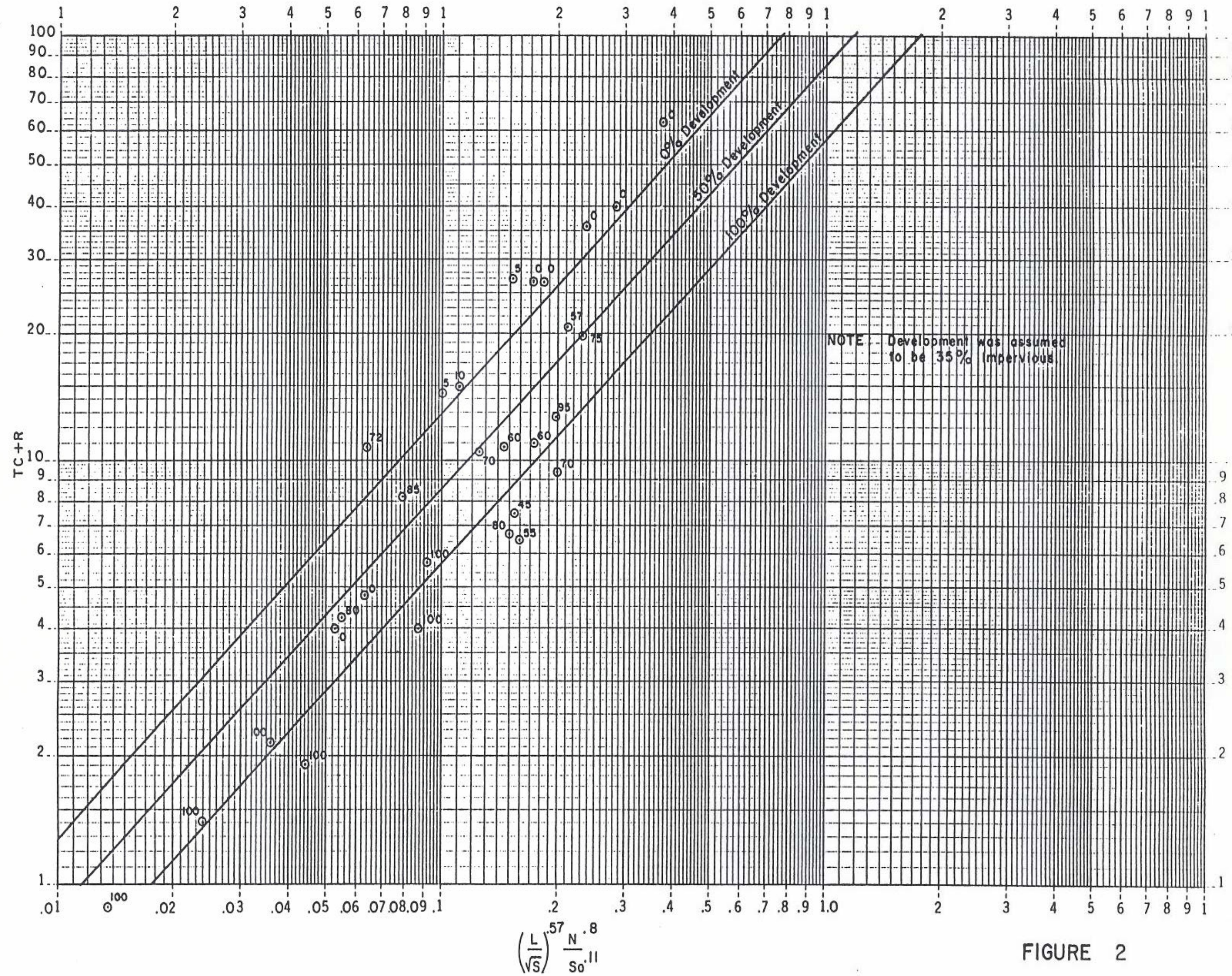
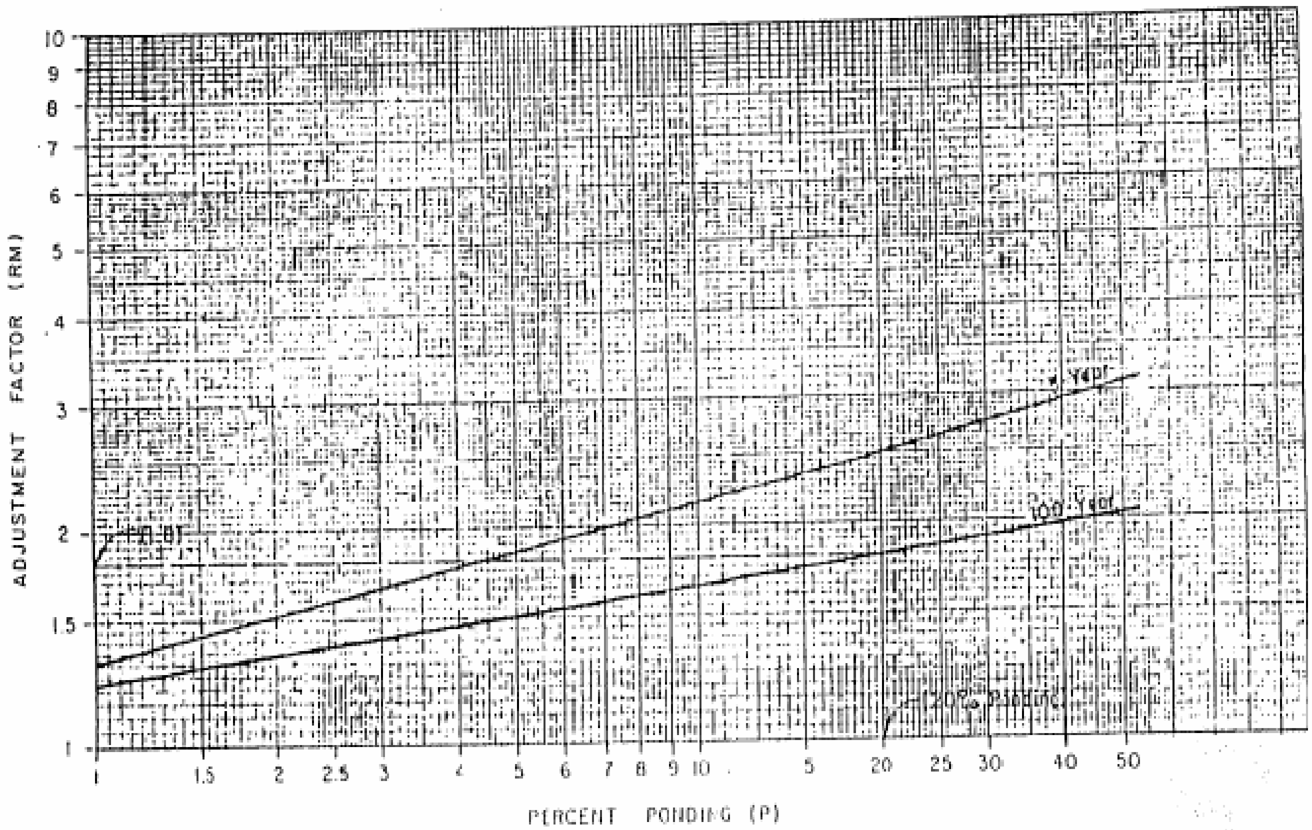

TC+R
0% Development
50% Development
100% Development
NOTE - Development was assumed to be 35% impervious
$\left(\frac{L}{\sqrt{S}}\right)^{.57} \frac{N^{.8}}{So^{.11}}$

FIGURE 2



PONDING ADJUSTMENT FACTOR FOR CLARKS STORAGE COEFFICIENT (R) FOR FORT BEND COUNTY, TEXAS

STORM EVENT	ADJUSTMENT FACTOR (RM) EQUATION
5 YEAR	$RM = 1.31\ P^{0.214}$
10 YEAR	$RM = 1.28\ P^{0.199}$
25 YEAR	$RM = 1.25\ P^{0.171}$
50 YEAR	$RM = 1.23\ P^{0.153}$
100 YEAR	$RM = 1.21\ P^{0.132}$
500 YEAR	$RM = 1.17\ P^{0.086}$

Source:
Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas, February, 1984.

August 1986	FIGURE 3

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TABLE 1
STORMS ANALYZED IN CORRELATION STUDIES

Station	Storm No.	USGS Sta. I.D. No.	Station Name	Storm Date
1	1a	0807 3630	Bettina St. Ditch at Kimberly St.	7/20/79
	1b	"	"	4/23/81
2	2a	0807 4250	Brickhouse Gully at Costa Rica St.	3/20/72
	2b	"	"	4/19/79
3	3a	0807 4500	Whiteoak Bayou at Heights Blvd.	3/20/72
	3b	"	"	1/6/79
	3b	"	"	5/13/82
4	4a	0807 4540	Little Whiteoak Bayou at Trimble St.	5/3/81
	4b	"	"	8/30/81
	4c	"	"	5/13/82
5	5	0807 4760	Brays Bayou at Alief	5/13/82
6	6	0807-4780	Keegans Bayou at Keegan Road	8/30/81
7	7a	0807-4800	Keegans Bayou at Roark Road	8/30/81
	7b	"	"	5/13/82
8	8a	0870 4810	Brays Bayou at Gessner Dr.	10/31/81
	8b	"	"	5/13/82
9	9a	0807 4910	Hummingbird St. Ditch at Mullins St.	7/1/79
	9b	"	"	5/13/82
10	10a	0807 5000	Brays Bayou at Main St.	4/19/79
	10b	"	"	5/13/82
11	11a	0807 5400	Sims Bayou at Hiram Clarke St.	6/11/73
	11b	"	"	5/13/82
12	12a	0807 5500	Sims Bayou at Highway 35	6/11/73
	12b	"	"	8/30/81
	12c	"	"	10/5/81
13	13a	0807 5550	Berry Bayou at Gilpin St.	5/19/79
	13b	"	"	5/13/82
14	14a	0807 5650	Berry Bayou at Forest Oaks St.	7/25/79
	14b	"	"	5/13/82
15	15a	0807 5730	Vince Bayou at Pasadena	3/19/79
	15b	"	"	7/25/79
	15c	"	"	5/13/82
16	16	0807 5770	Hunting Bayou at IH 610	5/17/82

TABLE 1 (Conclude)
STORMS ANALYZED IN CORRELATION STUDIES

Station	Storm No.	USGS Sta. I.D. No.	Station Name	Storm Date
17	17	0807 5900	Greens Bayou at US 75	9/19/79
18		0806 8450	Panther Br. nr Spring (Montgomery Cty)	CE Data[a]
19		0811 4900	Seabourne Cr. nr Rosenberg (Ft Bend Cty)	CE Data[a]
20		0811 6400	Dry Cr. nr Rosenberg (Ft Bend Cty)	CE Data[a]
21		0811 5500	Fairchild Cr. nr Needville (Ft Bend Cty)	CE Data[a]
22		0811 5000	Big Cr. nr Needville (Ft Bend Cty)	CE Data[a]
23		0806 7550	Welch Cr. (Montgomery Cty)	TC&B Data[b]
24		0806 8300	Mill Cr. Trib. nr Dobbin (Montgomery Cty)	CE Data[a]
25		0807 0500	Caney Cr. nr Splendora (Montgomery Cty)	CE Data[a]
26		0807 1000	Peace Cr. at Splendora (Montgomery Cty)	CE Data[a]
27		0806 8500	Spring Cr. nr Spring (Montgomery Cty)	CE Data[a]
28		0807 4400	Lazybrook	TC&B Data[b]
29		0807 3750	Stoneybrook Street Ditch	TC&B Data[b]

Note: All gages in Harris County unless otherwise noted.
a. Corps of Engineer's data for this station.
b. Turner, Collie & Braden, Inc. data used for this station.

(Ref. for a & b: "Harris County Flood Hazard Study Final Report", dated September 1984, prepared by TC&B and Pate Engineers, Inc.)

TABLE 2
BASIN CHARACTERISTIS FOR GAGES ANALYZED

Station No.	Drainage Area (mi^2)	Basin Length (mi)	Length to Centroid (mi)	Channel Slope (ft/mi)	Watershed Slope (ft/mi)	Weighted Manning's "n" value	Percent Development (@ 1980)
1	1.37	1.00	0.50	2.50	3.0	.025	100
2	11.40	6.35	3.55	7.90	8.0	.02	80
3	86.30	21.30	12.10	5.50	8.0	.025	60
4	18.00	7.89	3.39	14.40	8.0	.04	100
5	14.10	8.76	3.79	2.00	7.0	.04	60
6	7.47	6.50	2.90	2.35	3.0	.04	45
7	11.50	8.70	3.03	2.35	10.0	.04	55
8	53.20	13.80	7.50	3.30	6.0	.04	70
9	0.32	0.80	0.40	3.00	3.0	.04	100
10	94.90	21.60	11.30	3.10	6.0	.02	80
11	202.0	6.06	3.41	2.90	7.40	.04	70
12	63.00	18.20	7.95	3.10	7.20	.04	75
13	2.56	2.00	1.00	4.00	5.0	.4	72
14	10.70	5.00	2.50	10.00	8.0	.04	85
15	7.32	5.25	1.75	4.90	3.0	.03	100
16	15.80	6.55	3.00	2.20	7.0	.06	95
17	36.10	11.70	5.20	4.20	4.0	.05	57
18	34.50	13.10	7.80	6.30	57	.06	0
19	5.70	5.30	2.60	4.42	9.0	.04	5
20	8.60	6.63	3.06	5.08	8.0	.04	10
21	24.90	7.70	3.80	4.10	6.0	.06	0
22	42.80	14.00	7.42	3.20	10.0	.03	5
23	2.35	4.00	2.60	19.5	73	.06	0
24	4.07	3.60	1.70	33.0	53	.06	0
25	105	35.40	13.10	7.90	59	.06	0
26	117	26.80	15.40	7.70	80	.06	0
27	409	52.40	27.00	6.90	50	.06	0
28	0.13	0.66	0.27	5.20	8.0	.015	100
29	0.50	0.76	0.33	2.40	6.0	.020	100

TABLE 3
OPTIMIZATION RESULTS

Storm No.	% Imperv. [1]	First Run TC+R	First Run R/(TC+R)	First Run STRKR	First Run RTIOL	First Run ERAIN	Second Run[2] TC+R	Second Run[2] R/(TC+R)	Final Study[2,3] TC+R	Final Study[2,3] STRKR Storm	Final Study[2,3] STRKR Station
1a	35	1.68	0.78	0.47	1.90	0.62	1.71	.79	1.77	0.50	.55
1b	.35	2.14	.069	.071	1.90	0.62	2.12	.68	2.50	0.60	
2a	21	5.81	0.83	0.20	1.00	0.82	6.14	.86	5.56	0.32	.32
2b	28	2.93	0.54	0.32	1.00	0.82	2.92	.52	2.89	0.32	
3a	9.1	12.01	0.53	0.55	2.71	0.64	12.2	.53	13.7	0.56	
3b	20	8.65	0.70	0.28	2.71	0.64	8.62	.70	8.39	0.33	.37
3c	21	10.55	0.59	0.20	2.71	0.64	10.2	.55	10.5	0.21	
4a	35	6.79	0.76	0.48	3.88	0.51	6.55	.73	6.36	0.53	
4b	35	4.92	0.74	1.57	3.88	0.51	4.89	.75	4.77	1.92	1.02
4c	35	5.96	0.62	0.60	3.88	0.51	5.85	.63	5.84	0.62	
5	21	9.36	0.43	0.47	5.23	0.12	10.0	.53	11.0	0.50	0.50
6	15.8	7.75	0.97	0.81	2.11	1.00	7.57	.96	7.53	0.95	0.95
7a	19.3	8.70	0.97	0.52	1.73	0.87	9.24	.97	7.60	0.63	0.62
7b	19.3	5.54	0.81	0.63	1.73	0.87	6.13	.85	5.42	0.62	
8a	24.5	9.43	0.63	0.97	2.66	0.39	9.72	.65	10.2	0.98	0.73
8b	24.5	7.59	0.58	0.44	2.66	0.39	7.80	.58	8.38	0.48	
9a	35	1.68	0.67	0.92	3.26	0.80	1.8	.81	1.92	0.94	0.84
9b	35	1.84	0.59	0.79	3.26	0.80	1.87	.67	1.93	0.73	
10a	28	7.00	0.70	0.13	1.88	0.65	7.01	.70	6.86	0.12	0.34
10b	28	6.70	0.66	0.51	1.88	0.65	6.68	.64	6.65	0.56	
11a	19.3	12.86	0.84	0.02	3.20	0.42	13.2	.80	11.5	0.04	0.45
11b	24.5	9.08	0.69	0.84	3.20	0.42	8.65	.66	9.21	0.86	
12a	21	20.84	0.72	0.08	2.92	0.50	20.5	.70	20.4	0.08	

TABLE 3
OPTIMIZATION RESULTS

Storm No.	% Imperv. [1]	First Run TC+R	First Run R/(TC+R)	First Run STRKR	First Run RTIOL	First Run ERAIN	Second Run[2] TC+R	Second Run[2] R/(TC+R)	Final Study[2,3] TC+R	Final Study[2,3] STRKR Storm	Final Study[2,3] STRKR Station
12b26.3	19.64	0.42	0.59	2.92	0.50	19.3	.43	21.4	0.59	0.52	
12c	26.3	15.87	0.56	0.89	2.92	0.50	16.0	.59	17.4	0.90	
13a	24.5	13.17	0.97	0.14	1.08	0.89	13.2	.97	14.0	0.16	0.39
13b	24.5	6.33	0.50	0.59	1.08	0.89	6.17	.48	7.74	0.62	
14a	29.8	9.58	0.83	0.02	4.64	0.71	9.61	.82	9.83	0.04	0.41
14b	29.8	6.23	0.60	0.82	4.64	0.71	5.97	.57	6.59	0.78	
15a	35	3.62	0.84	0.31	2.07	0.92	3.85	.87	3.76	0.29	
15b	35	5.68	0.77	0.00	2.07	0.92	5.67	.77	5.57	0.00	0.28
15c	35	2.82	0.91	0.49	2.07	0.92	2.85	.91	2.69	0.55	
16	33.3	12.10	0.54	0.15	2.00	0.48	12.0	.51	12.9	0.13	0.13
17	3.5	-	-	-	-	-	21.2	.80	20.8	0.37	0.37

1 Assumes % Imperv. = 0.35 x % Development for the storm date.
2 Using RTIOL = 3.0 and ERAIN = 0.6 for all storms.
3 Using TC/(TC+R) = 0.38 log S_0 (See Figure 1).

TABLE 3A
FINAL VALUES ADOPTED
FOR OTHER STATIONS

Station No.	TC+R	R/(TC+R)
18	27	.3
19	14.5	.8
20	15.0	.5
21	26.5	.5
22	27.0	.5
23	4.8	.2
24	4.0	.5
25	40	.3
26	36	.3
27	63.8	.2
28	0.90	.9
29	1.40	.8

Reference: "Harris County Flood Hazard Study Final Report", dated September 1984 by TC&B and Pate Engineers, Inc.

TABLE 4
ANALYSIS OF VARIANCE – LOSS COEFFICIENT

Station	Values of X (STRKR) Storm a	Storm b	Σ X	$\sum(X^{2)}$	$\sum X^2 - \frac{(\sum x)^2}{2}$
1	0.50	0.60	1.1000	0.6100	0.00500
2	0.32	0.32	0.6400	0.2048	0.0000
3	0.56	0.33	0.89	0.4225	0.02645
4	0.53	1.92	2.4500	3.9673	0.96605
7	0.63	0.62	1.2500	0.7813	0.00005
8	0.98	0.48	1.4600	1.1908	0.12500
9	0.94	0.73	1.6700	1.4165	0.02205
10	0.12	0.56	0.6800	0.3280	0.09680
11	0.04	0.86	0.9000	0.742	0.33620
12	0.08	0.59	0.6700	0.3545	0.13005
13	0.16	0.62	0.7800	0.4100	0.10580
14	0.04	0.78	0.8200	0.6100	0.27380
15	0.29	0.00	0.2900	0.0841	0.04205
			13.6000	11.1210	2.12930

Total variance = 11.1210 – $(13.60)^2$ .26) = 4.0072

Average variance between storms = 2.1293/13 = 0.1638

Average variance between stations = (4.0072 – 2.1293)/12 = 0.1565

Average value of STRKR = 13.60/26 = 0.52

TABLE 5
COMPARISON OF 100-YEAR DISCHARGE COMPUTATIONS

		METHODOLOGY					
				U.S.G. WRI 80-17			
U.S.G.S. ID No.	D.A $(mi)^2$	Ft Bend County	Harris County	Frequency Analysis from observed data	Frequency Analysis with simulated data	Regional Equation	USGS WRI 3-73 Frequency Analysis with Simulated Data
0807 4250	11.4	10,100	8,200	8,210	6,500	4,600	7,110
0807 4500	86.3	26,600	33,800	23,600	23,960	27,550	22,600
0807 4780	7.5	2,000	3,300	1,250	870	1,400	600
0807 4800	11.5	3,860	4,780	1,880	1,740	2,190	1,790
0807 5000	94.9	32,300	39,100	40,600	33,700	36,670	20,700
0807 5400	20.2	8,730	7,430	5,680	5,590	6,330	5,750
0807 5500	63.0	15,600	17,410	16,140	15,300	14,400	16,300
0807 5550	2.56	1,900	-	1,000	870	1,000	880
0807 5650	10.7	7,800	6,230	8,280	6,020	6,220	7,570
0807 5730	7.3	4,520	8,400	4,620	5,000	3,850	-
0807 5770	15.8	5,770	4,650	4,900	4,910	5,820	5,930

Note: The drainage area indicated for each location is that as determined for the Fort Bend County analysis and in some instances may differ from previously published data. A slight variation in discharge may occur attributable to the differences in drainage area.

TABLE 6
COMPARISON OF 100-YEAR DISCHARGE (cfs) FOR DIFFERENT METHODOLOGIES

			Methodology			
Watershed	Drainage Area (mi^2)	Percent Developed	Fort Bend County	Harris County	Cypress Creek	Johnson-Sayre Nomograph
Clear Creek	1.61	10	597	610	501	400
	3.99	7	1662	1282	899	800
	6.71	8	1433	1213	1395	1450
Keegans Bayou	0.43	75	464	334	260	330
	1.16	30	578	273	450	500
	3.41	80	1830	914	1335	2000
	4.93	75	3134	4034	1689	2670
	5.50	70	3598	5034	1782	3000
Long Point Slough	0.44	0	174	183	164	80
	1.49	5	530	740	454	300
	3.72	0	1298	1540	857	500
	10.66	0	2469	2592	1827	1350
Willow Fork	0.43	25	254	192	192	200
	1.25	0	782	682	385	190
	5.01	0	1665	1420	1062	650

APPENDIX B

(see http://www.tceq.state.tx.us/compliance/field_ops/dam_safety/damsafetyprog.html)

INDEX

DAMS AND RESERVOIRS

Subchapter A
General Provisions
§§299.1-299.5

These new sections are adopted under the Texas Water Code, 12.052, which provides that the Texas Water Commission shall adopt any regulations necessary to provide for the safe construction, maintenance, repair and removal of dams located in this state.

§299.1. Definitions. The following words and terms, when used in this chapter shall have the following meanings, unless the context clearly indicates otherwise:

Dam – Any barrier, including one for flood detention, designed to impound liquid, volumes and which has a height of dam greater than six feet. This does not include highway, railroad or other roadway embankments, including low water crossing that may temporarily detain floodwater, levees designed to prevent inundation by floodwater, closed dikes designed to temporarily impound liquids in the event of emergencies, or off-channel impoundments authorized by the commission in accordance with Texas Water Code, Chapter 26, or the Texas Solid Waste Disposal Act, Texas Civil Statutes Article 4477-7.

Effective crest of the dam – The elevation of the lowest point on the crest of the dam excluding spillways.

Existing Dam –

(A) Any dam constructed in accordance with necessary authorizations of the commission;

(B) Any existing dam exempt under Texas Water Code §11.142;

Height of dam – The vertical distance from the effective crest of the dam to the lowest elevation on the centerline or downstream toe of the dam including the natural stream channel.

Maximum storage capacity – The volume of the impoundment created by the dam at the effective crest of the dam, usually expressed in acre-feet.

Normal storage capacity – The volume of the impoundment created by the dam, at the lowest controlled spillway crest, usually expressed in acre-feet.

Probably maximum flood (PMB) – The flood magnitude that may be expected from the most critical combination of meteorologic and hydrologic conditions that are reasonable possible for a given watershed.

Probable maximum precipitation (PMP) – Theoretically the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year.

Proposed Dam – Any dam, constructed or to be constructed, which is not included in the definition of Existing Dam.

Spillway design flood (SDF) – The flood criteria that needs to be considered in the design of a proposed project.

Spillway evaluation flood (SEF) – The flood criteria that needs to be considered in the hydrologic evaluation of an existing structure.

§299.2. General.

(a) When the executive director finds that a dam or reservoir poses a level of danger to the public which is unacceptable when evaluated in accordance with commission rules, he may either refer the matter directly to the attorney general for injunctive relief or he may seek an order from the commission to direct the owner to take appropriate action to remove the danger to life and property. An owner who willfully fails or refuses to take appropriate action is liable for a penalty of not more than $1,000 a day for each day the violation continues.

(b) In determining whether an existing or proposed dam and reservoir constitutes an unacceptable danger to life or property, the commission shall evaluate both the hydrologic and, if possible, the structural adequacy of the dam. The commission may take into consideration conditions, including but not limited to, the possibility that the dam might be endangered by overtopping, seepage, piping, settlement, erosion, cracking, earth movement, uplift, overturning, or failure of bulkheads, flashboards, gates, spillways and conduits.

(c) Dams and associated facilities must be adequately maintained throughout their lives, including as necessary, the operation and maintenance of surveillance and monitoring devices to detect changes in the dam and/or its foundation and appurtenant facilities. If abandoned at any time, a dam must be removed or breached in a manner to eliminate any hazard to life and property downstream.

(d) Dam and spillway adequacy shall be evaluated utilizing standard engineering procedures and techniques including, but not limited to, those employed and recommended by the Corps of Engineers, Soil Conservation Service, Bureau of Reclamation, and the American Society of Civil Engineers.

§299.3. Duties, Obligations and Liabilities of Dam Owners. Nothing in these rules or orders made by the commission shall be construed to relieve an owner or operator of a dam or reservoir of the legal duties, obligations or liabilities incident to ownership or operation.

§299.4. Registered Engineer. Preparation of all plans and specifications, and the construction, enlargement, alteration, repair or removal of dams subject to commission review shall be under the supervision of an engineer registered in this state, unless a waiver of this requirement is authorized pursuant to §299.5 of this title (relating to Exception).

§299.5. Exception. Written approval of the executive director is required for exception from any or all of the requirements of §299.4 of this title (relating to Registered Engineer), §299.22 of this title (relating to Approval of Plans and Specifications), §299.23 of this title (relating to Content of Construction Plans and Specifications), §299.24 of this title (relating to Maintenance of Records), §299.25 of this title (relating to Construction Progress Report), §299.26 of this title (relating to Construction Inspection), §299.27 of this title (relating to Plan and/or Specification Changes and Amendments), §299.28 of this title (relating to Noncompliance with Approved Plans and Specifications), §299.29 of this title (relating to Deliberate Impoundment), and §299.31 of this title (relating to As-built Drawings and Permanent Reference Mark). The executive director may grant exception if he determines that the physical conditions involved, when evaluated using standard engineering procedures and techniques, render the requirements unnecessary. Written approval will specify the extent of the exception granted and the executive director's reasons for granting it. This rule does not limit the executive director's authority under §299.27 of this title (relating to Plan and/or Specification Changes and Amendments) to require amendments, modifications or changes to ensure the safety of a structure.

Subchapter B
Design and Evaluation of Dams
§§299.11-299.18

These new sections are adopted under the Texas Water Code, 12.052, which provides that the Texas Water Commission shall adopt any regulations necessary to provide for the safe construction, maintenance, repair and removal of dams located in this state.

§299.11. Classification of Dams. All dams will be classified or reclassified as necessary to assure appropriate safety considerations. The three size classifications (small, intermediate and large), based on height of dam or impoundment capacity, and the three hazard classifications (low, significant and high), are combined to indicate a dam's downstream hazard potential. Thus, the classification assignment reflects the hazard potential associated with assumed failure of the dam. For example, dams located such that resulting failure could be catastrophic are classified so as to require a higher degree of design consideration than would be required for similar dams located in remote areas. Classification does not indicate the physical condition of a dam.

§299.12. Size Classification Criteria. The classification for size based on the height of the dam or maximum reservoir storage capacity, shall be in accordance with Table 1 of this subsection. The appropriate size is the largest category determined for either storage or height.

TABLE 1
SIZE CLASSIFICATION

Category	Impoundment Storage (Ac-Ft)	Height (Ft.)
Small	Less than 1000	Less than 40
Intermediate	Equal to or Greater than 1000 & Less than 50,000	Equal to or Greater than 40 & less than 100
Large	Equal to or Greater than 50,000	Equal to or Greater than 100

§299.13. Hazard Classification Criteria. The hazard potential classification shall be in accordance with Table 2 of this subsection. Hazard classification pertains to potential loss of human life and/or property damage within either existing or potential developments in the area downstream of the dam in event of failure or malfunction of the dam or appurtenant facilities. Hazard classification does not indicate any condition of the dam itself. Dams in the low hazard potential category are normally those in rural areas where failure may damage farm buildings, limited agricultural improvements and county roads. Significant hazard potential category dams are usually those in predominantly rural areas where failure would not be expected to cause loss of human life, but may cause damage to isolated homes, secondary highways, minor railroads, or cause interruption of service or use (including the design purpose of the facility) of relatively important public utilities. Dams in the high hazard potential category are usually those in or near urban areas where failure would be expected to cause loss of human life, extensive damage to agricultural, industrial, or commercial facilities, important public utilities (including the design purpose of the facility), main highways or railroads.

TABLE 2
HAZARD POTENTIAL CLASSIFICATION

Category	Loss of Human Life	Economic Loss
Low	Non expected (No permanent Structures for human habitation	Minimal (Undeveloped to occasional structures or agricultural improvements)
Significant	Possible, but not expected (A small number of inhabitable structures)	Appreciable (Notable agricultural, industrial or commercial development)
High	Expected (Urban development or large number of inhabitable structures)	Excessive (Extensive public, industrial, commercial or agricultural development)

§299.14. Hydrologic Criteria for Dams.

(a) The hydrologic criteria contained in Table 3 are the minimum acceptable spillway design flood (SDF) for proposed dams as defined in §299.1 of this title (relating to Definitions), including those to be constructed in accordance with Texas Water Code, §11.142.

(b) Exemptions to Minimum Hydrologic Criteria – Proposed low hazard dams exempt under Texas Water Code, §11.142 are exempt from the minimum criteria. Any other proposed structure may be exempt from the minimum criteria if properly prepared dam breach analyses show that existing downstream improvements are known or planned future improvements will not be adversely affected. A properly prepared breach analysis should include at least three events, the normal storage capacity non-flood event, the barely overtopping event and the PMF event. Data on additional flood magnitudes may be provided as necessary to document other conditions or conclusions. Downstream flooding differentials of one-foot or less between breach and non-breach simulations are not considered to be adverse.

TABLE 3
HYDROLOGIC CRITERIA FOR DAMS

Classification		Minimum Flood Hydrograph
Hazard	Size	
Low (No. 3)	Small	¼ PMF
	Intermediate	¼ PMF to ½ PMF
	Large	PMF
Significant (No. 2)	Small	¼ PMF to ½ PMF
	Intermediate	½ PMF to PMF
	Large	PMF
High (No. 1)	Small	PMF
	Intermediate	PMF
	Large	PMF

NOTE: The flood hydrograph in this table is the minimum required flood for a given project, i.e., the project will be required to safely pass this hydrograph. Where a range is given, the minimum flood hydrograph will be determined by straight line interpolation within the given range. Interpolation shall be based on either hydraulic height or impoundment size (§229.12, Table 1 of this title (relating to Size Classification Criteria), whichever is greater. The minimum flood hydrograph is computed as a percentage of the PMB hydrograph.

§299.15. Evaluation of Existing Dams.

(a) Existing dams, as defined in §299.1 of this title (relating to Definitions), are subject from time to time to reevaluation in consideration of continuing downstream development. Hydrologic criteria contained in §299.14, Table 3 of this title (relating to Hydrologic Criteria for Dams) are the minimum acceptable spillway evaluation flood (SEF) for reevaluating dam and spillway capacity for existing dams to determine whether upgrading is required. Dams not meeting minimum criteria are considered to be blow acceptable limits and are subject to action as necessary under §299.2 of this title (relating to General).

(b) Exemptions from Minimum Hydrologic Criteria – Existing low hazard dams are exempt from the minimum hydrologic criteria as given in table 3 and any other existing structure may be exempt from the minimum hydrologic criteria is properly prepared dam breach analyses show that existing downstream improvements are known or planned future improvements will not be adversely affected. A properly prepared breach analysis should include at least three events, the normal storage capacity non-flood event, the barely overtopping event and the PMF event. Data on additional flood magnitudes may be provided as necessary to document other conditions or conclusions. Downstream flooding differentials of one-foot or less between breach and non-breach simulations are not considered to be adverse.

(c) Structural Evaluation – Evaluating the structural condition of an existing dam includes, but is not limited to, visual inspections and evaluations of potential problems such as seepage, cracks, slides, conduit and control malfunctions and other structural and maintenance deficiencies which could lead to failure of a structure. An active and progressive deteriorating condition is sufficient for a finding that an existing dam is structurally inadequate.

§299.16. Interim Alternatives. At the time the commission considers the permanent upgrading or removal of an inadequate dam, the dam owner may request the commission to consider interim alternatives including but not limited to temporary repairs, reservoir dewatering, insurance coverage, and/or downstream warning and evacuation plans. Consideration shall be given to the time required to overcome economic, physical and legal restraints to upgrading, the prospect of permanent repair, current use of the facility, degree or risk and public welfare.

§299.17. Emergency Management. As required for emergency management planning, the executive director may request, and/or the commission may order a dam owner to provide sufficient data to plan for potential effects of failure or malfunction of a dam and/or associated appurtenant facilities.

§299.18. Variance. The owner of an existing dam that does not meet the hydrologic criteria of §299.14, Table 3 of this title (relating to Hydrologic Criteria for Dams) may request the commission to consider a variance from this criteria, based upon but not limited to the owner's evaluation of the consequences of potential dam failure, proposals to reduce potential hazard, and/or the economic and physical limitations to upgrading.

Subchapter C
Construction Requirements
§§299.21-299.31

These new sections are adopted under the Texas Water Code, §12,052, which provides that the Texas Water Commission shall adopt any regulations necessary to provide for the safe construction, maintenance, repair and removal of dams located in this state.

§299.21. Applicability. This subchapter applies only to engineering plans and specifications for the construction, enlargement, repair, or alteration of dams requiring commission authorization, except as follows:

(1) Exceptions approved in accordance with §299.5 of this title (relating to Exception);

(2) Dams designed by and constructed under the supervision of federal agencies such as the Corps of Engineers, Bureau of Reclamation and the Soil Conservation Service.

§299.22. Approval of Plans and Specifications. Construction of a dam or the enlargement, repair, or alteration of an existing dam requiring commission authorization shall not be commenced prior to the executive director's written approval of final construction plans and specifications. Construction plans and specifications shall be submitted to the executive director and shall be as completely detailed as necessary for submission to the contractors bidding on the proposal. Contractors shall not commence construction until provided with a copy of the plans and specifications evidencing the approval. This does not apply to ordinary maintenance or emergency repair. The executive director may require the filing of additional information and data which, in his opinion, may be necessary for determining the adequacy of operational functions and safety of the structures and works related thereto. The official name of the dam and reservoir by resolution of the governing body or by certificate if individually owned, shall be submitted to the department as early as possible, preferably with the construction plans.

§299.23. Content of Construction Plans and Specifications.

(a) Construction plans requiring approval by the executive director may include the following, as determined by the executive director:

(1) A topographic map of the dam site with contour intervals of not to exceed five (5) feet. A plan of the dam shall be superimposed on this map showing the location of spillways, outlet conduit, cutoff walls and other structures;

(2) A profile of the dam site taken on the long axis of the dam and a profile of each spillway along its long axis. The profile shall also show the location of the outlet conduit and spillway. A log showing the classification of materials encountered below the surface as shown by test pits or borings should be included;

(3) A cross section of the dam at maximum section showing complete details and dimensions;

(4) Detailed plans showing sections of outlet conduits, control works and spillways. These sections should be of sufficient number and detail to delineate clearly all features of the structure; and

(5) The location of all permanent instrumentation shall be shown on the plans. All pressure cells, settlement plates, piezometers, slope indicator casing or other devices shall be noted.

(b) Construction plans shall be accompanied by specifications which may include, but are not limited to the following:

(1) The requirements for the various types of materials to be used in the constructions of all pertinent works;
(2) A specified time of completion, i.e., a requirement that the contractor's bid contain a time of completion;
(3) A provision to the effect that plans and specifications shall not be substantially or materially altered without prior written approval of the executive director.

(c) Other engineering reports and additional information are sometimes prepared and may be required by the executive director for review. These reports, applicable to the type of structure (earthfill, rockfill or concrete) in question, may include details such as geology of the project site and vicinity, location and logs of test borings, pits and shafts, results of field and laboratory tests on structural and foundation materials; seepage studies, and stability analyses of embankments, spillways, retaining walls, etc. Additional information required may include recommendations concerning embankment slopes, crest width, berms, core trench depths, moisture-density and strength requirements, minimum compressive strength for concrete, construction sequence procedures and/or techniques for excavations and embankments, and types of compaction equipment, borrow excavation techniques and sequence of fill placement.

§299.24. Maintenance of Records.

(a) The owner shall continuously maintain records to insure compliance with the approved plans and specifications during construction. Copies of these records shall be furnished to the executive director at monthly intervals during the construction period, and may include but not necessarily be limited to such items as soil moisture-density test results, and concrete trial batch designs test and compression test results.

(b) Other observations which may be recorded include final bottom width and elevations of core and cutoff trenches, structural excavations, permanent sheet piles or bearing piles, and documentation of foundation groutings, dewatering problems, or observations during the construction period of any instruments installed to measure movements, stresses and pore pressure.

§299.25. Construction Progress Report. Within 10 days after beginning actual construction of a project, the executive director shall be notified in writing of the date work began. Thereafter, monthly reports of progress shall be forwarded to the executive director by the 10th of each month during construction. The report shall show the work accomplished during the month, the percent of time used and the percentage of completion of the project as of the close-out date of the report. In addition, the report shall show the inclusive dates of the reporting period.

§299.26. Construction Inspection. Inspection of construction work shall be conducted by a registered professional engineer experienced in the construction of dams and responsible directly to the owner. Continuous daily inspections shall be made and may be delegated to a qualified technician (inspector) provided he is under the supervision of the owner's engineer. The executive director may make periodic inspections for the purpose of ascertaining compliance with approved plans and specifications. The executive director shall require the owner, at his expense, to perform the work or tests necessary and to disclose information sufficient to enable the executive director to determine that conformity with approved plans and specifications is accomplished.

§299.27. Plan and/or specification Changes and Amendments. If after inspection, investigation or examination, or at any time as the work progresses, the executive director finds that changes or amendments are necessary to insure safety, he may request the owner to revise his plans and/or specifications. Alterations of the plans and specifications must be approved by the executive director before work commences under the changes, except in emergencies requiring immediate action of which the executive director shall be immediately notified. If the proposed alterations would result in deviation from the permitted right, amendment of the permit must be obtained from the commission.

§299.28. Non-Compliance with Approved Plans and specifications. If at any time during construction, enlargement, repair, or alteration of any dam or reservoir the executive director finds that the work is not being done in accordance with approved plans and specifications or in accordance with approved revised plans and specifications, he shall give written notice thereof and direct compliance by certified mail to the owner. If the owner fails to comply with the directive, the executive director may take appropriate action to assure compliance. Failure to comply with approved plans and specifications will be grounds for revocation of the permit and/or civil penalty as provided by law. The commission may order the structure removed to eliminate any safety hazard to life and property.

§299.29. Deliberate Impoundment. Written approval of the executive director must be obtained prior to deliberate impoundment of water in a partly or newly completed reservoir designed to impound more than 100 acre-feet at normal storage capacity. Deliberate impoundment shall mean any act which results in the intentional impoundment of water in the reservoir and includes but is not limited to closure of the lowest planned outlet or spillway serving the reservoir, blocking the diversion works used during the construction and beginning backfill within the closure section of a dam. Temporary closing of a valve or spillway gate for operational testing shall not be construed as an act of deliberate impoundment.

§299.30. Certificate of Completion. Immediately upon completion of a new dam and reservoir, or enlargement, repair or alteration of an existing dam and reservoir, the owner shall file a certificate with the executive director, signed by the responsible engineer supervising the work for the owner, certifying that, to the best of the engineers knowledge, the construction, alterations, or repairs were completed in accordance with the approved plans and specifications. In the case of projects excepted under §299.5 of this title (relating to Exception), the owner shall notify the executive director in writing that construction, alterations, or repairs were completed.

§299.31. Record Drawings and Permanent Reference Mark. As soon as possible after completion of construction, the owner or his engineer shall submit to the executive director a complete set of record drawings of the project for filing with the permanent records of the department. One or more permanent reference mark(s) shall be established for future use near but separate from the project. Accurate location(s) and elevation(s) above mean sea level shall be shown on the record drawings.

Subchapter D
Removal of Dams
§299.51

This new section is adopted under the Texas Water Code, §12.052, which provides that the Texas Water Commission shall adopt any regulations necessary to provide for the safe construction, maintenance, repair and removal of dams located in this state.

§299.51. Removal of Dams and Reservoirs. Removal or modification of a dam shall be done at the owner's expense, and except for emergency action required to protect lives and property, only after executive director approval. The executive director may require the owner to provide plans and specifications. The executive director may seek an order from the commission or an injunction through the attorney general requiring the removal or modification of dams and reservoirs which are not authorized by law or which have been determined to pose an unacceptable hazard to downstream lives or property.

Subchapter E
Emergency Action
§299.61

This new section is adopted under the Texas Water Code, §12.052, which provides that the Texas Water Commission shall adopt any regulations necessary to provide for the safe construction, maintenance, repair and removal of dams located in this state.

§299.61. Emergency Action. Pursuant to the provisions of Texas Water Code, §12.052, emergency orders may be issued, without notice to the owner, directing the owner of a dam to take immediate and appropriate action to remedy situations posing serious threat to human life, health, and/or property.

APPENDIX C

DEVELOPMENT
OF
PUMP STATION DESIGN CRITERIA

INTRODUCTION

To prevent flooding within leveed areas, pumps are recommended (instead of only providing storage) to handle interior drainage when the exterior river stage impedes gravity outflow. To determine the required pumping capacity and, therefore, the pump size needed for the pump station, design criteria were developed. One of the major components of the design criteria centers around the rainfall amount to be used in sizing the pump station that would be required to remove interior drainage during high water levels outside of the leveed area that submerge the gravity outlet.

The normal design criteria for large-scale drainage facilities (e.g. open channels, detention facilities) is based on the rainfall for the 100-year storm event. Large-scale interior drainage facilities for leveed areas should also be designed to the same standard (i.e. a condition that will be equaled or exceeded on the average only once in 100 years). For leveed areas providing flood protection from creeks and bayous (excluding the Brazos River), the drainage facilities (pumping and storage capacities) would need to be designed to handle the excess rainfall from a 100-year storm within the interior area, since the exterior creek/bayou would probably be at its 100-year flood stage as a result of the same storm event and thereby restrict any gravity outflow from the leveed area. For leveed areas providing flood protection from the Brazos River, however, the 100-year flood stage on the Brazos does not result from the same storm event as the 100-year rainfall over the leveed area. Thus, there are two different circumstances that would produce a critical design condition for interior drainage facilities. The first condition would be a 100-year storm event over the leveed area when the Brazos River water surface elevation is low, and the second condition would be a storm event over the leveed area when a large flood on the Brazos River is occurring that would restrict gravity outflow from the leveed area. Since these two conditions would occur from different storm events, the drainage facilities for the leveed area need to be designed so that the overall design capacities are equaled or exceeded on the average only once in 100 years. Thus, the pumping and storage facilities need to be designed for one storm event, and the channels under gravity flow conditions need to be designed for another storm event, so that together, the design would only be exceeded on the average once in 100 years. It was determined that a good combination for the pump station and gravity outlet designs would provide for exceedance frequencies of once in 1,000 years during high river states and nine times in 1,000 years during other periods, thus yielding a combined exceedance frequency of once in 100 years. This meant that the pump station and storage facilities would be designed to handle a coincidental probability of occurrence between high river stages and an event having local rainfall of once in 1,000 years. Also, the gravity outlet and internal channel system should be designed to handle the storm event with an exceedance frequency of nine times in 1,000 years; however, it was decided that a comparable design would be to design the gravity system for the 100-year storm event with one foot of freeboard.

DESIGN CRITERIA

The development of the pump station design criteria centers around establishing the percentage of time that a critical flood level on the Brazos River at the leveed area and a particular rainfall event over the leveed area occurring coincidentally will produce a frequency of occurrence that, together with the gravity outlet design criteria, would result in the entire design of the leveed area's drainage facilities being exceeded on the average once in 100 years. To establish this coincidental probability, the percent of time that a flood level on the Brazos River is equaled or exceeded needs to be related to the chance of occurrence of a storm event at the same time over the leveed area.

A flow-duration analysis for the Brazos River was performed using the flow records from the U.S.G.S. gauging station at Richmond in Fort Bend County. This analysis produced a curve showing the percent of time that a particular flow on the Brazos River has been equaled or exceeded based on the period of recorded data (approximately 67 years). Unfortunately, since the recorded data does not have any daily flow values greater than 125,000 cfs, this analysis indicates this value is never exceeded. However, the 100-year computed discharge value for the Brazos River is 181,000 cfs at Richmond. Therefore, the 10-, 50- and 100-year flood hydrographs generated at Richmond for the Fort Bend County Flood Insurance Study (dated 1986) were analyzed to produce some data at these higher flow values. Since such floods last almost a week at a flow within about five percent of the peak flow, a percent of time was determined that showed these flows would be equaled or exceeded during one week every 10-, 50- and 100-years, respectively. These percentages were used for adjusting the flow –duration curve to better represent the higher flow values. (See Figure 1).

The lower flow values from this curve also presented some concern, since the assumption of independency between the two events (i.e. Brazos River flooding and rainfall over leveed area) may not hold true for low flows on the Brazos River. Therefore, an analysis was made of actual recorded rainfall amounts at rain gages located in and around Fort Bend County during days when the Brazos River was flooding at or above 70,000 cfs (selected as approximately bankfill conditions). Discharge records for the USGS gage at Richmond (1923 to 1984) were used in the study along with three National Weather Service Cooperative rainfall gages. These rain gages included: Angleton (1923 to 1984), Sealy (1923 to 1984), and Thompson (1958 to 1984).

The discharge records were reviewed and the data and magnitude of all instances of flow above 70,000 cfs were documented. Daily rainfall totals were noted for the same dates and were tabulated, along with a 5-day antecedent period prior to each extreme flow event, in order to establish the correlation between Fort Bend County rainfall and Brazos River discharges. The total rainfall for the period of record was determined for each rain gage along with a total amount occurring simultaneous with Brazos River discharges above 70,000 cfs. Incomplete or mission data were replaced with estimates of rainfall using the other gages, as appropriate. From this, a daily percent of occurrence was found for each rain gage: Angleton 1.2%, Sealy 0.9%, and Thompson 1.05%. The TP-40 rainfall frequency could then be adjusted by these percentages of occurrence (exceedance) for each gage. Thus, the 1.2% exceedance for Angleton equates to a 1.2-year rainfall frequency to represent the 100-year coincidental event while the Brazos River flow is above 70,000 cfs. The corresponding rainfall amount for a 1.2-year storm event would be determined from the rainfall frequency curves developed from TP-40. Similarly, any coincidental frequency event could be computed. (For example, the 1,000-year event would be a 12-year rainfall event using the Angleton gage.) Similar TP-40 adjustment analyses were conducted for the same 3 rain gages based on discharges in the Brazos River exceeding 80,000 cfs, 90,000 cfs, and 100,000 cfs. The following table shows the percentages of occurrence for each gage for each discharge category:

Percentage of Rainfall Occurring
With Discharge (cfs) above:

Rain Gage	70,000	80,000	90,000	100,000
Angleton	1.2	0.36	.05	.01
Sealy	0.9	0.25	.06	.02
Thompsons	1.05	0.38	.02	0

The results of this analysis indicated the percent of rainfall that would occur during the time that the Brazos River had a flow of 70,000 cfs or greater in Fort Bend County. This analysis does not assume that the occurrence of rainfall in Fort Bend County is independent of river stage, since it is based on rainfall percentages actually observed during high river stages. It does assume, however, that the percentage of total rainfall observed is essentially the same as the percentage of high-intensity rainfall that would occur during high river stages. In order to check this assumption, a further study of the three rainfall records was made to determine the percentage of large storms (daily rainfalls exceeding one inch) that occurred historically during high river stages. Results from this further analysis confirmed this assumption.

The results of this percent of rainfall analysis were used along with the flow-duration curve, to develop a curve finally adopted for use in the pump station design criteria as shown on Figure 1, and was based upon the most conservative of the data available from the analyses discussed above. This adopted curve provided the means for generating the coincidental probability relationship between Brazos River flows and the rainfall event over a leveed area to be used in designing the pump station, having an exceedance frequency of once in 1,000 years. In order to simplify the use of these criteria, the curve adopted above was combined with the coincidental frequency curve to produce the curve as shown on Figure 2. It was determined however, that a 1-year frequency storm event should be set as the minimum allowable for designing the pump/storage facilities. Also, the curve should not be extended much below 70,000 cfs since such data were not utilized in deriving these coincidental relationships.

Although these criteria are considered to provide a conservative design, it is not an unacceptable over-design. Previous pump stations in Fort Bend County have used a fixed design rainfall of 7.55 inches in 24 hours, which equates to approximately a 7-year storm event. This compares to the 1-year storm event for highly protected areas through the 11-year storm event for low lying areas located along the Brazos River.

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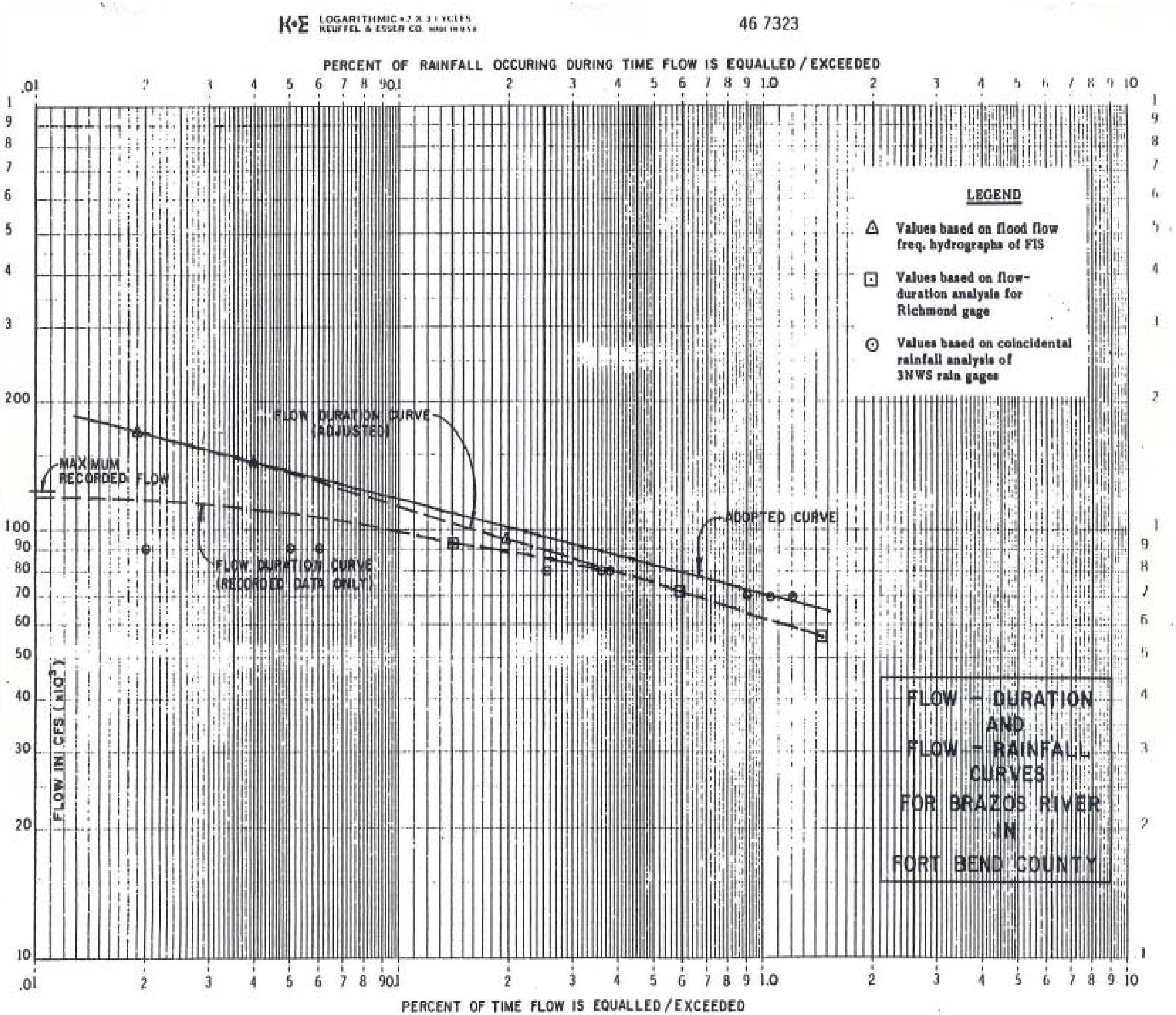

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46 7323
PERCENT OF RAINFALL OCCURING DURING TIME FLOW IS EQUALLED / EXCEEDED
LEGEND
Values based on flood flow freq. hydrographs of FIS
Values based on flow-duration analysis for Richmond gage
Values based on coincidental rainfall analysis of 3NWS rain gages
FLOW DURATION CURVE (ADJUSTED)
MAXIMUM RECORDED FLOW
ADOPTED CURVE
FLOW DURATION CURVE (RECORDED DATA ONLY)
FLOW IN CFS (x10³)
FLOW - DURATION AND FLOW - RAINFALL CURVES FOR BRAZOS RIVER IN FORT BEND COUNTY
PERCENT OF TIME FLOW IS EQUALLED / EXCEEDED