Wharton County Drainage Criteria Manual

Prepared by

HALFF

4030 West Braker Lane
Suite 450
Austin, TX 78759

Prepared for Wharton County, Texas
December 2008

WHARTON COUNTY DRAINAGE CRITERIA MANUAL
TABLE OF CONTENTS

Title **Page**

Title	Page

List of Tables

List of Figures

Appendices

CHAPTER 1 – Introduction

Section 1.1 – Purpose and Authority

Section 1.1.1 – Purpose

The purpose of this drainage manual is to establish standard principles and practices for the design and construction of drainage systems within Wharton County. The design factors, formulae, graphs and procedures are intended for use as engineering guides in the solution of drainage problems involving determination of the quantity, rate of flow, method of collection, storage and conveyance of stormwater.

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

Section 1.1.2 – 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 Wharton 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 Wharton County. This was especially true of Harris County's Criteria Manual for the Design of Flood Control and Drainage Facilities, the Drainage Manual for Fort Bend County, the Drainage Manual for Brazoria County and the Stormwater Design Guidelines for Bryan/College Station, which were used as guides in selecting drainage criteria and in preparing this Criteria Manual for Wharton County. A Storm Sewer Design example and guidelines for riprap and gabion design were excerpted from the City of Fort Worth Design Manual. 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 Wharton County. However, while there was obvious benefit for having consistency in the drainage criteria manuals of adjacent counties, this drainage criteria manual not only had to be an easy-to-use tool for solving drainage problems in Wharton County, but needed to contain standards and methodology that would be applicable to the specific problems and objectives of Wharton County. As a result, certain criteria and methodology were changed from those used in Fort Bend, Brazoria and Harris counties.

To assist design engineers in dealing with these three 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 total (hyetographs) for various events.
5. Drainage area - discharge curves
6. Detention criteria
7. Leveed areas
8. Rural subdivision criteria

Section 1.1.3 – Authority

This manual was developed with the intention that it could be adopted by various governmental entities. Therefore, the generic terms "Drainage Regulation Entity" and "Drainage Review Authority" are used throughout the manual.

Each entity adopting the manual can define those terms in a preface to the manual as appropriate for that entity. For instance, Wharton County would define "Drainage Regulation Entity" as "Wharton County" and "Drainage Review Authority" as "County Engineer".

It is anticipated that additional changes, modifications, additions or deletions will occur and that these items could be added to this manual through applicable means.

Section 1.2 – Policies

Section 1.2.1 – Stormwater Principles

Drainage System — For purposes of regulation, the drainage system shall be divided into geographical and functional groupings. The drainage system consists of all natural and man-made features that collect or receive concentrated stormwater flow. Examples are swales or channels (natural or man-made), streets, storm sewers, minor streams and major streams.

Primary and Secondary — Functional division is separation of the drainage system into its primary and secondary components. The Primary System consists of major streams that convey collected stormwater through Wharton County, including primary tributaries thereof. The Primary System is made up of the watercourses that are part of the FEMA-designated floodplain management network, the geographic limits of which may be amended from time to time by the County or cities. The Secondary System consists of all minor drainage ways, streets, storm sewers, and swales that collect stormwater and convey it to the Primary System. **It should be noted that the Primary System includes both "Primary and Secondary" Watercourses.**

Storm Duration — From a hydrologic standpoint, the Secondary System is sensitive to short duration, high intensity rainfall events. Flood effects occur suddenly and dissipate quickly, usually within a period of a few hours. By contrast the Primary System is sensitive to longer duration, moderate intensity rainfall events. Flood events occur over a longer period, with a slower rise to fall for peak flows and flood elevations. This fundamental difference between the

Primary and Secondary Systems forms the basis for strategies to manage stormwater and its effects within each.

Unique Characteristics Geographical division involves separating the various streams and land areas into broad drainage areas having unique characteristics in terms of land cover, pattern of development, governmental jurisdiction, proposed land uses, and system interconnection. Recognition of these differences allows for logical formulation of policies and standards tailored to specifics rather than generalities.

Known Problems Because the basic reason for regulating stormwater runoff and conveyance is to promote public safety, it must be emphasized that where persistent, known drainage problems exist, criteria more stringent than stated in these Guidelines may be necessary.

Section 1.2.2 – Framework of Stormwater Management Terms

A great variety of terms are used in the science and administration of managing stormwater. To foster clarity and expediency in use of these Guidelines, a limited series of terms has been specially defined. The focus is on the definitions of drainage areas, land proposed for development, and the purposes of detention. The diagram in Figure 1.2.2-1 offers a graphical representation supporting this framework of terms. The principal terms coined below are in bold print in this Section and are capitalized throughout these Guidelines.

1. Watersheds

Every land area in the Wharton County region is in a "**watershed**" of some description, each of which is associated with some kind of watercourse. For managing storm runoff in these areas it is useful to divide these areas according to the watercourses that drain them.

Named Streams For purposes of these Guidelines "**watersheds"** are all of the land areas contributing storm runoff to each of the principal watercourses making up the primary system. The primary system is divided into logical parts that are referred to as the **"Primary Watercourses",** (main channels) and **"Secondary Watercourses"** (major tributaries) presented in Figure A-1 and Table A-1 in Appendix A which are generally the watercourses which have been subjected to either "detailed studies" or "Limited Detailed Studies".

A hypothetical "Primary Watercourse" and the hypothetical watershed ("Watershed A") it drains are sketched in Figure 1.2.2-1.

2. Basins

Tributaries

For purposes of these Guidelines a "**basin**" is defined as the land area drained by a tributary of a "Primary Watercourse". Each "Primary Watercourse" has several tributaries (some possibly having localized names) that serve to help drain the **watershed**. Each **watershed** is made up of several **basins,** and all areas in a **watershed** are considered to be part of one of its **basins.**

Specific Limits

The specific geographic limits of any **basin** are a function of topographic features that can only be determined through engineering study. Such limits must be determined when dictated by the characteristics of a proposed land development project as determined by the Drainage Review Authority or his/her designee during project review processes.

Figure 1.2.2-1 illustrates the **basins** of a hypothetical **watershed**. In this sketch the "Primary Watercourse" has six tributaries, so the **watershed** is considered to have six **basins**. **Watershed** "A" has six identified **basins,** basins 1, 2, 6.

3. Land Development Projects

a. Land Areas

Enhanced Consistency

Land development projects occur in many shapes and sizes in a variety of locations. These Guidelines apply to all proposed projects but their application is a function of numerous variables. To enhance consistency in determining how these Guidelines apply to particular situations, the following land area terms will be used.

Project Area

Project Area: The entire land holding associated with any proposed land development project will be considered the **"Project Area"**. This is to include the largest acreage of any combination of: the entire ownership, the entire parent tract, and/or the entire purchase option acreage, if any. This is true for all contiguously owned tract(s) or lots regardless of whether platted or not platted. It is also irrespective of whether construction (buildings or infrastructure) is planned on portions of the land near term and/or at some future time, however well or poorly defined.

2-Phase Project

In Figure 1.2.2-1 hypothetical Project B is a two-phase project. Stormwater analysis and design for Phase 1 of Project B must consider Phase 2 to be part of the **project area**, even if Phase 2 facilities and/or buildings are planned for future construction. In addition, it must consider any "**Above-Project Area(s)"** and "**Pathway Area(s)**" as described below.

Above-Project Areas

Above-Project Areas: These are any land areas that contribute storm runoff onto or through the **project area**.

In Figure 1.2.2-1 schematic projects A, C, and E all have **“above-project areas”** since upland areas contribute storm runoff to the **project areas**. Schematic projects “B” and “F” may or may not receive runoff from limited upland areas. Schematic Project “D”, in Basin 1, borders the **basin** divide and receives no runoff from upland areas, so it has no **above-project area**.

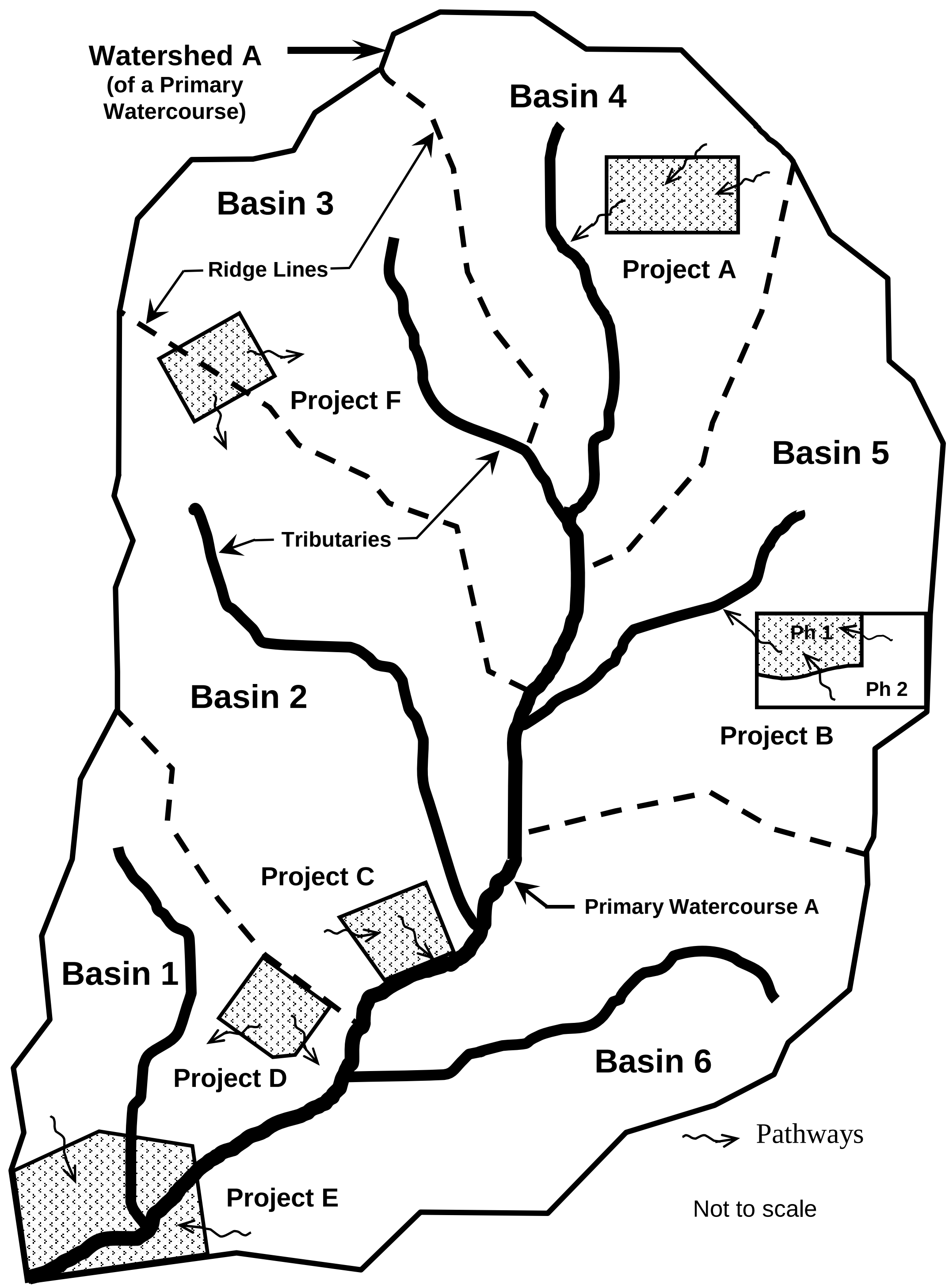


Figure 1.2.2-1: Watershed – Basin – Projects Diagram

Pathway Areas

Pathway Areas: As described in Section 1.2.4, Paragraph 2, "designated conveyance pathways", however simple or complex, must be identified for every land development project. Conveyance pathways downstream of a **project area** may carry runoff from land that is not part of the **project area** or the **above-project area**. Areas discharging to a "conveyance pathway" downstream of the **project area** are considered "**Pathway Areas"**.

Two Basins

In Figure 1.2.2-1 Projects "A", "B", and "D" each include **pathway areas** along the "conveyance pathway" that would extend from the **project area** to the tributary, then to **Watercourse A**. Project "F" straddles the divide between basins, so it will have two "conveyance pathways" and two sets of **pathway areas**, one in each of the two **basins**.

Drainage Study Area

Drainage Study Area: Every project will be considered as having a **"Drainage Study Area"** that is the **project area** at a minimum. As applicable, it may also include **above-project area(s)**, and/or **pathway area(s)**. To be considered complete, a "drainage study" must address all three components of a **drainage study area,** as well as the conveyance pathway itself. If such areas do not exist for a particular project, it shall be so stated in the drainage study report.

Design Drainage Area

Design Drainage Area: Every **drainage study area** will include any number of **"Design Drainage Areas"** that must be analyzed to determine the design storm flow for the purpose of sizing and placing stormwater management facilities of various types. This can vary widely, from a small area draining to a curb inlet, to many acres served by a channel and culvert.

b. Purposes of Detention

Two Purposes

Detention is a useful stormwater management technique. As fully addressed in Section 1.2.3, Paragraph 3, it can be used for managing flood control over a broad area such as an entire **basin** or **watershed**. It can also be used to manage property-to-property conveyance of stormwater.

If low enough in the **watershed**, **Detention** may be unnecessary, possibly even detrimental, to flood control objectives. Moreover, because it can drain directly into the principal watercourse, there may be no need for **Detention**.

Section 1.2.3 – Watershed Management

1. Primary Drainage System

a. Nature of Problems in Primary System

Floodplains

Stormwater problems in the primary drainage system result from floodwaters rising out of the banks of natural streams and inundating adjacent natural floodplains. Symptomatic problems are flooding of building structures, overflow of bridges and culverts hampering traffic access, and damage to public and private infrastructure (utilities, roads, etc).

Problem Causes

Problems in the primary system can be caused by the following:

- Inadequate capacity of crossing structures and failure to allow for overflow.
- Placing the finish elevation of the lowest floor of a structure situated adjacent to the Primary System below the existing or ultimate 100 year flood elevation.
- Inadequate or out-dated engineering studies that form the basis of the regulatory flood elevations.
- Failure to allow for increased discharge from, and resulting flood elevations in, upstream areas.
- Failure to control and limit increased stormwater discharge to downstream areas.
- Improper or ineffective alterations to natural channels that have the effect of “transferring” flood problems to upstream or downstream areas.

Resulting Hazards

The results are creation of hazards to life and damage to public and private properties. Remedial measures usually involve large capital improvements to channelize streams, create large detention facilities, or build larger crossing structures for roadways.

Hydrologic Studies

As a first step to dealing with these problems, the County has adopted comprehensive hydrologic and hydraulic engineering studies for most of the primary system and tributaries thereof. These identify the flood discharge and flood elevations within the primary system, for existing development conditions. Ultimate development conditions have not been currently defined because there is no adopted comprehensive land use plan and significant development is not anticipated in the foreseeable future. Duly adopted flood studies will govern actions and treatments (whether public projects or associated with land development projects) that affect the primary system and its tributaries, consistent with state and federal regulatory requirements.

Minimize Flooding

The policies of the Drainage Regulation Entity are to encourage the efficient conveyance of stormwater through and out of existing and future developed areas within the primary system. The lowest floor of all structures adjacent to the primary system shall be kept at a level above the ultimate 100-year flood level, and no structure will be allowed within the existing 100-year flood path defined as the "floodway." In order to eliminate sporadic and uncoordinated site improvements, modification of the floodway shall be restricted to projects engineered and treated in conformance with a comprehensive master plan established for regulatory channel reaches.

Encroachments

Unless stipulated otherwise in an ordinance or other design guidelines, minor encroachments in the floodway fringe will be allowed for individual sites and developments, provided they are clearly part of a "Drainage Development Permit" approved by the Drainage Regulation Entity. Crossing roadway structures are allowable to include encroachments, provided they are designed to accommodate the range of ultimate design flows through them (or through and over them) to eliminate formation of hazards and damage to private property or public infrastructure.

Regulations

To implement this policy, stormwater management ordinances and design guidelines may be adopted by each Drainage Regulation Entity. Requirements vary along each channel reach to recognize the differences related to development conditions, expected increases in flood elevations, and the potential for damages.

2. Secondary Drainage System

Typical Problems

Stormwater problems in the secondary system tend to be localized and scattered throughout the County. Typically they result from inadequate provision for streets, storm sewers, and collection channels. Examples include: excessive ponding in streets at low points, excessive storm flow through principal street intersections, overflow of streets, undersized drainage easements, facilities requiring excessive maintenance, and restriction of street uses due to excessive storm flow.

Problem Causes

Causes of problems in the secondary drainage system are listed as follows:

- Inadequate capacity for design flows.
- Inadequate allowance for increases in storm flow due to future development.

- No provision for containing and controlling (within designated easements or right of way) the discharge from the 100 year rainfall event.
- Failure to control discharge from new developments that exceeds the capacity of the receiving secondary system, existing or proposed.

Damage or Nuisances

The results are creation of nuisance problems and situations where damage to public and private property can occur. Remedial measures may be very difficult to achieve, and may range from expensive public improvement projects to situations where remedies are infeasible from a practical standpoint.

Drainage By Design

The policy of the Drainage Regulation Entity is to avoid formation of these problems through efforts at the design and development stage. Central to this strategy are the performance standards for drainage design contained in these Guidelines, including the "conveyance pathway" concept for containing the base flood discharge.

Performance Criteria

Based on this policy, performance criteria are set for design rainfall events. The emphasis at the performance level is to mitigate the nuisance aspect of storm drainage. An example of a performance standard would be: "design the street and attendant drainage system to carry the discharge from a ten-year rainfall event leaving an area approximately the width of one lane at the center free of any water flow". These Guidelines contain similar performance standards for various parts of the secondary and primary systems.

Conveyance Pathways

The secondary system is to be evaluated and designed for the stormwater conditions that will result for storms up to the magnitude of the 100-year rainfall event based on existing development within the applicable basin. From the location where storm flow is first introduced into a public easement or right of way near the upper end of any basin, a "conveyance pathway" shall be identified and provided to a discharge point at a main channel of the primary system. The designated "conveyance pathway" must follow or provide clearly identifiable watercourses. Needs for easements or ROW for conveyance pathways are to be assessed per the provisions of this Section. The purpose of providing for the 100-year storm level is to prevent the creation of situations hazardous to life, or harmful to public and private property. Accordingly, a major emphasis is on deliberately confining storm flow to designated conveyance pathways.

Watershed Diversion

Generally stormwater emitting from land drained by one watercourse of the primary system shall not be diverted to drain into a different named regulatory watercourse of the primary system.

3. Detention / Mitigation

Detention Purposes

Detention is an important mitigation measure. It can be used effectively for either or both of two fundamental purposes. As a tool for watershed management, it can be deployed with other features to minimize potential flooding along major watercourse(s). It can also be used to manage how stormwater is discharged from a property to adjacent properties. Thus, it can be an integral part of stormwater conveyance in route to the primary system or to a tributary thereof. Both are legitimate reasons for using detention facilities and any one detention facility might work toward both purposes, depending on its location in a watershed. The functional purposes for detention are further defined in foregoing Section 1.2.2, Paragraph 3.

a. Detention Requirements

Right Uses

For optimum results detention facilities must be deployed for the right reasons at the right locations. It is the intent of these Guidelines to stipulate the conditions under which detention must be used and why. These Guidelines are not intended to preclude the use of detention at locations where qualified engineers may deem it to be beneficial. Nevertheless, where detention is required by these Guidelines designed facilities must meet the criteria stipulated herein.

Peak Flow Regulated

Where detention facilities are required, peak stormflow rates from a project area resulting from the two (2), ten (10), and one hundred (100) year storm frequency events shall not be increased at any point of discharge. Regulation of peak flows to allowable levels, as determined by the provisions of these Guidelines, shall be achieved by storage facilities on, or away from, a project area, or by participation in an approved Regional Stormwater Management Program.

b. Detention Facilities May Be Optional

Detention Limited

At the discretion of the Drainage Review Authority, land development activity is not subject to the stormwater detention requirements of these Guidelines if one or more of the four conditions listed in Sub-paragraphs b(1) through b(3) before are satisfied, and an engineer registered in the State of Texas submits a signed, sealed, and dated letter addressed to the Drainage Review Authority, stating the following without qualification:

"*I have conducted a topographic review and field investigation of the existing and proposed flow patterns for stormwater runoff from* (name of subdivision or site project*) to the main stem of* (name of creek)*. At design conditions allowable by zoning, restrictive covenant, or plat note, the stormwater flows from the subject subdivision or site project will not cause any increase in flooding conditions to the interior of existing building structures, including basement areas, for storms of magnitude up through the 100-year event*":

(1). <u>Adjacent to Primary System</u>
The development is immediately adjacent to a designated primary system watercourse, and discharges directly into its lower reach (approximately the lower one third of the watercourse length).

(2). <u>One Existing Lot</u>
The proposed development project involves one single existing legal lot that is limited to single-family land use by zoning, restrictive covenant, or plat note.

(3). <u>Small Lot</u>
The size of a platted lot is equal to or less than one (1) acre for commercial use, or two (2) acres for detached residential use.

4. Water Quality

Concurrent Objectives

The intent of these Guidelines is to cause development of stormwater management facilities that effectively collect and convey stormwater without causing water damage impacts on life and property. A concurrent objective is to achieve facilities that minimize any adverse affect(s) on the quality of water conveyed into natural waterways that traverse and/or drain the developed areas within the County.

5. Master Drainage Plans

Plan Consistency

All land development projects and site re-development projects subject to the provisions of these guidelines must demonstrate that plans for managing the stormflow expected to emit from the project(s) are consistent with the County's Master Drainage Plan if available, or with any applicable publicly approved Watershed management master plan.

Section 1.2.4 – Extent of Design

1. Threshold for Engineered Design

Limited Exemptions

For purposes of these Guidelines, some land development projects may be exempted from requirements for drainage plans designed by a licensed engineer and approved by the County and/or Cities. However, in designated FEMA floodplain areas no construction of any kind, including clearing, grubbing or earthwork, may begin without fully approved engineering studies. Likewise, this provision shall not be construed to obviate any requirements of the Texas Professional Engineering Practices Act regarding engineering of facilities to be constructed for public use.

Possible Exemptions

Developments of the general nature listed below may be exempted from designs conforming with provisions of these Guidelines after appropriate review and approval by the Drainage Review Authority or his/her designee.

- A small lot less than one acre in size that does not receive stormwater from adjacent or nearby land areas.
- A platted lot set aside for construction of one single family residential unit.
- Any platted lot less than one (1) acre for commercial or multifamily use and two (2) acres for detached residential for which adequate stormwater management provisions can be administered through building permit requirements.
- Where, in the judgment of the Drainage Review Authority, development of a proposed project on a platted lot will have no appreciable down-steam effect**.**

2. Study Limits

Analysis Limits

Engineering for assessment of conditions resulting from a stormwater project shall include the **project area**, **above-project area(s)**, and **pathway area(s)** as necessary, and must extend upstream and/or downstream along designated **conveyance pathways** to a point where the applicant (or his engineer) can demonstrate to the Drainage Review Authority's satisfaction that there are no appreciable drainage effects caused by the proposed project. Downstream or upstream of these points the minimum responsibility of the project engineer is to merely document the location of the "conveyance pathway" to limits otherwise specified in these Guidelines.

3. Special / Alternate Designs

a. Drainage Review Authority Approval

Equivalent Safe Design

The Drainage Review Authority may, upon request, approve an alternate design or construction methodology that differs from the requirements in these Guidelines if the Drainage Review Authority determines that:

(1) The alternate design or construction methodology is equivalent or superior to the design that would result from using these Guidelines, and

(2) The alternate design or construction methodology is sufficient to ensure public health and safety.

b. Substantiation of Alternate Designs

Responsibility

It shall be the responsibility of the owner's/developer's (applicant's) engineer to substantiate that any proposed alternate design or construction methodology deviating from these Guidelines meets or exceeds designs or construction methodologies promulgated by these Guidelines.

4. Applicable Ordinance Requirements

Design Reviews

Nothing herein shall be construed to conflict with or supersede design review criteria otherwise established in applicable ordinances of the City of Wharton or the City of El Campo, the City of East Bernard or the City of Louise.

Section 1.2.5 – Public Facilities

1. Principles For Public / Private Facilities

Public/Private Mix

Stormwater management involves some combination of private and public facilities occurring on (or across) land, and in easements or ROW, in a mix of public and private holding (or ownership). The two-fold intent of these Guidelines is to regulate all such facilities as necessary to achieve specific objectives, while minimizing regulation where it is not fundamental to meeting those objectives.

Rural To Urban

Development activities either change the character (or use) of a previously developed site(s), or generally move land from rural to urban conditions. In the later case, storm runoff is necessarily directed into various types of concentrated flow that typically did not previously exist. This can tend to change both how and where flow is delivered to immediately adjacent properties or facilities. Because the new facilities are commonly situated in easements or ROW proposed to be conveyed to a public entity, the process may create a measure of public responsibility where none had previously existed.

Discharge Options

It is the responsibility of the owner/developer of any development project to properly provide for storm discharge from the project area. Where street or drainage ROW(s) or drainage easement(s) are to be dedicated to the public, and discharge is to drain across neighboring property(ies) before reaching a Primary Watercourse (or a recognized drainage way serving as a tributary thereof), it shall be the responsibility of the project owner/developer to accomplish one of the two following scenarios, or some combination thereof.

a. First Scenario: Establish Drainage Easement(s)

Receiving Easements

Drainage easements must be established across down stream properties as necessary along identified conveyance pathways. Such easements must be aligned and sized to safely accommodate the design discharge(s) from the project area, and must extend to a Named Regulatory Watercourse (or a tributary thereof). The easement(s) may be conveyed to a private party or to a public entity at the discretion of the Drainage Review Authority or her/his designee.

b. Second Scenario: Pre-Development Release

Designed Release(s)

Drainage facilities must be situated and designed so that discharge(s) are delivered to down stream properties with substantially the same flow characteristics (rate of flow, concentration, velocity, etc.) that existed in pre-development conditions. In addition, discharges are to be released at substantially the same locations that existed in pre-development conditions. Usually, all work necessary to accomplish this must be within the geographic limits of the project area.

2. Maintenance Considerations

A Design Function

All stormwater management projects subject to the provisions of these Guidelines that are to be dedicated to the public shall be designed with adequate provisions for maintenance of the designed facilities, regardless of their nature. Maintainability and access are important design objectives. These two factors must be an integral part of the design considerations for all stormwater facilities. The same principles must apply to the easements and/or right of way within which such facilities are to be placed.

Importance

Where, in the opinion of the Drainage Review Authority, design alternatives meet detention, flood level, and water quality criteria promulgated by these Guidelines and other regulatory requirements in essentially an equal manner, the option(s) offering lesser demand for maintenance work will be preferred. Likewise option(s) offering improved access will be preferred.

Justification Data

All information necessary to making such decisions shall be the responsibility of property owners proposing the land development project(s). Changes in proposed designs may be required in order to meet these objectives.

3. Easements and Right of Way

Drainage ROW

Where any part of a project area is traversed by a channel or stream, whether man-made or natural, an easement or drainage right of way (ROW) is to be provided for the watercourse. Likewise ROW is to be provided for drainage ways newly formed by runoff concentration within the project area of subdivision projects. In all cases ROW is required unless easements are specifically approved by the Drainage Review Authority. ROW will generally be required unless stormwater is conveyed via underground conduit, in which case easements will be considered.

Uses Limited

The purpose of easements or right of way (ROW) is to provide the necessary space for stormwater flow and for maintenance of drainage facilities. Any uses of such areas that are inconsistent with these purposes are prohibited. Prohibited uses include, but are not limited to, construction of fences or other obstructions, placement of building structures, or any uses that alter the required shape, configuration, or surface treatment needed for stormwater management functions.

a. Size Parameters

Approvals Needed

Decisions about the necessary alignment and extent of ROW and easements shall be subject to approval by the Drainage Review Authority or his/her designee, and shall be based, in part, on drainage information provided by the applicant. Criteria for this determination shall be based on the anticipated amount and spread of stormwater flow, the possibility of increased flow at some time in the future, any concurrent uses to be associated with the designated areas, the space required for the appropriate maintenance equipment and personnel, and the access necessary to conduct maintenance activities.

ROW For Channels

Where a land development project is traversed by a constructed swale, a constructed channel, a natural channel, or a stream, drainage ROW conforming substantially to the limits of such watercourse (plus additional width to accommodate flow from a 100-year frequency event*****) must be provided. Additional width may be required for maintenance purposes.

Conduit Easements

Where stormwater is to be conveyed in buried conduits, drainage facilities may be situated in drainage or utility easements provided flow from a 100-year frequency event***** will be wholly contained within the easement.

***** **In cases where the 100-year event cannot feasibly be conveyed, exceptions may be allowed with the approval of the Drainage Review Authority.**

b. Minimum Standards

The following minimum standards shall be used in determining the size and placement of drainage easements and ROW.

(1). The minimum width of any drainage easement shall be 15 feet.

(2). For buried conduit storm sewer, the minimum width for any drainage easement (or ROW) that is not congruent with any other pubic ROW or easement shall be 15 feet, and the centerline of the storm sewer shall not be closer than five (5) feet to either side of the easement. In addition, the easement or ROW (inclusive of the conduit capacity) must adequately convey the 100-year storm.

(3). For purposes of maintenance access for improved open channels, the minimum ROW width shall be the design top width of the channel plus an additional 20 feet (five feet along one side and 15 feet along the other side). However, where the design top width of the channel exceeds 30 feet, 15 feet of additional ROW shall be provided on both sides of the design channel width. Where special designs approved under the provisions of Section 1.2.3, Paragraph 3 of these Guidelines will obviate the need for easements of these widths, smaller or narrower easements will be considered. However, in no case shall adequate provisions for maintenance be seriously compromised.

(4). If access to a drainage easement or ROW is not available from public ROW, then an access easement having a width of 15 feet or more shall be provided from a public ROW to the easement or ROW containing drainage facilities.

(5). The width of all easements and ROW shall be sufficient to include areas that will be part of the designated conveyance pathways of the secondary system.

Section 1.2.6 – Private Facilities

1. Detention Systems

Guidelines Apply

All stormwater detention facilities required by these Guidelines shall be sized, designed, and constructed in conformance with the criteria stipulated herein and elsewhere in Drainage Regulation Entity regulations, whether to be retained as private facilities or dedicated to the public within an easement or ROW.

2. Conveyance Systems

The four conditions described in this sub-paragraph are illustrated in Figure 1.2.6-1.

a. Discharges Received By Private Land or Facilities

From Private

Stormwater conveyance features that will receive discharge only from private land or facilities at ultimate development conditions may be established as private conveyance systems at the discretion of the Drainage Review Authority or her/his designee. Design of such facilities in accordance with provisions of these Guidelines is generally at the discretion of the Registered Professional Engineer in charge of the work.

From Public

Where stormwater is proposed to discharge from existing or proposed public ROW(s) or easement(s) to private land or facilities it is the responsibility of the owner/developer (or applicant) to assure that the project discharge is compatible with the down stream land and conveyance features. This responsibility must be met as outlined in Section 1.2.5, Paragraph 1-a /or Paragraph 1-b, or via some combination of the two concepts.

b. Discharges Leaving Private Land or Facilities

To Private

In situations were conveyance facilities that are to be permanently held in private ownership will discharge to conveyance facilities that are likewise to be permanently held in private ownership, the design is generally at the discretion of the Registered Professional Engineer in charge of the work. At the discretion of the Drainage Review Authority or his/her designee, exceptions to this may apply for watershed management purposes.

To Public

Where private lands or facilities will discharge to publicly held lands or facilities, whether in fee simple or in easement(s) or ROW(s), the design, configuration, and construction of the upland facilities shall be in conformance with these Guidelines to the extent required by the Drainage Review Authority or her/his designee. Likewise, if private land or facilities are to discharge into floodplain areas or tributaries of a Primary Watercourse without first traversing public easements or ROW or publicly held land, they are subject to application of these Guidelines at the discretion of the Drainage Review Authority or his/her designee.

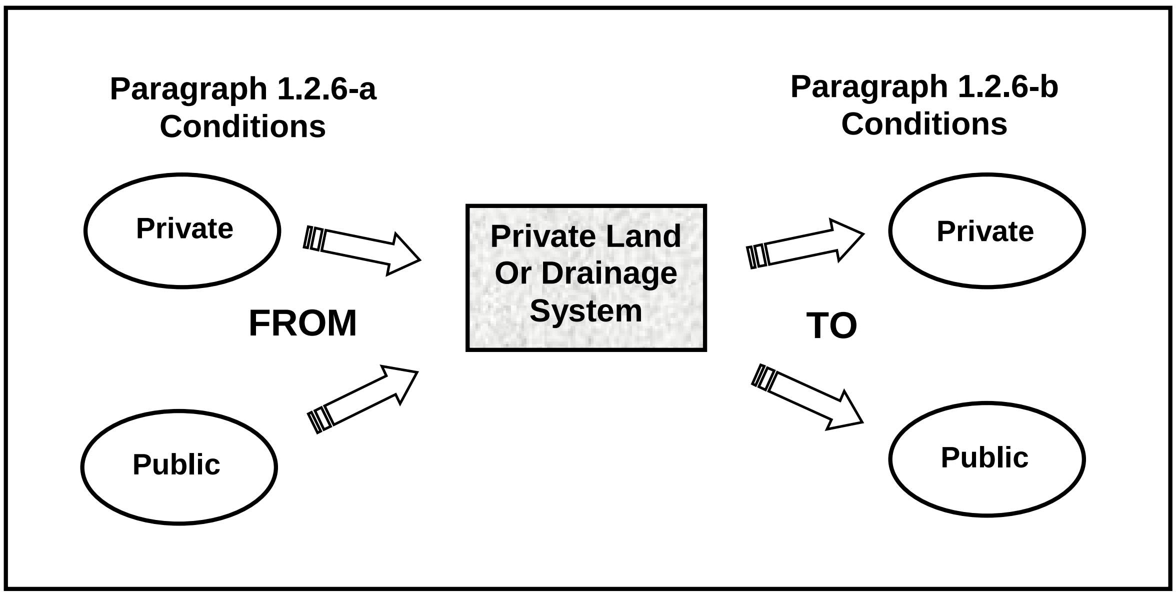


Figure 1.2.6-1: Public / Private Conveyance Systems Diagram (Paragraph 1.2.6)

Section 1.3 – Storm Water Administration

Section 1.3.1 – Permitting Process

The review process for any drainage plan must be in compliance with requirements of other entities with jurisdiction within the Drainage Regulation Entity as applicable. The following general process is recommended.

All developments shall be required to submit a Preliminary and Final Drainage Plan, prior to development, to the Drainage Review Authority.

Approval Process:

<u>Preliminary Plan Review</u>

The first step in the review and approval process for a proposed development is to submit a Preliminary Drainage Plan to the Drainage Review Authority demonstrating that adverse drainage or flooding conditions will not be created along any drainage outfall or adjacent property as a result of the development. The Preliminary Drainage Plan shall define the method of conveying rainfall runoff from the development to the appropriate drainage outfall. This will include showing sheet flow paths, outlet design, detention design, and addressing (if necessary) 100-year floodplain issues.

The Preliminary Drainage Plan will show the following as a minimum:

1. Name, address, phone number and Texas P.E. seal of the engineer preparing the plan.

2. Submittal and re-submittal dates.

3. Minimum scaled drawing on 24" x 36" sheet of 1"=200'.

4. Vicinity map and legend.

5. A primary bench mark referenced to a N.G.V.D. benchmark with elevation, datum, year of adjustment, and description is required on the plan.

6. North arrow on all sheets oriented upward or to the right.

7. All lot lines, property lines, rights-of-way lines, and easement lines.

8. Contour lines at 2 foot intervals covering the entire development including offsite elevations 100 feet around perimeter.

9. Cross-section of existing and/or proposed detention facility, swales, and ditches.

10. Drainage area boundaries for the project area, including off-site areas.

11. Location and dimensions of all existing and proposed drainage easements and reserves or fee strips.

12. Location of all drainage arteries adjacent to or crossing the development as determined by recent (within past year) ground survey. Stream alignment shall be shown 200 feet upstream and downstream of development.

13. Detention tabulations, including detention volume required and detention volume provided call-outs. Detention calculations shall be completed as outlined in Section 4.4.

14. Limits of the floodway and the 100-year flood plain scaled from the current FIRM, if applicable.

15. Location of existing drainage and other structures, pipelines, and other underground features.

<u>Final Plan Review</u>

The second step in the review and approval process for a proposed development is to submit a Final Drainage Plan to the Drainage Review Authority demonstrating that adverse drainage or flooding conditions will not be created along any drainage outfall or adjacent property as a result of the development. The Final Drainage Plan must be filed and approved prior to commencement of construction.

The Final Drainage Plan must include all of the items on the Preliminary Drainage Plan, as well as the following as a minimum:

1. Final detention calculations as outlined in Section 6.4.

2. Lot Grading plan, which provides for the passage of sheet flow from all adjacent properties.

3. A 100-year sheet flow analysis that provides direct access to the detention facility or main outfall.

4. Seal of a Registered Professional Engineer on all plans.

5. Approval of the drainage district(s) in which the project property is located.

<u>Time Limit for Approvals</u>

All approvals from the Drainage Review Authority shall be valid no longer than 12 calendar months. Failure to begin construction (building of roads, digging detention system) of an approved project or make full use of the approvals granted within that time period shall make such approvals null and void. Any fees associated with this process will be forfeited and will not be returned to the applicant. Request for a one-time extension, for a period not to exceed 12 months, may be granted by the Drainage Review Authority, at its discretion, providing good cause exists and the request is made prior to the expiration of the original approval.

The contractor shall have the construction time permitted as part of his bidding process plus any accepted time extensions. Should there not be time limitations relating to the contractor the Drainage Regulation Entity shall determine the applicable construction duration. Construction outside this time frame shall not be allowed without expressed written authorization from the Drainage Regulation Entity. Should the contractor not be complete within the permitted schedule he may be required to resubmit and obtain new construction permit(s).

<u>Acceptance Procedures:</u>

Prior to the Drainage Review Authority's approval of any drainage facilities in a development, the project engineer must certify that the drainage facilities were constructed in accordance with the approved plans and specifications.

The approval process will be accomplished by meeting the criteria of the governmental entities charged with drainage responsibilities, as well as the criteria set forth below.

1.3.2 Abbreviated Drainage Plan

a. Suitability

In certain situations, consistent with the policies and practices of the Drainage Review Authority, the owner/developer (or applicant) may provide an Abbreviated Drainage Plan in satisfaction of these Guidelines. This is applicable only to small site plan projects on platted lots, not involving the development of stormwater detention facilities, private or public. Although not precluding involvement of an engineer, the scope of such site projects generally does not involve hydrologic or hydraulic engineering analysis or the design of stormwater management facilities. Subdivision land development projects are specifically excluded from this type of submittal. As a function of the size, location, and hydrologic complexity of a project, the Drainage Review Authority or his/her designee may require submittal of an engineered drainage report.

b. Submittal Requirements

An Abbreviated Drainage Report is generally a very simple presentation of how stormwater is to be managed on a small project. At a minimum such a plan must include the information listed below. It must be accompanied by a letter of transmittal requesting approval, and all proposed site features must be subject to inspection via building permit processes.

- A site plan drawn to a standard engineering or architectural scale showing vertical dimensional controls and proposed site grading,
- Finish floor elevations of structures and illustration of how stormwater is to be routed around and away from them,
- Illustration of any flumes, walls, berms, and/or landscaping features proposed for the purpose of managing runoff,
- Brief discussion of how erosion and sedimentation will be prevented as a permanent part of the project,
- Description of how runoff is to be routed away from the property,
- Measures employed to preclude any negative affects on downstream properties, and
- Measures to preclude any negative effects on public or private watercourses to which runoff will be directed.

Section 1.4 – Related Permitting

Section 1.4.1 – FEMA-Designated Floodplains

1. Regulatory Floodplains

Named Watercourses

Based on long experience with helping offset the costs suffered by flood victims, The Federal Emergency Management Agency (FEMA) has developed a flood insurance program centered on the concept of floodplain management. Based on a series of engineering studies FEMA has mapped flood-prone areas along principal watercourses and their tributaries in urban areas nationwide. Termed “Flood Insurance Rate Maps”, these indicate areas where citizens may obtain flood insurance at favorable rates due to FEMA subsidies. For purposes of these Guidelines the FEMA-designated watercourses and their tributaries are designated as the “Named Regulatory Watercourses” of the County.

Floodplains

The County and Incorporated Cities administer FEMA regulation of the floodplains of the Named Regulatory Watercourses as necessary to ensure the availability of affordable flood insurance to area citizens.

2. Regulations

Minimize Flooding

FEMA has established certain criteria that must be met by the County and Incorporated cities along specific watercourses. The purpose is to minimizing flooding, so use of “flood fringe” areas is purposely limited. Complex criteria affect both mapped areas and, in some instances, areas that are not yet fully mapped based on engineering studies. Where a land development project or construction of any kind will have the effect of limiting the cross sectional area of a FEMA-designated watercourse, engineering studies are necessary to determine the hydraulic effects, and to assess whether flood stage water surface elevations will be affected outside of allowable criteria. Where the upper reaches of a FEMA-designated watercourse are not adequately mapped, engineering studies will be necessary to do so.

3. Managing Encroachment

Watersheds

Development of lands along FEMA-designated watercourses may involve the proposed use of “flood fringe” areas, overbank areas not usually involved with conveyance of stormwater during low flow conditions. Use of such areas is considered “encroachment” into regulated floodplains, and is therefore, limited. Encroachments generally have the affect of restricting the cross sectional area of a watercourse, so the objective is to avoid causing water surface elevations at flood stage to rise above certain predetermined levels as necessary to the characteristics of each watercourse.

4. Procedures

Other Sections

The possible need for engaging FEMA in review and approval of flood studies or crossings of FEMA-designated watercourses must be identified at the Stormwater Planning Conference outlined in Section III of these Guidelines. Different levels of FEMA approval are required as a function of the proposed activity and its potential impact on flood-prone areas. The approval appropriate to a project must be obtained and documented to the Drainage Review

Authority's satisfaction before authorization will be given to start construction.

Encroachments

The rationale for determining the extent of allowable encroachment and specific limitations are stipulated in Section 4.2 of these Guidelines.

Section 1.4.2 Stormwater Quality

Permits If Needed

There are a number of national and state regulations that have bearing on the quality of stormwater emitted from land development projects in Wharton County. These are principally focused on efforts to minimize the amount of sediments and pollutants carried into streams and waterways by storm runoff. Specific permitting requirements that may, from time to time, be required under any of the legislative provisions listed below must be met by owners/developers (or applicants) of land development projects. Proof that required permits have been issued by the appropriate authority must be provided before construction will be authorized by the County.

- Section 10 US Harbors and Rivers Act as administered by the US Army Corps of Engineers (USACE).
- Section 404 of the US Clean Water Act as administered cooperatively by the US Environmental Protection Agency (EPA) and the USACE.
- Section 401 of the US Clean Water Act as administered by the EPA.
- Section 402 of the US Clean Water Act as administered by the EPA in cooperation with the Texas Commission on Environmental Quality (TCEQ).
- Texas Administrative Code (30 TAC, Chapter 319) as administered by the TCEQ pursuant to the Texas Pollutant Discharge Elimination Program in cooperation with the EPA's Section 402 regulation of small MS4s.

Section 1.4.3 Governmental Entities In Wharton County Region

Planning Required

If a land development project of any size or complexity might possibly involve one or more of the entities listed in this paragraph that potential must be made known as early as possible in the development review process. Ideally the needed coordination and approvals will be fully discussed during the Stormwater Planning Conference outlined in Section III of these Guidelines. At the very least, such coordination must be identified as an open matter at that time and fully addressed in the project Drainage Report.

1. Wharton County

Approvals Required

Certain land development projects may directly or indirectly involve Wharton County governments. This may include site construction projects as well as subdivisions, and includes the creation of public drainage easements or ROW. Approvals by the office of the Drainage Review Authority must be substantiated in the form of letters or any documentation acceptable to the Drainage Review Authority or his respective designees.

Site Projects

Any site development project that is wholly or partially in the limits of the Drainage Regulation Entity (city or county) is subject to these Guidelines. Where a project will discharge stormwater directly or indirectly into roadway areas administered by Wharton County, it will be necessary for the project owner/developer (or applicant) to secure the necessary approvals by the office of the Drainage Review Authority, or his/her designee. Likewise, if stormwater is to be discharged into a drainage way of any character that is maintained or administered by the office of the Drainage Review Authority, approvals must be obtained. Approvals must be substantiated before site drainage plans will be approved by the County.

Subdivisions

Subdivisions are commonly proposed within the corporate limits or the Extra Territorial Jurisdiction (ETJ) of a city, and may be partially in both. Also, a subdivision project area may be partially in a city's ETJ and extend outside of the ETJ. Under any of these conditions stormwater facilities may be planned to discharge into roadside ditches or watercourses that are under the jurisdiction of Wharton County. In such circumstances County roadway facilities may be affected within or adjacent to the project area, or downstream thereof. For this reason the project owner/developer (or applicant) must secure the necessary approvals by the office of the Drainage Review Authority, or his/her designee whether they represent the city or the county

2. Texas Department of Transportation (TxDOT)

TxDOT Facilities

Any land development project that is adjacent to or astride a highway route administered by TxDOT must be fully coordinated with the office of the TxDOT Area Engineer or his/her designee. All ROW and drainage easements under TxDOT jurisdiction must be fully identified, as well as any stormwater discharge(s) received from TxDOT facilities. Likewise any proposed discharges to TxDOT facilities or easements must be identified in detail.

Documented Action

Evidence of adequate coordination with TxDOT must be provided to the Drainage Review Authority or her/his designee. Documentation of the necessary coordination must be to the mutual satisfaction of the offices of the TxDOT Area Engineer and the Drainage Review Authority. Approval of site construction projects and final plats is subject to satisfaction of this requirement by the project owner/developer (or applicant).

3. Lower Colorado River Authority

State Agency

The Lower Colorado River Authority is a State agency charged with overall management of the water resources of the lower portion of the Colorado River Watershed stretching from far west Texas to the Gulf of Mexico. The Agency's focus is on water and sewage treatment, and electric service for communities along the river's route. Its mission includes development and management of several water and flood control reservoirs.

Limited Role

During recent years the Agency has been given a broader role in support of the TCEQ's water quality mission. This largely parallels the Agency's other activities so it is focused on effluent point sources like sewage treatment and industrial processing enterprises. The Agency has no known role in reviewing or permitting stormwater facilities proposed in land development projects in the Wharton County. The one possible exception would be in situations where permanent water impoundment, as per State of Texas Water Rights Regulations, is proposed directly on tributaries to the Colorado River. The Agency should be contacted as early as possible if impoundment is proposed in order to determine the extent of permitting that might be required, if any. Any associated permitting requirements must be met by the project owner/developer (or applicant). Documentation thereof must be provided to the office of the Drainage Review Authority before design plans will be accepted for construction.

CHAPTER 2 – Hydrologic Analysis

Section 2.1 – General

Section 2.1.1 – Introduction to Hydrologic Methods

Hydrology deals with estimating flow peaks, volumes, and time distributions of storm water runoff. The analysis of these parameters is fundamental to the design of storm water management facilities, such as storm drainage systems and structural storm water controls. In the hydrologic analysis of a development/redevelopment site, there are a number of variable factors that affect the nature of storm water runoff from the site. Some of the factors that need to be considered include:

- Rainfall amount and storm distribution
- Drainage area size, shape, and orientation
- Ground cover and soil type
- Slopes of terrain and stream channel(s)
- Antecedent moisture condition
- Rainfall abstraction rates (Initial and continued)
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Characteristics of the local drainage system

There are a number of empirical hydrologic methods available to estimate runoff characteristics for a site or drainage subbasin; however, the following methods have been selected to support hydrologic site analysis for the design methods and procedures included in this Manual:

- Rational Method
- Clark's Unit Hydrograph Method
- SCS Unit Hydrograph Method
- Snyder's Unit Hydrograph Method
- Small Watershed Method
- Drainage Area - Discharge Curves

These methods were selected based upon a verification of their accuracy in duplicating local hydrologic estimates for a range of design storms throughout the state and the availability of equations, nomographs, and computer programs to support the methods. Table 2.1.1-1 summarizes the applicability of various hydrologic methods to Wharton County.

Table 2.1.1-1 Constraints on Using Recommended Hydrologic Methods		
Method	**Size Limitations[1]**	**Comments**
Rational Method (Section 2.1.3)	0-200 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems.
Small Watershed Method (section 2.1.5)	<2000 acres	Method can be used for estimating runoff volumes and hydrograph routing for detention planning and design. Basin sizes larger than 50 acres must utilize a hydrograph routing method for final design.
Drainage Area- Discharge Curves (Section 2.1.7)	200-2000 acres	Method can be used for estimating peak flows for smaller basins for planning applications and comparison purposes.
Unit Hydrograph (Clark's) (Section 2.1.4.3)	> 100 acres	Method can be used for estimating peak flows and hydrographs for all planning and design applications.
Unit Hydrograph (SCS)[2] (Section 2.1.4.4)	Any size	Method can be used for estimating peak flows and hydrographs in urbanized conditions
Unit Hydrograph (Snyder's)[3,4] (Section 2.1.4.5)	>100 acres	Method can be used for estimating peak flows and hydrographs with approval of Drainage Review Authority.
Detention Factor (Section 4.4.2)	0-50 acres	Method can be used for estimating detention volumes for basins of 50 acres or less.
Green and Ampt Loss Method (Section 2.1.4.1)	> 640 acres	Method can be used for determining losses in connection with Unit Hydrograph Methods for larger watersheds in the San Bernard watershed.
SCS Loss Method (Section 2.1.4.1)	Any Size	Method can be used for determining losses in connection with Unit Hydrograph Methods for smaller watersheds throughout the County and for larger Watersheds in the western portion of the County.
TXDOT Regression Equations[4] (Section 2.1.7)	10 to 100 mi^2	Method can be used for estimating peak flows for rural design applications for comparison purposes only.

[1] Size limitations refer to the drainage basin for the storm water management facility (e.g. culvert, inlet). These do not necessarily apply to master drainage plans.
[2] This refers to SCS routing methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology.
[3] This refers to the Snyder's routing methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology.
[4] Use only with approval of Drainage Review Authority.

If other hydrologic methods are to be considered and used by a local review authority or design engineer, the method should first be calibrated to local conditions and tested for accuracy and reliability. If local stream gage data are available, these data can be used to develop peak discharges and hydrographs. The user is referred to standard hydrology textbooks for statistical procedures that can be used to estimate design flood events from stream gage data.

Note: It must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of *precipitation on* a *drainage basin and the amount* of *runoff from the basin is complex and too little data are available on the factors influencing the rainfall-runoff relationship to expect exact solutions.*

Section 2.1.2 – Rainfall Estimation

The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

Duration (hours) - Length of time over which rainfall (storm event) occurs
Depth (inches) - Total amount of rainfall occurring during the storm duration
Intensity (inches per hour) - Depth divided by the duration

The Frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of *exceedance probability* or *return period.*

> *Exceedance Probability* - Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically in years
> Return Period - Average length of time between events, which have the same duration and volume

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedance probability of 0.01 and a return period of 100 years.

The statistical point rainfall data has been obtained from the *Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas, USGS Scientific Investigation Report 2004-5041, Asquith 2004*. The rainfall depths vary spatially throughout Wharton County and generally increase from north to south. The rainfall depths were determined with the aid of a computational procedure, developed by Asquith, to determine the statistical point rainfall values for each sub-basin. These point rainfall values were then reduced based on storm area reductions. For a general idea of the rainfall depths being considered, a central location in the county (29°18'30.67" latitude and 96°6'13.72" longitude) was used to determine the statistical point rainfall values shown in Table 2.1.2-1. Rainfall hyetographs for Wharton County may be developed using either the SCS Type III rainfall distribution or "Alternating Block" method to develop frequency rainfall patterns for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-yr events.

Table 2.1.2-1: Frequency Rainfall Depths for Central Wharton County

		Recurrence Interval (years)							
	Duration	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	250-yr	500-yr
Duration	**(hours)**	**Depth (inches)**							
5 min	**0.08**	0.80	1.07	1.25	1.47	1.66	1.85	2.11	2.33
15 min	**0.25**	1.11	1.42	1.62	1.88	2.09	2.31	2.61	2.86
30 min	**0.50**	1.57	1.94	2.18	2.50	2.74	3.00	3.36	3.65
60 min	**1.00**	1.97	2.55	2.94	3.48	3.93	4.43	5.16	5.78
2 hr	**2.00**	2.51	3.26	3.77	4.48	5.07	5.71	6.66	7.46
3 hr	**3.00**	2.59	3.48	4.13	5.06	5.87	6.79	8.20	9.45
6 hr	**6.00**	3.06	4.22	5.12	6.48	7.71	9.16	11.50	13.67
12 hr	**12.00**	3.42	4.71	5.71	7.23	8.60	10.21	12.82	15.23
24 hr	**24.00**	3.98	5.73	7.08	9.13	10.98	13.16	16.69	19.94
2 day	**48.00**	4.71	6.69	8.11	10.04	11.58	13.2	15.49	17.34
3 day	**72.00**	4.88	6.94	8.41	10.42	12.02	13.71	16.09	18.01

The point rainfall values can be reduced with the areal reduction analysis within HEC-HMS. This areal reduction analysis utilizes the TP-40 areal reduction curves developed by the National Weather Service. It was assumed that these TP-40 curves would be adequate for reducing the USGS point rainfall values, as the only known areal reduction method developed by the USGS is only applicable to the SCS 24-hr hypothetical storm. The storm duration for any hydrologic analysis should be at least longer then the time of concentration of the watershed. Figure 2.1.2-1 presents a set of rainfall intensity curves which were developed by plotting intensities based on the rainfall depths presented in Table 2.1.2-1 and manually smoothing these plots as described in the previously mentioned 2004 report by USGS. These curves should be used for future hydrologic studies within Wharton County.

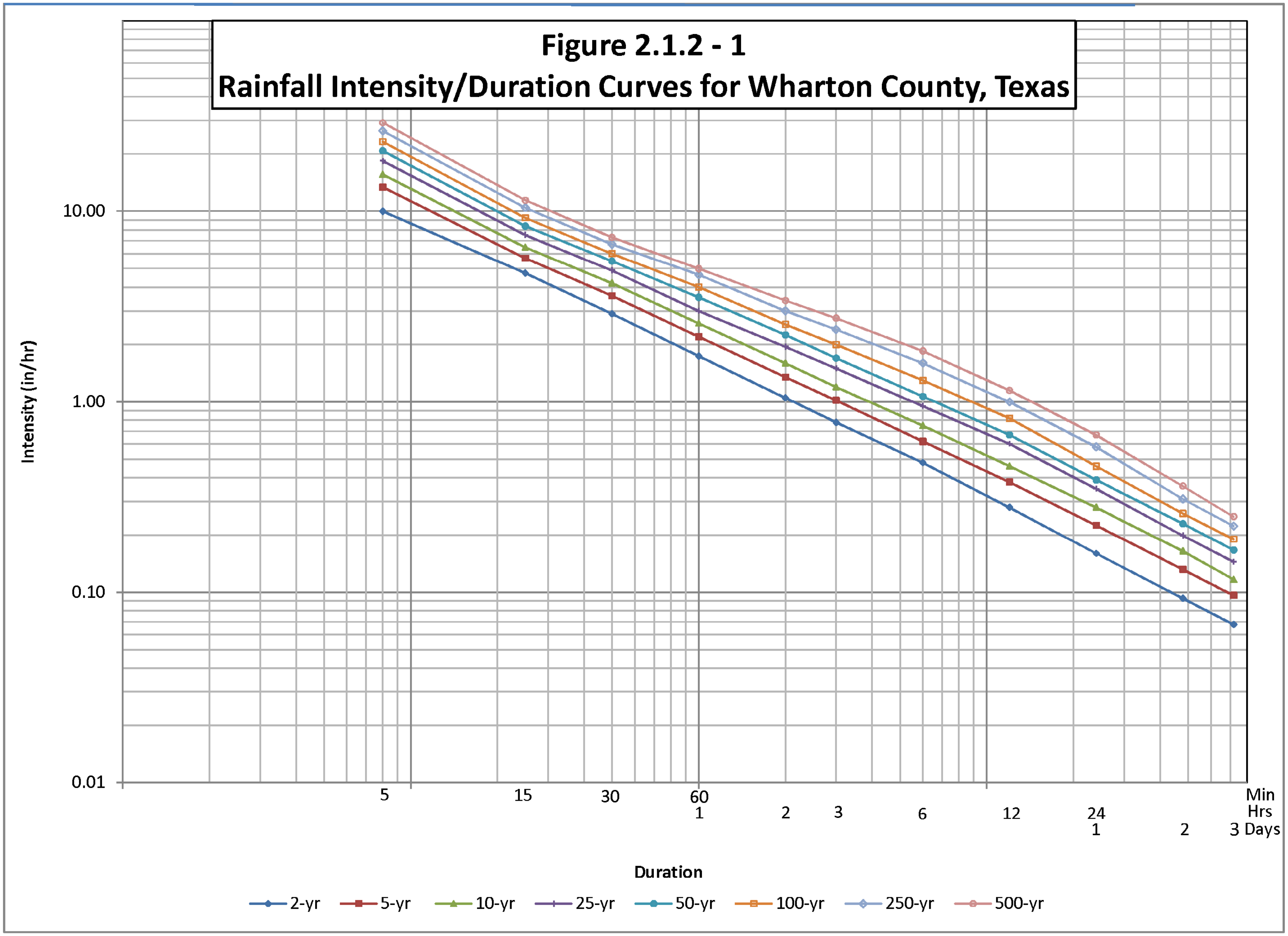


Figure 2.1.2 - 1
Rainfall Intensity/Duration Curves for Wharton County, Texas
Intensity (in/hr)
10.00
1.00
0.10
0.01
5
15
30
60
1
2
3
6
12
24
1
2
3
Min
Hrs
Days
Duration
2-yr
5-yr
10-yr
25-yr
50-yr
100-yr
250-yr
500-yr

Section 2.1.3 – Rational Method

Section 2.1.3.1 – Introduction

An important formula for determining the peak runoff rate is the Rational Formula. It is characterized by:

- Consideration of the entire drainage area as a single unit
- Estimation of flow at the most downstream point only
- The assumption that rainfall is uniformly distributed over the drainage area and is constant over time

The Rational Formula adheres to the following assumptions:

- The predicted peak discharge has the same probability of occurrence (return period) as the rainfall intensity (I)
- The runoff coefficient (C) is constant during the storm event

When using the Rational Method some precautions should be considered:

- In determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility.
- Since the Rational Method uses a composite C and a single tc value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), then the basin should be divided into sub-drainage basins.
- The charts, graphs, and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer should use sound engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate adjustments are appropriate.

Section 2.1.3.2 – Application

The Rational Method can be used to estimate storm water runoff peak flows for the design of gutter flows, drainage inlets, storm drainpipe, culverts, and small ditches. It is most applicable to small, highly impervious areas. The recommended maximum drainage area that should be used with the Rational Method is 200 acres.

The Rational Method should not be used for storage design or any other application where a more detailed routing procedure is required. However, the Small Watershed Method is used by some for design of small detention facilities, so the method has been included in Subsection 2.1.7.

Caution should be used in applying the Rational Method for analysis or design of bridges, culverts, or storm sewers that may act as restrictions causing storage, which could impact the peak rate of discharge.

Section 2.1.3.3 – Equations

The Rational Formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and the mean rainfall intensity for a duration equal to the time of concentration, t_c (the time required for water to flow from the most remote point of the basin to the location being analyzed).

The Rational Formula is expressed as follows:

$$Q = CIA \qquad (2.1.1)$$

where:

Q = maximum rate of runoff (cfs)

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the t_c (in/hr)

A = drainage area contributing to the design location (acres)

The coefficients given in Table 2.1.3-2 are applicable for storms with return periods less than or equal to 10 years. Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin Engineers, 1969). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f. The Rational Formula now becomes:

$$Q = C_f CIA \qquad (2.1.2)$$

The C_f values that can be used are listed in Table 2.1.3-1. The product of C_f times C shall not exceed 1.0.

Table 2.1.3-1 Frequency Factors for Rational Formula

Recurrence Interval (years)	**C_f**
10 or less	1.0
25	1.1
50	1.2
100	1.25

TABLE 2.1.3-2
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 1/2 acre)	0.30	0.35	0.40
Single Family Areas (Lots 1/4 - 1/2 acre)	0.40	0.45	0.50
Single Family Areas (Lots less than.1/4 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

Section 2.1.3.4 – Time of Concentration

Use of the Rational Formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The time of concentration consists of an overland flow time to the point where the runoff is concentrated or enters a defined drainage feature (e.g., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Table 2.1.3-3 - Minimum Times of Concentration

Land Use	Minimum (minutes)
Residential Development	15
Commercial and Industrial	10
Central Business District	10
Rural/Agricultural	20

The time of concentration (T_c) is the longest time of travel for water to flow from the upstream portion of the sub-basin to the downstream point of design. Typical site conditions will dictate that T_c is the minimum time to inlet per Table 2.1.3-3. In special cases, T_c in excess of those presented in Table 2.1.3-3 may be calculated with the following procedure and such calculations and flow paths should be included with the data submitted for review by the Drainage Review Authority. The procedures specified herein, are from NRCS *TR-55*, *Urban Hydrology for Small Watersheds*.

In using the calculated procedure for determining T_c the following issues shall be considered. First, care shall be taken to ensure that the longest time of travel chosen is characteristic of the overall drainage within the sub-basin. Second, the interface between overland flow and shallow concentrated flow shall be carefully evaluated considering shallow concentrated flow paths on lawns, in swales, between structures, etc.

T_c is composed of four basic components, overland flow, shallow concentrated flow, channelized flow to inlet, and channelized flow downstream of the inlet to the point of design. Either this method or the minimum time to inlet must be used when determining T_c downstream of an inlet. Time of concentration at a design point is calculated as:

$$T_c = T_0 + T_s + T_h + T_t$$

where:

T_c = Time of concentration, minutes (min);
T_0 = Overland flow travel time, min;
T_s = Shallow concentrated flow travel time, min;
T_h = Channelized flow travel time to inlet, min; and
T_t = Channelized flow time of travel downstream of inlet to the design point, min.

Overland Flow

The time of travel for the overland flow component (T_0) is computed using Manning's kinematic solution:

$$T_0 = 0.42 \frac{(nL)^{0.8}}{R_2^{\ 0.5} S^{0.4}}$$

where:

- T_0 = Overland flow time of travel, min;
- n = Manning's coefficient for sheet flow;
- L = Flow length, feet (ft);
- R_2= 4.9 inches which is the 2-year, 24-hour rainfall; and
- S = Slope of the hydraulic grade (assume it is equal the ground slope), ft/ft.

Manning's coefficient (n) for overland flow is based on soil cover. Values for n are presented in Table 2.1.3-4. Overland flow length (L) is based on City topographic maps (or more detailed site survey data) for pre-project conditions and proposed grading plan for post project conditions. L shall not exceed the lengths presented in Table 2.1.3-5. Larger L values for undeveloped and agricultural land use can be used for undeveloped pre-project conditions. The 300 feet maximum is set because after that distance, the flow is usually considered shallow concentrated flow.

Table 2.1.3-4
Manning's *n* for Overland Flow

Soil Cover	*n* Value
Undeveloped - Cultivated soil, dense grass, range, or woods	0.24 - 0.410
Developed - Lawns, dense grass, or woods	0.240
Concrete, asphalt, gravel, or bare soil	0.011

Shallow Concentrated Flow

Overland flow becomes shallow concentrated flow in reels, shallow gullies, or swales, such as those between houses or businesses. Such flow in undeveloped areas extends from the overland flow to a stream as defined on the most detailed topographic maps available. In developed areas, shallow concentrated flow extends from the overland flow to the curb. Flow in a gutter shall be treated as channelized flow. Areas with shallow concentrated flow with varying slopes or soil surfaces can be broken down into segments to better estimate the travel time. The total time of travel of the shallow concentrated flow is the sum of the times of travel for each segment.

Table 2.1.3-5
Maximum Overland Flow Lengths

Land Use	Maximum *L* (ft)
Undeveloped, agricultural*	300
Parks, permanent open space, playgrounds	60
Single family residential (less than 3 lots per acre)	50
Single family residential, schools	40
Multi-family residential, commercial, industrial, manufacturing	20
Central business district (CBD), strip centers	10

* This length is a minimum, unless there is a defined stream on detailed topographic maps. An undeveloped site can assume a minimum time of concentration at 20 minutes with a run-off coefficient of 0.20. (For events of 10 years or less, see Table 2.1.3-1).

Shallow concentrated flow is characterized by the soil cover as either paved or unpaved. The flow velocity is calculated using the following formula:

$$V_s = KS^{0.5}$$

where:
- V_s = Average velocity of flow, fps;
- K = 16.1 for unpaved and 20.3 for paved soil cover; and
- S = Slope of the watercourse, ft/ft.

The time of travel for shallow concentrated flow is calculated as:

$$T_s = \frac{L}{60V_s}$$

where:
- T_s = Shallow concentrated flow travel time, min;
- L = Flow length, ft; and
- V_s = Average velocity of flow, fps.

Channelized Flow

Channelized flow is drainage in gutters, storm drains, channels, and streams. Generally, in the analysis of channelized flow it is necessary to breakdown the flow into a series of reaches, each reach having its own characteristics, to better estimate the travel time. The total time of travel of the channelized flow is the sum of the times of travel for each segment. Flow velocities are calculated using the Manning equation with Q_p for the 2-year flood.

For natural and constructed channels and street gutters, the velocity (V_h) may be calculated by assuming uniform bank full flow. For closed conduit systems on flat grades not being hydraulically analyzed for the project, it may be reasonable to calculate V_h

assuming uniform half-full flow. After computing the velocity, the time of travel for channelized flow is calculated with the following equation:

$$T_h = \frac{L}{60V_h}$$

where:
T_h = Channelized flow travel time, min;
L = Flow length, ft; and
V_h = Average velocity of flow, fps.

Flow through ponds or lakes and where the calculated velocity for channelized flow for post project conditions is less than 3 fps, then the flow should be assumed to travel at wave celerity:

$$T_h = c = (g\ d_m)^{0.5}$$

where:
c = Wave celerity, fps;
g = 32.2 = Acceleration of gravity, feet per second per second (ft/sec^2); and
d_m = Average depth of flow, ft.

Time to Inlet

The time to inlet is the time of travel of the water flow to the inlet considering overland flow, shallow concentrated flow, and channelized flow. Minimum times of travel to the inlet are specified in Table 2.1.3-3. These minimum times to inlet may be used for T_c at inlets in lieu of calculating T_c for post project conditions. However, the calculated time to inlet shall be used when determining T_c downstream of an inlet.

For undeveloped pre-project conditions, T_c shall always be calculated and overland flow shall be assumed to occur for the first 300 feet of flow, unless there is a defined stream depicted on detailed topographic maps. If the calculated T_c is less than 20 minutes, then the 20-minute minimum time to inlet shall apply. This 20-minute minimum time to inlet shall only be used for undeveloped pre-project conditions.

Time of Travel

T_c for design points downstream of inlets shall be calculated using the time to inlet (i.e., the calculated T_c, minimum times to inlet shall not be used) plus the time of travel (T_t) of the flow through the channelized flow segments downstream of the inlet. For small drainage systems with short times of travel, the channelized flow segments downstream of the inlet for post project conditions may be neglected for design purposes. Time of travel (T_t) downstream of inlets shall be computed using the hydraulic procedures as previously specified for channelized flow (T_h).

Section 2.1.3.5 – Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in in/hr for a duration equal to the time of concentration for a selected return period. (Intensity equals depth divided by duration.) Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration data given in the rainfall values in Figure 2.1.2-1.

Section 2.1.3.6 – Runoff Coefficient (C)

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 2.1.3-2 gives the recommended runoff coefficients for the Rational Method.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 2.1.3-2 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as rooftops, asphalt, and concrete streets and sidewalks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

It should be remembered that the Rational Method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area, then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

It may be that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site. This should be checked particularly in areas where the overland portion is grassy (yielding a long t_c) to avoid underestimating peak runoff.

Section 2.1.4 – Unit Hydrograph Methods

A Unit Hydrograph model is designed to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a portion of the basin, commonly referred to as a subbasin. A component may represent a surface runoff entity, a stream channel, or a reservoir. Representation of a component requires a set of parameters which specify the particular characteristics of the component and mathematical relations which describe the physical processes. The result of the modeling process is the computation of streamflow hydrographs at desired locations in the river basin. 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 Wharton County. 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 detail, the widely used computer program HEC-HMS is recommended as the primary tool for modeling storm runoff hydrographs in Wharton County. 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's, SCS and Snyder's unit hydrograph method, design storms and rainfall loss rates. A methodology for deriving the parameters used to compute the Clark's Unit Hydrograph was developed from optimization studies utilizing U.S. Geological Survey 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 (see Table 2.1.1-1 for size criteria). In situations requiring determination of a complete flood hydrograph, and not just a peak discharge for small areas, the Small Watershed Method should be utilized. If the engineer wishes to use an alternative design technique, it is required that the Drainage Review Authority be consulted prior to design.

Section 2.1.4.1 – 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. Two commonly used loss methods in Wharton County are the Green and Ampt Method and the SCS Method.

1. Green and Ampt Loss Method

All rainfall-runoff losses computed using the Green and Ampt loss method should follow these guidelines. Parameters required for loss calculations using Green and Ampt include:

- Initial Loss (inches)
- Volume Moisture Deficit
- Wetting Front Suction (inches)
- Hydraulic Conductivity (in/hr)
- Imperviousness (percent)

The Green and Ampt Parameters were determined similar to the guidelines set forth in a Tropical Storm Allison Recovery Project (TSARP) paper titled “Replacing HEC-1 Exponential Loss Function in HEC-HMS.” The Initial Loss and Hydraulic Conductivity were determined from the predominant hydrologic soil group. The Volume Moisture Deficit and Wetting Front Suction were determined from parameters related to soil texture. The values used for each of these parameters were originally developed as part of TSARP and results were favorable using the same values for this study. The Green and Ampt loss parameters are shown in Table 2.1.4-1. The Volume Moisture Deficit and Wetting Front Suction were both applied based on the soil texture for each sub-basin using a weighted average based on area. The Hydraulic Conductivity parameter should be applied uniformly to individual sub-watersheds based on the predominant soil type.

Table 2.1.4-1: Green and Ampt Loss Parameters

	Volume Moisture Deficit	**Wetting Front Suction (in)**	**Hydraulic Conductivity (in/hr)**
Soil Texture			
Sand	0.402	2.41	2.354
Loamy Sand	0.402	2.41	2.354
Sandy Loam	0.412	4.33	0.858
Loam	0.436	3.5	0.52
Silt Loam	0.486	6.57	0.268
Sandy Clay Loam	0.33	8.6	0.118
Clay Loam	0.389	8.22	0.079
Silty Clay Loam	0.431	10.75	0.079
Sandy Clay	0.321	9.41	0.047
Silty Clay	0.423	11.5	0.039
Clay	0.385	12.45	0.024
Hydrologic Soil Group			
A	0.417	1.95	2.354
B	0.436	3.5	0.52
C	0.389	8.22	0.079
D	0.385	12.45	0.024

2. SCS Method

This Method which is also commonly used to compute losses for unit hydrograph methods was developed by the Soil Conservation Service (SCS), now called the Natural Resources Conservation Service (NRCS). Runoff Factors, known as "Curve Numbers" or "CN" values are computed based on a standardized methodology. In addition to rainfall losses, the CN values can also be applied to the SCS Unit Hydrograph Method as described in Section 2.1.4.4.

The hydrologic soil textures and types can be obtained from the U.S. Natural Resources Conservation Service, Soil Survey Geographic databases (SSURGO) for Wharton, Fort Bend, Colorado, Austin, and Brazoria Counties. The predominant soil type within the San Bernard Watershed was Hydrologic Soil type D with some various combinations of Soil Type A, B, and C in the upper watershed particularly in the areas of Austin and Colorado County. A comprehensive discussion of the derivation of SCS "CN" values follows.

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the SCS has divided soils into four hydrologic soil groups.

Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

A list of soils throughout the State of Texas and their hydrologic classification can be found in the publication *Urban Hydrology for Small Watersheds, 2nd Edition, Technical Release Number 55, 1986*. Soil Survey maps can be obtained from local USDA Natural Resources Conservation Service offices for use in estimating soil type. Appendix B contains hydrologic soils classification data for Wharton County. Specific data can be found on-line through NRCS at http://soils.usda.gov/.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions. Average antecedent soil moisture conditions (AMC II) are recommended for most hydrologic analysis. Areas with high water table conditions may want to consider using AMC III antecedent soil moisture conditions. This should be considered a calibration parameter for modeling against real calibration data. Table 2.1.4-2 gives recommended curve number values for a range of different land uses.

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by using the weighted method as presented below.

The different land uses within the basin should reflect a uniform hydrologic group represented by a single curve number. Any number of land uses can be included, but if

Composite Curve Number Calculation Example			
Land Use	Percent of Total Land Area	Curve Number	Weighted Curve Number (% area x CN)
Residential 1/8 acre Soil Group B	0.80	0.85	0.68
Meadow Good condition Soil Group C	0.20	0.71	0.14
Total Weighted Curve Number = 0.68 + 0.14 = 0.82			

their spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

Urban Modifications of the SCS Method

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The Curve Number values given in Table 2.1.4-2 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system. It is possible for curve number values from urban areas to be reduced by not directly connecting impervious surfaces in the drainage system, but allowing runoff to flow as sheet flow over significant pervious areas.

The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

The CNs provided in Table 2.1.4-2 for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

1. Pervious urban areas are equivalent to pasture in good hydrologic condition, and

2. Impervious areas have a CN of 98 and are directly connected to the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 2.1.4-2 are not applicable, use Figure 2.1.4-1 to compute a composite CN. For example, Table 2.1.4-1 gives a CN of 70 for a 1/2-acre lot in hydrologic soil group B, with an assumed impervious area of 25%. However, if the lot has 20% impervious area and a pervious area CN of 61, the composite CN obtained from Figure 2.1.4 -1 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Table 2.1.4-2: CN Values

Cover Description		Curve numbers for hydrologic soil groups			
Cover type and hydrologic condition	*Average percent impervious area*[2]	A	B	C	D
Cultivated Land:					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land:					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow:					
Good condition		30	58	71	78
Wood or forest land:					
Thin stand, poor cover		45	66	77	83
Good cover		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town house)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (previous areas only, no vegetation)		77	86	91	94

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

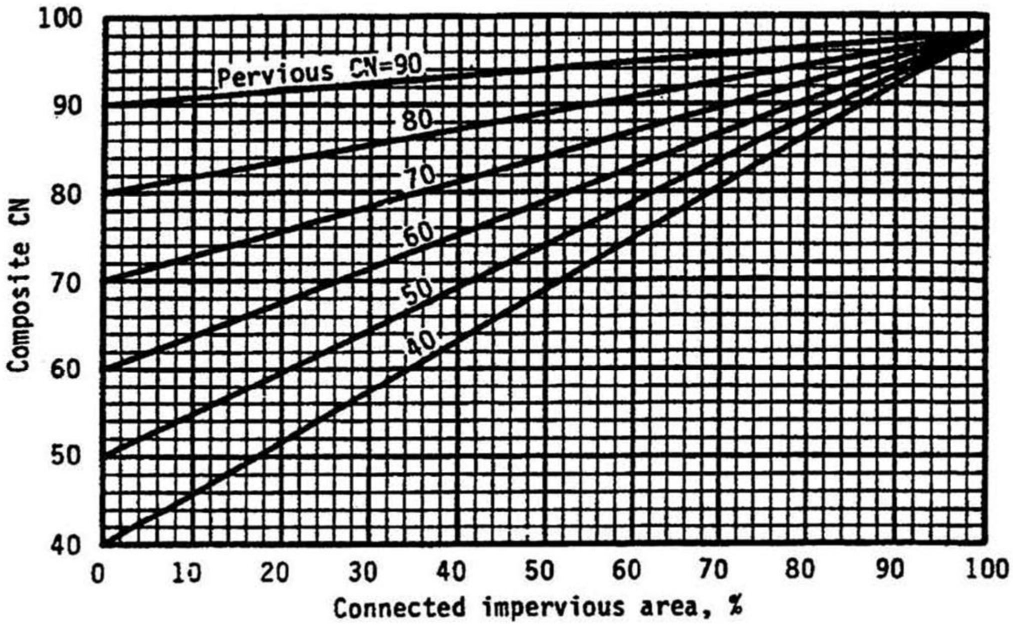


Figure 2.1.4-1 Composite CN with Connected Impervious Areas
(Source: SCS, TR-55, Second Edition, June 1986)

Section 2.1.4.2 – 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 stormwater detention generally used in reducing downstream peak flood flow rates. Open channel routing is a refinement of the description of an area's rainfall-run-off 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 detailed 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.

Section 2.1.4.3 – Clark's Unit Hydrograph Method

Hydrographs and corresponding peak discharges for each sub-basin using Clark's Unit Hydrograph Method should follow these procedures. The Clark Time of Concentration (Tc) and Storage Coefficient (R) for each sub-basin were calculated using formulas derived by the Harris County Flood Control District (HCFCD) in the mid- 1980s. Please note that Clark's Tc is defined differently than the Time of Concentration for the Rational or SCS methods. Ponded areas required for determining percent ponding can be calculated by delineating rice fields and farm ponds from aerial photos. The percent urbanization parameter can be determined by using existing land use to locate areas of urbanization. Other parameters used in this method such as percent channel improvement and percent channel conveyance were calculated using channel data but are not always necessary for rural watersheds. The equations HCFCD developed for calculating Tc and R are as follows.

$$Tc = D*[1-(0.0062*(0.30*(DLU)+0.70*(DCI)))]*(Lca/\sqrt{S})^{1.06} \qquad (2.1\text{-}4)$$

D = 2.46 if So<=20 ft./mi.

D = 3.79 if So>20 ft./mi/ but So<40 ft./mi.

D = 5.12 if So>40 ft./mi.

$$Tc+R = 7.25*(L/\sqrt{S})^{0.706} \qquad (\text{if DLU} <= 18\%) \qquad (2.1\text{-}5)$$

$$Tc+R = (4295[DLU]^{-0.678}*[DCC]^{-0.967})*(L/\sqrt{S})^{0.706} \ (\text{if DLU} > 18\%) \qquad (2.1\text{-}6)$$

Tc = Time of Concentration
DLU = % Land Urbanization
DCI = % Channel Improvement
Lca = Length to Centroid
S = Channel Slope
So = Watershed Slope
L = Watershed Length
DCC = % Channel Conveyance
R = Storage Coefficient

Figure 2.1.4-3 graphically illustrates the application of this method.

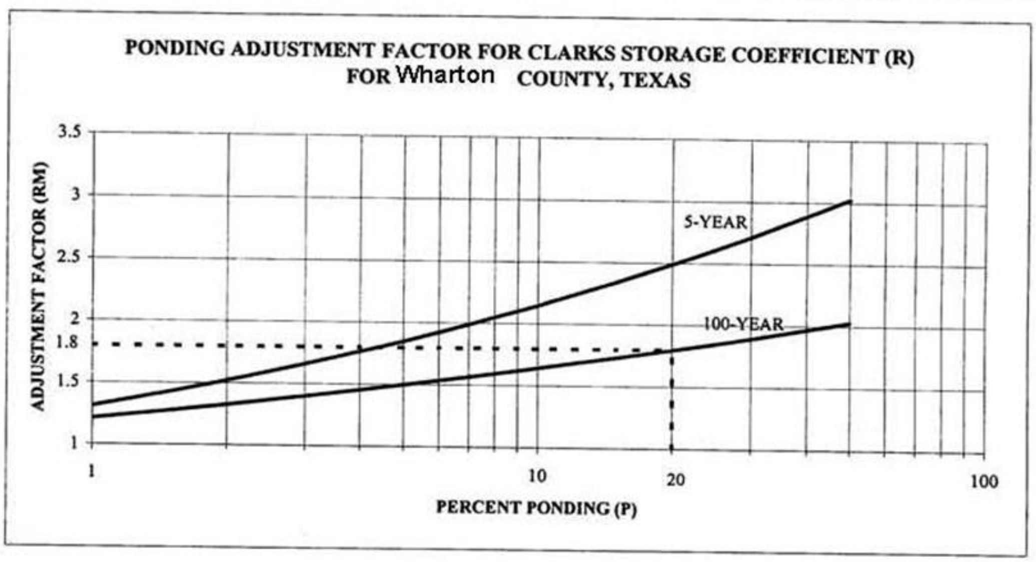
Percent Ponding

Percent ponding (DPP) is the portion of a subarea where runoff is retarded from reaching a watercourse due to obstructions or natural storage. Such obstructions include leveed fields (rice farms), swamps, etc. It is expressed as a percent of the total drainage area.

An adjustment factor can be applied to the storage coefficient if percent ponding is greater than 20%. The adjustment factor in the HCFCD method varied for each frequency storm being considered. For Wharton County, the 100-year adjustment factor can be used for all storm events to limit the repetitive models that would be needed to vary this factor for different frequency events. The other frequency events are also presented in Figure 2.1.4-2 if needed.

Percent ponding is used to increase Clark's storage coefficient (R), after its value has been calculated through the unit graph parameter equations. The adjustment of R due to the percent ponding is dependent upon which storm frequency is being analyzed. Figure 2.1.4-2 graphically illustrates the relationship between percent ponding and the adjustment factor for R. The equations for this relationship are also given on the exhibit. The percent ponding factor should only be used when the ponded areas cover at least 20% of the watershed.

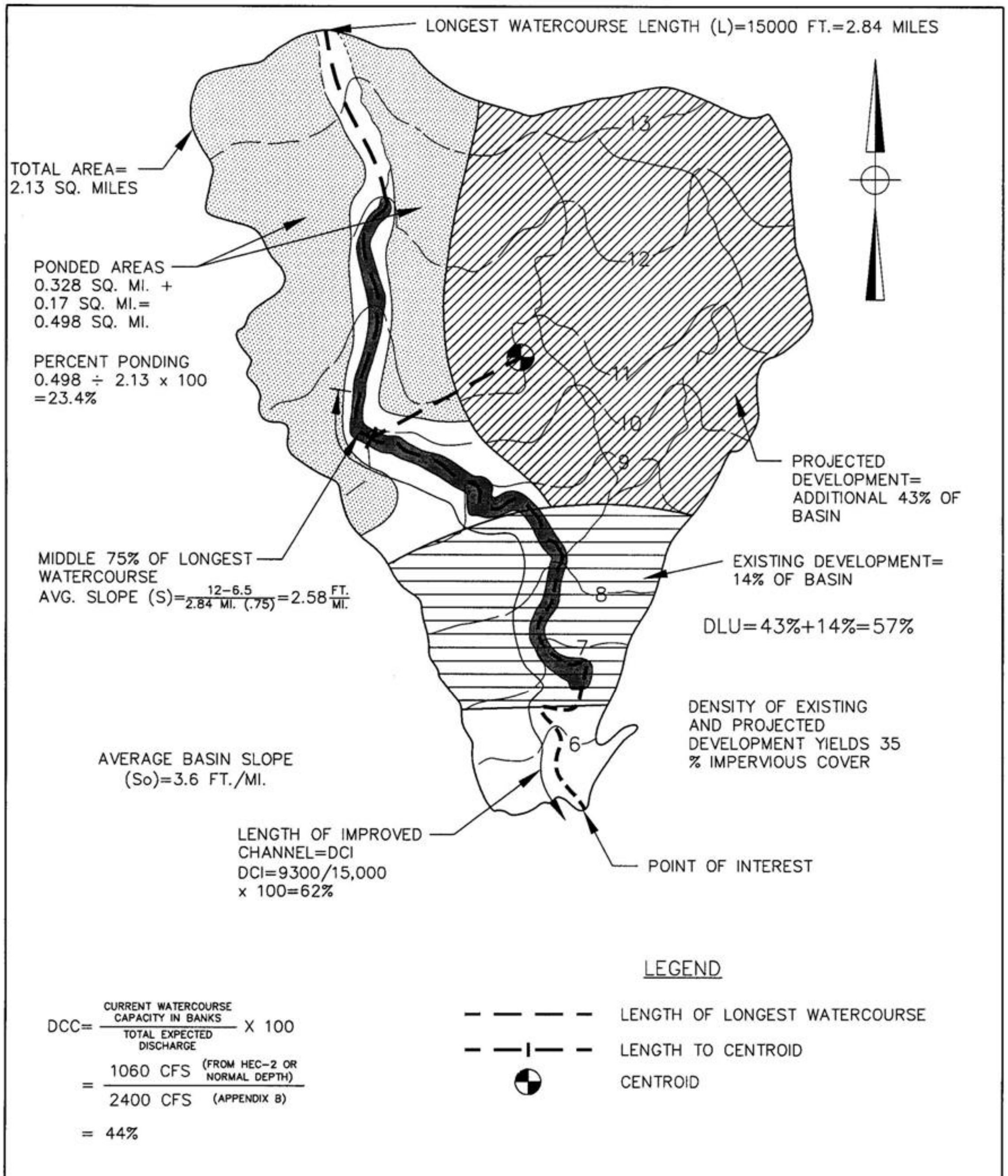
The flooded portion of a reservoir has 0% ponding. This is because the runoff will not be delayed from reaching a watercourse. The flooded portion of a reservoir will actually produce more runoff than the surrounding area since all of the rainfall is converted to runoff and none is lost to infiltration. Reservoir attenuation is accounted for in storage routing computations.



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

Note:
Percent ponding factor is generally not used when the ponded areas cover less than 20 % of subarea.

Figure 2.1.4-2



EXAMPLE WATERSHED FOR
DETERMINATION OF BASIN
PHYSICAL PARAMETERS

Figure 2.1.4-3

Section 2.1.4.4 – SCS Unit Hydrograph Method

The Soil Conservation Service (SCS) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4, Hydrology.

A typical application of the SCS method includes the following basic steps:

1. Determination of curve numbers that represent different land uses within the drainage area.

2. Calculation of time of concentration to the study point.

3. Using the Type II rainfall distribution, total and excess rainfall amounts are determined. Note: See Figure 2.1.4-4 for the geographic boundaries for the different SCS rainfall distributions.

4. Using the unit hydrograph approach, the hydrograph of direct runoff from the drainage basin can be developed.

Application

The SCS method can be used for both the estimation of storm water runoff peak rates and the generation of hydrographs for the routing of storm water flows. The SCS method can be used for drainage areas up to 2,000 acres. Thus, the SCS method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches, open channels, and energy dissipators.

Equations and Concepts

The hydrograph of outflow from a drainage basin is the sum of the elemental hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical characteristics of the basin including shape, size, and slope are constant, the unit hydrograph approach assumes there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the runoff volume. Therefore, a storm that produces two (2) inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the flow of the hydrograph would be one-half of the flow of the unit hydrograph.

The following discussion outlines the equations and basic concepts used in the SCS method.

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, combine hydrographs from different sub-basins as applicable, and/or route flows to points of interest.

Rainfall - The SCS method applicable to South Texas is based on a storm event that has a Type III time distribution. This distribution is used to distribute the 24-hour volume of rainfall for the different storm frequencies (Figure 2.1.4-4).

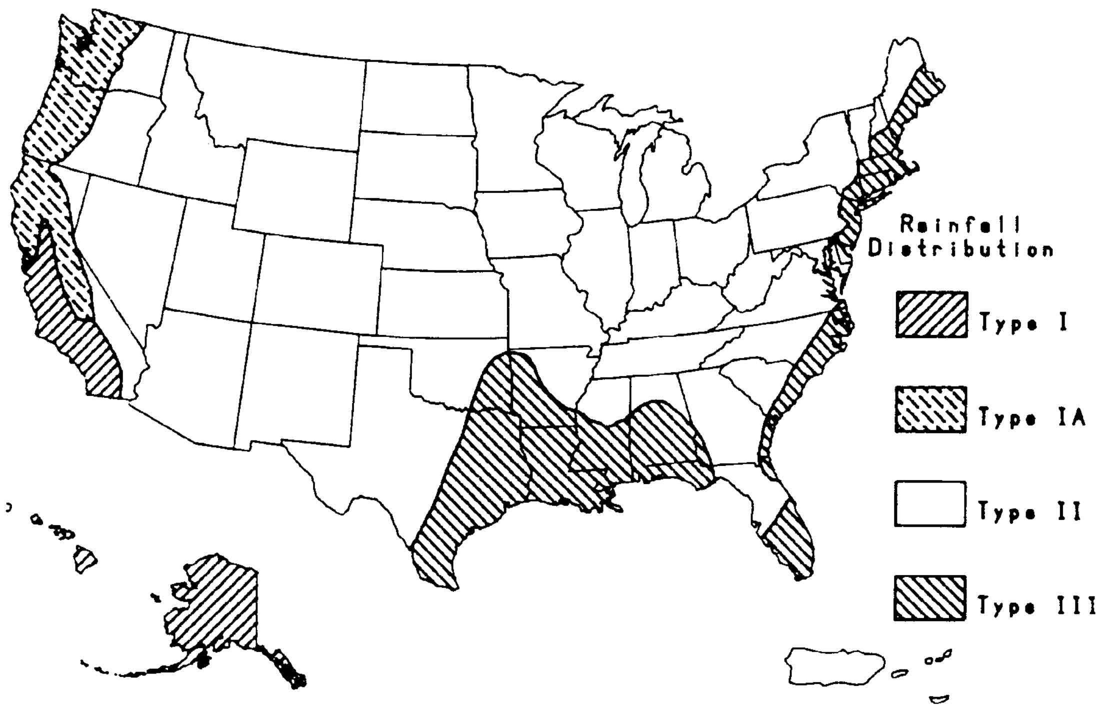


Figure 2.1.4-4 Approximate Geographic Boundaries for SCS Rainfall Distributions

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The following SCS runoff equation is used to estimate direct runoff from the 24-hour storm rainfall. The equation is:

$$Q = (P - I_a)2 / [(P - I_a) + S]$$

where:

Q = accumulated direct runoff (in)
P = accumulated rainfall (potential maximum runoff) (in)
I_a = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)
S = 1000/CN - 10 where CN = SCS curve number

An empirical relationship used in the SCS method for estimating I_a is:

$$I_a = 0.2S$$

This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment. Table 2.4.1-3 provides values of I_a for a wide range of curve numbers (CN).

Substituting $0.2S$ for I_a, the equation becomes:

$$Q = (P - 0.2S)^2 / (P + 0.8S)$$

Figure 2.1.4-5 shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurred on a watershed with a curve number of 85. The curve number can be estimated if rainfall and runoff volume are known with the following equation (Pitt, 1994):

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}]$$

Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another within a watershed, through the various components of the drainage system. Time of concentration (T_c) is computed by summing all the travel times for consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed. Following is a discussion of related procedures and equations (USDA, 1986). Section 2.1.3.4, "Time of Concentration" under "Rational Method" presents the recommended method for calculating this time of concentration.

Table 2.1.4-3 I_a Values for Runoff Curve Numbers			
Curve Number	**I_a (in)**	**Curve Number**	**I_a (in)**
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.74	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Source: SCS, TR-55, Second Edition, June 1986

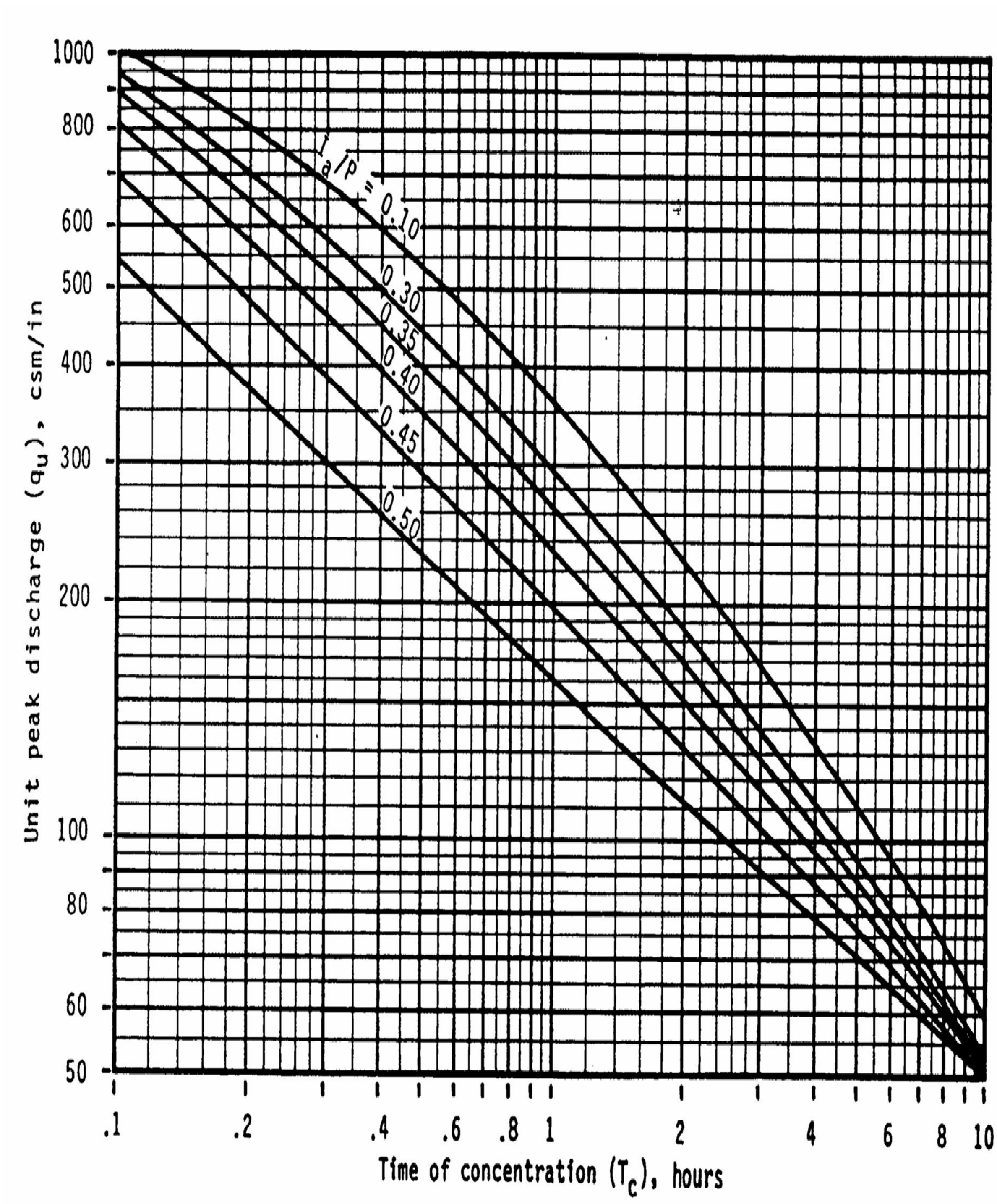


Figure 2.1.4-5 SCS Type II Unit Peak Discharge Graph

(Source: SCS, TR-55, Second Edition, June 1986)

Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The SCS has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for small drainage areas (less than 2,000 acres). The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, SCS has developed hydrograph procedures to be used to generate composite flood hydrographs. For the development of a hydrograph from a homogeneous developed drainage area and drainage areas that are not homogeneous, where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the SCS in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

The unit hydrograph equations used in the SCS method for generating hydrographs includes a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. SCS indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat (coastal) areas.

A value of 300 should be used for most areas of Wharton County which are relatively flat.

The development of a runoff hydrograph from a watershed is a laborious process not normally done by hand calculation. For that reason, only an overview of the process is given here to assist the designer in reviewing and understanding the input and output from a typical computer program. There are choices of computational interval, storm length (if the 24-hour storm is not going to be used), and other "administrative" parameters, which are peculiar to each computer program.

The development of a runoff hydrograph for a watershed or one of many sub-basins within a more complex model involves the following steps:

1. Development or selection of a design storm hyetograph. Often the SCS 24-hour storm described in the Equations and Concepts portion of this subsection is used.
2. Development of curve numbers and lag times for the watershed using the methods described in this manual.
3. Development of a unit hydrograph using the standard (peaking factor of 484) dimensionless unit hydrograph. See discussion below.
4. Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation.
5. Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of rainfall (this is called "convolution").

6. Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

To assist the designer in using the SCS unit hydrograph approach with a peaking factor of 484, Figure 2.1.4-6 and Table 2.1.4-4 have been developed. The unit hydrograph with a peaking factor of 300 is shown in the figure for comparison purposes, and may be typical of areas in Wharton County.

The procedure to develop a unit hydrograph from the dimensionless unit hydrograph in the table below is to multiply each time ratio value by the time-to-peak (T_p) and each value of q/q_u by q_u calculated as:

$$q_u = (PF\ A) / (T_p)$$

where:

q_u	=	unit hydrograph peak rate of discharge (cfs)
PF	=	peaking factor (484)
A	=	area (mi^2)
d	=	rainfall time increment (hr)
T_p	=	time to peak = d/2 + 0.6 T_c (hr)

For ease of spreadsheet calculations, the dimensionless unit hydrograph for 484 can be approximated by the equation:

$$q / q_u = \left(t/T_p e^{[1-(t/T_p)]}\right)^{x}$$

where X is 3.79 for the PF=484 unit hydrograph and higher for lower PF values.

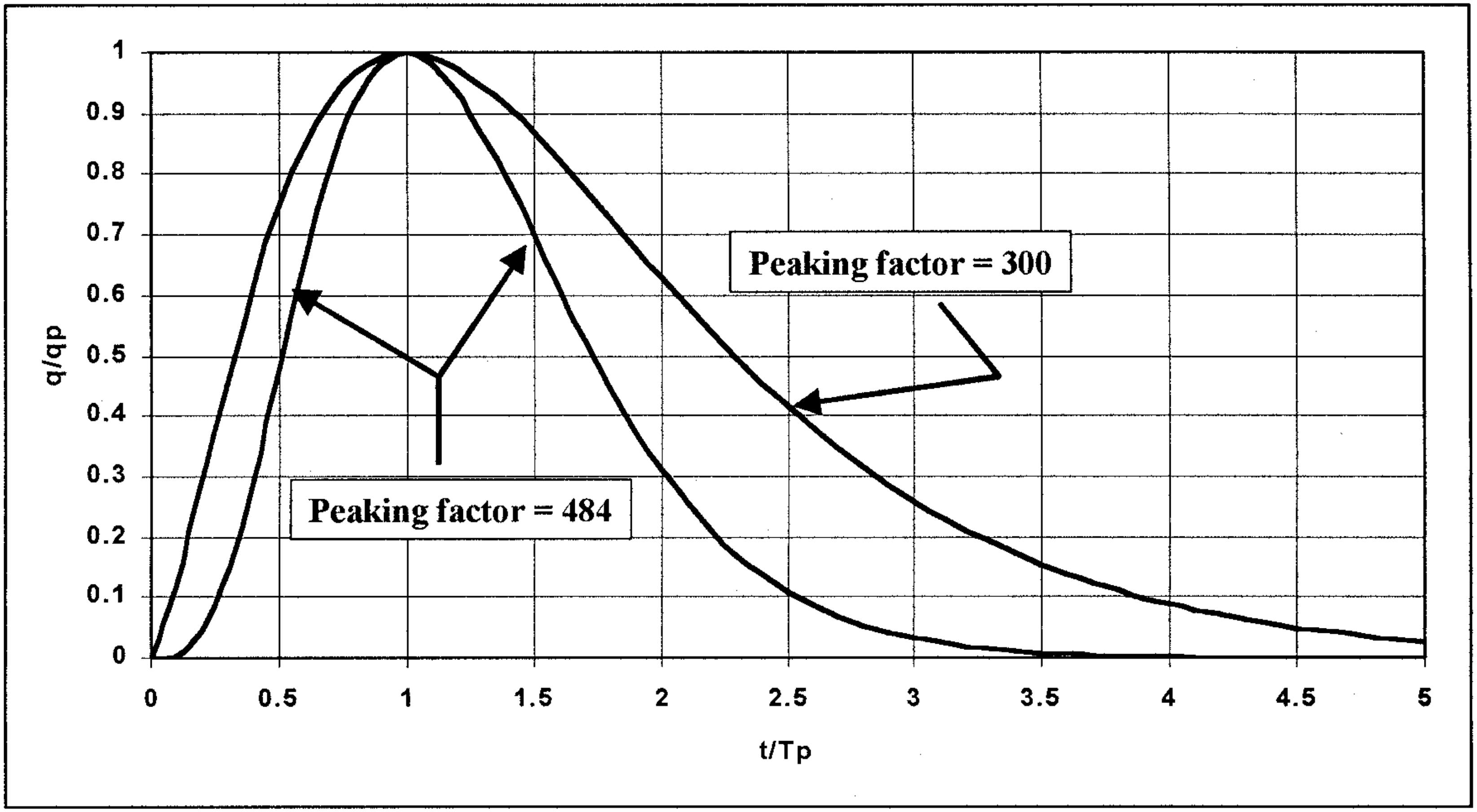


Figure 2.1.4-6 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

Table 2.1.4-4 Dimensionless Unit Hydrograph With Peaking Factor of 484

Time Ratio t/T_t	Discharge Ratio 484 q/q_u	Mass CurveRatio Q/Q_p
0.0	0.0	0.0
0.1	0.005	0.000
0.2	0.046	0.004
0.3	0.148	0.015
0.4	0.301	0.038
0.5	0.481	0.075
0.6	0.657	0.125
0.7	0.807	0.186
0.8	0.916	0.255
0.9	0.980	0.330
1.0	1.000	0.406
1.1	0.982	0.481
1.2	0.935	0.552
1.3	0.867	0.618
1.4	0.786	0.677
1.5	0.699	0.730
1.6	0.611	0.777
1.7	0.526	0.817
1.8	0.447	0.851
1.9	0.376	0.879
2.0	0.312	0.903
2.1	0.257	0.923
2.2	0.210	0.939
2.3	0.170	0.951
2.4	0.137	0.962
2.5	0.109	0.970
2.6	0.087	0.977
2.7	0.069	0.982
2.8	0.054	0.986
2.9	0.042	0.989
3.0	0.033	0.992
3.1	0.025	0.994
3.2	0.020	0.995
3.3	0.015	0.996
3.4	0.012	0.997
3.5	0.009	0.998
3.6	0.007	0.998
3.7	0.005	0.999
3.8	0.004	0.999
3.9	0.003	0.999
4.0	0.002	1.000

Section 2.1.4.5 – Snyder's Unit Hydrograph Method

Introduction

Snyder's Unit Hydrograph Method is a method sometimes utilized by the Corps of Engineers Galveston District for hydrologic studies in the region, and is also used by consultants and other entities within the region. It is similar in nature to the SCS method, in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. This method was used for the most recent study of the main stem of the Colorado River through Wharton County but should be used for new watershed studies only with approval of the Drainage Review Authority.

Application

Snyder's unit hydrograph method may be used for drainage areas 100 acres or larger. This method, detailed in the U.S. Army Corps of Engineers Engineering Manual (EM 1110-2-1405), *Flood-Hydrograph Analysis and Computations* and The Bureau of Reclamation's "Flood Hydrology Manual, A Water Resources Technical Publication." utilizes the following equations:

$$t_p = C_t (L\, L_{ca})^{0.3} \quad (2.1.7)$$

$$t_r = t_p \div 5.5 \quad (2.1.8)$$

$$q_p = C_p 640 \div t_p \quad (2.1.9)$$

$$t_{pR} = t_p + 0.25(t_R - t_r) \quad (2.1.10)$$

$$q_{pR} = C_p 640 \div t_{pR} \quad (2.1.11)$$

$$q_{pR} = q_p\, t_p \div t_{pR} \quad (2.1.12)$$

$$Q_p = q_p\, A \quad (2.1.13)$$

The terms in the above equations are defined as:

- t_r = The standard unit rainfall duration, in hours.
- t_R = The unit rainfall duration in hours other than standard unit, t_r, adopted in specific study.
- t_p = The lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours.
- t_{pR} = The lag time from midpoint of unit rainfall duration, t_R, to peak of unit hydrograph in hours.
- q_p = The peak rate of discharge of unit hydrograph for unit rainfall duration, t_r, in cfs/sq. mi.
- q_{pR} = The peak rate of discharge in cfs/sq mi. of unit in hydrograph for unit rainfall duration, t_R.

Q_p = The peak rate of discharge of unit hydrograph in cfs.
A = The drainage area in square miles.
L_{ca} = The river mileage from the design point to the centroid of gravity of the drainage area.
L = The river mileage from the given station to the upstream limits of the drainage area.
C_t = Coefficient depending upon units and drainage basin characteristics.
C_p = Coefficient depending upon units and drainage basin characteristics.

Table 2.1.4-5 Typical Values of C_p

Typical Drainage Area Characteristics	Value of C_p
Undeveloped Areas w/ Storm Drains	
Flat Basin Slope (less than 0.50%)	0.55
Moderate Basin Slope (0.50% to 0.80%)	0.58
Steep Basin Slope (greater than 0.80%)	0.61
Moderately Developed Area	
Flat Basin Slope (less than 0.50%)	0.63
Moderate Basin Slope (0.50% to 0.80%)	0.66
Steep Basin Slope (greater than 0.80%)	0.69
Highly Developed/Commercial Area	
Flat Basin Slope (less than 0.50%)	0.70
Moderate Basin Slope (0.50% to 0.80%)	0.73
Steep Basin Slope (greater than 0.80%)	0.77

The coefficient C_t is a regional coefficient for variations in slopes within the watershed. Typical values of C_t range from 0.4 to 2.3 and average about 1.1. The value of C_t for the Lower Colorado River is 2.2. C_t for a watershed can be estimated if the lag time, t_p stream length, L, and distance to the basin centroid, L_{ca}, are known. The coefficient C_p is the peaking coefficient, which typically ranges from 0.3 to 1.2 with an average value of 0.8, and is related to the flood wave and storage conditions of the watershed. The C_p value for the Colorado River is 0.70. Larger values of C_p are generally associated with smaller values of C_t. Typical values of C_p are listed in Table 2.1.4-5

Section 2.1.5 – Small Watershed Method

A technique for hydrograph development which is useful in the design of detention facilities serving relatively small watersheds (up to approximately 2000 acres) has been presented by H.R. Malcom. 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 curvalinear design hydrograph which peaks at the design flow rate and which contains a runoff volume consistent with the design rainfall. The pattern hydro graph is a step function approximation to the dimensionless hydrograph proposed by the Bureau of Reclamation and the SCS (Now NRCS).

Malcom's Method consists of the following equations:

(1) $$T_p = \frac{V}{1.39Q_p} \tag{2.1.14}$$

(2) $$q_i = \frac{Qp}{2}\left[1 - \cos\left(\frac{\pi ti}{T_p}\right)\right] \tag{2.1.15}$$

for $t_i \leq 1.25\ T_p$

(3) $$q_i = 4.34Q_p e^{-1.30\frac{t_i}{T_p}} \tag{2.1.16}$$

for $t_i > 1.25\ T_p$

*Calculator must be in radian mode.

where:

Qp = peak design flow rate in cfs

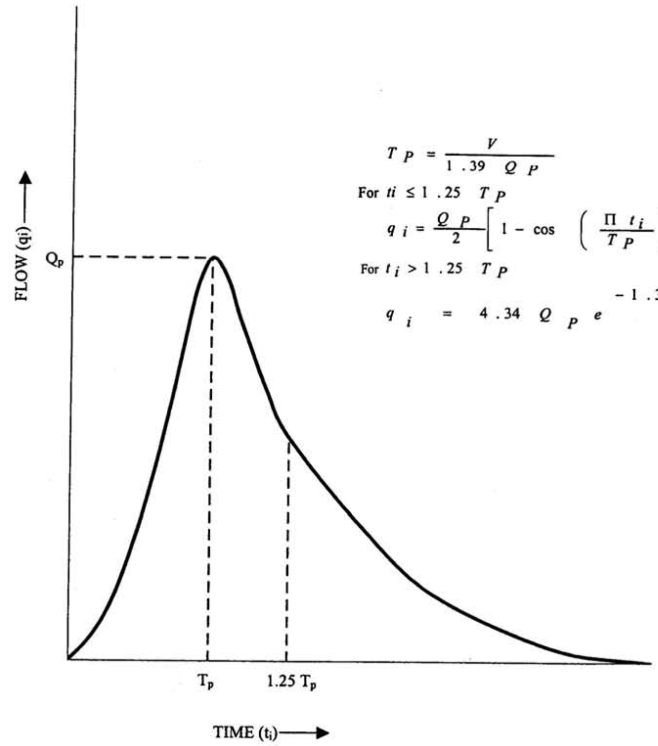
Tp = time to Qp in seconds

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 2.1.5-1.

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).



$$T_P = \frac{V}{1.39\,Q_P}$$

For $t_i \le 1.25\,T_P$

$$q_i = \frac{Q_P}{2}\left[1 - \cos\left(\frac{\Pi\,t_i}{T_P}\right)\right]^*$$

For $t_i > 1.25\,T_P$

$$q_i = 4.34\,Q_P\,e^{-1.30\frac{t_i}{T_P}}$$

* 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 .

NOV 2003 **Figure 2.1.5-1**

Section 2.1.6 – 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 hydrologic features such as ponding areas, storage basins or watershed overflows. The curves developed for this manual for the 25- and 100-year events, respectively, are shown in Figures 2.1.6-1 and 2.1.6-2, and are applicable to drainage areas between 200 and 2000 acres. These curves are based on "best fit" plots of typical Unit Hydrograph and Rational method studies with comparisons to similar curves used in other area counties. 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 condition in the design of detention facilities should be determined on a case-by-case basis working closely with the Drainage Review Authority (See Chapters 2 and 6). Whenever the situation requires the determination of complete flood hydrograph, and not just a peak discharge, the Small Watershed Method or one of the Unit Hydrograph methods, as described in Chapter 2, should be used.

Section 2.1.7 – TxDOT Regression Equations

Regional regression equations are the most commonly accepted method for establishing peak flows at larger ungauged sites (or sites with insufficient data for a statistical derivation of the flood versus frequency relation). Regression equations have been developed to relate peak flow at a specified return period to the physiography, hydrology, and meteorology of the watershed.

Regression analyses use stream gauge data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar watershed, channel, and meteorological characteristics; they are often termed hydrologically homogeneous geographic areas. It may be difficult to choose the proper set of regression equations when the design site lies on or near the hydrologic boundaries of relevant studies. Another problem occurs when the watershed is partly or totally within an area subject to mixed population floods.

The following suggestions should be considered when using regression equations:

Conduct a field visit to compare and assess the watershed characteristics for comparison with other watersheds.

Collect all available historical flood data.

- Use the gathered data to interpret any discharge values.

Figure 2.1.6-1 25-Year Drainage Area - Discharge Curves for Wharton County, Texas

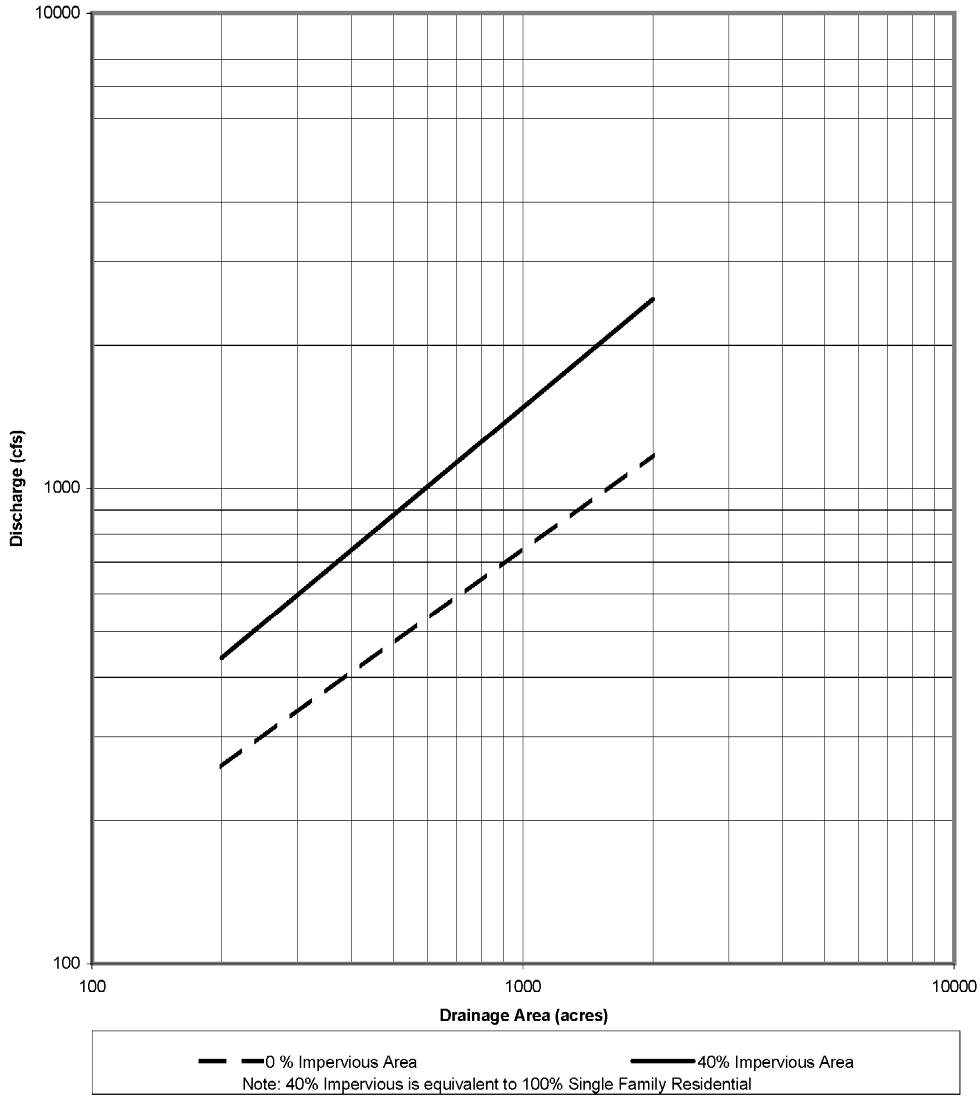
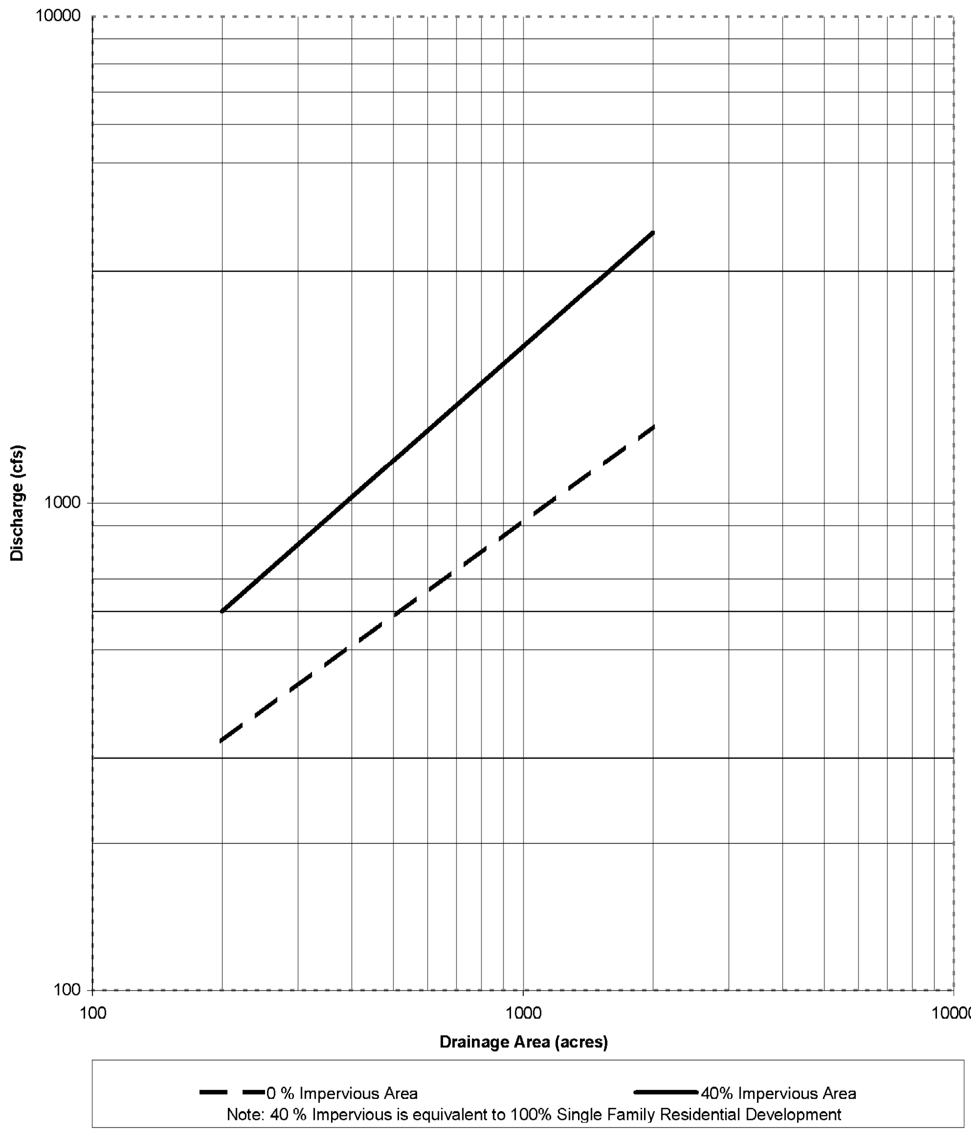


Figure 2.1.6-2 100-Year Drainage Area-Discharge Curves for Wharton County, Texas



Section 2.1.8 – Downstream Hydrologic Assessment

Storm drainage from a development must be carried to an "adequate outfall" or "acceptable outfall." An adequate outfall is one that does not create adverse flooding or erosion conditions downstream and is in all cases subject to the approval of the Drainage Review Authority. A “zone of influence” from a proposed development extends to a point downstream where the discharge from a proposed development no longer has a significant impact upon the receiving stream or storm drainage system. Downstream impacts due to a development must be analyzed and mitigated for the 2-, 10-, and 100-year floods for the entire zone of influence, as determined by the engineer’s analysis. The Zone of Influence for any proposed development must be defined by the engineer, based on a drainage study that determines the specific location along the drainage route where “no adverse impacts” from the new development exist**.**

The Drainage Plan (see description in chapter 1) will include the necessary hydrologic and hydraulic analyses to clearly demonstrate that the limits of the Zone of Influence have been identified, and that along the drainage route to that location, these parameters are met:

- No new or increased flooding of existing insurable (FEMA) structures (habitable buildings),
- No significant (0.1’) increases in flood elevations over existing roadways for the 2-, 10- and 100-year floods.
- No significant rise (0.1’ or less) in 100-year flood elevations, unless contained in existing channel, roadway, drainage easement and/or R.O.W.
- No significant increases in channel velocities for the 2-, 10- and 100-year floods. Post-development channel velocities shall not exceed the pre-development velocities (maximum increase of 5% allowed), but will not exceed six (6) feet per second, or the applicable maximum permissible velocity shown in Table 4.2.3-1, whichever is lower. Exceptions to this criteria will require certified geotechnical/geomorphologic studies that provide documentation that higher velocities will not create additional erosion.
- No increases in downstream discharges caused by the proposed development that, in combination with existing discharges, exceeds the existing capacity of the downstream storm drainage system.

CHAPTER 3 – Hydraulic Design of Streets and Closed Conduits

Section 3.1 – Street Drainage

Section 3.1.1 – Storm Water System Design

Street Purposes

The primary purpose of streets is transportation: to offer effective mobility for all users, and to ensure that each land parcel has reasonable access. Stormwater collection and conveyance is an important, but secondary purpose. Consequently, designs for handling storm flow should minimize interference with transportation uses. In general, the more important the street (in terms of functional classification) the more important it is that stormwater design not interfere with transportation uses. Conversely, moderate interference with transportation uses is more acceptable on lower class streets.

Flow Parameters

The design flow of water in streets shall be related to the extent and frequency of interference with traffic as related to street functional class and the chance of flood damage to surrounding properties. Interference with traffic is regulated by design limits of the spread of water into traffic lanes. Flooding of surrounding properties is regulated by limiting the depth of flow at the curb and by containment of the 100-year design storm flow within the street right of way.

Section 3.1.2 – Performance Standards and Limitations

a. Velocity of Flow

(1). The maximum velocity of street flow shall not exceed 10 feet/second. At “T” street intersections flow velocity must be checked on the stem of the “T” to ensure that flow will not traverse the crown and opposing curb of the crossing street and enter onto private property.

(2). A minimum velocity shall be maintained to ensure cleansing flushes at low flows by keeping the minimum gutter slope to five tenths of one percent (0.005 ft/ft) without specific approval of the Drainage Review Authority.

b. Allowable Depth of Flow

Top of Curb

The depth of flow shall be limited to the top of curb for a design storm having a return period of five (5) years.

Within ROW

Design flows for storms with an average return period up to and including 100 years* shall be confined within the limits of the street right-of-way until discharged into a drainage easement or drainage ROW that is part of the designated Conveyance Pathway system, or directly into a main channel of the primary drainage system. The capacity of the storm drain system shall be increased beyond other design criteria in these Guidelines as necessary to ensure this objective. Design computations shall demonstrate satisfaction of this criterion.

*100-year capacity unless a lesser magnitude storm is specifically approved by the Drainage Review Authority.

c. Grades and Cross-slopes

Street grades and cross-slopes shall be consistent with Wharton County or City Technical Specifications.

d. Allowable Water Spread

(1). Local Streets – The design storm flow in local streets shall be limited to the top of crown or the top of curb, whichever is less. Stormwater shall be removed from the streets by inlets or openings into adjacent drainage systems. These shall be placed at low points and as frequently as necessary to avoid exceeding water spread and depth criteria. The design storm shall have a return period of five (5) years.

(2). Collector Streets – Design storm flow in collector streets shall be limited so that one 12-foot wide area (one traffic lane width) at the center of the street will remain clear of water. Stormwater shall be removed from the street by inlets or openings into adjacent drainage systems. These shall be placed at low points and as frequently as necessary to avoid exceeding water spread and depth criteria. The design storm shall have a return period of ten (10) years.

(3). Arterial and Parkway Streets – Design storm flow in arterial and parkway streets (any street having a raised median regardless of classification) shall be limited so that one (1) twelve-foot traffic lane each direction at the center of the street (or one on each side of a raised median) will remain clear of water. Stormwater shall be removed from the street by inlets or openings into adjacent drainage systems. These shall be placed at low points and as frequently as necessary to avoid exceeding water spread and depth criteria. The design storm shall have a return period of ten (10) years.

(4). Intersections – Inlet placement and storm sewer size shall ensure that design storm flows are intercepted ("dried up") along street legs entering the intersection in advance of the curb returns connecting the streets based on the criteria provided below. In no case shall inlets be placed in the curved portion of curbs connecting intersecting streets. Where storm flow is allowed to pass through an intersection, valley gutter design must provide for smooth, uninterrupted traffic flow as stipulated by TxDOT Technical Specifications.

Intersection Pair	Intercept	Valley Gutter Criteria
Arterial – Arterial	All legs	No valley gutters
Arterial – Collector	All legs	No valley gutters
Arterial – Local	All legs	No valley gutters
Collector – Collector	All legs	No valley gutters
Collector – Local	Local legs	Valley gutters can parallel Collector
Local – Local	Two legs preferred	Valley gutters acceptable

(5). Mid block Cross Drainage – Where storm drainage is collected on one side of a street and must be conveyed to the other side, it shall be accomplished via underground conduit unless the roadway is functionally classified as a local street. Where storm flow is to cross such a local street the preferred conveyance is via underground conduit, however, at the discretion of the Drainage Review Authority, very low design flow may be conveyed in a valley gutter that satisfies TxDOT Technical specifications.

Section 3.1.3 – Design Procedure

a. Straight Crowns

Flows in streets which have a straight crown will be calculated using the following equation for triangular channels:

$$Q = 0.56\frac{z}{n}S^{0.5}Y^{2.67}$$

where,

Q = gutter discharge (cubic feet per second)

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

S = street or gutter slope (ft/ft)

n = Manning's roughness coefficient

Y = depth of flow (ft)

When flows over concrete or asphalt pavement are being calculated, the valve of "**n**" shall be taken as 0.016.

b. Parabolic Crowns

Flows in streets which have a parabolic crown become complicated and difficult to precisely solve for each design case. Design equations must be used to determine gutter flow when street design is to include parabolic crown sections. If parabolic crowns are planned, the concept is to be discussed during an initial meeting with the Drainage Review Authority or her/his designee.

Section 3.2 – Storm Drain Inlets

Section 3.2.1 – Principles

The purpose of a storm drain inlet is to intercept street or surface runoff and direct it into another component of the drainage system, usually an underground conduit. Inlets are typically of the curb opening type for streets and grate type for area drainage. Curb inlets occur at low points or on grade, and can have a throat that is either depressed or flush with the gutter invert grade. Grate inlets can occur at low points or on grade and may or may not be depressed.

Section 3.2.2 – Street Inlet Criteria

Recessed Inlets One concept for placemental inlets along arterial or major or minor collector streets is recession (horizontally displacement) away from the line of the curb so that any depression at the mouth of the inlet occurs wholly within the limits of the gutter, with no irregularity of elevation extending into the travel lane. A diagram of a recessed inlet is illustrated in Figure 3.2.2-1. Recessed inlets shall only be used with specific approval of the Drainage Review Authority.

Optional Design Inlets along streets classified as "local" may only be recessed with approval of Drainage Review Authority. See diagram in Figure 3.2.2.

Inlet Length Curb opening inlets shall have a minimum length of five (5) feet, and construction details shall conform to TxDOT Specifications.

Section 3.2.3 – Stormwater Inlets

Standard Inlets Standard inlets are classified into two groups: inlets in sumps (Type A) and inlets on grade (Type B). These are further subdivided as follows:

Inlets in Sumps

- Curb openings (with or without gutter depression) Type A-1
- Grate inlet; Type A -2

Inlets on Grade

- Curb openings with gutter depression Type B-1
- Curb openings without gutter depression Type B-2
- Grate Inlet

Combination Inlets A combination inlet is a side-by-side placement of a standard curb inlet and a grate inlet. The upstream inlet may be a standard curb inlet or simply part of an inlet. The benefit is that the curb opening tends to intercept debris that might otherwise clog the grate inlet. Such arrangements typically offer very little additional capacity over standard depressed inlets. In order to determine the capacity of a combination inlet on grade, it is recommended that the capacity of each (standard and grate) be calculated and the greater capacity be assumed for the pair for design purposes.

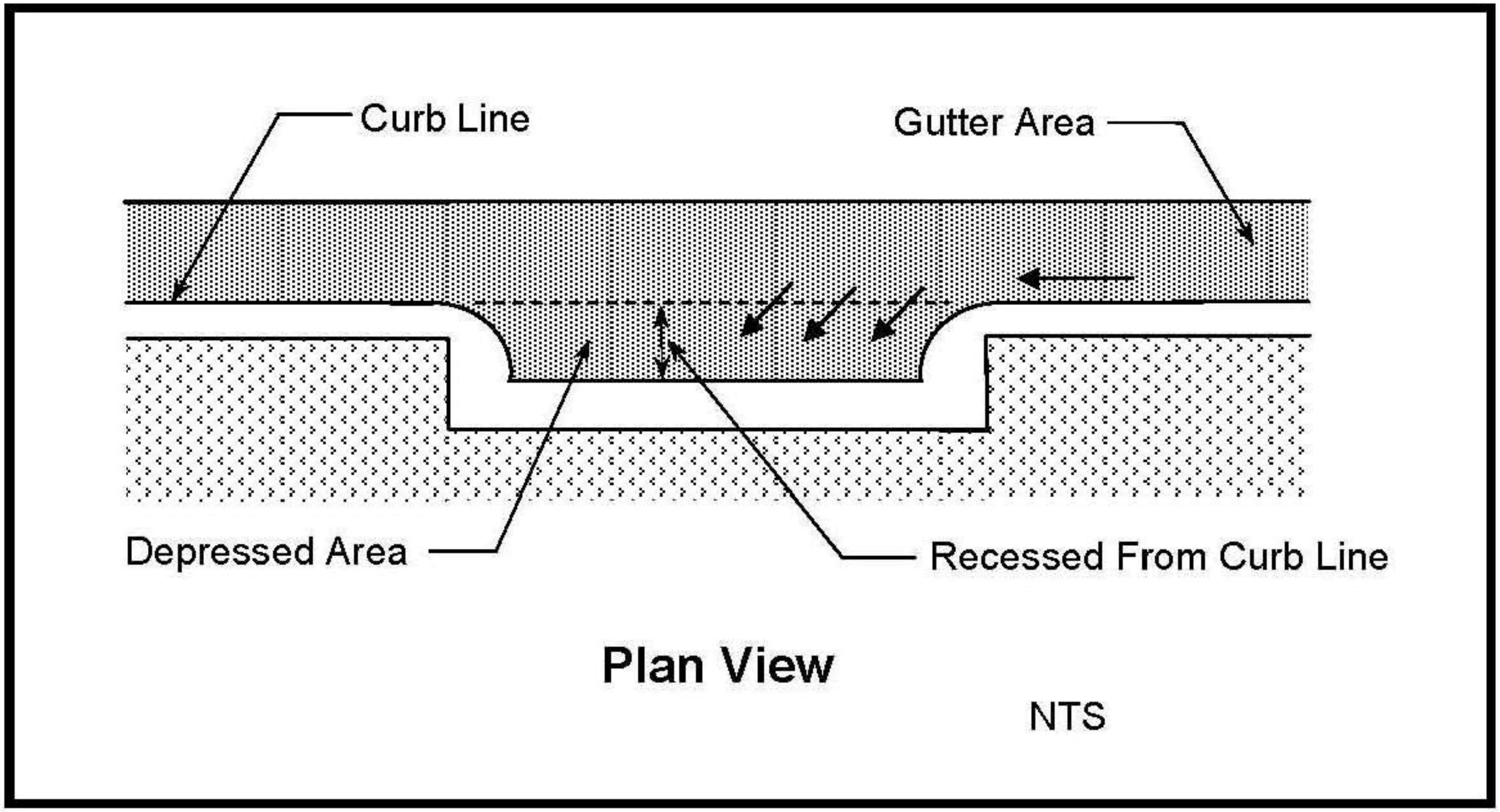


Figure3.2.2-1:Recessed Curb Inlet Diagram

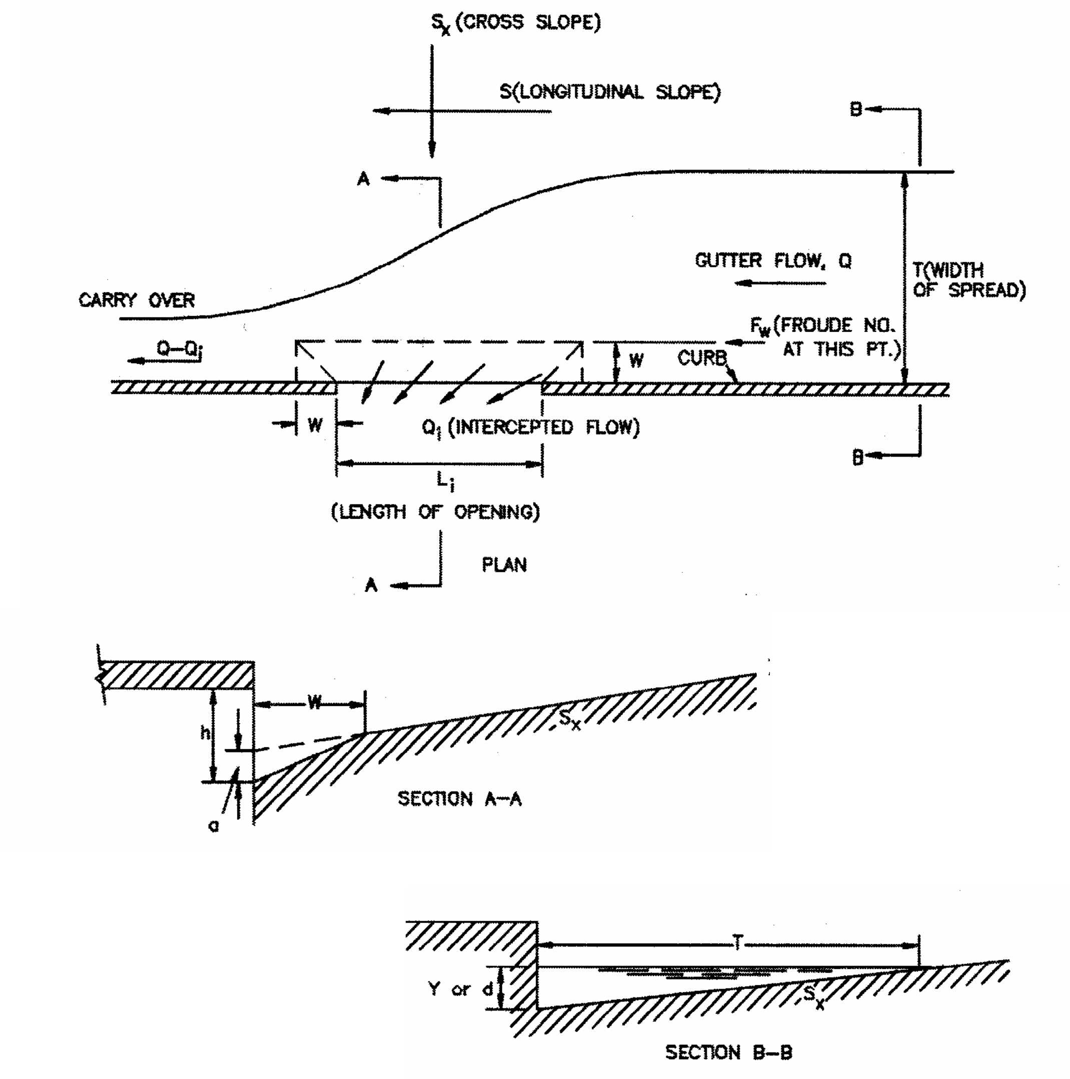


Figure 3.2.2-2: Non-Recessed Curb Inlet Diagram

Section 3.2.4 – Inlet Location

Limit Conflicts Inlet locations shall conform to the requirements of paragraph A of this section of these Guidelines, and shall be located as feasible to limit conflicts (caused by the inlet itself or associated stormwater) with vehicle, bicycle, or pedestrian traffic.

Limit Cross-Flow Inlets shall be located along streets to prevent concentrated stormwater flow from crossing traffic lanes, except as outlined in paragraph A of this section. Typical locations for these conditions are at transitions to super elevated sections, at the ends of long traffic islands, or at the ends of medians in super elevated sections.

Meet Standards Specific configuration and exact location of inlets shall be consistent with requirements of the Texas Department of Transportation Technical Specifications but shall not be in conflict with provisions of Section 3.1.2, Paragraph d.

Section 3.2.5 – Inlet Sizing

a. Inlets in Sumps

Minimize Ponding These inlets are placed at low points to relieve ponding of surface water. For purposes of design, inlets having a gutter depression greater than five (5) inches on streets with less than a one percent (1%) grade may be considered as inlets in sumps.

Maximum Depth Under no circumstances shall inlets at low points in streets allow water to pond to a depth exceeding 24 inches above the gutter flow line for up to 100-year frequency design storms based on project buildout and ultimate development conditions. Where computations show that this would be exceeded, provision must be made for an overflow outlet designed to handle the excess flows. This can take the form of a flume draining the street or a swale in an adjacent drainage easement, provided neither present an obstruction to non-motorized travel. Alternatively, the inlet system and receiving facilities shall be oversized as necessary.

(1). Curb Openings Inlets (Type A-1) that are not submerged are considered to function as a rectangular weir with a discharge coefficient of **3.0**. The capacity of a curb opening inlet is found by the following equation:

$$Q = 3.0 L y^{1.5}$$

where:

Q = capacity in cubic feet per second (cfs)

L = length of the opening which water enters into the inlet

y = total depth of water or head on the inlet

Clogging Factor Because of the tendency for curb opening inlets in sumps to collect debris, their calculated capacity shall be reduced by ten percent (10%) to compensate for potential clogging.

(2). Grate Inlets (Type A-2) are considered to function as an orifice with a discharge coefficient of 0.60. The capacity of a grate inlet is based on the following equation:

$$Q = 4.82 A_g y^{0.5}$$

where:

Q = capacity in cubic feet per second

$\mathbf{A_g}$ = clear opening area in square feet

y = total depth of water or head on the inlet in feet.

Clogging Factor Because of the tendency for grate inlets to collect debris, their calculated capacity shall be reduced by twenty-five percent (25%) to compensate for potential clogging, except where used as a controlling device in a detention facility.

b. Inlets on Grade

(1). Curb Inlets (without gutter depression) Type B-1

The capacity of such inlets is based on the weir equation, reduced to account for street grade and cross-flow effects. The head, "**y**", shall be taken as the depth of flow at the upstream end of the opening determined via criteria stipulated in Section 3.1.2 and Section 3.1.3. Equation 1 in Table 3.2.5-1 shall be used to determine the capacities of these inlets on grade, with the value for "**a**" set equal to zero.

(2). Curb Opening Inlets (with gutter depression) Type B-2

The same guidelines and criteria apply as for those inlets without a gutter depression, except the value "**a**" shall be taken as the gutter depression. The gutter depression is defined as the difference in elevation from the normal gutter grade line to the pavement grade at the throat or entry of the inlet (see Figure 3.2-2).

(3). The equations in Table 3.2.5-1 are to be used to determine the necessary size of curb inlets on grade. The applicable determinates and variables are defined in the table and the purpose of each equation is described.

(4). Grate Inlets

The capacity of a grate inlet on-grade depends on its geometry and cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness.

The depth of water next to the curb is the major factor affecting the interception capacity of grate inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 ft. long, interception flow is small. Agencies and manufacturers of grates have investigated inlet interception capacity. For inlet efficiency data for various sizes and shapes of grates, refer to Hydraulic Engineering Circular No. 22 Urban Drainage Design Manual by the Federal Highway Administration.

Section 3.3 – Storm Drainage Systems

Section 3.3.1 – Principles

Conduit System Storm Drain systems are conduits for the collection and conveyance of surface water to desired points of discharge. Design is accomplished by application of the Manning equation either directly, or through charts and nomographs derived from the equation. The following general conditions apply to the design.

Accept Design Flow The system must be designed to accommodate all intercepted flow for the design storm at each inlet and opening that allows stormwater into the system. Preferably the system shall operate “flowing full” and within the theoretical limits of open channel flow for the required design flows.

Future Runoff Design and construction shall take into account any stormflow from future subdivision areas contributing to the system. No existing system shall have flows added (or directed to it) that will exceed its theoretical design capacity.

100-Year Runoff The system shall be evaluated with associated drainage systems for the flow conditions that will result from a 100-year frequency rainfall event under existing development conditions over the Design Drainage Area. Lower magnitude storm design may be utilized with specific approval of the Drainage Review Authority. Design shall be revised as required to prevent formation of any conditions that could be considered hazardous to life, property, or public infrastructure, or that could create conditions inconsistent with the requirements of other sections of these Guidelines.

Table 3.2.5-1
Equations for Sizing Inlets On Grade

Reference Section 3.2.5, Paragraph b

Ref. No.	Equation	Use
1	$L_x = K_c Q^{0.42} S^{0.3} \left(\frac{1}{nS_x} \right)^{0.6}$	Calculating length of curb inlet (without gutter depression) required for total interception of gutter flow.
2	$E = 1 - \left[1 - \frac{L_i}{L_T} \right]^{1.8}$	Calculating efficiency of curb inlet shorter than required length.
3	$E_o = \frac{Q_w}{Q} = 1 - \left[1 - \frac{W}{T} \right]^{2.67}$	Calculating E_o, the ratio of the frontal flow to total gutter flow for a straight roadway cross slope; used in equation 4.
4	$S_e = S_x + \frac{a}{W} E_o$	Calculating S_e to substitute for S_x in Equation 1 to determine length of curb inlet (with gutter depression) for total interception of gutter flow.
NA	**Where symbols are as follows:** E_o = Ratio of frontal flow to total gutter flow Q_w = Flow in width W, cfs Q = Total gutter flow, cfs W = Width of depressed gutter, feet T = Total spread of water in gutter, feet K_c = 0.6 (in English measure) L_x = length of curb inlet required, feet S = longitudinal slope, (ft/ft) n = Manning's roughness coefficient S_x = cross slope of road surface, (ft/ft) E = Efficiency of inlet or percentage of interception L_i = Curb-opening length, ft L_T = Curb-opening length required for 100% interception, ft S_e = equivalent cross slope, (ft/ft) a = gutter depression depth, ft	
Note:	The length of a recessed inlet is to be determined in the same manner as inlets having a depressed gutter section, because a depressed section is to be provided at the throat of the inlet but behind the curb line (Fig. 3.2-1).	

Section 3.3.2 – Initial Design Considerations

a. Minimum and Maximum Velocities

Minimum velocities are necessary to prevent excessive deposits of sediment that could lead to clogging. The minimum design velocity for conduits flowing full shall be 2.5 feet per second.

Maximum velocities are necessary to prevent excessive erosion of the inverts. The maximum design velocity for conduits flowing full shall be 15 feet per second.

b. Roughness Coefficients, "n"

Selection of a roughness coefficient should reflect the average condition present during the life of the conduit. Factors to consider are erosion of the interior surface, displacement of joints, and introduction of foreign material and deposits. The following values shall be used for the materials listed:

Reinforced Concrete: 0.013

Ductile Iron or steel (Smooth): 0.010

Corrugated Metal: 0.024

Smooth lined High Density Poly-Ethylene (HDPE): 0.012

Non-lined High Density Poly-Ethylene (HDPE): 0.020

c. Location of Manholes and Junction Boxes

(1). Junction boxes shall be provided at all changes in conduit size and grade, and where changes in alignment are made at pipe joints. Manhole access shall be provided as part of the design of all junction boxes unless otherwise approved by the Drainage Review Authority.

(2). Manholes shall be provided at intervals not to exceed 600 feet for conduits 60 inches in diameter or smaller. For conduits exceeding 60 inches in diameter, the interval between openings shall not exceed 1000 feet. "Pipe to pipe" connections between manholes must be specifically approved by the Drainage Review Authority.

d. Minimum and Maximum Grades

(1). The minimum grade for conduits shall be that necessary to produce the minimum acceptable velocity per Section 3.2, Paragraph a.

(2). In order to prevent formation of a hydraulic jump conditions at the terminus of a conduit, the maximum grade along the outfall shall be less than the calculated grade that would result in supercritical flow, except where approved energy dissipation measures are used.

e. Minimum Pipe Diameter

18-Inch Usual

In most instances conduit that will become an integral part of the public storm sewer system shall have a diameter of 18 inches or greater. For design purposes, conduits having a diameter of less than 24 inches shall be assumed to have a twenty-five percent (25%) reduction of cross-sectional area to compensate for potential partial blockage.

f. Other Considerations

(1). Pipe sizes shall increase in the downstream direction, regardless of additional capacity developed by increased grade, and pipe soffit (inside top) elevations shall be aligned whenever practicable.

(2). Pipe shall be placed on the design friction slope as much as is practical.

Section 3.3.3 – Hydraulic Design Requirements

a. Flow Assumptions and Manning's Equations

Design shall be by application of the Continuity equation and Manning's Equation as follows:

$$Q = AV$$

$$Q = \frac{1.49}{n} AR^{0.67} S_f^{\ 0.5}$$

where :

Q = flow in cubic feet per second

A = cross sectional area in square feet

V = velocity of flow in feet/sec

n = roughness coefficient of conduit

R = hydraulic radius = **A/WP** in feet.

WP = wetted perimeter in feet **S_f** = friction slope of conduit in feet/foot

Capacity of a given size conduit is based on an assumption that it is "flowing full". Thus, **R** is equivalent to the cross sectional area divided by the inner circumference, while a value for **n** and **S_f** must be chosen.

b. Head Losses and Friction Losses

Head losses computed at junctions, inlets, and manholes shall be determined using the following equation:

$$h_j = (V_2^2 - K_j V_1^2)/2g$$

where:

h_j = head loss in feet at structures

V_1 = velocity at upstream entrance of structure (feet per second)

V_2 = velocity at downstream exit of structure (feet per second)

k_j = structure coefficient of loss (Table 3.3.2-1)

g = 32.2 feet per second per second

Head losses due to friction for open channel flow conditions are found by the following equation:

$$h_f = S_f L$$

where:

h_f = head loss due to friction in feet

S_f = friction slope (normally equal to the slope of the conduit, **S_o**), in feet per foot

L = length of conduit in feet

Table 3.3.2-1

Coefficient of Loss, K_j*

Design Condition	K_j *
Inlet on Main Line	0.50
Inlet on Main Line with Branch Lateral	0.25
Junction or Manhole on Main Line with 45 degree Branch Lateral	0.05
Junction or Manhole on Main Line with 90 degree Branch Lateral	0.25
Inlet or Manhole at Beginning of Line	1.25
Conduit on Curve for 90 degree	
Curve Radius = Diameter	0.05
Curve Radius = (2 to 8)	0.04
Curve Radius = (7 to 8)	0.25
** Where bends other than 90 Degree are used, then 90 Degree bend coefficient can be used with the following percentage factor applied:	60^o Bend – 85% 45^o Bend – 70% 22.5^o Bend – 40%

* From City of Austin Drainage Criteria Manual

c. Computation of Hydraulic Grade Line

All designs shall verify the elevation of the hydraulic grade line by calculation along the length of the system for two conditions. For the design storm the theoretical hydraulic grade line shall be verified as being at least one half foot (0.5 feet) below the inlet opening elevation, the gutter elevation, or the ground surface which ever is lowest. The hydraulic grade line shall also be calculated for the 100-year frequency storm assuming existing development conditions in the Design Drainage Area, and must be kept within the limits specified in all other sections of these Guidelines.

d. Allowance for Surcharging

Design of the system and evaluation of hydraulic grade lines shall take into account the tail water elevation at the outlet or final discharge point. Discharge at free outfalls shall assume a starting water surface elevation at the soffit of the conduit. For outlets that might operate in a submerged or partially submerged condition, the starting water surface elevation shall be taken as the coincident water surface elevation of the receiving (See Table 3.3.4-1).

e. Use of WINSTORM Program

Use of the WinStorm computer program is acceptable for calculating the capacity of inlets and storm drain systems. The program is available at no cost through TxDOT's web site. If WinStorm is used as a design aid for a project, the complete report the program can generate shall be submitted as part of the drainage report. In addition, both an analysis layout and an electronic medium (diskette or CD) of the analysis shall be provided.

Section 3.3.4 - Storm Drain Outfalls

All storm drains have an outlet where flow from the storm drainage system is discharged. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the storm water. The procedure for calculation the energy grade line through a storm drainage system begins at the outfall. Therefore, consideration of outfall conditions is an important part of storm drain design.

Several aspects of outfall design must be given serious consideration. These include the flowline or invert (inside bottom) elevation of the proposed storm drain outlet, tailwater elevations, the need for energy dissipation, and the orientation of the outlet structure.

The flowline or invert elevation of the proposed outlet should be equal to or higher than the flowline of the outfall. If this is not the case, there may be a need to pump or otherwise lift the water to the elevation of the outfall.

The tailwater depth or elevation in the storm drain outfall must be considered carefully. Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation. For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth of the outlet. The tailwater may also occur between the critical depth and the invert of the outlet. However, the starting point for the hydraulic grade line determination should be either the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, $(d_c + D)/2$, whichever is greater.

An exception to the above rule would be for a very large outfall with low tailwater where a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the design tailwater elevation, whichever was highest.

If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by a comparison of the drainage area of the receiving stream to the area of the storm drainage system. For example, if the storm drainage system has a drainage area much smaller than that of the receiving stream, the peak discharge from the storm drainage system may be out of phase with the peak discharge from the receiving watershed. Table 3.3.4-1 provides a comparison of discharge frequencies for coincidental occurrence for the 2-, 5-, 10-, 25-, 50-, and 100-year design storms. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. (For receiving streams which do not have a 1-year water surface profile, the 1-year water surface elevation can be estimated by extrapolation as the larger events 2-year through 100-year on log-probability graph plots.) For example, if the receiving stream has a drainage area of 200 acres and the storm drainage system has a drainage area of 2 acres, the ratio of receiving area to storm drainage area is 200 to 2 which equals 100 to 1. From Table 3.3.4-1 and considering a 10-year design storm occurring over both areas, the flow rate in the main stream will be equal to that of a five year storm when the drainage system flow rate reaches its 10-year peak flow at the outfall. Conversely, when the flow rate in the main channel reaches its 10-year peak flow rate, the flow rate from the storm drainage system will have fallen to the 5- year peak flow rate discharge. This is because the drainage areas are different sizes, and the time to peak for each drainage area is different.

Table 3.3.4-1

Frequencies for Coincidental Occurrence				
Area ratio	2-year design		5-year design	
	main stream	tributary	main stream	tributary
10,000:1	1	2	1	5
	2	1	5	1
1,000:1	1	2	2	5
	2	1	5	2
100:1	2	2	2	5
	2	2	5	5
10:1	2	2	5	5
	2	2	5	5
1:1	2	2	5	5
	2	2	5	5
	10-year design		25-year design	
	main stream	tributary	main stream	tributary
10,000:1	1	10	2	25
	10	1	25	2
1,000:1	2	10	5	25
	10	2	25	5
100:1	5	10	10	25
	10	5	25	10
10:1	10	10	10	25
	10	10	25	10
1:1	10	10	25	25
	10	10	25	25
	50-year design		100-year design	
	main stream	tributary	main stream	tributary
10,000:1	2	50	2	100
	50	2	100	2
1,000:1	5	50	10	100
	50	5	100	10
100:1	10	50	25	100
	50	10	100	25
10:1	25	50	50	100
	50	25	100	50
1:1	50	50	100	100
	50	50	100	100

Section 3.3.5 - Storm Drain Design Example

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

An example of the use of the method used in the manual for the design of a storm drainage system is outlined below and shown on Figure 3.3.5-1 Computation Sheet. The design theory has been presented in the preceding sections with their corresponding tables and graphs of information.

Preliminary Design Considerations

A. Prepare a drainage map of the entire area to be drained by proposed improvements. Contour maps serve as excellent drainage area maps, when supplemented by field reconnaissance. The scale of the map shall not be less than 1" = 200' for project area although smaller scale maps for large offsite drainage areas.

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

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

D. Indicate on each drainage area the code identification number and the direction of surface runoff by small arrows. Provide a runoff table showing area, "C" factor for each portion and composite "cA", TA, I_5, Q_5, I_{100} and Q_{100}.

E. Show all existing underground utilities.

F. Establish design rainfall frequency.

G. Establish minimum inlet time of concentration.

H. Establish the typical cross section of each street.

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

J. Plot profile of existing natural ground along the center line of the proposed storm drain.

K. Extend downstream plan and profile to a point of acceptable outfall.

Runoff Computations

The runoffs are shown on the Storm Drain Figure 3.3-1 Computation Sheet at the end of this section. The first 15 columns of the computation sheet cover the tabulation for runoff computations.

Column 1 Enter the storm drain inlet point station number. Design should start at the farthest upstream point.

Column 2 Enter the storm drain inlet point station number of inlet point immediately downstream.

Column 3 Enter the distance (in feet) between storm drain inlet point shown in Columns1 and 2. Column 1 stationing minus Column 2 stationing.

Column 4 Record the identification code number of each different drainage area to correspond to the numbers shown on the drainage area map.

Column 5 Record the area in acres for each of the individual areas of Column 4.

Column 6 Record the total drainage area in acres within the system corresponding to storm drain inlet point shown in Column 1.

Column 7 Record the coefficient of runoff "C" for each drainage area shown in Column 5.

Column 8 Multiply Column 5 by Column 7 for each area.

Column 9 Determine the total "CA" for the drainage system corresponding to the inlet or manhole shown in column 1.

Column 10 Determine inlet time of concentration (See Section 2.1.3.4).

Column 11 Determine flow time in sewer in minutes. The flow time in sewer is equal to the length from Column 3 divided by 60 times the velocity of flow through the sewer.

Column 12 Total time of concentration in minutes. Column 10 plus Column 11. (Times of concentration and rainfall intensities shall not be modified except at downstream inlets and junctions. The junction of paired inlets does not constitute a downstream junction.)

Column 13 Design frequency established by Design Criteria from Section 3.1.2.

Column 14 Intensity of rainfall in inches per hour corresponding to time of concentration shown in Column 12. Use Table 2.1.2-1. Rainfall Intensity-Duration-Frequency Curves.

Column 15(15, 15X, 15A, 15B) Design Discharge or Runoff in c.f.s. Column 9 times Column 14. (This column is split to include "detain", "bypass" and "pipe" discharges.)

Hydraulic Design

After the computation of the quantity of storm runoff entering each inlet, the size and gradient of pipe required to carry off the design storm are determined. It should be born in mind that the quantity of flow to be carried by any particular section of storm drain is not the sum of the inlet design quantities of all inlets above that section of pipe, but is less than the straight total. This situation is due to the fact that as the time of concentration increases the rainfall intensity decreases. Columns 16 through 29 of the computation sheet cover the minimum necessary hydraulic requirements to establish the hydraulic grade line for a storm drain. The ground line profile is now used in conjunction with the previous runoff calculations. The elevation of the hydraulic gradient is arbitrarily established approximately two (2') feet below the ground elevation at the first pickup point (Column 18). A tentative gradient from this point to the second inlet point is determined to maintain the gradient feet below the ground surface. When this tentative gradient is set and the design discharge is determined as shown in Column 15, a Manning flow chart may be used to determine the pipe size and velocity which will result. It is probable that the tentative gradient will have to be adjusted at this point since the intersection of the discharge and the slope on the chart will likely occur between standard pipe sizes. The smaller pipe should be used if the design discharge and corresponding slope does not result in an encroachment on the ground surface. If there is encroachment, use the larger pipe which will establish a capacity somewhat in excess of the design discharge. Velocities can be read directly from a Manning flow chart based on a given discharge, pipe size, and gradient slope. For situations where a circular conduit is flowing partially full refer to FHWA Hydraulic Engineering Circular, Chart 26 (August, 2001).

Column 16 Record the selected pipe size.

Column 17 Record the required slope of the frictional gradient (hydraulic gradient) such that the pipe when flowing full, will carry an amount of flow equal to or greater than the computed discharge. The pipe shall be constructed on a grade such that the inside crown of the pipe coincides with hydraulic gradient or is below the developed hydraulic gradient when flowing full.

Column 18 Record the hydraulic gradient elevation at the upstream end.

Column 19 The head loss due to friction, the product of Columns 3 and 17 is subtracted from Column 18 and entered in Column 19 which is the elevation of the hydraulic gradient at the lower end of the particular section of the line being designed.

Column 20 Velocity of flow in incoming pipe (main line) at junction, inlet or manhole at design point (Column 1).

Column 21 Velocity of flow in outgoing pipe (main line) at junction, inlet or manhole at design point (Column 1).

Column 22 Velocity head loss for outgoing velocity (main line) at junction, inlet or manhole at design point (Column 1).

Column 23 Velocity head loss for incoming velocity (main line) at junction, inlet or manhole at design point (Column 1).

Column 24 Head loss coefficient "Kj", at junction, inlet or manhole at design point from Table 3.3.2-1.

Column 25 Multiply Column 23 by Column 24 (0.10' minimum).

Column 26 Column 22 minus Column 25.

Column 27 Column 18 plus Column 26.

Column 28 Invert elevation at design point for incoming pipe.

Column 29 Invert elevation at design point for outgoing pipe.

Column 30 Top of curb elevation.

Column 31 Column 30 minus Column 27.

The above procedure is followed for each section of the storm drain. At the outfall, the hydraulic gradient of the line must be at the same elevation or above the gradient of the conduit or channel receiving the storm runoff discharge.

With the hydraulic gradient established for a particular line, considerable latitude is available for the physical placement of the pipe flow line elevations. The inside top of the pipe must be on or below the hydraulic gradient, thus allowing the pipe to be lowered where necessary to maintain proper cover and to minimize grade conflicts with existing utilities.

COMPUTATION SHEET NC

HYDRAULIC COMPUTATIONS FOR STORM DRAINS

FROM	TO	Pipe	Drainage Area			Runoff	Incr.	Total	Time of Concentration			Design	Rain	Total	Q	Q	Q	Pipe			HGL		HEAD LOSS CALCULATIONS							Design	Invert Elev.		T/C	T/C -	
		Length	Incremental		Total Area	"c"	cA	cA	Inlet	Travel	Total	Frequency	Intensity	Runoff	detain	bypass	pipe	Size	n	Sf	U/S	D/S	V1 (in)	V2 (out)	$V1^2/2G$	$V2^2/2G$	Kj	$KjV1^2/2G$	Hk	HGL	FROM	TO	ELEV.	HGL	
		feet	No.	Area					min.	min.	min.	years		cfs	cfs	cfs	cfs	in.		ft/ft	Elev.	Elev.	ft/sec	ft/sec	ft.	ft.		ft.	ft.	Elev.	ft.	ft.	ft.	ft.	COMMENTS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	15X	15A	15B	16		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	
LINE A																																			
4+00	5+42	142	A1	35.00	35.00	0.50	17.50	17.50	15.00		15.00	100.00	9.60	168.00		14.50	153.50	48	0.013	0.0114	824.35	822.73	12.22	12.22	2.32	2.32	0.00	0.00	0.00	824.35	815.00	813.30			
1+86	4+00	214	A2	0.50	35.50	0.50		17.75	15.00		15.00	100.00	9.60	170.40		11.80	158.60	48	0.013	0.0122	821.41	818.80	12.22	12.62	2.32	2.47	0.50	1.16	1.31	822.73	813.30	810.73			
1+43	1+86	43	A3	0.18	35.68	0.50	0.09	17.84	15.00	0.48	15.48	100.00	9.40	167.70		0.00	167.70	48	0.013	0.0136	817.27	816.69	12.62	13.34	2.47	2.77	0.50	1.24	1.53	818.80	810.73	810.22			
0+50	1+43	93	A4	0.56	36.24	0.50	0.28	18.12	15.00	0.53	15.53	100.00	9.40	170.33		0.00	170.33	48	0.013	0.0141	815.91	814.60	13.34	13.55	2.77	2.85	0.75	2.07	0.78	816.69	810.22	809.11			
0+00	0+50	50			36.24	0.50		18.12	15.00	0.53	15.53	100.00	9.40	170.33		0.00	170.33	54	0.013	0.0075	813.11	812.73	13.55	10.71	2.85	1.78	0.10	0.29	1.50	814.60	808.61	808.23			
LINE A-3																																			
A 1+86	n. inlet	22	see note	0.18	0.18	0.50	0.09	0.09	15.00		15.00	100.00	9.60	14.01		0.00	14.01	21	0.013	0.0078	818.97	818.80	0.00	5.83	0.00	0.53	1.25	0.00	0.66	819.63	812.98	817.00	821.50	1.87	half (A1+A2 bypass) + half A3
A 1+86	s. inlet	24	see note	0.18	0.18	0.50	0.09	0.09	15.00		15.00	100.00	9.60	14.01		0.00	14.01	21	0.013	0.0078	818.99	818.80	0.00	5.83	0.00	0.53	1.25	0.00	0.66	819.65	812.98	817.00	821.50	1.85	half (A1+A2 bypass) + half A3
CORRECTED CALCULATIONS																																			
LINE B																																			
0+00	4+78	478										100.00		191.56			191.56	72	0.013	0.0020	809.17	808.19	8.09	6.78	1.02	0.71	0.75	0.76	0.10	809.27			810.31		
LINE A																																			
15+16	16+75	159										100.00		290.84	202.00		88.84	54	0.013	0.0020	807.59	807.26		5.59		0.48	1.25	0.00	0.61	808.19			808.66		end of street elev.
12+37	15+16	279										100.00		399.36	202.00		197.36	72	0.013	0.0022	806.80	806.19	5.59	6.98	0.48	0.76	0.60	0.29	0.47	807.26			807.92		rim elev.
9+74	12+37	263										100.00		425.49	202.00		223.49	72	0.013	0.0028	805.79	805.06	6.98	7.90	0.76	0.97	0.75	0.57	0.40	806.19			806.52		
8+78	9+74	96										100.00		434.42	202.00		232.42	72	0.013	0.0030	804.74	804.44	7.90	8.22	0.97	1.05	0.75	0.73	0.32	805.06			805.21		
8+63	8+78	14										100.00		465.74	202.00		263.74	72	0.013	0.0039	803.88	803.83	8.22	9.33	1.05	1.35	0.75	0.79	0.56	804.44			804.83		
7+03	8+63	160										100.00		465.74	202.00		263.74	72	0.013	0.0039	803.15	802.53	9.33	9.33	1.35	1.35	0.50	0.68	0.68	803.83					
6+78	7+03	25										100.00		465.74	202.00		263.74	72	0.013	0.0039	802.26	802.16	9.33	9.33	1.35	1.35	0.20	0.27	0.27	802.53					
CORRECTED RUNOFF Q's, HOLDING TOTAL Q																																			
LINE B																																			
0+00	4+78	478						21.52			17.27	100.00	8.90	191.56			191.56	72	0.013	0.0020	808.08	807.10	8.09	6.78	1.02	0.71	0.75	0.76	0.10	808.18			810.31		
LINE A																																			
15+16	16+75	159						36.35			20.83	100.00	8.05	292.49	265.52		26.97	54	0.013	0.0002	807.05	807.02		1.70		0.04	1.25	0.00	0.06	807.10			808.66		end of street elev.
12+37	15+16	279						57.87		0.68	21.51	100.00	7.90	457.34	265.52		191.82	72	0.013	0.0021	806.73	806.16	6.78	6.78	0.71	0.71	0.60	0.43	0.29	807.02			807.92		rim elev.
9+74	12+37	263						61.67		0.58	22.09	100.00	7.79	480.17	265.52		214.65	72	0.013	0.0026	805.80	805.12	6.78	7.59	0.71	0.89	0.75	0.54	0.36	806.16			806.52		
8+78	9+74	96						63.89		0.20	22.29	100.00	7.75	494.96	265.52		229.44	72	0.013	0.0029	804.77	804.49	7.59	8.11	0.89	1.02	0.75	0.67	0.35	805.12			805.21		
8+63	8+78	14						68.49		0.03	22.32	100.00	7.74	530.25	265.52		264.73	72	0.013	0.0039	803.89	803.84	8.11	9.36	1.02	1.36	0.75	0.77	0.59	804.49			804.83		
7+03	8+63	160								0.29	22.60	100.00	7.74	530.25	265.52		264.73	72	0.013	0.0039	803.16	802.53	9.36	9.36	1.36	1.36	0.50	0.68	0.68	803.84					
6+78	7+03	25								0.04	22.65	100.00	7.74	530.25	265.52		264.73	72	0.013	0.0039	802.26	802.16	9.36	9.36	1.36	1.36	0.20	0.27	0.27	802.53					
CORRECTED RUNOFF Q's, HOLDING PROPOSED DETENTION																																			
LINE B																																			
0+00	4+78	478						21.52			17.27	100.00	8.90	191.56			191.56	72	0.013	0.0020	812.39	811.41	8.09	6.78	1.02	0.71	0.75	0.76	0.10	812.49			810.31		
LINE A																																			
15+16	16+75	159						36.35			20.83	100.00	8.05	292.49	202.00		90.49	54	0.013	0.0021	810.78	810.45		5.69		0.50	1.25	0.00	0.63	811.41			808.66		end of street elev.
12+37	15+16	279						57.87		0.51	21.34	100.00	7.94	459.42	202.00		257.42	72	0.013	0.0037	809.59	808.56	6.78	9.10	0.71	1.29	0.60	0.43	0.86	810.45			807.92		rim elev.
9+74	12+37	263						61.67		0.44	21.78	100.00	7.85	484.03	202.00		282.03	72	0.013	0.0044	807.98	806.81	9.10	9.97	1.29	1.55	0.75	0.97	0.58	808.56			806.52		
8+78	9+74	96						63.89		0.15	21.93	100.00	7.82	499.49	202.00		297.49	72	0.013	0.0049	806.25	805.77	9.97	10.52	1.55	1.72	0.75	1.16	0.56	806.81			805.21		
8+63	8+78	14						68.49		0.02	21.95	100.00	7.81	535.18	202.00		333.18	72	0.013	0.0062	804.91	804.82	10.52	11.78	1.72	2.16	0.75	1.29	0.87	805.77			804.83		
7+03	8+63	160								0.23	22.18	100.00	7.81	535.18	202.00		333.18	72	0.013	0.0062	803.74	802.75	11.78	11.78	2.16	2.16	0.50	1.08	1.08	804.82					
6+78	7+03	25								0.04	22.22	100.00	7.81	535.18	202.00		333.18	72	0.013	0.0062	802.32	802.16	11.78	11.78	2.16	2.16	0.20	0.43	0.43	802.75					

Figure 3.3.5-1 Computation Sheet

CHAPTER 4 – Hydraulic Design of Open Channels, Culverts, Bridges and Detention Structures

Section 4.1 – General

This section summarizes the practical consideration, 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. 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 this manual is to be used, it should be reviewed and discussed with the Drainage Review Authority prior to commencing significant portions of the design effort.

Section 4.2 – Open Channel Design

Section 4.2.1 – Principles

Analysis of open channels is necessary to determine the depth and velocity of a given flow for an established cross-section. Typical uses are to determine the tail water and/or the back water condition(s) at a culvert structure, flood elevation for selected discharge of natural streams and watercourses, and discharge capacities for existing or proposed designed channels.

Design Objectives

Design of open channels involves the selection of a cross-section, surface treatment, and alignment to accommodate some series of design discharges. A successful channel design can take one of two basic forms. It can replicate the features and characteristics commonly found in natural streams, or it can provide the characteristics of traditional constructed channels. In either case the design objective is to provide stable structural components that will not develop excessive sediment deposits or erosive cuts that will maintain a stable cross-section that will minimize the need for maintenance, and that will not be damaged by entry of uncontrolled surface flows.

Natural Designs

Leaving streams in their natural state offers numerous advantages, so this practice is preferred. Designs that replicate the characteristics of natural streams are encouraged, provided they meet the objectives of the provisions in these Guidelines. Such a design approach may be required at the discretion of the Drainage Review Authority. Where plant growth and hydro-environments can be created or maintained to accomplish stabilized channels they are encouraged. Such designs must ensure that long term maintenance costs are not likely to be greater than would be expected from the use of traditional channel lining treatments.

Section 4.2.2 – Determination of Water Surface Profiles

a. Methods of Analysis

(1). Manning's Equation

The equation is expressed as follows:

$$Q = \frac{1.49}{n} AR^{0.67} S^{0.5}$$

where:

Q = the discharge in cubic feet per second

n = Manning's Roughness Coefficient

A = cross-sectional area representing the depth of flow in feet

R = hydraulic radius = **A/WP** in feet.

WP = wetted perimeter of channel section for area "**A**" in feet

S = slope of channel bed in feet/foot

The equation is applied to a single cross-section and assumes a uniform channel cross-section and slope as well as steady, uniform flow in the channel. Consequently, its use shall be limited to designed channels and suitable natural channels in the secondary drainage system.

(2). Standard – Step Procedure

This procedure shall be used in analyzing natural or man-made channels of the primary drainage system. It may also be applied to open channels in the secondary drainage system.

Bernoulli's Equation

The procedure involves application of Bernoulli's Equation to a series of stream cross-sections using the continuity equation, the velocity head, and Manning's Equation as inputs. A detailed description is beyond the scope of these Guidelines.

HEC-RAS Software

The method shall be applied using the HEC-RAS software endorsed by the Hydraulic Engineering Center of the U.S. Army Corps of Engineers, or other computer analysis programs employing the same methodology. The application shall be according to the recommendations contained in the user's manual for the program.

b. Primary Design Parameters

(1). Channel Section

Cross-section(s) should be representative of the channel reach being studied.

(2). Manning's Roughness Coefficients ("**n**" values)

Section of values for “n” shall fall within the range of values and descriptions given in Table 4.2.2-1 and the design values presented below.

<u>Manning's Roughness Coefficients for Channel Design</u>

i.	**Grass lined**	**=**	**0.035 (velocity check)**
ii.	**Grass lined**	**=**	**0.050 (capacity check)**
iii	**Concrete lined**	**=**	**0.015**
iv.	**Gabions**	**=**	**0.030**
v.	**Rock Riprap**	**=**	**0.040 (or use Corps' equation n=0.038$D_{90}^{1/6}$)**
vi.	**Grouted Riprap**	**=**	**0.028 (FHWA)**

(3). Channel Slope

The slope of the channel shall be taken as the average slope along the reach being studied.

c. Determination of Flow Character

In order to prevent formation of areas of supercritical flow and hydraulic jumps except where planned, flow must be kept within the limits of sub-critical flow. To do this, design flow depth must be greater than critical depth. For non-rectangular channels, the critical depth can be found through application of trial depths and the following relationship:

Table 4.2.2-1
Manning's Roughness Coefficients (n)[1]

Conditions	Coefficients Min.	Coefficients Max.
Natural Stream Channels		
Minor Streams With Fairly Regular Section, and:		
1. Some grass and weeds, little or no brush	0.030	0.035
2. Dense weeds, flow depth materially exceeds weed height	0.035	0.050
3. Some weeds, light brush on banks	0.035	0.050
4. Some weeds, heavy brush on banks	0.035	0.050
5. Some weeds, dense willows on banks	0.050	0.070
6. Trees in channels & branches submerged at high stage, Use 1 to 5 above and increase values by:	0.010	0.020
Minor Streams With Irregular Section (pools, slight channel meander): use 1 to 5 above, and increase values by:	0.010	0.020
Flood Plain (adjacent to natural streams)		
Pasture: no brush, short grass	0.030	0.035
Pasture: no brush, tall grass	0.035	0.050
Heavy weeds, scattered brush	0.050	0.070
Wooded: Varies depending on undergrowth, height of foliage on trees, etc. The area of "n" = 0.10 and greater indicated extremely heavily wooded condition.	0.075	0.120
Lined Channels		
Metal corrugated	0.021	0.024
Neat concrete lined	0.012	0.018
Concrete	0.012	0.018
Concrete rubble	0.017	0.030
Grass Covered Small Channels, Shallow Depth		
No rank growth	0.035	0.045
Rank growth	0.040	0.050
Unlined Channels		
Earth, straight and uniform	0.017	0.025
Dredged	0.025	0.033
Winding and sluggish	0.022	0.030
Stony beds, weeds on bank	0.025	0.040
Earth bottom, rubble sides	0.028	0.035

[1] From "Hydraulic Design Manual" of Texas Depart of Transportation, 2004

$$\frac{Q^2}{g} = \frac{A_c^3}{T_c}$$

where:

Q = discharge in cubic feet per second

g = 32.2 feet per second per second

$\mathbf{A_c}$ = cross-sectional area of flow at critical depth in square feet

$\mathbf{T_c}$ = top width of critical flow in feet.

For non-uniform cross sections, a rating curve of critical depth versus discharge shall be constructed.

Once the discharge **Q**, area **A**, and depth **d** are determined, the slope necessary to produce these conditions in a channel can be computed from Manning's Equation.

Section 4.2.3 – Design of Open Channels

Traditional Designs The criteria outlined in this section are intended to guide the development of traditional designed/constructed open channels. Roadside ditches shall be designed as open channels per the Guidelines in Section 4.2.4 of this Section. Alternate channel designs will be considered by the City Engineer provided they are shown to meet the intent of these Guidelines.

Natural Designs Encouraged Designs intended to replicate the characteristics of natural streams are encouraged but must be shown to satisfy the essential purposes of the provisions of this paragraph. Example features that might be considered for such designs are among those outlined in Appendix E.

a. Physical Considerations

(1). Cross-Section Geometry

- Earthen Channels

  An earthen trapezoidal channel shall have a trapezoidal shape with side slopes not steeper than a 4 to 1 ratio and a channel bottom at least six feet in width.

- Lined Channels

  A lined trapezoidal concrete channel shall have side slopes of two (2) foot horizontal to one (1) foot vertical or otherwise to such standards, shape and type of lining as may be approved by the Drainage Review Authority. The channel bottom must be a minimum of 6' in width.

- Swales

A swale is an earthen V-bottom channel. Its side slopes shall not be steeper than three foot horizontal to one foot vertical and its maximum depth shall be three (3) feet unless approved by the Drainage Review Authority.

(2). Minimum and Maximum Grades

The minimum longitudinal slope shall be 0.001 foot per foot (0.1 percent) for earthen or vegetative lined channels to prevent formation of standing water. The maximum allowable grade shall be a function of allowable flow velocity as related to channel lining materials stipulated in Table 4.2.3-1. If the proposed maximum grade will exceed 70 percent of the calculated critical slope values for the required range of design flows, special channel linings and energy dissipation features must be used to compensate for the high velocities and hydraulic jumps associated with supercritical flow. Designs for supercritical flow are limited to straight sections having a minimum grade that is at least 130 percent of the critical slope values calculated for the required range of design flows.

Table 4.2.3-1

Maximum Design Velocities (V) [1]

Surface Treatment	Max. Design Velocity
Grass: seeded with erosion mat	6.0 ft./sec.
Grass: established sod	6.0 ft./sec.
Rubble (Riprap): placed rock or concrete	12.0 ft./sec.
Impermeable: (concrete, Gunite, etc.)	15.0 ft./sec.
*Note: Velocities in excess of 12 feet per second shall require additional methods such as baffles, stilling basins, and/or drop structures to reduce velocities to levels stipulated.	

[1]Higher velocities in grass channels with erosion mat require specific approval of Drainage Review Authority.

(3). Bends and Horizontal Curves

The maximum allowable deflection angle for bends in designed channels shall be 45 degrees. The outside of horizontal curves shall provide additional channel bank height and surface treatment as necessary to fully contain the design flow and prevent erosion and overtopping.

(4). Erosion Protection Measures

Measures required for protection of earthen channels are described in Section 4.6 of these Guidelines.

(5). Berms

If earthen berms are proposed as permanent features for stormwater management they shall meet a structural compaction of 95 percent Standard Proctor. Berm side slopes shall be a maximum of three horizontal to one vertical (3:1) if to be privately maintained and four horizontal to one vertical (4:1) if to be publicly maintained. As a function of height, berms shall have a minimum top width as follows:

- Height 2 feet or less — 3 feet top width
- Height between 2 and 6 feet — 5 feet top width
- Height exceeding 6 feet — 10 feet top width

b. Flow Considerations

(1). Design Flows

a). Channel capacity shall be determined to accommodate the discharge from a 100-year storm assuming buildout conditions for all of the Project Area of a land development project that can be foreseen to discharge to the channel, plus the 100-year storm from existing conditions on all other land areas served by the channel. Channel designs based on smaller magnitude storms must have specific approval of the Drainage Review Authority.

b). Channels shall be designed to flow either sub-critical or supercritical for the range of discharges resulting from the 5-year, 10-year, 25-year, and 100-year design storms on the Design Drainage Area.

c). When a low-flow pilot channel is to be constructed in an open channel, an invert section of concrete or other materials acceptable to the Drainage Review Authority must be designed to carry the peak design discharge of a 2-year storm for the channel as stipulated in the previous sub-paragraph.

(2). Velocity Limitations

a). It is recommended that the average flow velocity not be less than two (2.0) feet per second for the design storm. Lower velocities shall require specific approval of the Drainage Review Authority.

b). Maximum velocities for the design flow shall be less than the values given in Table 4.3.2-1 for the type of surface treatment(s) specified.

(3). Freeboard Requirements

Channels shall be designed with a minimum freeboard equal to one foot above the design depth of flow.

c. Outfall Junctures

Junctures Important

Where part of a storm drainage system discharges into another part of the system, on-going long-term maintenance difficulties can result, particularly where the receiving facility is an open channel. The complexity and importance of these junctures warrants careful design attention.

Juncture Categories

Junctures can be grouped into three categories: discharge from an under ground storm sewer conduit into the secondary or primary drainage system; discharge of an open flume into the secondary or primary drainage system; and the confluence of two channels (secondary/secondary or secondary/primary).

Public System The following guidelines apply to points of discharge into the public stormwater conveyance system, whether from a private or public drainage facility.

(1). Storm Sewer Outfall Points

Acute Connections Where storm sewer lines are to discharge directly into culverts or channels they must do so at an acute angle (preferably not exceeding 45 degrees) so that flow is generally in the same direction as the flow of the receiving facility. Where discharge is into a culvert, the connection should match the soffit elevation of the two facilities as closely as practical.

Match Inverts Where discharge is directly into a designed or natural watercourse, the discharge invert elevation should match that of the receiving facility as closely as practical. Alternatively, special channel treatment designs may be proposed so that the outfall discharge will not inhibit or obstruct flow in the receiving channel. In either case, the design must work to manage the velocity of the outfall discharge to prevent scour of the bottom or sides of the receiving channel.

(2). Flume Outfall Points

No Erosion, Scour Flumes that convey stormflow into a natural or designed watercourse shall be designed to prevent storm flow from interfering with the integrity of the bottom or sides of the receiving facility. This will necessarily involve managing discharge velocity to avoid scour, as well as possible treatment of portions of the receiving water course. No such connection shall inhibit or obstruct conveyance of the design storm flow of the receiving water course.

(3). Points of Channel Confluence

Control Turbulence Channel confluences should be at 45 degrees or less, and the design should bring flows together as nearly as possible at the same velocity in order to minimize turbulence. The design must include treatments to ensure adequate erosion control consistent with provisions in Section 4.6 of these Guidelines.

Section 4.2.4 – Roadside Ditches

Where the use of roadside ditches is approved by the Drainage Review Authority, the design shall be governed by provisions for open channel flow as set out in the forgoing paragraphs of this Section, unless superseded by higher or more explicit standards as outlined below.

a. Hydraulic Design of Ditches

(1). Ditches must, as a minimum, contain the flow from the design 5-year storm with a water surface elevation six (6) inches below the top of the ditch. The 100-year storm shall be contained in the roadway ROW unless specifically approved by the Drainage Review Authority.

b. Ditch Geometry

(1). Culverts must be at least 18 inches in diameter.

(2). The top of the ditch bank must be separated laterally from the roadway shoulder (edge of base course) by at least two (2) feet.

(3). Ditch sections shall be flat bottom and trapezoidal in shape with a minimum depth of one and one half (1.5) feet.

(4). Side slopes shall be no steeper than four horizontally to one vertical (3:1).

c. Ditch Construction

(1). Culverts and grading shall be constructed in compliance with TxDOT Technical Specifications.

(2). All ditches must be completely vegetated in accordance with TxDOT Technical Specifications.

Section 4.2.5 – Modification of Natural Watercourses

a. FEMA and "Non-FEMA" Systems

Both the Primary and Secondary Systems include natural watercourses of various sizes and capacities. Several of these watercourses form the FEMA-designated Floodplains as defined in Section 4.2.6. Most of the remaining natural watercourses are generally upstream extensions of those forming the FEMA-designated system. For purposes of these Guidelines natural watercourses shall be considered to be in one of two categories: as part of the Primary Watercourses defined in Chapter 1 (the "FEMA-Designated Flood Plain System"), or as "Non-FEMA" watercourses.

b. FEMA-Designated Flood Plain System

Watercourses making up the FEMA-Designated Flood Plain System must be in compliance with the requirements of Section 4.2.6, in addition to provisions of this Section (4.2.5) and its subparagraphs.

c. Principles

(1). Modifications shall be defined as physical changes in a watercourse's vertical and/or horizontal alignment, cross-section geometry, surface characteristics, or over-bank areas. Movement or addition of earthen materials, grubbing, and wholesale removal of vegetation is considered modification activity, but trimming of vegetation is considered maintenance and is not governed by these Guidelines.

(2). At a minimum, all modifications to natural watercourses shall meet the requirements governing design or improvement of open channels stipulated elsewhere in these Guidelines.

(3). Changes to natural watercourses must be consistent with an approved master plan for modification of a complete reach of the Primary System if such a master plan exists. If no plan exists, one may be required at the discretion of the Drainage Review Authority. Changes to short parts of a natural watercourse must demonstrate compatibility with similar modifications along the length of that reach, whether existing or planned.

(4). On any site that is a single platted lot, minor encroachments, consisting of fill and earthwork changes in existing defined floodway fringe areas may be allowed at the discretion of the Drainage Review Authority. Any encroachments shall meet all requirements listed in the following sub-paragraphs.

d. Determination of Floodway and Floodplain Areas

(1). For streams forming the primary drainage system, a comprehensive hydraulic model, referred to as the County's Flood Study, has been adopted. This study shall be used as the principal source defining floodway and floodplain areas for streams and channels making up the primary system.

(2). Along streams and channels lacking an existing study, floodway and floodplain areas shall be determined by extending the adopted Flood Study using the standard step procedure. Where new flood discharges must be determined, they shall be computed using the methods outlined in Chapter 2 of these Guidelines.

(3). Land development projects proposing to use land filling or berms or structural features to raise existing floodplain areas above flood levels are considered encroachments into floodplain areas. Because this will raise the base flood elevations (BFE) in the vicinity of the proposed work the extent of encroachment must be limited so that the BFE is not raised by more than 0.5 foot. These geographic limits will define resulting “floodway” for that Watercourse, or tributary thereof. **This "floodway" is more restrictive than the Standard FEMA regulatory floodway.**

(4). The floodway shall be determined using an encroachment method based on proportionate conveyance reduction (as a function of hydraulic cross sectional areas) from both sides of the channel over-bank. However, the limits of encroachment shall not extend into the designated channel area. The engineering studies necessary to identify “floodways” rests with the owner/developer (or the applicant) of the proposed project at the discretion of the Drainage Review Authority or his/her designee.

e. Design Considerations

(1). Analysis for System Impacts

Modified Channels When existing channels are straightened, improved in cross-section, and/or lined, their hydraulic efficiency increases. Such action results in reduced travel times and reduced times of concentration. It can also result in loss of over bank storage capacity. These factors cause higher flood discharges and higher flood elevations downstream of the area of improvement. Any changes to channels within the Primary System shall be accompanied by a revised analysis of the hydrologic model (both current condition and ultimate condition) of the adopted Flood Study. The changes will be reflected in the routing reaches and lag factors for affected channel reaches and s.

Downstream Effects Downstream impacts shall be reviewed to prevent damage to existing property and structures. Key items shall include the effect of higher discharges at bridges and culverts, and the changes in flood elevations. Channel improvements shall not cause increases in flood discharges that will exceed the capacity of downstream crossing structures, and shall not raise ultimate 100-year flood elevations.

(2). Transition Sections

Smooth Transitions Modification of any channel section shall include designs to affect smooth transitions with the existing channel features, both upstream and downstream. These transitions should be gradual to prevent the formation of excessive energy losses and turbulence, or the creation of inappropriate velocities in upstream or downstream sections of the channel. Any proposals for abrupt changes in section, profile, or alignment must be accompanied by engineering studies demonstrating that planned energy dissipation measures will preserve the long term integrity of channel elements. Energy dissipation measures must be acceptable to the Drainage Review Authority.

Section 4.2.6 – Floodplains

1. Principles

Floodplain Definition A “floodplain” is generally land areas along and near a waterway that are inundated during large and relatively infrequent storm events. The runoff from smaller, more frequent storm events is generally contained within the main channel of the waterway and has little to no effect on adjacent “over-bank” land areas.

Width Varies Fundamentally, every watercourse has attendant floodplain areas that can be situated along one or both sides of the main channel, depending on topographic features. Along smaller streams or channels there may be little distinguishable difference between the main flow area and the floodplain. However, on larger streams or channels floodplain areas may be very broad and shallow, and may provide for very little conveyance of stormwater.

2. Identification of Floodplains

Identified Floodplains

Floodplains are principally associated with the primary drainage system. The primary system and its tributaries make up the Named Regulatory Watercourses listed in Table (Appendix) of these Guidelines. The over-bank areas of these waterways are considered to be the "identified" floodplains, even though the specific geographic limits of some reaches of each watercourse system are not dimensionally defined in hydrologic and/or topographic terms.

Floodplain Limits

As land development occurs along the Watercourses identified in Table (Appendix) of these Guidelines, and along upstream extensions thereof, it will be necessary to fully define the geographic limits of the attendant floodplains. This will allow application of these Guidelines to those areas in a precise manner, thus defining hydraulic engineering needs, land development parameters, and private/public interests.

3. Regulations

FEMA Flood Studies

A series of several FEMA-approved hydrologic studies have been conducted to determine the floodplain areas along the majority of the reaches of the Named Regulatory Watercourses listed in Table (Appendix). These are the FEMA-designated watercourses in the Bryan-College Station area. Taken together, the flood studies conducted for these Watercourses represent the "Flood Study" of the County.

Areas Not Defined

In some instances the floodplain areas along upper reaches of a Watercourse are undefined even though the floodplain clearly extends beyond areas shown on FEMA maps. In other instances floodplain areas may be ill-defined due to topographic or other constraints.

Define Limits

Land development or building projects proposed on properties astride of, or adjacent to, the Watercourses listed in Table (Appendix) may require flood studies in order to precisely identify the elevation and geographic limits of potential floods, and thus the mitigation measures necessary for the project(s). If a proposed development will involve more than 50 lots or five (5) acres at buildout, a comprehensive flood study may be required at the discretion of the Drainage Review Authority.

Special Use Areas In land areas set aside for parks or other recreational or green space uses, or proposed for such uses, special regulations by the Drainage Regulation Entity may require adjustments in how these Guidelines are applied. Any deviation from provisions of these Guidelines in such areas will be at the discretion of the Drainage Review Authority or his/her designee.

4. Procedures

If Study Needed When a comprehensive flood study is needed for a land development or building project, a number of procedures are required. The hydrologic analyses criteria and methods stipulated in Chapter 2 (Hydrology) of these Guidelines. For minor streams or channels that are tributaries to the Primary Watercourses, existing and ultimate flood elevations shall be established by extending the adopted Flood Study as described in foregoing Section 4.2.5, Paragraph d.

Plot Limits Water surface elevations based on the configuration and limitations of the existing channel shall be determined for the ultimate development conditions planned by the City for the Watershed involved. The resulting geographic limits of projected flooding will be plotted by the engineer conducting the study.

Channel Changes When existing channels are straightened, improved in cross-section and/or lined, existing floodplain and floodway areas are likely to be altered. Redefinition shall follow the methodology for floodway determination outlined in Paragraph 2 of this Section.

Limited Effects Proposed changes in channel section or alignment shall not increase the flood elevations (established by the adopted Flood Study) within, or upstream or downstream of, the area of modification, more than allowed by these Guidelines. Any changes shall be made part of the adopted Flood Study and submitted to the required authorities for approval prior to construction work in floodway or floodplain areas.

Section 4.3 – Culvert and Bridge Design

Section 4.3.1 – Principles

Transportation Purpose The purpose of a culvert or bridge is to allow a transportation facility to cross a drainage way. Consequently, its primary function is to satisfy transportation purposes. Designs to accomplish this end necessarily involve satisfying both hydrologic and transportation parameters.

Design Objectives Hydrologic parameters are established to achieve important design objectives: safety of transportation users; safety of surrounding properties; long term integrity of constructed facilities;

minimum maintenance costs; and integrity of the natural environment.

Parameters Vary

Not all parameters are universally applicable to drainage way crossings. Because transportation facilities (roadways) vary in their function and importance, related hydrologic parameters are varied accordingly. Conversely, parameters relating to the integrity and maintenance of constructed facilities, and those relating to potential flooding of adjacent properties cannot vary.

Section 4.3.2 – General Parameters

100-Year Discharge

The design storm discharge shall be based on the ultimate development conditions that are projected to exist in the Watershed or served by the watercourse to be crossed. In addition to satisfying parameters for passing the design discharge, the 100-year storm flow must be accommodated. Arterial and major collector roadways are not to be toped by flow from the 100-year design storm. Certain minor roadways may be topped according to criteria set out in Section 4.3.3, Paragraph c below.

Minimize Erosion

Structures shall include design features that can receive the discharge of street or storm drain flow in a manner that will prevent erosion or scour of adjacent embankments or the floor or walls of the channel. Typically, a concrete apron or other suitable surfacing shall be provided to receive the discharge.

Flood Hazard Areas

Structures within established areas of special flood hazard as defined by the flood plain management ordinance(s) of the Drainage Regulation Entity shall meet all the requirements for those areas as a minimum. These Guidelines supersede provisions for such areas only to the extent that more stringent requirements are promulgated.

Section 4.3.3 – Design Limitations and Performance Criteria

a. Determination of Design Discharges

(1). For structures over Named Regulatory Watercourses or their direct tributaries, the design discharges shall be determined from the adopted Flood Study of the County per Chapter 1 of these Guidelines.

(2). For structures over watercourses making up the secondary system, the design discharges shall be determined using the appropriate methods outlined in Chapter 2 of these Guidelines.

b. Maximum Operating Headwater

(1). For all discharges up to and including the 100-year frequency storm culverts shall be designed to limit upstream headwater to elevations that will not endanger their structural integrity or cause flooding to adjacent structures or properties.

(2). At Country Road bridge crossings the water surface elevation of the 5-year storm flow shall not be higher than one (1.0) foot below the lowest bridge support stringers unless specifically approved by the Drainage Review Authority.*

(3). For culvert crossings the upstream headwater elevation for the design discharge shall be at least one (1.0) foot below the lowest top of curb at the crossing.*

* It is recommended, subject to TxDOT confirmation, that bridges and culverts on TxDOT roadways be designed for the 25-year frequency storm but must meet the above mentioned upstream 100-year headwater criteria.

c. Allowable Over-Road Flow

Over Minor Roads Where a roadway classified as a local street or minor collector will be topped by flow from a 100-year frequency storm due to allowable lesser design storms for a culvert, the excessive storm flow may be conveyed over the roadway provided the following criteria are met.

(1). Roadway and storm drainage features must be designed so that all over-road storm flow is conveyed across the road and routed to the downstream watercourse without endangering adjacent properties or structures.

(2). The maximum depth of over-road flow shall be two (2.0) feet, measured from the roadway crown at the lowest point in the roadway profile.

(3). Considered together, the velocity and depth of over-road flow provide an indication of the potential detriment to the structural integrity of the roadway. Therefore, the product of the velocity of the overflow discharge (in feet per second) and the maximum depth of flow (in feet) as described in the foregoing paragraph shall be less than six (6) [Vx2<6], a dimensionless number. The overflow velocity shall be determined from the continuity equation as follows.

$$V = \frac{Q_{over}}{A}$$

where:

V = velocity in the overflow discharge, feet per second.

$\mathbf{Q_{over}}$ = maximum discharge over roadway, cubic feet per second.

A = area of the overflow section described by the headwater elevation and roadway profile at the crown.

d. Maximum Discharge Velocities

The velocity of discharge through a structure shall be limited based on channel conditions immediately downstream of the structure. Reference is made to Table 4.2.3-1. For discharges from the design storm, downstream conditions will be evaluated to the point where normal flow characteristics are re-established in the receiving channel, but not less than a distance that is four (4) times the difference between the width of the downstream flow and the width of the structure opening. This does not apply for discharges from less frequent storms.

4.3.4 – Physical Configuration

a. Alignment Criteria

Match Flow Lines Bridges and culverts beneath roadways should provide flow lines that match, as closely as possible, the alignment of the watercourse they are to serve. At the same time, it is desirable for watercourses to cross roadways in a perpendicular manner. Where both of these design objectives can not be reasonably satisfied, the amount of skew in crossing a roadway should be minimized. In addition, the hydraulic demands resulting from introducing any artificial turns in a watercourse must be fully accommodated by the design.

Driveway Culverts Where driveways must cross roadside ditches, culverts shall be placed in public right-of-way, generally parallel to the street, and aligned with the flow line of the ditch.

Straight Structures Changes in bridge or culvert alignment shall not occur within the right-of-way of the roadways they cross.

b. Culvert Ends

The following guidelines shall be used in designing culvert end treatments. Figure 4.3.4-1 shows a schematic diagram illustrating terms commonly used to describe a typical culvert structure.

(1) Concrete headwalls and end-walls shall be provided to be functionally monolithic with the culvert conduit and must generally be parallel with the alignment of the crossing roadway. Related wing walls shall generally be oriented according to the flow characteristics of the crossing watercourse. In no case shall headwalls or wing walls restrict the clear opening of the structure.

(2) Flared wing-walls shall be used where both of the following conditions apply:

- Approach velocities exceed six (6) feet per second for the design discharge
- The approach channel is irregular and not well defined.

(3) Wing-walls parallel to the flow line of a watercourse may be used where all of the following conditions are met:

- Approach velocities are less than six (6) feet per second for the design discharge, and
- The channel is well defined and regular in cross section, and
- Downstream channel surface protection is not necessary.

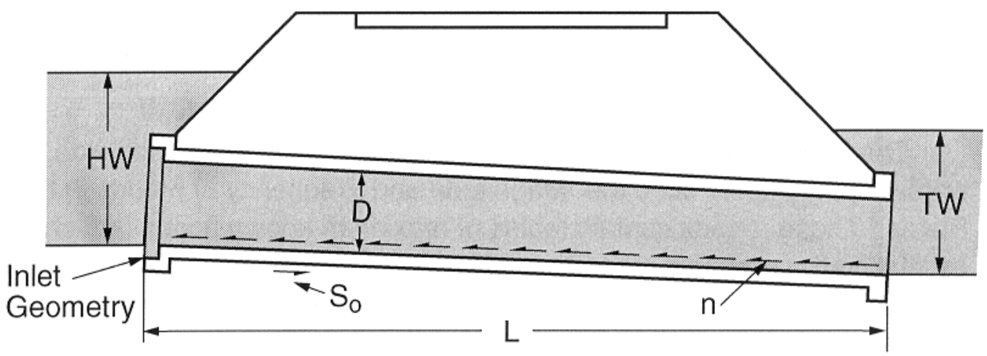
(4) The maximum side slopes for all grading in the vicinity of culvert headwalls shall be six horizontal to one vertical (6:1), unless 4:1 or flatter is approved via a design exception approved by the City Engineer.

Section 4.3.5 – Bridge and Culvert Hydraulic Design

a. Analysis Methodology

Bridge Hydraulics

The following items shall be addressed as part of the engineering design and analysis of crossing structures. Bridges shall be analyzed for hydraulic conditions using the HEC-RAS Water Surface Profiles computer program applied using the guidelines and recommendations of the U.S. Army Corps of Engineers. Unless other parameters can be substantiated to the satisfaction of the Drainage Review Authority, discharges shall be determined based on the methodology in Chapter 2 of these Guidelines



Where:

D = inside diameter for circular pipe (ft.)
HW = headwater depth at culvert entrance (ft.)
L = length of culvert (ft.)
n = Manning's surface roughness (dimensionless)
S_o = slope of the culvert pipe (ft./ft.)
TW **= tailwater depth at the culvert outlet (ft.)**
K_e = Entrance Loss (dimensionless)

Figure 4.3.4-1: Factors Influencing Culvert Discharge

Culvert Hydraulics

Culverts may be analyzed using the same method as for bridges. Additionally, they may be analyzed using accepted charts and nomographs for the type of structure and material proposed for use. TxDOT's Hydraulic Manual contains a complete treatment of culvert analysis and design, including nomographs. The latest version of TxDOT's Hydraulic Manual shall be considered the standard for analysis of culverts by these Guidelines.

b. Culvert Operations

The rate of flow through a culvert barrel is limited by several direct factors such as slope, length, and surface roughness. Where conditions at the culvert entrance (inlet) prevent optimum flow based solely on these factors, the culvert is considered to operate under "inlet control". When the flow permitted through the barrel is less than the flow allowed at the upstream entrance, the culvert is considered to operate under "outlet control" (sometimes referred to as "barrel control"). For each design discharge the type of control shall be determined.

c. Headwater and Tail Water Elevations

(1). Tail water elevations shall be determined using one of the methods described in the portion of the Guidelines for open channel design (see Section 4.2).

(2). Headwater elevations shall be determined by adding the total head losses through the structure to the tail water elevation, for the given discharge.

d. Head Losses

The total head losses, **H**, on a structure is the sum of all losses due to exit, friction, and entrance conditions for the given discharge.

(1). Entrance losses are caused by the narrowing of flow from the normal channel width to the structure opening (predominant for bridges), or to the shape or condition of the actual inlet or opening (predominant for culverts). Channel losses of this type must be computed using a standard step procedure as outlined in the part of this Section dealing with open channels. Entry losses shall be computed using the following equation:

$$H_e = k_e\left[\frac{V_2^{\ 2}}{2g} - \frac{V_1^{\ 2}}{2g}\right]$$

where:

$\mathbf{H_e}$ = entrance head loss, feet

$\mathbf{V_2}$ = velocity of flow in culvert, feet per second

$\mathbf{V_1}$ = velocity of flow in approach channel, feet per second

g = 32.2 feet per second per second

$\mathbf{k_e}$= entry loss coefficient from Table C-12, Appendix C.

(2). Exit losses are caused by the expansion of flow from the structure opening to the normal downstream channel width. The same equation for entrance losses applies to those for exit losses except $\mathbf{k_e}$ may be taken as **1.0** and $\mathbf{V_1}$ shall be defined as the velocity of flow in the downstream receiving channel after full expansion.

(3). Friction losses are those that occur within the structure itself. These can range from open channel flow losses, and pressure flow losses, to losses caused by physical obstructions within the structure (bridge piers for example). All friction losses shall be accounted for in the analysis and design of crossing structures.

e. Erosion and Scour Protection

(1). All culverts determined to be functioning under inlet control for the design discharge shall have an energy dissipation structure at the outlet of the culvert or meet the requirements of Paragraph (2) below.

(2). The velocity of the design stormflow in the structure shall not exceed the requirements for the downstream channel condition stipulated in Table 4.2.3-1

Section 4.3.6 – Bridge Design Procedure

The following is a general bridge hydraulic design procedure:

1. Determine most efficient alignment of proposed roadway, attempting to minimize skew at the proposed stream crossing.
2. Determine design discharge from hydrologic studies or available data (City, FEMA, Corps, TxDOT, or similar sources).
3. If available, obtain effective FEMA hydraulic backwater model. Note: it is assumed that if a bridge is required instead of a culvert, the drainage area would exceed one square mile and could already be included in a FEMA study. If effective FEMA model or other model is not available, prepare a basic hydrologic model (discharges) and backwater model for the stream. The Drainage Regulation Entity requires a backwater analysis for bridges on unstudied streams, either by hand computations or HEC-RAS.
4. Using Corps or FEMA guidelines, compute or duplicate an existing conditions 100-year design profile. (Note: see section on exceptions.) Compute profile for design 100-year flood, to use as baseline for design of new bridge/roadway crossing.
5. Use the design discharge to compute an approximate opening that will be needed to pass the design storm (for this preliminary sizing, use the procedures for a normal-depth design or simply estimate required trapezoidal opening).
6. Prepare a bridge crossing data set in the hydraulic model to reflect the preliminary design opening, which includes the required freeboard, and any channelization from downstream to upstream to transition the floodwaters through the proposed structure.
7. Compute the proposed bridge flood profile and design parameters (velocities, flow distribution, energy grade). Review for criteria on velocities and freeboard, and revise model as needed to accommodate design flows.
8. Review the velocities and determine erosion control requirements downstream, through the structure, and upstream.
9. Finalize the design size and erosion control features, based on comparing model with existing conditions profiles, impacts on other properties, FEMA guidelines, and city criteria.
10. Exceptions/Other Issues

A. Conditional Letter of Map Amendment (CLOMR) will be needed for new crossings of FEMA streams and if required by Wharton County.
B. Coordinate with Corps of Engineers' Section 404 Wetland/Waters of US permit requirements.
C. Evaluate the project with respect to Drainage Regulation Entity drainage policy regarding upstream and downstream impacts.
D. Design should be for fully developed watershed conditions, but available discharges could be FEMA, existing conditions hydrology. This will require either: 1) New hydrology; 2) Extrapolation of fully developed from existing data; or 3) Variance from city on design discharge.
E. Freeboard requirements could require an unusually expensive bridge or roadway elevation that is impractical. Some reasonable variance criteria should be considered.

The procedures in this section are acceptable. In addition, backwater analysis will be required either hand computed or HEC-RAS, for any proposed bridge, to determine accurate tailwater elevations, velocities, headlosses, headwater elevations, profiles and floodplains affected by the proposed structure. If the current effective FEMA model is a HEC-2 model, the engineer has the option to either use that model, or convert to HEC-RAS for analysis of proposed conditions.

Section 4.3.7 – Culvert Design Procedures

The following paragraphs supplement the description in Section 4.3.5.

A culvert conveys surface water through a roadway embankment or away from the street right-of-way. The hydraulic and structural designs must be such that minimal risks to traffic, property damage, and failure from floods prove the results of good engineering practice and economics. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Design considerations include site and roadway data, design parameters (including shape, material, and orientation), hydrology (flood magnitude versus frequency relation), and channel analysis (stage versus discharge relation).

The culvert design process includes the following basic stages:

1. Define the location, orientation, shape, and material for the culvert to be designed. In many instances, consider more than single shape and material.
2. With consideration of the site data, establish allowable outlet velocity (V_{max}) and maximum allowable depth of barrel.
3. Based upon subject discharges (Q), associated tailwater levels (TW), and allowable headwater level (HW_{max}), define an overall culvert configuration to be analyzed (as part of the design process of trial and error)—culvert hydraulic length (L), entrance conditions, and conduit shape and material.
4. Determine the flow type (supercritical or subcritical) to establish the proper path for determination of headwater and outlet velocity.
5. Optimize the culvert configuration.

6. Treat any excessive outlet velocity separately from headwater.

A backwater analysis is required, either hand computed or HEC-RAS, to determine accurate tailwater elevation, headlosses, headwater elevations and floodplains affected by the proposed structure. If the current effective FEMA model for the stream is a HEC-2 model, the engineer has the option to either use that model, or convert to HEC-RAS for analysis of proposed conditions.

Nomographs

Note: Nomographs are not allowed by the Drainage Regulation Entity for final sizing of culverts. The reference for nomographs is FHWA HDS-5. A backwater analysis using HEC-RAS is required.

Section 4.4 – Storage Design

Section 4.4.1 – General Storage Concepts

Controlled Discharge

The purpose of a detention facility is to store excess stormwater runoff and discharge it at a predetermined controlled rate. Typically, this is done so that discharge rates from a development site will be limited to those that existed prior to any land development activities. This is accomplished for a range of design storms.

Facility Types

As a function of how they are designed to operate, detention facilities can be grouped into three categories. One type is effectively a permanent pond. That is, it retains a significant water pool on a year-round basis, but acts to detain stormflow, metering water release until some predetermined pool level is reached. This might be termed a “pool-type” (retention) facility. Another type might be termed a “wetland-type” facility. This type retains storm flow and meters its release, but is not intended to drain fully dry. Rather, an aquatic ecosystem is specifically designed into part or all of the facility so that it is sustained by the storm flow that passes through the facility. The third type is designed to drain fully dry between storm events, a “dry-type” facility.

Detention Philosophy

These Guidelines are largely oriented toward development of “dry-type” facilities. However, where topographic, water, and other physical characteristics make it feasible to design viable “wetland-type” facilities, they are encouraged. Successful “wetland-type” or “pool-type” facilities can be difficult to establish and are highly dependent on an expert multi-discipline design team for their success. Use of a “wetland-type” or “pool-type” facility will be considered a special design, and must be approved by the Drainage Review Authority on a case-by-case basis. The Drainage Review Authority must be informed early during the planning of a project. In addition, the design must be handled by qualified professionals, experienced in establishing self-sustaining wetland environments. The stormwater detention function shall not be compromised by such special designs.

Drained Areas Detention facilities may be site-specific, or may be designed for a specific land development project comprised of multiple lots, streets, utilities, and other infrastructure elements. In any case, their primary purpose is to protect immediate downstream properties and drainage system from excessive stormflow. One detention facility, or a system of facilities, may be necessary to meet stormwater management objectives for an entire Project Area. A site-specific example would be using a detention facility in a large parking area to avoid overwhelming adjacent streets and storm sewers of the secondary system. Common methods include use of depressions in parking lots and/or landscaped areas that drain dry between rainfall events.

Regional Detention Detention facilities also may be regional in scope, receiving stormwater from many land development Project Areas and/or sites. In such situations a limiting capacity is often that of the drainage system that traverses an exiting developed area.

Multi-Purpose Areas A regional facility requires a large land area for the required storage and, thus, is usually designed for multiple uses compatible with its stormwater purpose. For best results, these are permanent storage ("pool-type") facilities designed to hold water between rainfall events, and may be combined with green-space and recreation areas.

"Regional" Limited Detention facilities will only be considered "Regional" at the discretion of the Drainage Review Authority.

Section 4.4.2 – General Storage Design Procedures

a. Design Storm

Secondary System Any detention facilities to be located in the Secondary Drainage System that are site-specific, or will serve a specific development project, shall use a maximum design storm based on specific detention requirements stipulated in these Guidelines. The following sequence of design storms shall be used until the maximum design storm is reached: 2-year, 10-year, and 100-year. Full consideration must be given to the receiving facilities of the secondary system relative to performance standards and Conveyance Pathway requirements. In addition, the 100-year design storm shall be evaluated to check emergency overflow requirements of the detention facility and the effects of resulting flows on downstream drainage systems.

Primary System Where detention facilities are required to be located in the primary drainage system, either on-line (astride the main channel) or as an adjacent flood relief feature, they shall use a maximum design storm having an average return period of 100 years or greater as determined by the Drainage Review Authority.

b. Delineation of Drainage Area

Each detention facility shall serve a Design Drainage Area that contributes (or will contribute) runoff to the facility. The Design Drainage Area and the runoff computations shall be determined for existing pre-development conditions and for expected post-development conditions.

c. Pre-development and Post-development Hydrographs

A pre-development hydrograph representing the Design Drainage Area and land cover conditions existing prior to the proposed development shall be determined. Likewise, a post-development hydrograph shall be determined representing the Design Drainage Area and land cover conditions proposed to exist after buildout of the Project Area that contributes runoff to the detention facility.

Hydrographs shall be determined using the appropriate methods from Chapter 2 (Hydrology) of these Guidelines.

d. Determination of Storage Volume

Pre/Post Flows Storage volume shall be adequate to ensure that the peak discharges from the detention facility determined via the post-development hydrographs will be limited to values equal to, or less than, the peak discharges determined by the pre-development hydrographs for the design storms.

Existing Storage Any land features, such as low areas or ponds, having the effect of storing or detaining stormwater during pre-development conditions shall not be ignored in determining the required post-development storage volume. If such features are to be altered or eliminated, then the required storage volume must be increased to account for their pre-development detention characteristics. The existence and effects of such features shall be disclosed during the design review process.

Storage Routing All detention facilities shall have the necessary storage volume determined from storage routing analysis procedures.

e. Storage Routing Analysis

The basis of this method is the continuity equation modified to yield the following:

$$(I_1 + I_2) + \left(\frac{2S_1}{dt} - O_1\right) = \left(\frac{2S_2}{dt} + O_2\right) \quad (4.4.1)$$

where:

I = the inflow over time period t,

O = the outflow over time period t,

S = the storage volume,

dt = the designated time period, and

subscripts 1 and **2** represent the beginning and end of time period respectively.

The use of this procedure is based the following assumptions:

- The inflow hydrograph is known.
- The starting conditions of storage volume and outflow are known at the beginning of the routing.
- The discharge rate at the outlet structure(s) is only a function of the head available.
- The relationship between depth and storage are known.
- The time period "**dt**" shall be taken as less than or equal to 1/5 $\mathbf{t_c}$ (time of concentration).

f. Outlet Structures

(1). Design of outlet structures shall consider the conditions for all required design storms. The structure shall limit the peak discharge to be equal to, or less than, the peak discharge that existed under pre-development conditions for all design storms.

(2). Except for facilities designed to have a permanent storage component, outlet structures shall be designed to allow the facility to be drained dry by gravity.

(3). An emergency overflow outlet shall be provided with a capacity to carry the peak discharge from a 100-year frequency storm for buildout conditions over the entire Design Drainage Area. This discharge must be limited and directed in a manner that will: prevent damage to adjacent properties or public infrastructure; avoid damaging the structural integrity of any element of the detention facility; and present no hazardous conditions. In addition, the discharge shall be evaluated for its effect on the downstream receiving drainage system, and shall not exceed its capacity to control and contain the storm discharge assuming ultimate conditions.

(4). Analysis and design of outlet works shall use the methods promulgated by these Guidelines, namely those dealing with drainage inlets, drainage conduit, open channel flow, and culverts.

g. Procedure for Drainage Areas Less Than 50 Acres

The following is an alternative procedure applicable to smaller developments and drainage areas (Total of drainage area less than 50 acres).

The maximum allowable release rate from the detention facility during the 100-year storm event is the rate of runoff from the drainage area prior to development.

The acre-feet of flood control storage, S, to be provided by the facility for the 100-year storm event is:

$$S=I^{1/2}xA$$

where,

I = the average percent imperviousness of the developed area served by facility divided by 100.

A = the developed area served by the facility (in acres).

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 4.3.5), 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.

Section 4.4.3 – Physical Characteristics For Detention Storage Facilities

a. Side and Bottom Slopes

(1). Side slopes shall not exceed 4:1 for vegetative cover and 2:1 for non-vegetative (concrete, riprap, etc.) cover.

(2). Bottom slopes must be 0.2 percent (0.2%) or steeper and directed to the low flow outlet.

(3). A low-flow invert section of concrete or other materials acceptable to the Drainage Review Authority shall be provided for all facilities proposed to have a bottom with vegetative cover. To minimize the need for these sections, designs are encouraged to locate the inflow and outflow points as close to each other as practical.

b. Emergency Overflow Requirement

(1). All detention facilities shall be fitted with an emergency overflow feature that discharges into a recognized drainage facility acceptable to the Drainage Review Authority.

(2). The geometry of an emergency overflow structure shall be that of a rectangular or trapezoidal weir.

(3). The surface treatment of the structure and its discharge path to a recognized drainage facility shall give due regard to maintenance. Velocities shall be limited to be consistent with the proposed surface treatments to prevent erosion, prevent undercutting of structural components, and avoid other maintenance difficulties.

(4) The elevation of the weir crest shall not be less than the water surface elevation resulting from the design 100-year storm, assuming a fully operating discharge structure. See diagram presented in Figure 4.4.3-1.

(5). The entire perimeter of the facility shall have at least one half (0.5) foot of freeboard above the water surface elevation generated by the 100-year storm assuming buildout conditions of the Design Drainage Area, a completely clogged discharge structure, and a fully functioning spillway.

c. Storage Depth

In parking areas the maximum design storage depth, based on site buildout conditions, shall not exceed six (6) inches.

d. Retention (Permanent Storage) Facilities

All facilities located astride natural streams or water courses that are designed with a permanent storage component shall meet all design and construction criteria for dams and reservoirs as required by the Texas Commission on Environmental Quality (TCEQ).

e. Allowance For Sedimentation

No allowance for sedimentation is required as long as adequate maintenance procedures are followed.

Measures to mitigate the effects of erosion and resulting sedimentation are divided into two categories: temporary (non-permanent) and permanent.

f. Non-Permanent Measures

Non-permanent (temporary) measures are designed to manage soil materials in a manner that will minimize their migration away from any land development or site improvement project during clearing, grubbing, grading, excavation, filling, and construction activities. This includes capturing sediments eroded by stormwater that traverses areas where established vegetation has been disturbed or removed, or that impacts loose materials, including stockpiles. The emphasis is on preventing sediment from being transported and deposited, by wind, water, or actions of man, onto adjacent properties, or into the primary or secondary drainage systems.

g. Permanent Measures

Permanent measures are designed to prevent erosion and resulting sedimentation from occurring over time, whether within earthen channels, in various facilities constructed for purposes of managing storm flow, or across unpaved land areas. Properly conceived, designed, and constructed, permanent measures can also promote the proper management of storm flow.

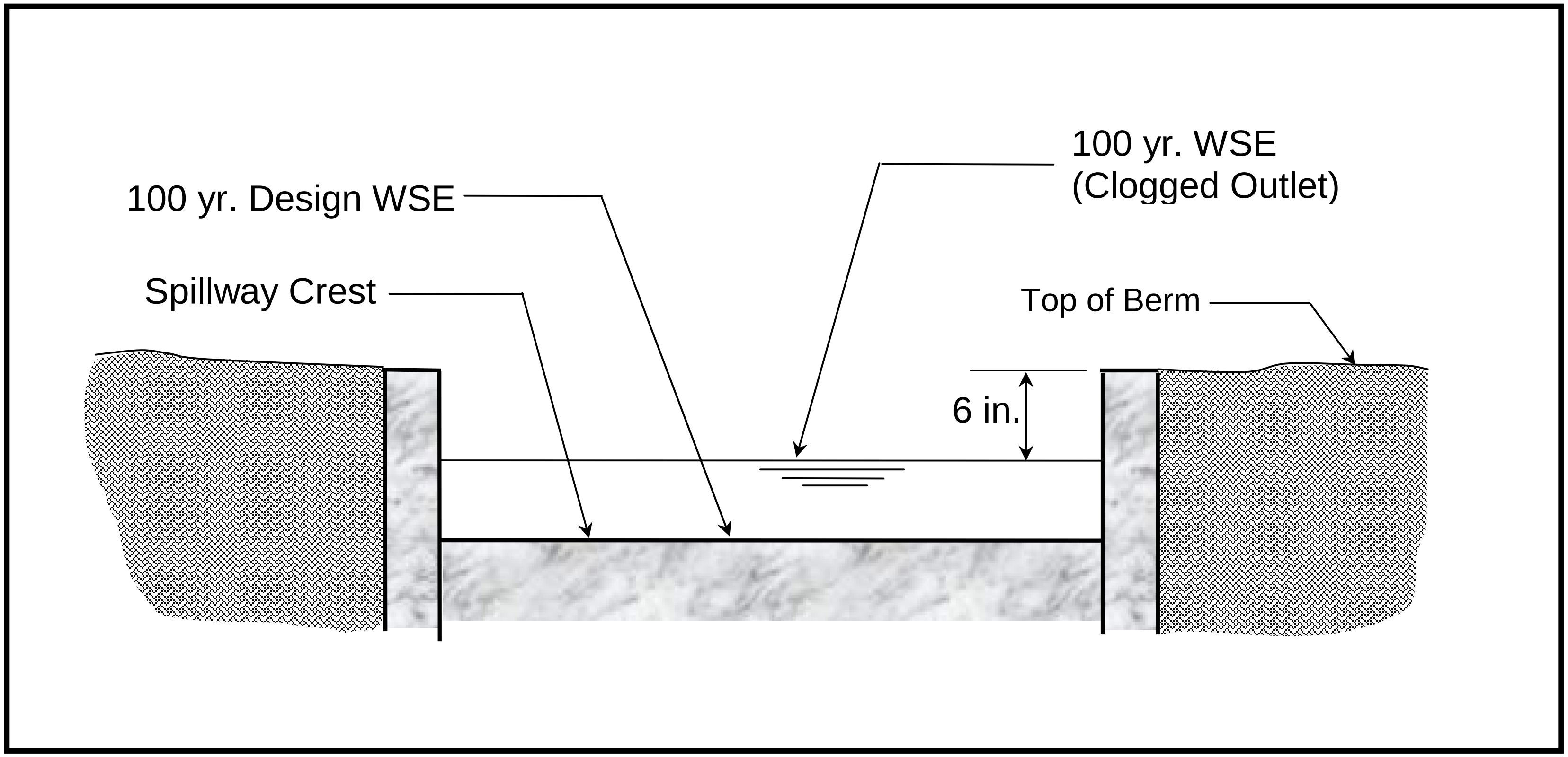


Figure 4.4.3-1: Diagram of Detention Spillway Section

Section 4.5 – Outlet Structures

Section 4.5.1 – Introduction

Primary outlets provide the critical function of the regulation of flow for structural storm water controls. There are several different types of outlets that may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the storm water facility can be designed as a simple pipe or culvert. For multistage control structures, the inlet is designed considering a range of design flows.

Section 4.5.2 – Outlet Structure Types

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in storm water facility design:

- Orifices
- Perforated risers
- Pipes / Culverts
- Sharp-crested weirs
- Broad-crested weirs
- V-notch weirs
- Proportional weirs
- Combination outlets

The design professional must pay attention to material types and construction details when designing an outlet structure or device. Non-corrosive material and mounting hardware are key to device longevity, ease of operation, and low cost maintenance. Special attention must also be paid to not placing dissimilar metal materials together where a cathodic reaction will cause deterioration and destruction of metal parts.

Protective coatings, paints, and sealants must also be chosen carefully to prevent contamination of the storm water flowing through the structure/device. This is not only important while they are being applied, but also as these coating deteriorate and age over the functional life of the facility.

A range of design flood flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway through the embankment. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

1. Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice.

For a single orifice, the orifice discharge can be determined using the standard orifice equation below.

$Q = CA\,(2gH)^{0.5}$ (4.5.1)

where:

Q = the orifice flow discharge
(cfs) C = discharge coefficient
A = cross-sectional area of orifice or
pipe (fe) g = acceleration due to gravity
(32.2 ft/s^2)
D = diameter of orifice or pipe (ft)
H = effective head on the orifice, from the center of orifice to the water surface

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces.

When the pipe thickness is thinner than the orifice diameter, with sharp edges, a coefficient of 0.6 should be used. For square-edged entrance conditions the generic orifice equation can be simplified:

Q = O.6A (2gH)o.5 = 3.780^{2}Ho.^{5} (4.5.2)

where:

D = diameter of orifice or pipe (ft)

When the pipe thickness is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used.

Flow through multiple orifices, can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

2. Perforated Risers

A special kind of orifice flow is a perforated *riser*. In the perforated riser, an orifice plate at the bottom of the riser, or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so as not to become the control.

A shortcut formula has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988):

$$Q = C_p \frac{2A_p}{3H_s} \sqrt{2g} H^{3/2}$$

(4.5.3)

where:

Q = discharge (cfs)
C_p = discharge coefficient for perforations (normally
0.61) Ap = cross-sectional area of all the holes *(tr)*
Hs = distance from *8/2* below the lowest row of holes to *8/2* above the top row (ft)

3. Pipes and Culverts

Discharge pipes are often used as outlet structures for storm water control facilities. The design of these pipes can be for either single or multi-stage discharges. Minimum pipe diameter is 18 inches. If design outflow requires a smaller diameter pipe a “weir box” structure may be designed to regulate the outflow.

Pipes should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in Section 4.3, *Culvert Design,* or by using equation 4.5.4 (NRCS, 1984).
The following equation is a general pipe flow equation derived through the use of the Bernoulli and continuity principles.

$$Q = a[(2gH) / (1 + k_m + k_p L)]0.5 \qquad (4.5.4)$$

where:

- Q = discharge (cfs)
- a = pipe cross sectional area *(ft^2)*
- g = acceleration of gravity (ft/s^2)
- H = elevation head differential (ft)
- k_m = coefficient of minor losses (use 1.0)
- kp = pipe friction coefficient = $5087n^2/D^{4/3}$
- L =pipe length (ft)

4. Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear as it overflows, the overflow is termed a *sharp-crested* weir. If the sides of the weir also cause the through flow to contract, it is termed an *end-contracted* sharp-crested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a measurement device. The discharge equation for this configuration is (Chow, 1959) which can also be used for circular pipe risers:

$$Q = [(3.27 + 0.4(H/H_c)] LH^{1.5} \qquad (4.5.5)$$

where:

- Q = discharge (cfs)
- H = head above weir crest excluding velocity head (ft)
- He = height of weir crest above channel bottom (ft)
- L = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Qs = Qf(1 - (H_z/H_1)^{1.5})^{0.385} \qquad (4.5.7)$$

where:

- Q_s = submergence flow (cfs)
- Q_f = free flow (cfs)
- H_1 = upstream head above crest (ft)
- H_2 = downstream head above crest (ft)

5. Broad-Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a *broad-crested* weir. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner. If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

The equation for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \qquad (4.5.8)$$

where:

Q = discharge (cfs)
C = broad-crested weir coefficient
L = broad-crested weir length perpendicular to flow (ft)
H = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.0. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used.

6. V-Notch Weirs

The discharge through a V-notch weir can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan(\theta/2)\, H^{2.5}$$

where:

Q = discharge (cfs)
θ = angle of V-notch (degrees)
H = head on apex of notch (ft)

7. Combination Outlets

Combinations of orifices, weirs, and pipes can be used to provide multi-stage outlet control for different control volumes within a storage facility (Le., water quality protection volume, streambank protection volume, and flood control volume).

They are generally two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs, or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe

Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

The use of a combination outlet requires the construction of a composite stage-discharge curve suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed in the next section.

Section 4.5.3 – Multi-Stage Outlet Design

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements for storm water ponds, storm water wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms.

A design engineer may be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routing are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite of the different outlet that are used for different elevations within the multi-stage riser.

Section 4.5.4 – Trash Racks and Safety Grates

1. Introduction

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions:

- Keeping debris away from the entrance to the outlet works where they will not clog the critical portions of the structure
- Capturing debris in such a way that relatively easy removal is possible
- Ensuring that people and large animals are kept out of confined conveyance and outlet areas
- Providing a safety system that prevents anyone from being drawn into the outlet and allows them to climb to safety

When designed properly, trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985; Allred Coonrod, 1991). The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety and size of outlet. Well-designed trash racks can also have an aesthetically pleasing appearance.

An inclined vertical bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

2. Trash Rack Design

Trash racks must be large enough so that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used “rule-of-thumb” is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level - the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb are found in literature. Figure 4.5.4-1 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack ..

Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USSR, 1978; UDFCD, 1999). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% is chosen as a working assumption. These clogging percentages are recommended (but <u>not</u> required) by the Drainage Review Authority.

$$H_g = K_{g1}\,(W/X)^{4/3}\,(V_u^2/2g)\,\sin 0_g \qquad (5.10)$$

Where:

- H_g = head loss through grate (ft)
- K_{g1} = bar shape factor:
  - 2.42 - sharp edged rectangular
  - 1.83 - rectangular bars with semicircular upstream faces
  - 1.79 - circular bars
  - 1.67 - rectangular bars with semicircular up- and downstream faces
- w = maximum cross-sectional bar width facing the flow (in)
- x = minimum clear spacing between bars (in)
- V_u = approach velocity (ft/s)
- g = acceleration due to gravity (32.2 ft/s^2)
- 0_g = angle of the grate with respect to the horizontal (degrees)

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$H_g = \frac{K_{g2} V_u^2 /}{2g} \tag{5.10}$$

Where:

K_{g2} is defined from a series of fit curves as:

- sharp edged rectangular (length/thickness = 10)

$K_{g2} = 0.00158 - 0.03217\, A_r + 7.1786\, A_r^2$

- sharp edged rectangular (length/thickness = 5)

$K_{g2} = -0.00731 + 0.69453\, A_r + 7.0856\, A_r^2$

- round edged rectangular (length/thickness = 10.9)

$K_{g2} = -0.00101 + 0.02520\, A_r + 6.0000\, A_r^2$

- circular cross section

$K_{g2} = 0.00866 + 0.13589\, A_r + 6.0357\, A_r^2$

and A_r is the ratio of the area of the bars to the area of the grate section.

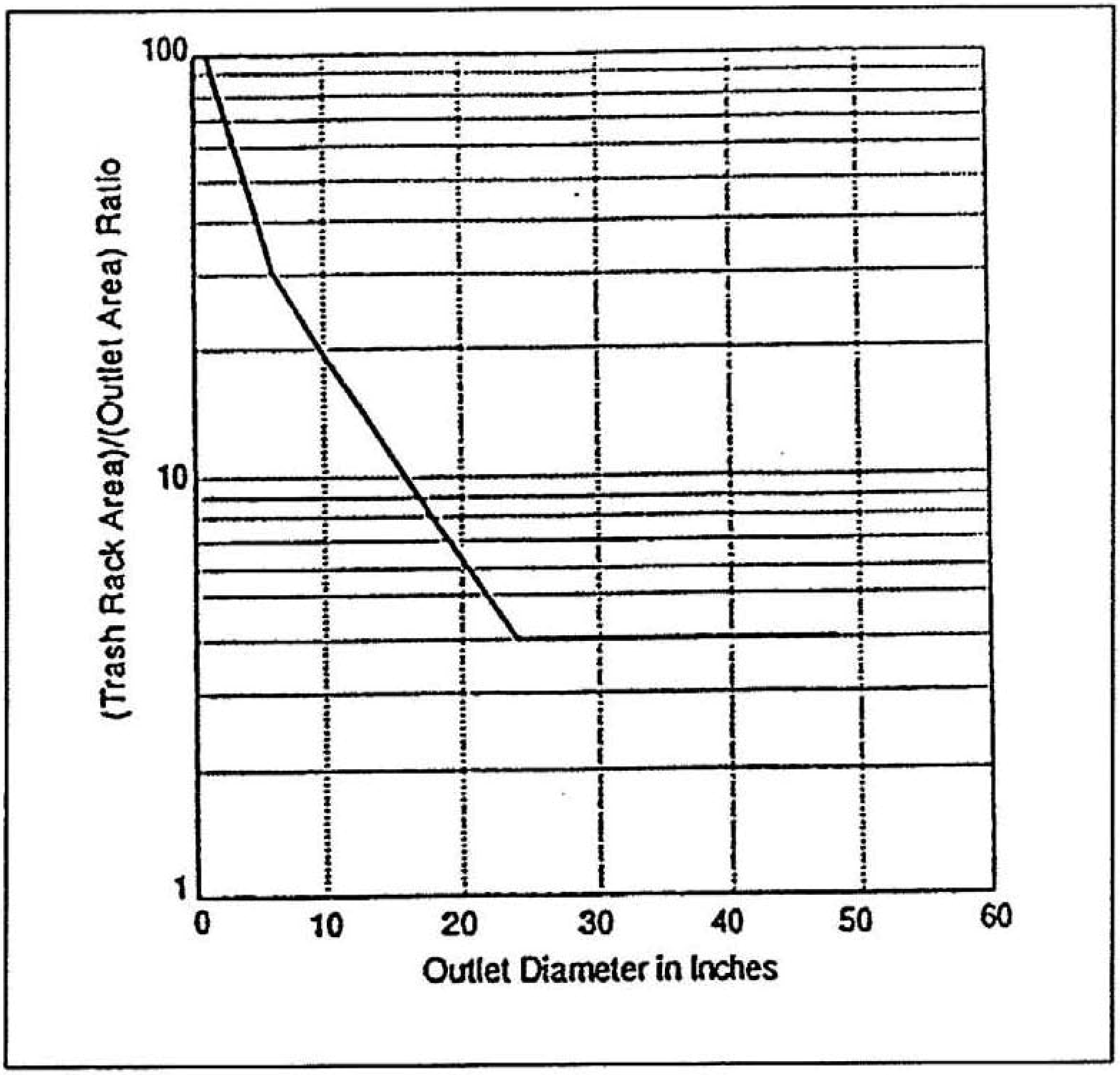


Figure 4.5.4-1 **Minimum Rack Size vs. Outlet Diameter**
(Source: UDCFD, 1992)

Section 4.5.5 – Secondary Outlets

1. Introduction

The purpose of a secondary outlet (emergency spillway is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 4.5.5-1 shows an example of an emergency spillway.

In many cases, on-site storm water storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

2. Emergency Spillway Design

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 4.6-19). The emergency spillway is proportioned to pass flows in excess of the design flood (typically the 100-year flood or greater) without allowing excessive velocities and without overtopping of the embankment. Any dam, six feet or higher, must meet appropriate state and federal design standards, especially those regarding spillway design requirements related to passage of the probable maximum flood. In any case, the 100-year flood discharge, assuming blockage of outlet works, must be conveyed with some freeboard as specified by local criteria. Flow in the emergency spillway is open channel flow (see Section 4.2, Open Channel Design, for more information). Normally, it is assumed that critical depth occurs at the control section.

NRCS (SCS) manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper than 3:1 horizontal to vertical.

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated on the next page.

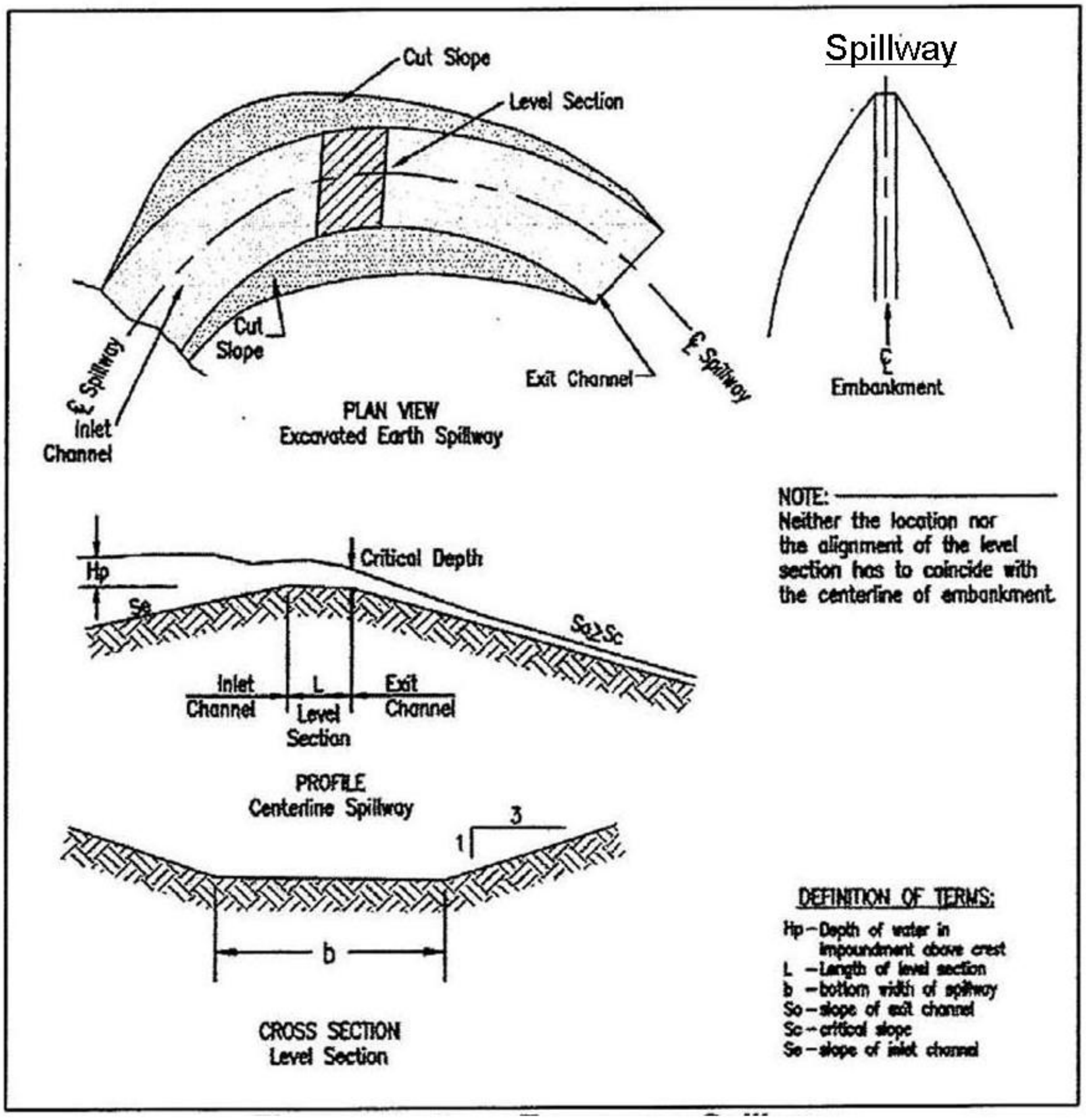


Figure 4.5.5-1 Emergency Spillway

(Source: VDCR, 1999)

Section 4.6 – Erosion and Sedimentation

Section 4.6.1 – Principles

1. Temporary and Lasting Measures

Measures to mitigate the effects of erosion and resulting sedimentation are divided into two categories: temporary (non-permanent) and permanent.

a. Non-Permanent Measures

Non-permanent (temporary) measures are designed to manage soil materials in a manner that will minimize their migration away from any land development or site improvement project during clearing, grubbing, grading, excavation, filling, and construction activities. This includes capturing sediments eroded by stormwater that traverses areas where established vegetation has been disturbed or removed, or that impacts loose materials, including stockpiles. The emphasis is on preventing sediment from being transported and deposited, by wind, water, or actions of man, onto adjacent properties, or into the primary or secondary drainage systems.

b. Permanent Measures

Permanent measures are designed to prevent erosion and resulting sedimentation from occurring over time, whether within earthen channels, in various facilities constructed for purposes of managing storm flow, or across unpaved land areas. Properly conceived, designed, and constructed, permanent measures can also promote the proper management of storm flow.

2. Erosion Reference

A general guide and reference service for erosion and sediment control methods and protection is published by the National Resource Conservation Service (formerly Soil Conservation Service). The publication entitled "Erosion and Sediment Control Guidelines for Developing Areas in Texas" is adopted as the definitive reference for purposes of these Guidelines, and can be obtained at the address listed below. The agency can also be reached through its web site at: www.NRCS.USDA.gov.

U.S. Natural Resource Conservation Service
P.O. Box 6567
Fort Worth, TX 76115

3. Scope of Actions

Measures to prevent the movement of sediment by erosion or action of man shall be implemented at all areas undergoing development or construction. Positive steps shall be taken by those conducting such work to accomplish the following:

a. Prevention

Prevent the transport of sediment from all work areas onto adjacent properties or into any part of the primary or secondary drainage systems.

b. Clean Up

Promptly remove all sediment resulting from their activities if it enters onto adjacent properties or into any part of the primary or secondary drainage systems.

Section 4.6.2 – Non-Permanent Erosion Control Measures

Methods

Non-permanent methods to control or contain sediment materials generally fall into two categories: sediment basins and barriers. One or more methods shall be used on areas where construction activity of any kind results in earthen soils that are not covered by vegetation or impervious surfaces prior to final completion of a project.

Regulations

Non-permanent erosion control measures as required by the latest regulations of the Texas Commission on Environmental Quality (formerly the Texas Natural Resource Conservation Commission) shall be used on all applicable land development or site projects approved for construction in the Drainage Regulation Entity. Compliance with such regulations during project construction shall be a requirement for continued operation of construction activities. Construction plans for grading, excavation, and street and utility construction in subdivision projects must include stormwater pollution prevention plans (SW3Ps). Appendix B has additional discussion of these regulations and BMP's.

Section 4.6.3 – Permanent Erosion Control Measures

The following actions shall be incorporated into the design and construction of permanent land development or permanent improvements to properties.

1. Land Grading

a. Excavation

The cut face of earth excavation that will be in publicly maintained areas and is to be vegetated shall not be steeper than four horizontal to one vertical (4:1). Such excavated areas that will be vegetated and remain privately owned shall not be steeper than three horizontal to one vertical (3:1). Cut slopes that will not be vegetated shall be protected by approved surface treatments to protect them from erosion.

b. Earthen Fills

Permanent exposed faces of fills shall be no steeper than three horizontal to one vertical (3:1) and shall be vegetated or otherwise surfaced to protect them from erosion.

c. Runoff Management

Provisions are to be made to safely convey surface water to storm drains or suitable natural water courses and to prevent surface runoff from damaging cut faces and fill slopes.

d. Adjoining Properties and Facilities

Near Property Lines Excavations shall not be made so close to property lines as to endanger adjoining property without protecting such property from erosion, sliding, settling, or cracking. No fill is to be placed where it will slide or wash onto adjacent or down stream properties, including structures.

Near Channels/Streams No fill shall it be placed adjacent to the bank of a channel or natural stream in a manner that will allow it to migrate into the channel or stream, cause bank failure, or reduce the capacity of the channel or stream in any way.

2. Unpaved Areas and Swales

a. Stripped Areas

All areas that are graded and stripped of natural vegetative cover shall receive at least a finish layer of topsoil at least six (6) inches in depth and be seeded or covered with sod according to approved plans. The result shall be reestablishment of a protective vegetative cover capable of resisting the erosive effects of surface flow.

b. Swale Treatments

Earthen swales that will not be lined with hard surfaces shall be seeded or covered with sod according to approved plans.

3. Channels

a. Banks and Inverts

Earthen channel banks and inverts shall be treated with vegetative materials according to the requirements of the TxDOT Construction Standards.

b. Surface Treatments

Design velocities shall be less than the recommended maximum velocity acceptable for the proposed surface treatment as outlined in Table 4.2.3-1. Where multiple surface treatments are to be situated in a length of channel in close enough proximity to have interactive effects, the limiting velocity shall be the minimum recommended value among those representing the proposed surface treatment types.

c. Supercritical Flow

Channels designed to function with supercritical flow shall be fitted with lining and energy dissipation features adequate to handle the resulting velocities and hydraulic jumps.

d. Channel Protection

The integrity of channel linings and cross sections shall be protected at all locations where stormwater enters a channel from other stormwater facilities. See "Outfall Junctures" in Section 4.2.3, Paragraph c of these Guidelines.

4. Energy Dissipation

Energy dissipation features are required at any point where stormflow design velocities are expected to exceed the surface erosion characteristics of the receiving facility, or empirical criteria established elsewhere in these Guidelines.

a. Allowable Velocities

Design velocities shall be less than the recommended maximum velocity acceptable for the proposed surface treatment as outlined in Table 4.2.3-1.

b. Examples Designs

Acceptable configurations for energy dissipation structures at outfall structures and channels are in Appendix F and presented in TxDOT Technical Specifications, but other special designs will be considered. Designs suitable to specific situations are encouraged. Reinforcing steel shall be designed to resist the anticipated hydraulic, hydrostatic, dead, and live loads for the structures.

c. Natural Dissipation Features

Energy dissipation features designed to replicate those occurring due to interaction between stormflow and the stream bed along natural streams are encouraged. Plunge pools in series, stilling "basins", surfaces, and vegetative materials are examples of elements that might be used in combination to achieve such designs.

Section 4.6.4 – Stone Riprap Design

A number of agencies and researchers have studied and developed empirical equations to estimate the required size of rock riprap to resist various hydraulic conditions, including the U.S. Army Corps of Engineers, Soil Conservation Service and Federal Highway Administration. As with all empirical equations based on the results of laboratory experiments, they must be used with an understanding of the range of data on which they are based.

A paper prepared by Garry Gregory in June of 1987 has been widely used in Texas for riprap design. He recommends estimating $D_{50} = \tau/[0.04(\gamma_s-\gamma)]$, including similar adjustments for bends and channel side slopes. Excerpts from this paper are presented below. Also see Appendix F for gradation curves (Plates 1 through 6) and stone riprap sections (Plates B-41 and B-43). Regardless of computed thickness the minimum allowable riprap thickness is 12 inches.

Design Criteria

Natural or Construction Channel Protection:

1. Calculate boundary shear (tractive stress or tractive force) by:

$T_o = \gamma RS$ (Eq. 4.6.4-1)

Where: T_o=average tractive stress on channel bottom, PSF
γ=unit weight of water (62.4pcf)
R=hydraulic radius of channel
S=slope of energy gradient

$T_o^1 = T_o (1- \sin^2 \varphi/\sin^2 \theta)^{0.5}$ (Eq. 4.6.4-2)

Where: T_o^1=average tractive stress on channel side slopes, PSF
T_o=same as in equation (Eq. 4.6.4-1)
φ =angle of side slope with the horizontal
θ =angle of repose of riprap (approx. 40°)

The greater value of T_o or T_o^1 governs.

2. Determine the tractive stress in a bend in the channel by:

$T_b = T \times 3.15 (r/w)^{-0.5}$ (Eq. 4.6.4-3)

Where: T_b=local tractive stress in the bend, PSF
T=the greater of T_o or T_o^1 from equations (Eq. 4.6.4-1) & (Eq. 4.6.4-2)
r=center-line radius of the bend, feet
w=water surface width at upstream end of bend, feet

3. Determine D_{50} size of riprap stone required from:

$D_{50} = T/0.04 (\gamma_5 - \gamma)$ (Eq. 4.6.4-4)

Where: D_{50}=required average size of riprap stone, feet
(size at which 50% of the gradation is finer weight)
T=the greater of T_o or T_o^1 from equations (Eq. 4.6.4-1) & (Eq. 4.6.4-2)
or for a bend in the channel
a=a constant = 0.04
γ_S=saturated surface dry (SSD) specific weight of stone
γ=unit weight of water (62.4pcf)

4. Select minimum riprap thickness required from GRAIN SIZE CURVES, PLATES 1 through 6 (Appendix I). Select from smaller side of band at 50% finer gradation.

5. Select RIPRAP GRADATIONS table (Tables 4.6.4-1 and 4.6.4-2) based upon riprap thickness selected in step 4.

6. Select bedding thickness from the GRAIN SIZE CURVES, PLATES 1 through 6 which was used to select the riprap thickness in step 4. <u>NOTE:</u> the bedding thicknesses included on Plates 1 through 6 are based upon using a properly designed geotextile underneath the bedding. If a geotextile is not used the bedding thickness must be increased to a minimum of 9 inches for 24 inch and 30 inch thickness of riprap and a minimum of 12 inches for the 36 inch thickness of riprap.

7. To provide stability in the riprap layer the riprap gradations should meet the following criteria for GRADATION INDEX:

GRADATION INDEX: $[D_{85}/D_{50} + D_{50}/D_{15}] \leq 5.5$ (Eq. 4.6.4-5)

Where: D85, D50 and D15 are the riprap grain sizes in MM of which 85%, 50% and 15% respectively are finer by weight.
The mid-band gradations of Plates 1 through 6 meet this criteria.

8. To provide stability of the bedding layer the bedding should meet the following filter criteria with respect to the riprap

$D_{15}/d_{85}<5<D_{15}/d_{15}<40$ (Eq. 4.6.4-6)

$D_{50}/d_{50}<40$ (Eq. 4.6.4-7)

Where: D refers to riprap sizes in MM
d refers to bedding sizes in MM
The mid-band gradations of Plates 1 through 6 meet this criteria.

9. The geotextile underneath the bedding should be designed as a filter to the soil.

10. Plates B-41 and B-43 in Appendix I present typical riprap design sections. These figures are from EM1110-2-160 by the U.S. Army Corps of Engineers.

<u>Culvert Outlet Protection:</u>

1. Determine the D_{50} size of riprap required from:

$D_{50}=\sqrt{V}[C\ [2g\ (\gamma_5-\gamma_w/\gamma_w)]]^{1/2}$ (Eq. 4.6.4-8)

Where: D_{50}=Required average size of riprap stone, feet
V=water velocity at culvert outlet, FPS
g=acceleration of gravity, 32.2 feet per sec/sec
γ_5=saturated surface dry (SSD) specific weight of stone
γ_w=unit weight of water, 62.4 pcf
C=a stability coefficient determined by the author to be 1.8
For culvert outlets based upon experience and observation

NOTE: For a SSD specific weight of stone of 160 pcf and C=1.8 equation (8) reduces to:

$D_{50}= (V/18)^{1/2}$ (Eq. 4.6.4-9)

2. Select riprap and bedding from Plates 1 through 6 using D_{50} determined from equation (Eq. 4.6.4-8) or (Eq. 4.6.4-9).

3. Select gradations from gradation tables (Figures 4.6.4-1 and 4.6.4-2).

Examples

<u>Cross Section 73920 (HEC 2)</u>
<u>Subject: Erosion Protection @ Bend</u>

1. $T_o = \gamma RS = 62.4 \text{ X } 25 \text{ X } 0.0015 = 2.34\text{PSF}$
2. $(r/w)^{-0.5} = (545/470)^{-0.5} = 0.93$
3. $T_b = T_o \text{ X } 3.15\ (r/w)^{-.05}$
   $T_b = 2.34 \text{ X } 3.15 \text{ X } 0.93 = 6.86\text{PSF}$
4. $D_{50} = T/0.04(\gamma_S - \gamma_w)$
   $D_{50} = 6.86/0.04\ (165-62.4) = 1.67 \text{ Ft} \approx 20 \text{ inches}$
5. From plate 6: min $D_{50} \approx$ 500MM
   ≈19.7 inches therefore Select 36 inch
   Thickness of Riprap
6. Use 9 inch thickness of bedding with geotextile
7. Select riprap and bedding gradations from tables.

<u>Subject: Erosion Protection – Culvert/Storm Dr. Outlet</u>

1. Pipe size=36 inch Diameter; Min Tailwater
   Outlet velocity=15 FPS; Q=60 CFS
2. $D_{50} = (V/18)^{½}$
   $D_{50} = (15/18)^{½} = 0.91 \text{ FT} \approx 11 \text{ inches}$
3. From plate 4: Min D_{50} = 12 inches
   Therefore select 24 inch thickness of riprap.
4. Use 6 inch thickness of bedding with geotextile.
5. Select riprap apron size from iSWM Figure 4.7-2 (min tailwater conditions):
   a. 3 D_o=3 X 3 Ft = 9 Ft
   b. L_a=20 Ft (from Fig 7.45) MIN
   c. W_d=20 + 9=29 Ft (D_o + 2:1 sides)
      29 > 23 therefore use W_d=29 Ft; Say 30 Ft

Table 4.6.4-1

RIPRAP GRADATIONS 36" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
44 INCH	100
36 INCH	65 - 100
30 INCH	50 - 80
18 INCH	25 - 45
12 INCH	10 - 25
8 INCH	0 - 10

RIPRAP GRADATIONS 24" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
30 INCH	100
24 INCH	65 - 100
18 INCH	45 - 75
12 INCH	25 - 50
8 INCH	10 - 30
6 INCH	0 - 15

RIPRAP GRADATIONS 30" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
36 INCH	100
30 INCH	65 - 100
24 INCH	45 - 75
18 INCH	25 - 50
12 INCH	10 - 25
8 INCH	0 - 10

RIPRAP GRADATIONS 18" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
21 INCH	100
18 INCH	65 - 100
12 INCH	35 - 65
8 INCH	15 - 40
6 INCH	5 - 25
4 INCH	0 - 15

Table 4.6.4-2

RIPRAP GRADATIONS	
12" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
15 INCH	100
12 INCH	70 - 100
8 INCH	45 - 75
6 INCH	30 - 55
3 INCH	10 - 30
1-1/2 INCH	0 - 10

RIPRAP GRADATIONS	
8" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
10 INCH	100
8 INCH	70 - 100
6 INCH	50 - 75
3 INCH	20 - 40
1-1/2 INCH	0 - 15

BEDDING GRADATIONS	
6" THICKNESS OF BEDDING	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
3 INCH	100
1-1/2 INCH	55 - 100
3/4 INCH	25 - 60
3/8 INCH	5 - 30
No.4	0 - 10

BEDDING GRADATIONS	
9" THICKNESS OF BEDDING	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
6 INCH	100
3 INCH	65 - 100
1-1/2 INCH	40 - 60
3/4 INCH	25 - 40
No.4	0 - 12

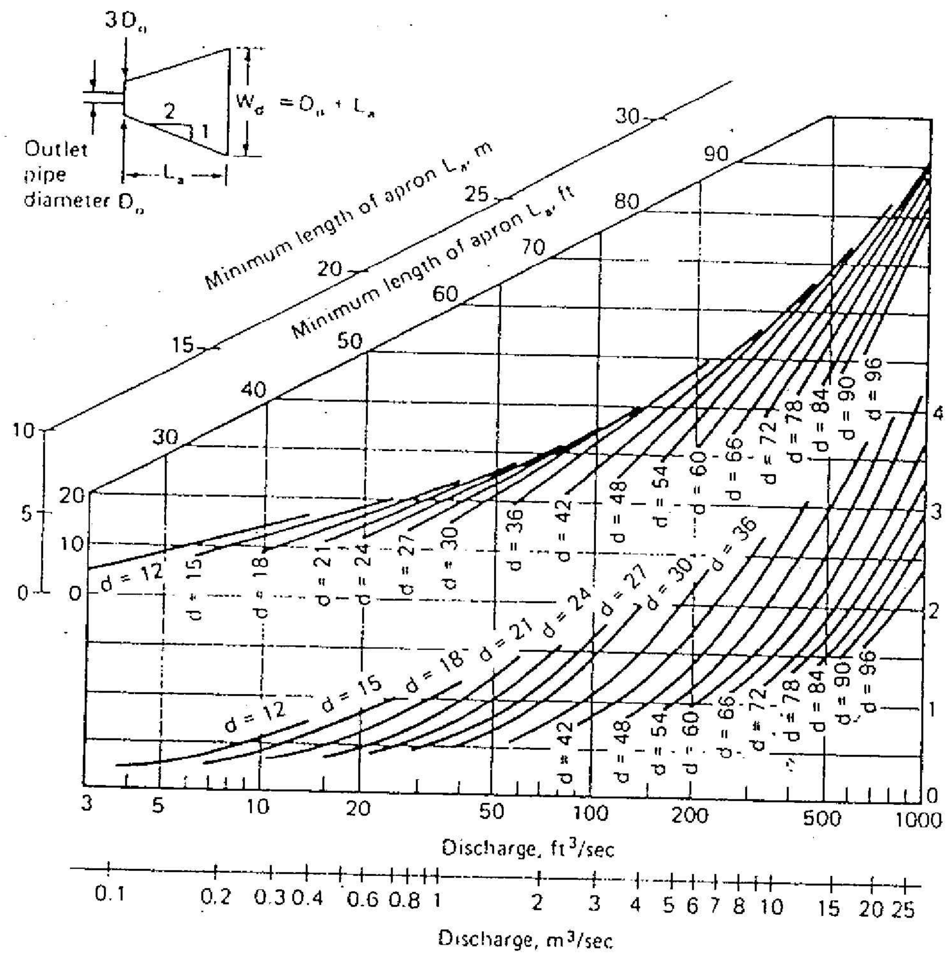


Fig. 7.45 Design of riprap outlet protection from a round pipe flowing full; minimum tailwater conditions. (6, 14)

TAILWATER < 0.5 D_o

Figure 4.6.4-1

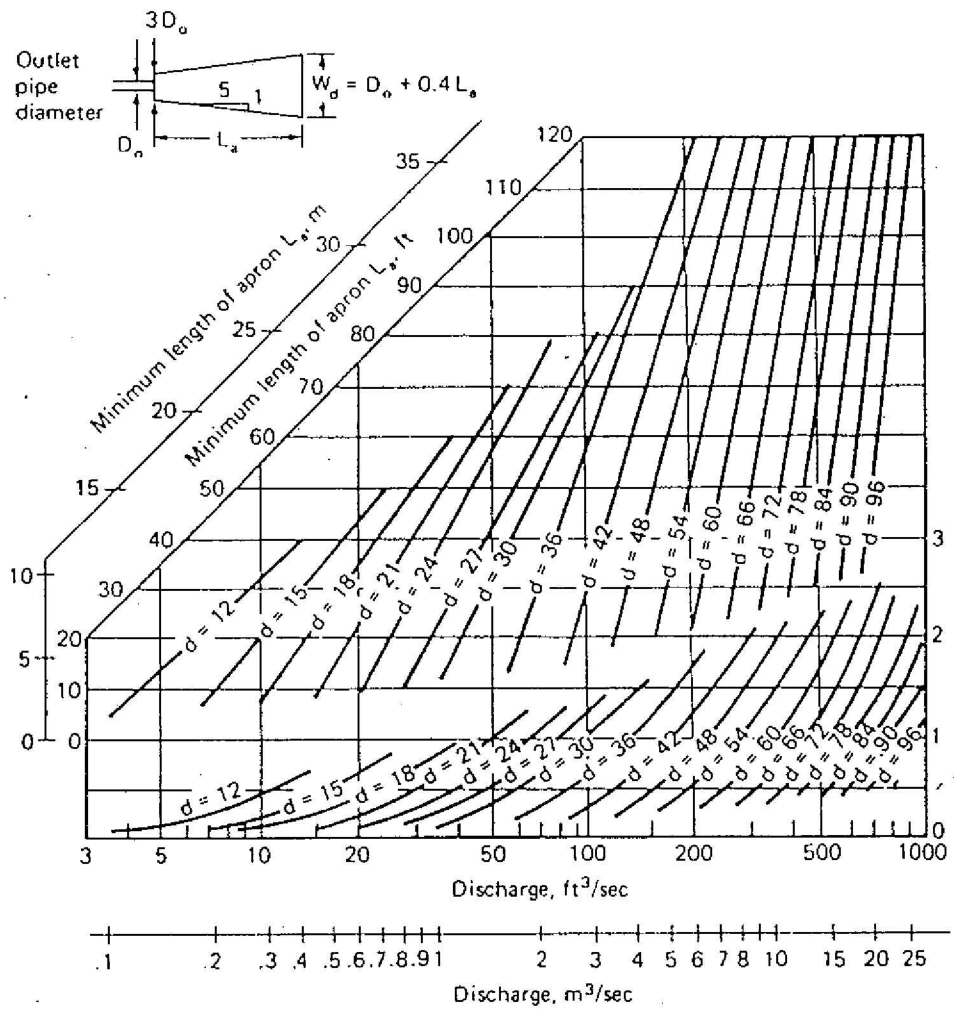


Fig. 7.46 Design of riprap outlet protection from a round pipe flowing full; maximum tailwater conditions. (6, 14)

TAILWATER $\geq 0.5\ D_o$

Figure 4.6.4-2

Grouted Riprap – The Drainage Regulation Entity will allow grouted stone riprap as an erosion control feature. However, the design thickness of the stone lining will not be reduced by the use of grout. See the Corps' design manual ETL 1110-2-334 on design and construction of grouted riprap which should be an available option for certain applications.

Section 4.6.5 – Gabion Design

This section is excerpted from "Gabions for Streambank Erosion Control" EMRR Technical Notes Collection (ERDC TN-EMRRP-SR-22), U.S. Army Engineer Research and Development Center, Vicksburg, MS, 2000.

1. OVERVIEW

Gabions come in three basic forms, the gabion basket, gabion mattress, and sack gabion. All three types consist of wire mesh baskets filled with cobble or small boulder material. The baskets are used to maintain stability and to protect streambanks and beds.

The difference between a gabion basket and a gabion mattress is the thickness and the aerial extent of the basket. A sack gabion is, as the name implies, a mesh sack that is filled with rock material. The benefit of gabions is that they can be filled with rocks that would individually be too small to withstand the erosive forces of the stream. The gabion mattress is shallower (0.5 to 1.5 ft) than the basket and is designed to protect the bed or banks of a stream against erosion.

Gabion baskets are normally much thicker (about 1.5 to 3 ft) and cover a much smaller area. They are used to protect banks where mattresses are not adequate or are used to stabilize slopes, construct drop structures, pipe outlet structures, or nearly any other application where soil must be protected from the erosive forces of water. References to gabions in this article refer generally to both mattresses and baskets.

Gabion baskets can be made from either welded or woven wire mesh. The wire is normally galvanized to reduce corrosion but may be coated with plastic or other material to prevent corrosion and/or damage to the wire mesh containing the rock fill. New materials such as Tensar, a heavy-duty polymer plastic material, have been used in some applications in place of the wire mesh. If the wire baskets break, either through corrosion, vandalism, or damage from debris or bed load, the rock fill in the basket can be lost and the protective value of the method endangered.

Gabions are often used where available rock size is too small to withstand the erosive and tractive forces present at a project site. The available stone size may be too small due to the cost of transporting larger stone from remote sites, or the desire to have a project with a smoother appearance than obtained from riprap or other methods. Gabions also require about one third the thickness of material when compared to riprap designs. Riprap is often preferred, however, due to the low labor requirements for its placement.

The science behind gabions is fairly well established, with numerous manufacturers providing design methodology and guidance for their gabion products. Dr. Stephen T. Maynord of the U.S. Army Engineer Research and Development Center in Vicksburg, Mississippi, has also conducted research to develop design guidance for the installation of gabions. Two general methods are typically used to determine the stability of gabion baskets in stream channels, the critical shear stress calculation and the critical velocity calculation. A software package known as CHANLPRO has been developed by Dr. Maynord (Maynord et al. 1998).

Manufacturers have generated extensive debate regarding the use and durability of welded wire baskets versus woven wire baskets in project design and construction. Project results seem to indicate that performance is satisfactory for both types of mesh.

The rocks contained within the gabions provide substrates for a wide variety of aquatic organisms. Organisms that have adapted to living on and within the rocks have an excellent home, but vegetation may be difficult to establish unless the voids in the rocks contained within the baskets are filled with soil.

If large woody vegetation is allowed to grow in the gabions, there is a risk that the baskets will break when the large woody vegetation is uprooted or as the root and trunk systems grow. Thus, it is normally not acceptable to allow large woody vegetation to grow in the baskets. The possibility of damage must be weighed against the desirability of vegetation on the area protected by gabions and the stability of the large woody vegetation.

If large woody vegetation is kept out of the baskets, grasses and other desirable vegetation types may be established and provide a more aesthetic and ecologically desirable project than gabions alone.

2. DESIGN

Primary design considerations for gabions and mattresses are: 1) foundation stability; 2) sustained velocity and shear-stress thresholds that the gabions must withstand; and 3) toe and flank protection. The base layer of gabions should be placed below the expected maximum scour depth. Alternatively, the toe can be protected with mattresses that will fall into any scoured areas without compromising the stability of the bank or bed protection portion of the project. If bank protection does not extend above the expected water surface elevation for the design flood, measures such as tiebacks to protect against flanking should be installed.

The use of a filter fabric behind or under the gabion baskets to prevent the movement of soil material through the gabion baskets is an extremely important part of the design process. This migration of soil through the baskets can cause undermining of the supporting soil structure and failure of the gabion baskets and mattresses.

3. PRIMARY DESIGN CONSIDERATIONS

The major consideration in the design of gabion structures is the expected velocity at the gabion face. The gabion must be designed to withstand the force of the water in the stream.

Since gabion mattresses are much shallower and more subject to movement than gabion baskets, care should be taken to design the mattresses such that they can withstand the forces applied to them by the water. However, mattresses have been used in application where very high velocities are present and have performed well. But, projects using gabion mattresses should be carefully designed.

The median stone size for a gabion mattress can be determined from the following equation:

$$d_m = S_f C_s C_v d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{0.5} \frac{V}{\sqrt{gdK_1}} \right]^{2.5} \qquad (1) \qquad \text{(Eq. 4.6.5-1)}$$

The variables in the above equation are defined as:

C_s = stability coefficient (use 0.1)
C_v = velocity distribution coefficient
= 1.283-0.2 log *(R/W)* (minimum
of 1.0) and equals 1.25 at end
of dikes and concrete channels d_m = average rock diameter in gabions
d = local flow depth at *V*
9 = acceleration due to gravity $K_,$ = side slope correction factor (Table 4.6.5-1)
R = centerline bend radius of main
channel flow
Sf = safety factor (1.1 minimum) *V* = depth-averaged velocity
W = water surface width of main channel
,¥s = unit weight of stone ,¥w = unit weight of water

Table 4.6.5-1 K1 Versus Side Slope Angle

Side Slope	K1
1V : 1H	0.46
1V : 1.5H	0.71
1V : 2H	0.88
1V : 3H	0.98
<1V : 4H	1.0

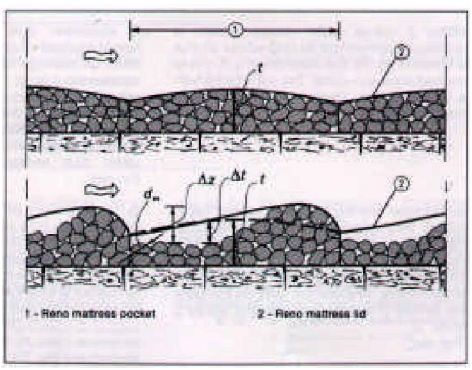


Figure 4.6.5-1. Gabion mattress showing deformation of mattress pockets under high velocities (courtesy Maccaferri Gabions)

This equation was developed to design stone size such that the movement of filler stone in the mattresses is prevented. This eliminates deformation that can occur when stone sizes are not large enough to withstand the forces of the water. The result of mattress deformation (Figure 4.6.5-1) is stress on the basket wire and increases in resistance to flow and the likelihood of basket failure. The upper portion of Figure 4.6.5-1 shows an undeformed gabion, while the lower portion shows how gabions deform under high-velocity conditions.

Maccaferri Gabions offers a table in their materials giving guidance on sizing stone and allowable velocities for gabion baskets and mattresses. This is shown in Table 4.6.5-1.

Table 4.6.5-2 Stone Sizes and Allowable Velocities for Gabions (courtesy of and adapted from Maccaferri Gabions)

Type	Thickness (ft)	Filling Stone Range	D50	Critical* Velocity	Limit** Velocity
Mattress	0.5	3 - 4"	3.4"	11.5	13.8
	0.5	3 - 6"	4.3"	13.8	14.8
	0.75	3 - 4"	3.4"	14.8	16
	0.75	3 - 6"	4.7"	14.8	20
	1	3 - 5"	4"	13.6	18
	1	4 - 6"	5"	16.4	21
Basket	1.5	4 - 8"	6"	19	24.9
	1.5	5 - 10"	7.5"	21	26.2

When the data in Table 4.6.5-2 are compared to Equation 4.6.5-1, if V = 11.5, C_s = 0.1, C_v = 1.0, K_1 = *0.71*, γ_w = 150 and *Sf*= 1.1, the local flow depth must be on the order of 25 ft in order to arrive at the stone diameter of 3.4 in. shown in Table 4.6.5-2. Designers should use Equation 4.6.5-1 to take the depth of flow into account. Table 4.6.5-2 does, however, give some general guidelines for fill sizes and is a quick reference for maximum allowable velocities.

Maccaferri also gives guidance on the stability of gabions in terms of shear stress limits. The following equation gives the shear for the bed of the channel:

$$\tau_b = \gamma_w S d \qquad \text{(Eq. 4.6.5-2)}$$

with the bank shear τ_m taken as 75 percent of the bed shear, i.e. $\tau_m = 0.75\tau_b$. (S is the bed or water surface slope through the reach.) These values are then compared to the critical stress for the bed calculated by the following equation:

$$\tau_c = 0.10(\gamma_s - \gamma_w)d_m$$

(Eq. 4.6.5-3)

with critical shear stress for the banks given as:

$$\tau_s = \tau_c \sqrt{1 - \frac{\sin^2\theta}{0.4304}}$$

(Eq. 4.6.5-4)

where θ = the angle of the bank rotated up from horizontal.

A design is acceptable if $\tau_b < \tau_c$ and $\tau_m < \tau_s$. if either $\tau_b > \tau_c$ or $\tau_m > \tau_s$, then a check must be made to see if they are less than 120 percent of τ_b and τ_s. If the values are less than 120 percent of τ_b and τ_s, the gabions will not be subject to more than what Maccaferri defines as "acceptable" deformation. However, it is recommended that stone size be increased to limit deformation if possible.

Research has indicated that stone in the gabion mattress should be sized such that the largest stone diameter is not more than about two times the diameter of the smallest stone diameter and the mattress should be at least twice the depth of the largest stone size. The size range should, however, vary by about a factor of two to ensure proper packing of the stone material into the gabions. Since the mattresses normally come in discrete sizes, i.e. 0.5, 1.0, and 1.5 ft in depth, normal practice is to size the stone and then select the basket depth that is deep enough to be at least two times the largest stone diameter. The smallest stone should also be sized such that it cannot pass through the wire mesh.

4. Stability of Underlying Bed and Bank Materials.

Another critical consideration is the stability of the gabion foundation. This includes both geotechnical stability and the resistance of the soil under the gabions to the erosive forces of the water moving through the gabions. If there is any question regarding the stability of the foundation, i.e. possibility of rotational failures, slip failures, etc., a qualified geotechnical engineer should be consulted prior to and during the design of the bank/channel protection. Several manufacturers give guidance on how to check for geotechnical failure (see Maccaferri Gabions brochure as an example).

Stacked gabion baskets used for bank stability should be tilted towards the soil they are protecting by a minimum of about 6 deg from vertical. Gabions are stacked using two methods. These are shown in Figure 4.6.5-2. While the gabions can be stacked with no tilt, it is recommended that some tilt into the soil being protected be provided.

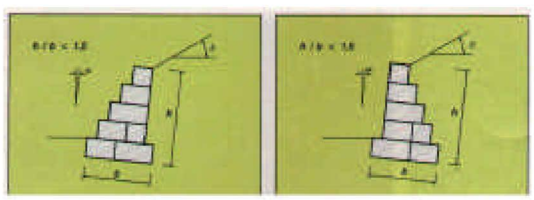


Figure 4.6.5-2 Front step and rear step gabion layout (courtesy of Maccaferri Gabions)

One of the critical factors in determining stability is the velocity of the water that passes through the gabions and reaches the soil behind the gabion. The water velocity under the filter fabric, i.e. water that moves through the gabions and filter fabric, is estimated to be one-fourth to one-half of the velocity at the mattress/filter interface. (Simons, Chen, and Swenson 1984) The velocity at the mattress/filter interface *(Vb)* is estimated to be

$$V_b = \frac{1.486}{n_f}\left(\frac{d_m}{2}\right)^{2/3} S^{1/2} \qquad \text{(Eq. 4.6.5-5)}$$

where n, = 0.02 for filter fabric, 0.022 for gravel filter material and S is the water surface slope (or bed slope) through the reach. If the underlying soil material is not stable, additional filter material must be installed under the gabions to ensure soil stability. Maccaferri also provides guidance on the stability of soil under the gabions in terms of velocity criteria.

The limit for velocity on the soil is different for each type of soil. The limit for cohesive soils is obtained from a chart, while maximum allowable velocities for other soil types are obtained by calculating V_e, the maximum velocity allowable at the soil interface, and comparing it to V'' the residual velocity on the bed, i.e. under the gabion mattress and under the filter fabric.

V_e for loose soils is equal to 16.1 $d^{\prime/2}$ while V, is calculated by:

where *Va* is the average channel velocity and d_m is (Eq. 4.6.5-6)

the average rock diameter.

If V_f is larger than two to four times V_e, a gravel filter is required to further reduce the water velocity at the soil interface under the gabions until V_f is in an acceptable range. To check for the acceptability of the filter use the average gravel size for d_m in Equation 4.6.5-2. If the velocity V_f is still too high, the gravel size should be reduced to obtain an acceptable value for V_f.

Other Design Considerations

It may be possible to combine gabions with less harsh methods of bank protection on the upper bank and still achieve the desired result of a stable channel. Provisions for large woody vegetation and a more aesthetically pleasing project may also be used on the upper banks or within the gabions However, the stability of vegetation or other upper bank protection should be carefully analyzed to ensure stability of the upper bank area. A failure in the upper bank region can adversely affect gabion stability and lead to project failure. sump

$$V_f = \frac{1.486}{n_f}\left(\frac{d_m}{2}\right)^{2/3} S V_a^{1/2}$$

Section 4.6.6 – Energy Dissipation

The following requirements are specific to Wharton County:
Channel Transitions, Energy Dissipation Structures, or Small Dams – A backwater analysis is required, either hand computed or HEC-RAS, to determine accurate tailwater elevation and velocities, headlosses, headwater elevations, velocities and floodplains affected by the proposed transition into and out of 1) An improved channel, 2) Any on-stream energy dissipating structures, and 3) Small dams (less than 6 feet). If the current effective FEMA model for the stream is a HEC-2 model, the engineer has the option to either use that model, or convert to HEC-RAS for analysis of proposed conditions. For larger dams, a hydrologic routing will be required, as well as hydraulic analysis, to determine impacts of the proposed structure on existing floodplains and adjacent properties.

Examples of Open Channel Transition Structures - See drawings in Appendix F and application guidance for Bureau of Reclamation Baffled Chute (Basin IX) and Harris County Flood Control District Straight Drop. The computer program associated with FHWA Hydraulic Engineering Circular No. 14 is “HY8Energy” dated May 2000. This program provides guidance in the selection and sizing of a broad range of energy dissipaters.

CHAPTER 5 – Leveed Areas

Floodplains cover a significant area within Wharton 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 Colorado River or San Bernard 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 Drainage Regulation Entity 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, and the design standards established herein. In the event there is a conflict between FEMA's minimum requirements and the design standards established herein, the more stringent of the two requirements shall apply. The engineer is advised to check the current FEMA rules and regulations. Maintenance of these facilities generally will not be the responsibility of The Drainage Regulation Entity and no right of way on easement will be dedicated to the Drainage Regulation Entity.

Section 5.1 – 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 4.2 Open Channel Design, Section 3.3 Storm Drain Systems and Section 4.4 Storage Design for The Drainage Regulation Entity design criteria.

Section 5.2 – Levee System

Section 5.2.1 – Frequency Criteria

The levee system shall include a levee embankment that will protect the development from the 500-year frequency flood event on the adjacent watercourse. Protection will include the 500-year water surface elevation on the watercourse, plus protection from any associated wind and wave action. The levee crown will include an additional 3 feet of free board above the specific level of protection.

Section 5.2.2 – Design Criteria

The following specific criteria and requirements shall apply to the. design and construction of a levee in Wharton County, Texas:

1. A geotechnical investigation shall be 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 lilts of not more than 12 inches thick.
5. The levee top and slide 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. The minimum top of levee elevation shall be the 500-year water surface elevation on the adjacent watercourse plus l' above FEMA's minimum requirements.
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 flood plain.
10. All pipes and conduits passing through the levee shall have anti-seep collars, flap gates and slope protection.
11. The minimum base width of the levee shall be from toe to toe.

    In addition, the establishment of an additional area for maintenance and access, outside of the levee footprint, shall be required. Access shall be provided with either a minimum 10-foot width 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 area should be established in at least two locations to provide access to the levee right-of-way from a nearby public road.

Section 5.3 – Pump Station

Section 5.3.1 – 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 acreage 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. If the two events are independent of each other, then a coincidental probability relationship exists and the first set of criteria (Section 5.3.1.1) should be utilized. Since high exterior flood stages requiring the pumping of interior drainage can exist independent of rainfall occurring over the leveed area (e.g. high water levels on the Colorado River or San Bernard River versus rainfall in Wharton County), the probability of these two independent severe storm events occurring at the same time is much smaller than their individual probabilities. As a result, the design rainfall used in determining the required pumping capacity can be reduced below the design 100-year" frequency rainfall by an amount related to the frequency that flood stages in the receiving watercourse impede gravity outflow. For a detailed discussion of the development of this criteria, see U.S. Army Corps of Engineers Manual EM 1110-2-1410. If the two events are dependent (i.e. they result from the same storm event), the second set of criteria (Section 5.3.1.2) based on the design I00-year frequency rainfall should be utilized.

Section 5.3.1.1 – Design Criteria Assuming Coincidental Events

This criteria presumes that the storm even 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 Colorado River or San Bernard River in Wharton 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.
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.
4. Use rainfall criteria in Section 2.1.2 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.

Section 5.3.1.2 – 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 even 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 2.1.2-1 for rainfall amounts.)

Section 5.3.2 – Design Criteria

All leveed areas within The Drainage Regulation Entity 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 5.3.1.1 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 5.3.1.1.

2. A minimum of 1-1/2 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 5.3.1.2 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 station in The Drainage Regulation Entity shall be equipped with auxiliary power for emergency usage.

Section 5.4 – Gravity Outlet and Outfall Channel

An outlet shall be required to release the gravity flow 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 4.2 Open Channel Design. The velocities within the outfall channel at the adjacent river shall not exceed 5.0 feet per second.

Section 5.5 – Review Process

When a levee system is required for development, the following information shall be submitted to the Drainage Review Authority 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 calculation showing that the maximum ponding elevation is not exceeded within the leveed area more than once in 100 years on the average.

CHAPTER 6 – Rural Subdivisions Criteria

Section 6.1 – Purpose

The purpose of this chapter is to make available an alternative drainage procedure that can be used in the design of detention facilities for such rural-type subdivision.

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 Wharton County. Individual circumstances may warrant an enhanced drainage and/or detention system.

Section 6.2 – 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 criteria:

A. Lot size of 1 acre or greater;
B. Maximum percent impervious cover based upon lot size (see Figure 6.4-1);
C. Roadside ditch drainage system (v.s. curb and gutter); and
D. No major drainage improvements that would significantly alter the natural drainage patterns in the area for large flood events;

Section 6.3 – Design Criteria

The following design criteria shall be utilized for rural subdivisions:

A. Minimum slab elevations – two (2) feet above natural ground, or one (1) foot above the 100-year floodplains, or one (1) foot above the crown of any down gradient roadway, whichever is higher.
B. Roadways –
   (1) R.O.W. – Seventy (70) feet wide.
   (2) Crown – Maximum of one (1) foot above natural ground.
   (3) Roadside drainage system – Open ditch with 3:1 side slopes; equalizer pipes under roadway at least every 1000 feet (min. 24 in. – diameter RCP) if roadway blocks natural drainage path.
C. Lot drainage – Swales may be constructed along lot lines to provide for minimal drainage of lots.
D. Detention Requirements – See Section 4.4.2 for amount of on-site detention required. Discharge pipe to be maximum 18-inch diameter RCP, or equivalent.

Section 6.4 – Submittals

A. Drainage area map showing existing drainage ways on or adjacent to property.
B. 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.

C. Preliminary (and eventually final) plat with the following plat notes:

(1) Land use within the subdivision is limited to an average imperviousness of no more than 25 percent. (Obtain maximum percent imperviousness from Figure 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.

(2) The minimum slab elevation shall be one foot above the FEMA Effective 100-year flood elevation, or at least 2 feet above natural ground, whichever is higher.

(3) 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 condition. 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.

CHAPTER 7 – References

The following sources were consulted directly or indirectly by reference in the development of these Guidelines:

1. Atlas of Depth – Duration Frequency of Precipitation Annual Maxima for Texas, USGS Report 2004-5041, 2004.

2. Brazoria County Drainage Criteria Manual, November 2003.

3. Code of Ordinances, City of Wharton, Appendix A, Subdivisions, October 2004.

4. Development Standards for the City of El Campo, September 2006.

5. Drainage Criteria Manual, City of Temple, November 1996.

6. Drainage Manual, City of Austin, June 1993.

7. Erosion and Sediment Control Guidelines for Developing Areas in Texas, Soil Conservation Service, US Department of Agriculture.

8. Fort Bend County Drainage Criteria Manual, Revised April 1999.

9. Harris County Flood Control District, Policy Criteria and Manual, October 2004.

10. Hydraulic Design Manual, Texas Department of Transportation, March 2004

11. "Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5," Federal Highway Administration, FHWA-ID-85-15, Washington, D.C., September 1985.

12. Hydrology for Harris County, Seminar sponsored by the American Society of Civil Engineers, March 1988

13. Mitigation Guidelines Regulatory Program, Fort Worth District, US Army Corps of Engineers, December 2003.

14. National Menu of Best Management Practices For Stormwater Phase II, US Environmental Protection Agency, August 2002.

15. "Rainfall Frequency Atlas of the United States, Technical Paper No. 40," U.S. Department of Commerce, Washington, D.C., May 1961.

16. Regulatory Program Overview, Fort Worth District, US Army Corps of Engineers, March 2003.

17. “Urban Drainage Criteria and Design Manual, Hydraulic Engineering Circular No. 22,” Federal Highway Administration, FHWA-NH1-01-021, Washington, D.C., August 2001.

18. “Urban Hydrology for Small Watersheds,” U.S. Soil Conservation Service, Technical Release No. 55, June 1986.

19. Unified Stormwater Design Guidelines, City of College Station, City of Bryan, February 2007.

20. U.S. Army Corps of Engineers Institute of Water Resources, “HEC-HMS Technical Reference Manual”, Hydraulic Engineering Center, April 2006.

21. U.S. Army Corps of Engineers, River Analysis System, HEC-RAS, Version 3.0, January 2001.

APPENDICES

Appendix A
Primary and Secondary Watercourses

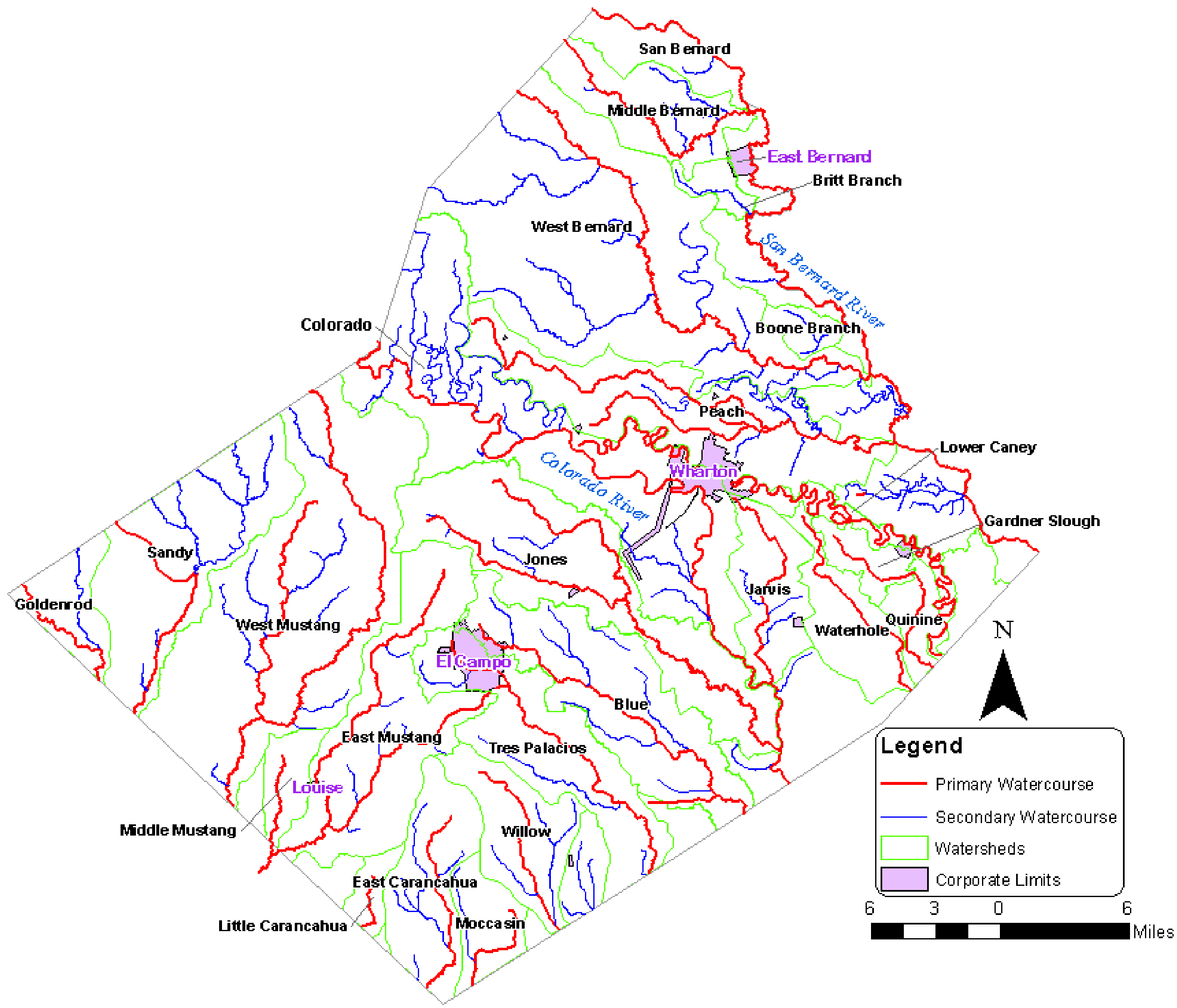


Figure A-1 Primary and Secondary Watercourses in Wharton County

Table A-1
Primary and Secondary Watercourses in Wharton County *

	Watercourse Name	Watershed
Primary Water course	Baughman Slough	Upper Peach
	Blue Creek	Blue
	Colorado River	Colorado
	East Carancahua Creek	East Carancahua
	East Fork Jones Creek	Jones
	East Mustang	East Mustang
	Gardner Slough	Gardner
	Goldenrod Creek	Sandy
	Jarvis Creek	Jarvis
	Jones Creek	Jones
	Little Carancahua Creek	Little Carancahua
	Lower Caney Creek	Lower Caney
	Middle Bernard Creek	Middle Bernard
	Middle Mustang	Middle Mustang
	Moccasin Creek	East Carancahua
	Peach Creek	Upper Peach
	Porters Creek	West Mustang
	Quinine Slough	Gardner
	San Bernard River	San Bernard
	Sandy Creek	Sandy
	Stage Stand Creek	East Mustang
	Tres Palacios Creek	Tres Palacios
	Upper Caney Creek	Upper Caney
	Water Hole Creek	Waterhole
	West Bernard Creek	West Bernard
	West Mustang	West Mustang
	Willow Creek	Willow
Secondary Water course	Blossom Slough	West Bernard
	Blue Creek Trib	Blue
	Blue Creek Tributary	Blue
	Boatwright Branch	Middle Bernard
	Boone Branch	Boone
	Boone Branch Tributary	Boone
	Bosque Slough	Colorado
	Britt Branch	Britt Branch
	Caney Creek	Caney
	Chaco Slough	West Mustang
	Chitland Creek	West Mustang
	Clarks Branch	West Bernard
	Cypress Slough	San Bernard
	Davis Branch	San Bernard
	Dewberry Branch	West Bernard
	Dry Branch	West Bernard
	Dry Creek	Colorado
	Eagle Branch	Middle Bernard
	East Carancahua Trib 1	East Carancahua

Table A-1
Primary and Secondary Watercourses in Wharton County *

Secondary Water Course	East Carancahua Trib 2	East Carancahua
	East Carancahua Trib 3	East Carancahua
	East Carancahua Trib 4	East Carancahua
	East Fork Blue Creek	Blue
	East Fork Jones Ck Trib	Jones
	East Mustang Trib	East Mustang
	East Turkey Creek	Sandy
	Gobbler Creek	West Mustang
	Gumtree Branch	West Bernard
	Gumtree Branch Tributary	West Bernard
	Horseshoe Slough	San Bernard
	Jarvis Creek Trib 1	Jarvis
	Jones Creek Trib 1	Jones
	Jones Creek Trib 2	Jones
	Jones Creek Trib 3	Jones
	Juanita Creek	Juanita
	Lone Tree Creek	West Bernard
	Lookout Creek	West Mustang
	Lost Fork Goldenrod	Sandy
	Middle Bernard Trib 2	Middle Bernard
	Middle Turkey Creek	Sandy
	Mott Branch	West Mustang
	Mud Creek	Blue
	Peach Creek Trib 2	Lower Peach
	Peach Creek Trib 3	Lower Peach
	Peach Creek Trib 4	Lower Peach
	Peach Creek Trib 5	Upper Peach
	Pinoak Creek	Sandy
	Plainview Outflow Ditch	East Mustang
	Porters Creek Trib	West Mustang
	Robb Slough	Colorado
	San Bernard River Trib 3 Unnamed Trib	San Bernard
	San Bernard River Trib 1	San Bernard
	San Bernard River Trib 2	San Bernard
	San Bernard River Trib 3	San Bernard
	San Bernard River Trib 3A	San Bernard
	San Bernard River Trib 3B	San Bernard
	San Bernard River Trib 3C	San Bernard
	San Bernard River Trib 3D	San Bernard
	San Bernard River Trib 4	San Bernard
	San Bernard RiverTrib 3 Unnamed Trib	San Bernard
	San Bernard Tributary 2 Trib	San Beernard
	Sandy Branch	West Bernard
	Stage Stand Creek Trib	East Mustang
	Tres Palacios Trib 1	Tres Palacios
	Tres Palacios Trib 2	Tres Palacios
	Tres Palacios Trib 6	Tres Palacios
	Trib to Peach Creek Trib 4	Lower Peach

Table A-1
Primary and Secondary Watercourses in Wharton County *

Secondary Water Course	Turkey Slough	San Bernard
	Unnamed Canal	Blue
	Unnamed Creek	Tres Palacios
	Unnamed Stream	East Carancahua
	Unnamed Trib	West Mustang
	UNT to Baughman Slough	Upper Peach
	UNT to Colorado River	Colorado
	UNT to East Carancahua	East Carancahua
	UNT to Jarvis Creek	Jarvis
	UNT to Middle Bernard Creek	Middle Bernard
	UNT to Mud Creek	Blue
	UNT to Pinoak Creek	Sandy
	UNT to Sandy Creek	Sandy
	UNT to Tres Palacios	Tres Palacios
	UNT to Willow Creek	Willow
	West Bernard Tributary 1	West Bernard
	West Bernard Tributary 2	West Bernard
	West Bernard Tributary 3	West Bernard
	West Bernard Tributary 4	West Bernard
	West Bernard Tributary 5	West Bernard
	West Bernard Unnamed Tributary	West Bernard
	West Mustang Creek Trib	West Mustang
	West Mustang Trib	West Mustang
	West Turkey Creek	Sandy

* This table lists watercourses included in the 2008 studies by Halff Associates. For other watercourses included in FEMA sponsored studies see the currently effective FIS and FIRMS.

Appendix B
Federal, State, and Regional Regulations and Programs

Table of Contents

B.1 Overview

As a result of the need to address the potential negative of development and storm water runoff, numerous federal, state, and regional programs and regulations have been created to deal with the problems of urban runoff and pinpoint source pollution. A storm water management program is best implemented at a local level, where the local community directly influences land use and development related decisions. Federal and state legislation greatly influences and supports local government in their efforts to manage storm water runoff in their communities.

The purpose of this section is to provide a brief overview of many of the regional, state, and federal laws, regulations, and programs that are required to local governmental entities in Texas that may impact local storm water management programs and activities.

As this is not intended to be detailed analysis of each requirement, it would be advisable to obtain a copy of the specific administrative rules for each program from the appropriate regulatory agency.

B.2 Federal Initiatives

B.2.1 National Flood Insurance Program

Established under the National flood Insurance Act of 1968 and broadened with the passage of Flood Disaster Protection Act of 1973, the National Flood Insurance Program (NFIP) provides federally supported flood insurance to residents in communities that voluntarily adopt and enforce regulations to reduce future flood damage. As part of the program, the Federal Emergency Management Agency (FEMA) defines minimum standards for floodplain development that the local communities must adopt to be eligible for program benefits. New construction and substantial improvements must be built at or above the baase flood elevation, which is the computed elevation of the “100 year flood”. Also, new development that would result in an increase in flood heights is prohibited in the defined floodways. More information on the NFIP and floodplain management in general is available from the FEMA – Region VI office in Denton, TX.

Contact Agency	Phone	Address
Federal Emergency Management Agency (FEMA) Region VI	940-898-5399	800 N. Loop 288 Denton, TX 76209

Website: http://www.fema.gov/regions/vi/index.shtm

B.2.2 U.S. Army Corps of Engineers and Environmental Protection Agency Programs

The U.S. Army Corps of Engineers (USACE) Regulatory Program implements Section 404 of the Clean Water Act and Section 10 of the Rivers and Harbors Act of 1899, through regulations that serve to protect the Nation’s valuable aquatic resources.

Under Section 10 of the Rivers and Harbors Act of 1899, the Corps regulates all work and structures in or affecting the course, condition, or capacity of navigable water of the U.S. Navigable waters of the U.S. are those waters that are subject to the ebb and flow of the tide shoreward to the mean high water mark and/or are presently used, have been used tin the past, or may be susceptible to use in the transport of interstate or foreign commerce.

Contact Agency	Phone	Address
Texas Commission on Environmental Quality	512-239-0400	MC 174 P.O. Box 13087, Austin, TX 78711-3087
US Army Corp of Engineers, Galveston District	409-766-3930	CESWG-PER-R 2000 Fort Point Road PO Box 1229 Galveston, TX 77553-1229

Website: www.tceq.state.tx.us and http://www.swf.usace.army.mil

a. Section 10 Rivers and Harbors Act

Navigable Waters

Section 10 of the Rivers and Harbors Act of 1899 places jurisdiction over certain waters squarely in the hands of the Federal Government. The US Army Corps of Engineers (USACE) operates a regulatory program under the authority of this and subsequent law. This deals with the "navigable waters of the United States". "Navigable waters" are those that are subject to the ebb and flow of the tide shoreward to the mean high water mark and/or are presently used, or have been used in the past or may be susceptible to use, to transport interstate or foreign commerce. The Brazos River and its tributaries (with some limitations) are included in this definition.

Basic Provisions

The Corps of Engineers regulates all work and structures in, or affecting, the course, condition, or capacity of navigable waters of the United States. Example activities and structures include dredging, filling, excavation, bulkheads, revetments, riprap, and pilings. This has obvious application to roadway crossings, on line or adjacent detention facilities, and most types of earthwork along the banks of applicable watercourses.

b. Section 404 Clean Water Act

Waters of The US

Administered jointly by the USACE and the Environmental Protection Agency (EPA), Section 404 has the objective of restoring and maintaining the chemical, physical, and biological integrity of the "waters of the United States". This deals with the surface water tributary system. It includes the smallest streams, any lake, pond, or other water body on those streams, and adjacent wetlands. Under this Act the US Corp Corps of Engineers has certain regulatory powers.

Basic Provisions

The Corps of Engineers' Wetland Delineation Manual provides guidelines for determining whether wetland areas are regulated by Section 404. Placement of dredged or excavated materials into waters of the US is regulated. This includes the addition of material associated with mechanized land clearing, ditching, channelization, sidecasting, temporary stockpiling and other ground-disturbing activities, especially if materials have the effect of replacing water or wetland environments, or changing the bottom elevation of waters of the US.

c. Section 401 Clean Water Act

Point Sources — Dating from 1977, Section 401 established permitting requirements for allowing discharges of effluent into navigable water of the US. The focus was on permitting for construction of plants or facilities that would discharge potentially polluted water, primarily from point sources, as from food processing industrial processes, or waste treatment. Later legislation began applying water quality regulation to stormwater runoff.

d. Section 402 Clean Water Act

Stormwater Quality — In1987 the US Congress amended Section 402 of the Clean Water Act regarding administration of the National Pollutant Discharge Elimination System (NPDES). As to the quality of stormwater discharge, a comprehensive two-phased permitted framework was initiated for dealing with “municipal separate storm sewer systems”. “Separate is important because it differentiates between systems that collect and discharge only storm runoff from those that may include effluents from such sources are sewage treatment or industrial processes. Fundamentally, it requires municipalities to initiate comprehensive program for minimizing pollutant loads discharged into streams and waterways.

Phases I & II — Phase I regulates large and medium “municipal separate storm sewer systems” (MSSS or MS4). Municipalities having a population in excess of 100,000 are known as “Phase I MS4s” These have been required to implement some system of practices designed to improve the quality of stormwater discharges. Under Phase II rules issued by the EPA in 1999, smaller MS4s must also be in compliance with NPDES requirements. Smaller MS4s are defined as municipalities having less than a population of 100,000 and located in “urbanized areas” as defined by the US Census. These are known as “Phase II MS4s”. No areas of Wharton County currently fall within this category.

BMPs & Six Measures — The EPA has published a menu of “Best Management Practices (BMPs) considered suitable for stormwater system management to use in achieving water quality goas. A brief description of several BMPs is included latter appendix . In addition, EPA has established six classes of control measures that smaller MS4s must address in local programs to improve the quality of storm runoff. Listed below, these control measures must be in place or in process in order for smaller MS4s to obtain the required permit when that becomes necessary.

1. Public education and outreach
2. Public participation and involvement
3. Illicit discharge detection and elimination
4. Construction site runoff control
5. Pos-construction runoff control
6. Pollution prevention/good housekeeping

B.3 State of Texas Initiatives

In 1998 administration of eh National Pollutant Discharge Elimination System (NPDES) was partially delegated by the Environmental Protection Agency, via a memorandum of understanding, to the State of Texas. However, the EPA retains its enforcement authority.

a. Texas Pollutant Discharge Elimination System (TPDES) MS4 Stormwater Permit Program

Texas Waters — The Texas Commission on Environmental Quality (TCEQ) is the State agency responsible for the quality of "Waters of the State", including stormwater quality. Since 1998 stormwater quality has been regulated pursuant to the Texas Pollutant Discharge Elimination Program administered by TCEQ. Prior to that, individual permits were issued to larger MS4s by the EPA, but since 2002 the TCEQ has issued renwal permits and addressed various issues for those MS4s. The TCEQ has responsibility for administering Phase II permitting. This will include designating small MS4s, developing a template general permit, providing suitable BMPs for use by municipal entities, and administering the permitting process.

Requirements — Under Phase II requirements, small MS4s are required to reduce the discharge of pollutants to the maximum extent practicable. MS4s are to accomplish this by developing and implementing a Stormwater Management Program (SWMP) for an individual permit from the TCEQ. Several example BMPs, are listed at the end of this Appendix.

b. Requirements Pending

Although Phase II requirements for small MS4s have been established by the EPA and TCEQ none of Wharton County has a large enough population or is in a "urbanized " area to fall within these requirements.

Until specific requirements are advanced through appropriate rule-making procedures, the drainage regulation entity land development projects that propose to use available Best Management Practices for improving water quality in the design of stormwater facilities, notwithstanding the limitations otherwise stated in these Guidelines.

B.3.1 TPDES Storm Water Permit for Construction Activities

The TPDES storm water permit for construction activities is directed toward controlling the quality of storm water runoff from construction activities. The permit requires the development of a construction storm water Pollution Prevention Plan (SWPPP) that emphasizes the application of BMPs to protect storm water quality from erosion and sedimentation processes, as well as construction material and wastes during the construction phase of development.

Operators of construction sites five acres or greater are required to obtain storm water permits from the TCEQ by developing a SWPP and filing a Notice of intent (NOI) 48 hours prior to initiating construction activities.

Construction sites one to five acres in size require a SWPPP to be developed, but an NOI is not required to be submitted to the TCEQ. Construction sites that are part of a larger common plan of development, such as a subdivision, that is collectively greater than one or five acres, must be evaluated according to the size of the larger common plan of development when considering permit requirements. For example, a person constructing on a ¼ -acre site located within a 10-acre subdivision under construction would be required to comply with the permit requirements for sites disturbing 5 acres or more.

If storm water runoff from the construction site enters a municipal storm sewer system, the construction site operator and /or owner is also required to notify the MS4 operator about the construction activity. For large construction sites, submitting a copy of the NOI to the MS4 operator is required, and small construction sites, a copy of the Construction Site Notice is required. Local ordinances should be submitted to the MS4 operator.

Construction site operators on sites with an NOI are further required to submit a Notice Termination (NOT) to TCEQ when final stabilization has been achieved on all portions of the site under their control. Refer to TPDES. General Permit TXR15000 for specific permit requirements.

B.3.2 Industrial TPDES Storm Water Permit Program

The TPDES program also requires that the discharge of storm water from certain types of industrial facilities be regulated under a permit program. Industrial storm water is defined as that discharged from any conveyance which is used for collection and conveying storm water and which is directly related to manufacturing, processing, or materials storage areas. Discharge of storm water from regulated industrial facilities in Texas is managed by TCEQ under a single general permit issued in 2001.

Currently, the following thirty categories of industrial facilities identified in the Multi-Sector General Permit are required to have a TPDES permit for their storm water discharge:

Timber Products

Paper and Allied Products

Chemical and Allied Products

Asphalt Paving and Roofing Material and Lubricants

Glass, Clay, Cement, Concrete, and Gypsum Products

Primary Metals

Metal Mining (Ore Mining and Dressing)

Coal Mines and Coal Mining Related Facilities

Oil and Gas Extraction

Mineral Mining and Dressing

Hazardous Waste Storage Facilities

Landfills and Land Application Sites

Automobile Salvage Yards

Scrap Recycling Facilities

Steam Electric Generating Facilities

Land Transportation and Warehousing

Water Transportation

Ship and Boat Building or Repairing Yards

Air Transportation

Treatment Works

Food and Kindred Products

Textile Mills, Apparel, and Other Fabric Product Manufacturing, Leather and Leather Products.

Furniture & Fixtures

Printing and Publishing

Rubber, Miscellaneous Plastic Products, and Miscellaneous Manufacturing Industries

Leather Tanning and Finishing

Fabricated Metal Products

Transportation Equipment, Industrial or Commercial Machinery

Electric, Electrical, Photographic, and Optical Goods

Miscellaneous Industrial Activities

Regulated Industrial facilities are required to develop a storm water pollution prevention plan (SWPPP) and submit a Notice of Intent (NOI) for permit coverage to the TCEQ.

Components of the SWPPP include identification and elimination of potential sources of storm water contamination, storm water monitoring at each storm water outfall, employee training, and storm water protection activities. New industrial facilities are required to submit an NOI 48 hours prior to conducting any new activity.

B.3.3 Dams and Reservoirs in Texas (See Appendix C)

B.4 Wharton County Flood Mitigation Plan

Wharton County has experienced major flooding from hurricanes and coastal storms in addition to flooding from both the Colorado and San Bernard Rivers. In 2005, the US Army Corps of Engineers, in cooperation with the Lower Colorado River Authority, published the Lower Colorado River Flood Damage Reduction Study which includes proposed activities to provide flood damage reduction and ecosystem improvement in the lower basin that includes Wharton County. In April 2006, FEMA published new Flood Insurance Rate Maps (FIRM's) for Wharton County as a result of the National Map Modernization Project. In 2006, the Texas Water Development Board approved a Flood Mitigation Assistance (FMA) Program Grant to Wharton County to prepare a Flood Mitigation Plan.

In March 2006, Wharton County selected Halff Associates, Inc. to prepare a county-wide Flood Mitigation Plan for Wharton County and all communities within the county for submittal to the Texas Water Development Board (TWDB) and the Federal Emergency Management Agency (FEMA). Halff Associates, Inc. was the planning consultant to assist in preparation of the Flood Mitigation Plan to meet the planning requirements mandated by FEMA's Flood Mitigation Assistance (FMA) Program and to satisfy the planning requirements for FEMA's Community Rating System (CRS) Program.

Wharton County is vulnerable to several types of natural and technological hazards which were identified in the Texas Colorado River Floodplain Coalition (TCRFC) all-hazards Regional mitigation plan which has been approved by both TxDEM and FEMA.

Wharton County is located within 30 miles of the Texas Gulf Coast and vulnerable to hurricanes, coastal storms, and flooding from both the Colorado and San Bernard Rivers. Through proper identification of hazards faced and assessment of the capability of Wharton County and participating communities to respond to those hazards, Wharton County planned to improve the overall disaster preparedness within the county.

Mitigation is defined as sustained actions taken to reduce or eliminate long-term risk to people and property from hazards and their effects. The purpose of mitigation activities is two-fold. The first goal involves protecting lives and property. The second goal seeks to minimize the costs associated with disaster response and recovery. In this manner, Wharton County and participating communities recognized the importance of mitigation efforts when they allocated the local match necessary for flood mitigation planning efforts.

A flood mitigation plan is more than just another planning document. The flood mitigation plan is a dynamic record of the community's recognition of vulnerability to flood hazard, determination of the risks associated with hazard effects, and commitment to reducing the long-term consequence of flood hazards. The flood mitigation plan outlines flood mitigation goals, identifies a risk reduction strategy for flood hazards that threaten the area, and discusses the on going risk reduction activities undertaken within the jurisdiction. The Wharton County Flood Mitigation Plan is designed to meet the planning requirements associated with participation in FEMA's Community Rating System (CRS) Program and to satisfy the requirements of the Flood Mitigation Assistance Program administered by the Texas Water Development Board. In addition, the completed flood mitigation plan will become a portion of the Wharton County all-hazard mitigation plan and Emergency Management planning efforts.

In 2006 the Wharton County Commissioners Court approved procedures for preparation of a flood mitigation plan for Wharton County. At that time, a Flood Mitigation Planning Committee was established that included: citizens, Wharton County and participating communities' staff, and consultants.

A flood mitigation plan questionnaire was distributed to all residents of Wharton County, via utility bill inserts, public meetings, and the local newspaper. The efforts of the questionnaire sought to increase public involvement regarding flood mitigation issues. Two public meetings were conducted to discuss the flood mitigation plan.

During the planning process, copies of the draft plan were submitted to outside organizations and "Other Agencies" for comment. The organizations contacted included: Lower Colorado River Authority, Texas Colorado River Floodplain Coalition, Wharton County Emergency Management Office, Texas Division of Emergency Management, Federal Emergency Management Agency, Texas Commission on Environmental Quality, Insurance Standards Office, Houston – Galveston Area Council, US Army Corps of Engineers, Natural resources Conservation Service, Texas Department of Transportation, and the Texas Water Development Board. Comments received from the "Other Agencies" were incorporated into the plan.

In developing the Flood Mitigation Plan, the committee identified numerous hazards to which community is subject. Each hazard was briefly discussed in the planning document. Of each of the hazards identified, the most common hazard events to affect the area were determined to be flooding events. To clarify the extent of flooding problems within the community, the plan identifies the number and types of buildings in the floodplain, the number of flood insurance policies within each participating community, and the number of flood losses within Wharton County. The procedures for warning and evacuation during flood events are included in the plan. Critical facilities located within the county and their proximity to the floodplain is also discussed. Finally, specific flood protection projects already completed within Wharton County are recognized.

After assessing the hazards and reviewing the alternatives, the Flood Mitigation Planning Committee established the flood mitigation goals for Wharton County. Current mitigation activities and other activities completed on an annual basis were reviewed. Documentation of each of the activities was included in the plan in order to receive CRS planning credits. Following identification of the goals and activities, the committee identified specific action items to be undertaken or continued as part of the flood mitigation planning effort. The action items are identified as follows:

- <u>Study and map unstudied streams in Wharton County</u>
- <u>Drainage System Maintenance and creation of the Wharton County Drainage District</u>
- <u>Increase Insurance Awareness by increasing flood insurance coverage</u>
- Annual review and update of the TCRFC (All Hazards) Mitigation Action Plan
- <u>Elevation, Relocation and Acquisition, "Demo-Rebuild" and small Flood Protection Projects to mitigate floodprone properties</u>
- <u>Obtain and Annually Improve Community Rating System (CRS) Ratings</u>
- Create a County-Wide Elevation Reference Mark Database
- Adopt "Higher Standard" Codes and Ordinances
- <u>Support USACE and LCRA structural and nonstructural Flood Protection Projects</u>

The final requirement of the Flood Mitigation Plan involved formal adoption and implementation of the plan by Wharton County Commissioners Court and City Councils of East Bernard, El Campo and Wharton.. The adopted plan was submitted to FEMA to fulfill the Community Rating System (CRS) planning requirements and to the Texas Water Development Board for approval as a Flood Mitigation Assistance (FMA) Plan. Wharton County and participating communities implemented the suggested actions as identified in the plan. The Plan includes a schedule for implementation, annual reviews and a Plan update every five years.

B.5 Best Management Practices

This section is provided in order to facilitate and foster design solutions that will help improve water quality. The effectiveness of the techniques outlined herein is very dependent on proper application and implementation, and is in no way assured. Likewise their use does not assure achieving public safety objectives, and can work against those objectives if improperly conceived or deployed.

Special designs may propose using any of the examples outlined herein or other techniques that may have been implemented in other jurisdictions. It is highly recommended that any special design concepts be carefully coordinated with the City Engineer or his/her designee as early as possible in design processes. It shall be the designers' responsibility to substantiate that the special design does not compromise public safety objectives or aggravate long term maintenance requirements. It should be emphasized that the information provided in this portion of Appendix B is for guidance and is not specific requirements.

In their publication "National Menu of Best Management Practices For Storm Water Phase II", the US Environmental Protection Agency (EPA) has advanced a number of concepts for managing urban stormwater runoff in a manner that will enhance water quality. The techniques are intended to provide guidance to regulated small MS4s. This Appendix provides a brief introduction to several of those techniques. They are offered only as examples. There is no requirement to use them, nor are they specifically recommended over other potential design solutions. Likewise, designers should not limit their thinking to only these examples.

All of the techniques offered by the EPA have been used at various locations and have been scientifically evaluated for their general effectiveness. The specific chemical or physical effectiveness of the techniques is beyond the scope of these Guidelines, as are their advantages and disadvantage in terms of initial cost, comparative costs, or maintenance ramifications. Nevertheless, these later issues must be addressed in technical reports substantiating special design proposals. The designers' attention is directed to the aforementioned publication for the information necessary to implement these and other techniques.

Retention / Irrigation Basins

Retention refers to the idea of capturing stormwater and retaining it, as opposed to simply collecting it and metering its release at some pre-determined flow rate. As suggested by the title, the concept of this technique is to collect runoff into a holding pond and then draw from it to irrigate landscaped areas. The intent is to replicate natural situations where the majority of rainfall is infiltrated into the soil or underlying groundwater, and pollutants are captured by soils. In addition, particles settle while the water is pooled.

Extended Detention Basins

A traditional detention facility captures storm flow and releases it at a pre-determined rate, one associated with pre-development conditions, with no particular consideration for water quality objectives. An "extended detention basin" functions in a similar way but is designed to release the collected water at a much slower rate, one that causes the water to remain pooled much longer, usually on the order of 24 hours. This allows time for suspended solids to settle, and can derive other water quality benefits. Such a facility should serve no more than 100 acres, and generally requires a slower release rate and a larger storage volume than a traditional detention facility.

Grassy Swales

A grassy swale is a specially designed channel. With very flat side slopes (4:1 or flatter), it is wider than it is deep. The flow line slope should be between one percent and five percent, and the surfaces must be covered with vegetation, generally close-growing, water-resistant grasses.

The idea is simple: as runoff flows over and through the grass at a shallow depth and slow rate, particles tend to settle and biological uptake of pollutants tends to occur.

Vegetative Filter Strips

As suggested by the name, this technique involves long strips of vegetated area placed so that runoff will traverse their length in route to lower areas. The idea is to bring runoff to the strips in broad sheet flow or in uniform shallow overland flow, not in a concentrated manner. As stormwater moves through the strip(s) in very shallow flow at a slow rate, the vegetation tends to cause particles to settle and biological filtration of pollutants.

Sand Filter Systems

These systems can vary widely in their design but in any case require carefully specified and constructed components in order to be effective. Generally, two chambers are required, one for sedimentation and another for filtration. Runoff first enters the sedimentation chamber where larger solids are collected. Next it seeps through the sand bed in the filtration chamber. There, a specially designed sand bed composed of sand, gravel, and filter fabric in just the right combinations and having just the right physical characteristics, captures a range of other pollutants. Water is finally released through perforated collection pipe(s) situated beneath the sand bed system.

A “full sedimentation” system includes a wall with a riser pipe between the two chambers. This type requires the first chamber to be sized for the entire design capture volume. A “partial sedimentation” system includes a porous separation between the two chambers so larger solids may not pass into the filtration chamber. In this type, the two chambers together are sized for the entire design capture volume.

Wet Basins

In simplest terms a wet basin is designed to retain a pool of water year-round. Whereas a traditional detention facility has an outlet near its bottom, a wet basin has an outlet located near its top. With no lower outlet, the facility must fill to the level of the top outlet before any water is released, and it does not drain. In addition, a wet basin typically has a standing crop of water-tolerant vegetation along its usual waterline.

A wet basin should have two components: a sediment forebay and a main pool. Runoff first moves through the forebay where gross solids are captured. It then fills the main pool basin until overflowing through an outlet spillway. Properly sized, such a basin will capture the desired volume of water before allowing discharge. In this way it acts as a stilling basin allowing solids to settle. One objective is for the aquatic environment to eliminate pollutants through wetland plant uptake and microbial degradation. In dry climates supplemental water sources may be necessary in order to maintain a pool level supportive of the aquatic environment.

Constructed Wetlands

The concept of a constructed wetland is to gain the pollutant removal characteristics of a natural wetland environment. Among these are settling of solids, wetland plant uptake, and microbial degradation. Extremely wide variations in design are possible. The facility is similar to a wet basin because it must be wet year-round, but it is shallow and marsh-like, creating conditions supporting abundant vegetation and microbial population. Micro-pools, small islands for waterfowl habitat, and multiple species of trees, shrubs, and plants are among the design elements that must be balanced for the facility to be successful.

A constructed wetland has four principal components: a splitter box, a sedimentation forebay, the wetland zone (“pond”), and the outlet structure. The splitter box diverts flow from the main flow path to the entrance, keeping away anything more than the design flow. From the splitter box, runoff moves into the forebay where gross solids are captured before flowing into the wetland zone. In the wetland zone, runoff moves through multiple irregular flow paths and micro-pool areas filling the wetland “pond” to no more than two feet above its usual water surface elevation. The outlet structure must allow the water level to gradually decrease to its normal elevation. If storm flow rushes through the facility or keeps it inundated too long, the aquatic echosystem can be damaged. In dry climates supplemental water sources may be necessary in order to maintain a water level supportive of the aquatic environment.

Appendix C
Dams and Reservoirs
In Texas

Dams and Reservoirs in Texas

The Texas Commission on Environmental Quality (TCEQ) regulates the construction of dams in Texas that are six feet or more in heights as per the Texas Administrative Code (Title 30, Part 1, Chapter 299-Dams and Reservoirs). Approval from the TCEQ of plans and specifications is required for construction of a dam. The TCEQ also has the authority to inspect existing dams, and if necessary, require unsafe dams to be upgraded or removed. The Dam Safety Program is administered under the Field Operations Division of the Office of Compliance and Enforcement. Forms, Guidelines, Rules, Regulations, and many other resources can be found on-line as www.tnrcc.state.tx.us/enforcement/dam_safety/intro2.html.

Structures constructed for the purpose of the impounding water either on a temporary or permanent basis, which are over six feet in height are regulated by the State. The Texas commission on Environmental Quality (TCEQ) is the regulatory agency responsible for administration of the State dam safety laws in Texas. Dams are classified according to size and the potential for loss of human life and/or properly damage within the are downstream of the dam. The State regulates the design, construction, operation, and maintenance of dams, and this chapter provides an overview of some of the pertinent criteria.

Proposed Rule Changes

The following changes have been proposed . These changes include only what would be different from what is in the current rules. As of November 2008 changes new ruels are scheduled to be adopted in early 2009.

- Current rules would be repealed with new rules. There is a need for better clarity and definition.

- Definitions

    - Dam would be defined as: having a height greater that or equal to 25 feet and a maximum storage (top of dam) capacity greater that 15 acre-feet; having a height greater than 6 feet and a maximum storage capacity greater than or equal to 50 acre-feet; or posting a threat to human life or property in the event of failure, regardless of height or maximum storage capacity.
    - Deficient dam. A dam that fails to meet the requirement of Chapter 299 and poses a threat to human life and property.
    - Capacity. Only water that can be stored above natural ground level would be considered in assessing the storage volume.
    - Owner. Any person who:
        - Holds legal possession or ownership of an interest in a dam;
        - Is the fee simple owner:
        - Has a contractual right to construct, operate, or maintain a dam; or
        - Has a lease or easement to construct, operate, or maintain a dam.

- Emergency repair would be defines as any repair, considered to be temporary in nature, and necessary to preserve the integrity of the dam and prevent a possible failure of a dam.

- As part of an evaluation to determine if a dam is a threat, the executive director may require the owner to obtain the services of an independent team of professional engineers or other dam experts (possibly only for large dams).

- The rules would specify the materials to be submitted and the review to be performed by the executive director when an owner submits an application for a water rights permit involving a dam.

- A requirement is proposed to be added that a new owner of a dam notify the executive director with the contact after purchasing the property with a dam.

- A requirement would be added for an inventory of dams.

- Classification of dams. Language would be added that the executive director may reclassify the hazard classification based on an inspection and downstream hazard evaluation by the executive director, a report of an inspection and downstream evaluation by the owner's professional engineer, or a breach analysis.

- The size classification would be changed from the current rules only because of the changes in the definition of dam.

- The hazard classification would be based on existing conditions at the time of the evaluation.

- The hydrologic and hydraulic criteria for a proposed dam would be the full criteria, but would allow overtopping if designed for overtopping.

- The following would be proposed for the hydrologic and hydraulic criteria for an existing dam:

    - A dam would be considered adequate if the dam and spillways pass 75: of the PMF and the owner has an emergency action plan, an operation and maintenance program and an inspection program, and submits an annual report documenting the programs.
    - If the hazard classification for an existing dam changes due to increased development downstream, the executive director may require either submission of plans for upgrading the dam; an analysis to request a reduction in the minimum hydrologic criteria (i.e., a breach analysis or a risk assessment); or an alternative to upgrading (i.e., remove the dam, lower the lake level, or meet the requirements in the bullet above).

- A requirement would be added for freeboard for proposed dams, possibly only for large dams.

- A requirement would be added for stability analyses for proposed large and intermediate sized dams and large and intermediate size dams proposed to be modified or rehabilitated. Factors of safety would be included.

- Requirements would be included for a professional engineer to determine if the safety of the dam would be compromised for the following:

- Any person proposing to dredge the reservoir within 200 feet of a dam;
  - A company proposing to install a utility line in a dam or in spillways;
  - A drilling company proposing to drill wells within 200 feet of a dam or spillways;
  - A company proposing to install a utility line in a dam or in spillways;
  - A drilling company proposing to drill wells within 200 feet of a dam or spillways;
  - A company proposing to blast within ½ mile of a dam.

- The subchapter on construction requirements (submittal of plans and specifications, inspections during construction, reports and records) would apply to dams:

  - Requiring a TCEQ authorization (would be defined);
  - Used for detention purpose and impounding a maximum storage capacity of 200 acre-feet or more;
  - Originally designed and constructed as NRCS-assisted project dams, but being proposed to be modified or rehabilitated without the assistance and approval of the NRCS;
  - That are small and classified as either high or significant hazard and exempt from a water rights permit under Texas Water Code §11.142.
  -
- The subchapter on construction requirements would not apply to dams:

  - That the owner has received an approval for an exception of the rules;
  - Proposed to be designed and constructed, or an existing dam proposed to be constructed for mining purposes and approved and inspected by the Mine Safety and Health Administration;
  - Small, low hazard dams exempt from a water rights permit under Texas Water Code §11.142

- Language will be added that would require the executive director not to approve construction plans and specifications unless a Stormwater Pollution Prevention Plan has been developed and implemented.

- As part of a review of construction plans and specifications for large proposed dams and large existing dams, a report on proposed instrumentation may be required.

- Plans that may be required during a review by the executive director include:

  - A quality control and assurance plan for proposed dams;
  - A plan for closure of a proposed dam:
  - A plan for addressing emergencies during construction.

- The review and approval process will be provided in the rules, giving the process and time frames for review.
- A time limit of 4 years will be proposed to be added for plans that have been approved by the executive director. If construction has not been started within four years of approval, the plans may have to be resubmitted for approval.

- Language would be added to require maintenance of construction records in secure location.

- Language would be added t describe the process for approval of a construction change order.

- Language will be added to include a time frame for submission of the engineer's notification of completion (30 days after completion).

- Language will be added to include a time frame for submission of the record drawings (6 months after completion). If no changes were made during construction, a signed, sealed, and dated letter from the engineer indicating that no changes would be accepted.

- A requirement is proposed to be added for a gate operation plan to be submitted for any proposed dam that will have a gated spillway.

- A requirement is proposed to be added for an operation and maintenance plan for any proposed dam to be submitted before completion of construction.

- Language is proposed to be added that owner shall provide the date when the dam will be turned over to a property owner association or other designated group.

- A new subchapter is proposed to be added to cover operation and maintenance of dams. This subchapter will include:

    - Owner responsibilities. The owner is responsible for operating and maintaining the dam in a safe manner, and the owner is responsible for addressing all maintenance and safety concerns identified during any inspection.

- Inspections.

    - The executive director may enter anyone's property for the purpose of inspecting the dam.
    - A frequency of engineering inspections will be proposed for a high and significant hazard dams and large, low hazard dams (5 years).
    - Small and intermediate size, low hazard dams will not be scheduled for inspection unless requested or there is a need to assess hazard classification.
    - The information to be developed during the inspection and the process for completing the report will be proposed to be added.
    - The requirement will be proposed to be added. A requirement will be proposed that the owner's engineer's report may be used in lieu of TCEQ making an inspection.

- Operation and Maintenance. The types of maintenance activities will be identified.

- Gate Operation Plan. Within 12 months of the effective date of the rules, the owner of dams with gated spillways shall notify the TCEQ that either a plan has been completed or that one already exists. The plan will be considered an appendix of the emergency action plan. If the plan is submitted to the TCEQ, it will be stored in the agency's confidential permanent records.

- Emergency Repairs. The requirements for emergency repair are proposed to be added.

- Records. Language would be added to require maintenance of operation and maintenance records in a secure location.

- The requirements for removal of dams are proposed to be added in a subchapter.

- A requirement will be proposed to be added for owners of all high and significant hazard dams and all large, low hazard dams to prepare an emergency action plan and submit the plan to the TCEQ. The type of information to be included in the plan will be identified.

- A requirement will be added to be added to address security for all critical infrastructure dams (only 66 identified at this time) and backup power to ensure operation of the dam. The plan will be filed in the agency's confidential, permanent files.

- A new subchapter is proposed to be added to cover enforcement procedures and emergency orders.

A revised version of the rules has scheduled to be adopted in the near future. The changes from the new rules are summarized in the next section.

Appendix D
Storm Water Computer Models

The Drainage Regulation Entity accepts appropriately versions of the following computer models.

1. HY8Energy (FHWA Hydraulic Engineering Circular No. 14) for selection and analysis of energy dissipation structure.

2. CHANLPRO by the US Army Corps of Engineers for analysis and design of streambank protection measures.

3. Culvert by TxDOT for hydraulic analysis and design of culverts.

4. HY8 by FHWA for hydraulic analysis and design of culverts. (This program is similar to CULVERT).

5. Winstorm by TxDOT for analysis and design of storm sewer.

6. Gabion Design Programs by Maccaferri:

    a. Macra 1 for Channel Design
    b. GawacWIN for Retaining Wall Design

7. HEC-1, HEC-2, HEC-HMS and HEC-RAS by the U.S. Army Corps of Engineers for hydrologic and hydraulic analysis and design..

8. TR-20 by NRCS for hydrologic analysis and design…

9. SWMM (and privately enhanced versions) by EPA for dynamically routed hydrology and hydraulics.

10. InfoWorks by Wallingford for dynamically routed hydrology and hydraulics.

Appendix E
Nomographs and Checklists

DESIGN CHARTS FOR CULVERTS

CHART 1B

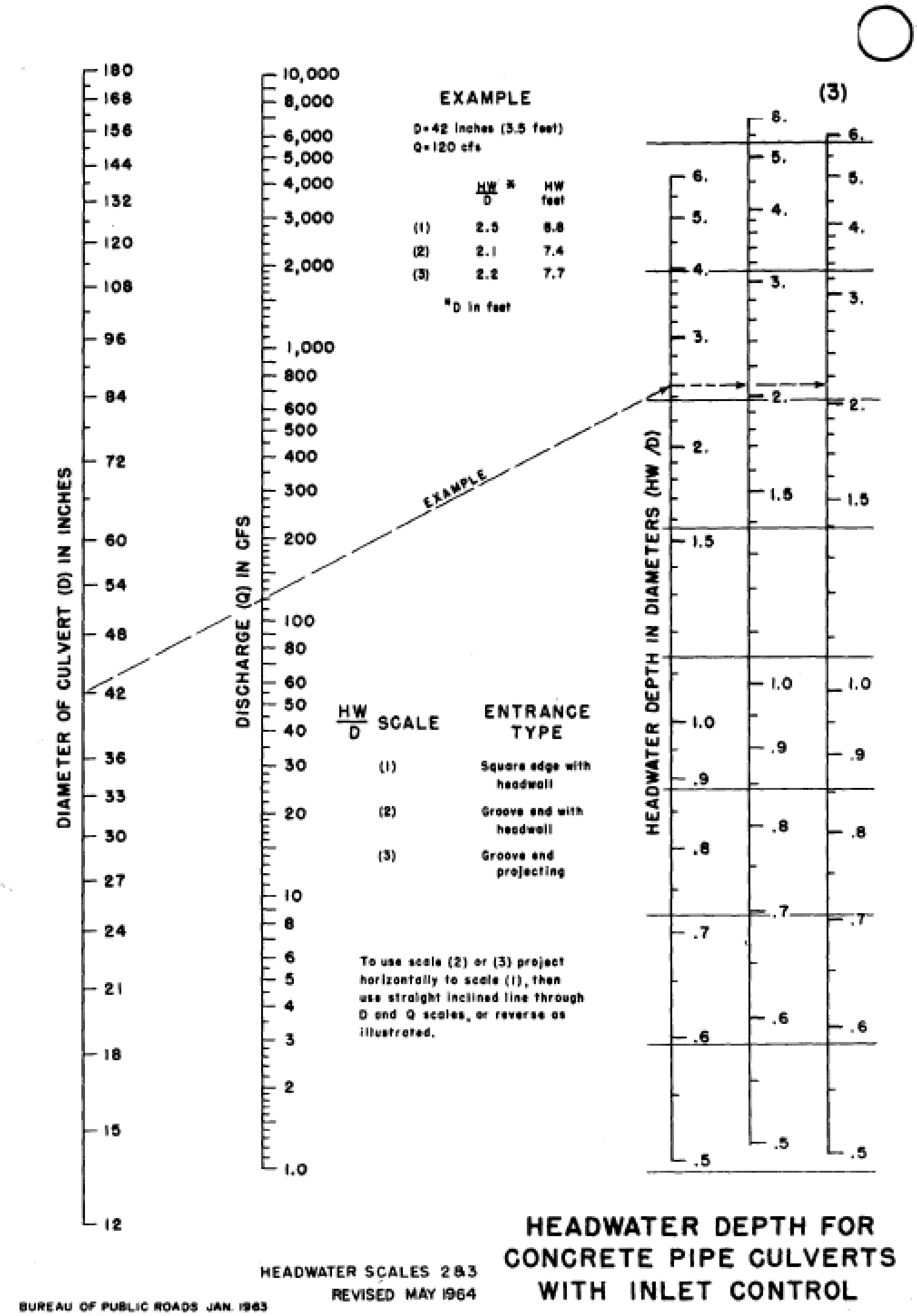
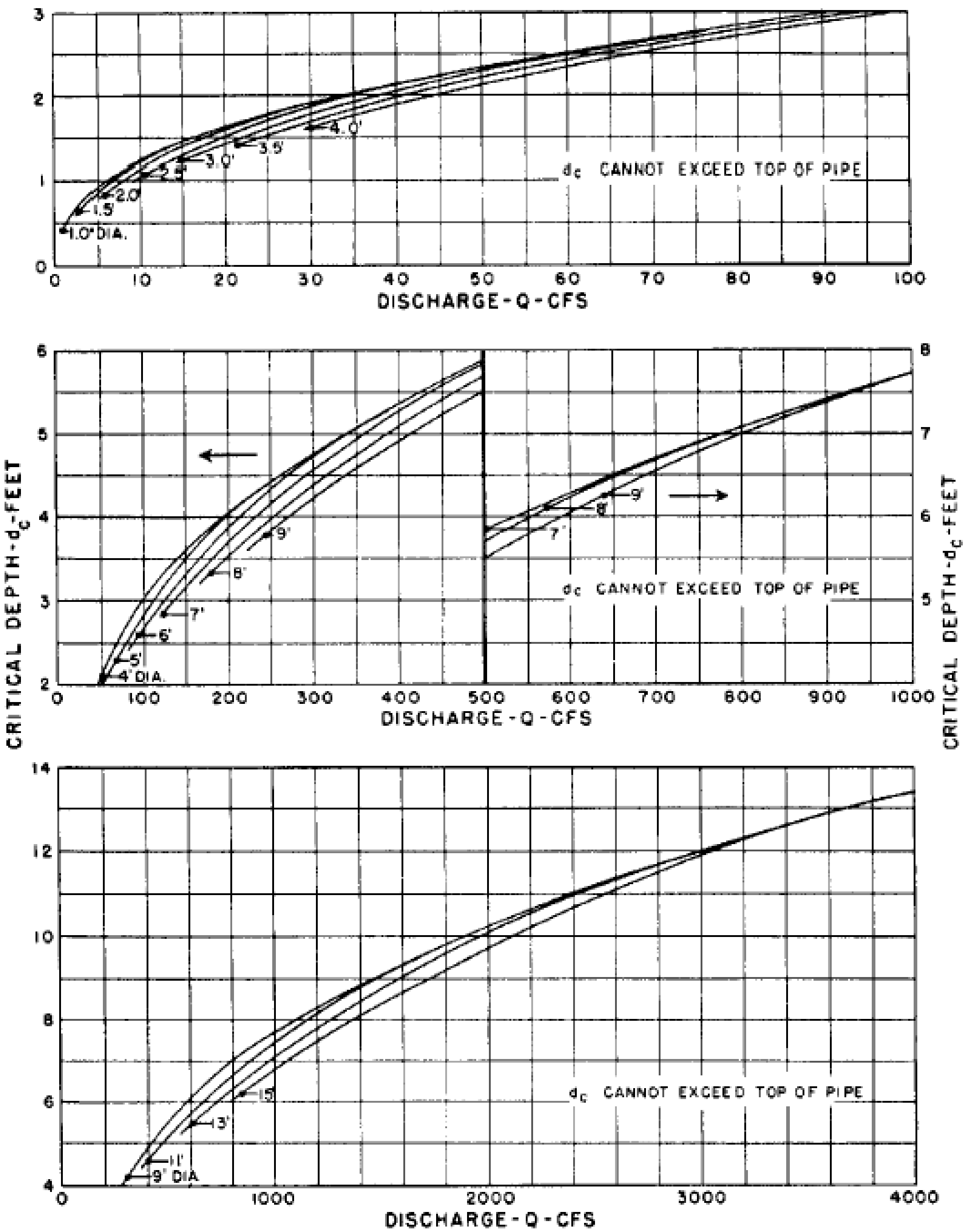

EXAMPLE
D=42 inches (3.5 feet)
Q=120 cfs
HW/D * HW feet
(1) 2.5 8.8
(2) 2.1 7.4
(3) 2.2 7.7
*D in feet
DIAMETER OF CULVERT (D) IN INCHES
180
168
156
144
132
120
108
96
84
72
60
54
48
42
36
33
30
27
24
21
18
15
12
DISCHARGE (Q) IN CFS
10,000
8,000
6,000
5,000
4,000
3,000
2,000
1,000
800
600
500
400
300
200
100
80
60
50
40
30
20
10
8
6
5
4
3
2
1.0
EXAMPLE
HEADWATER DEPTH IN DIAMETERS (HW/D)
(1)
(2)
(3)
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 DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL
HEADWATER SCALES 2&3 REVISED MAY 1964
BUREAU OF PUBLIC ROADS JAN. 1963

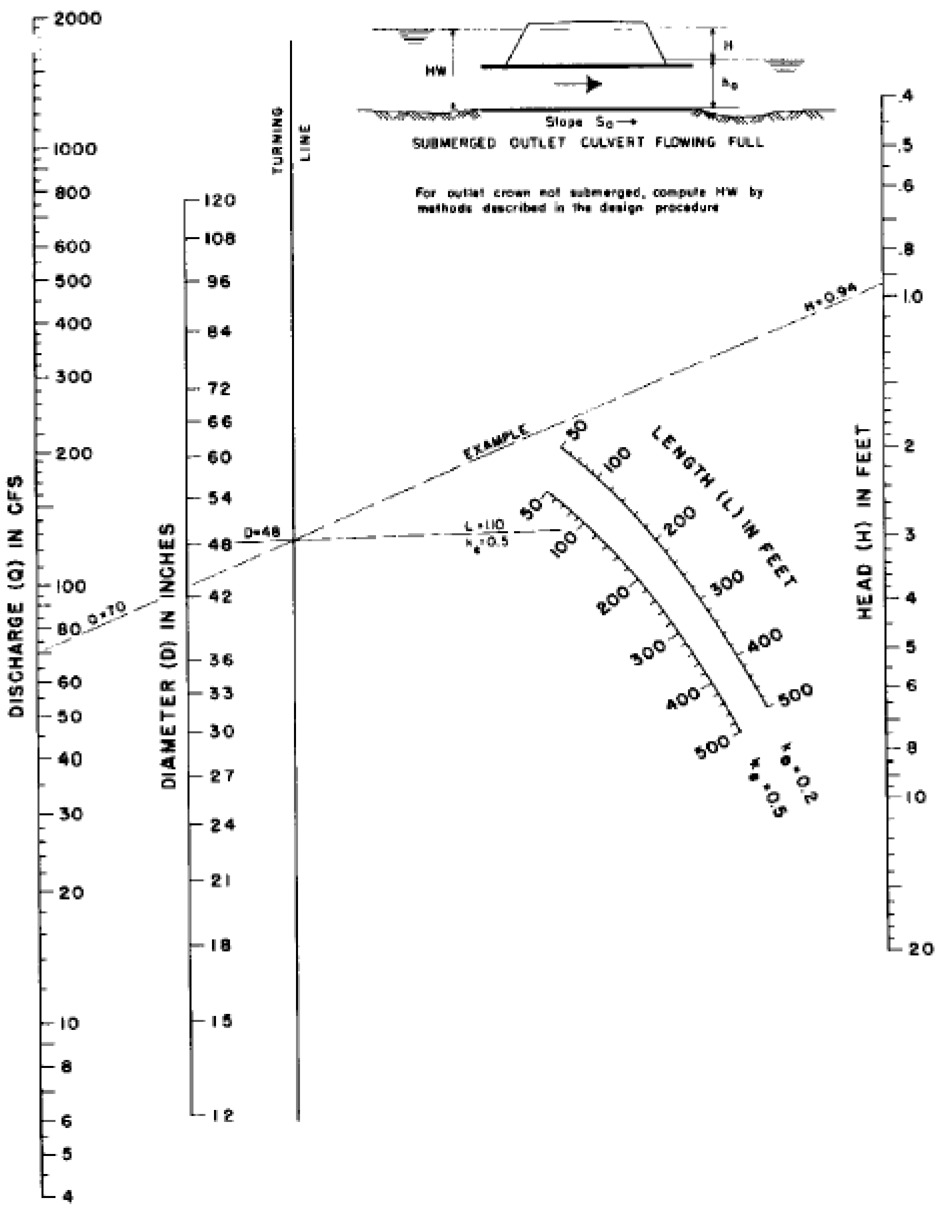
CHART 4B



BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

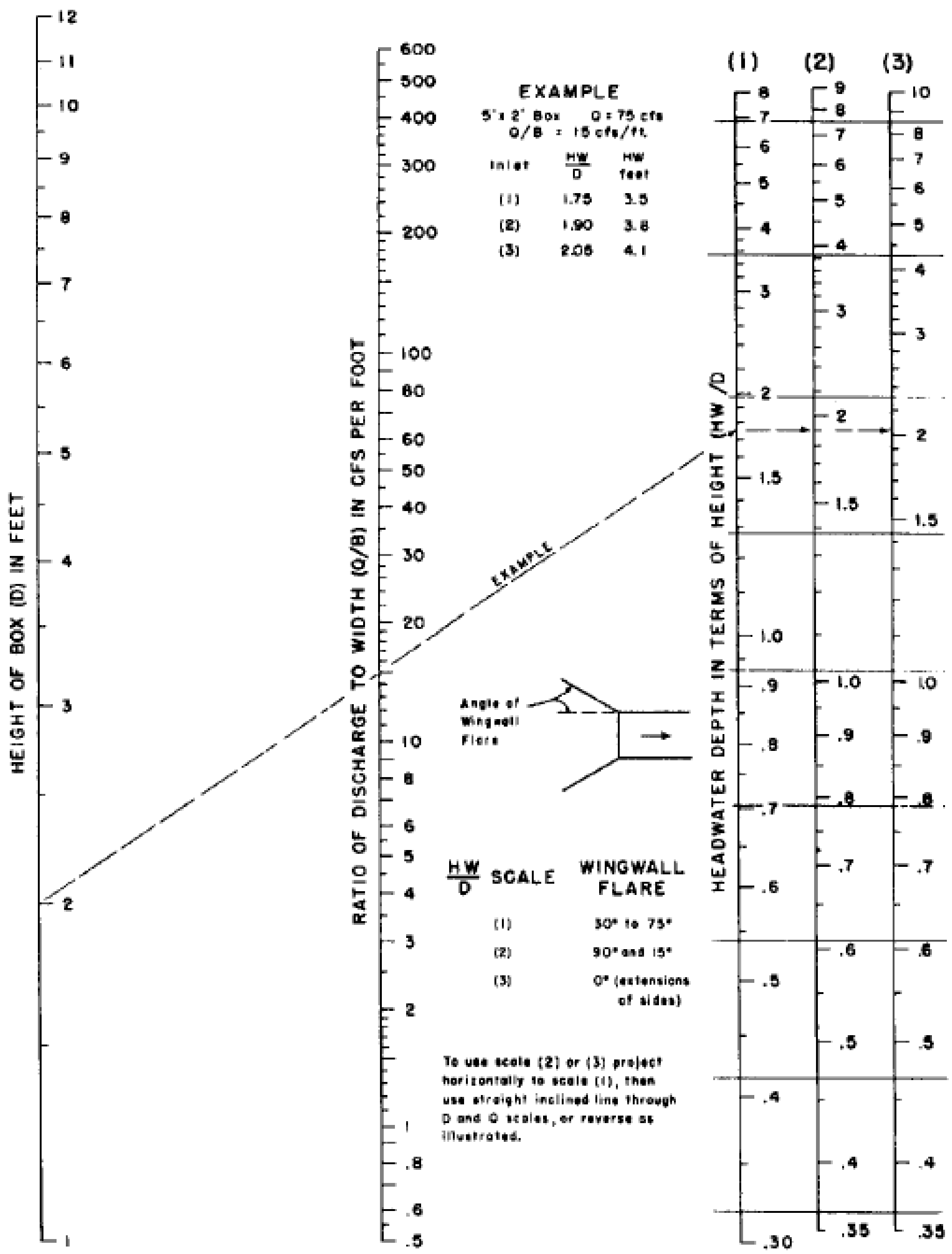
CHART 5B



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

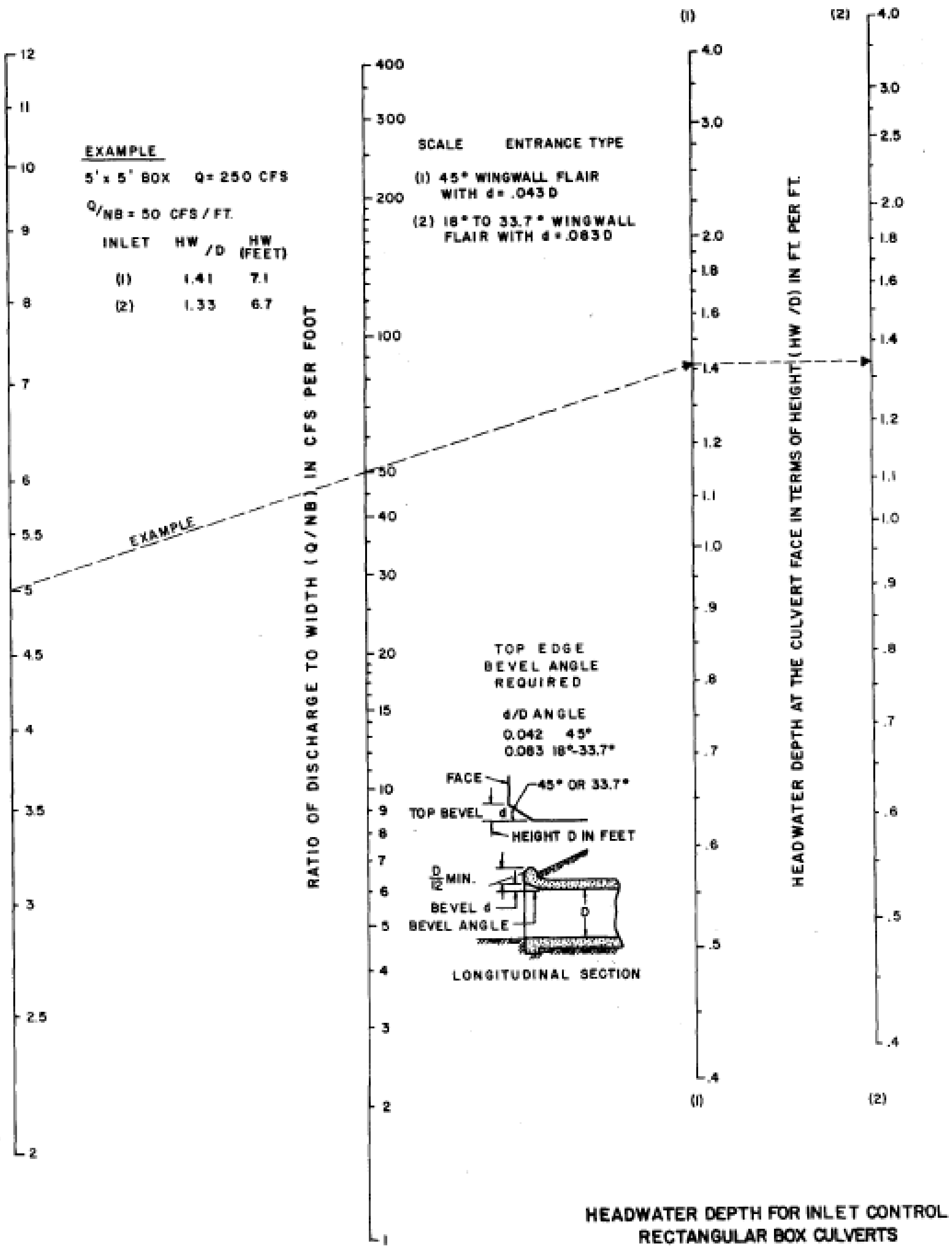
CHART 8B



HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

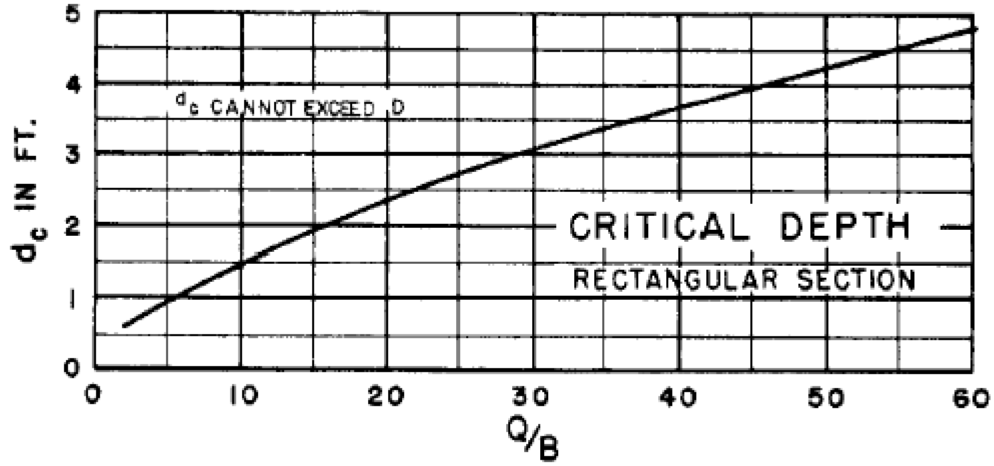
BUREAU OF PUBLIC ROADS JAN. 1963

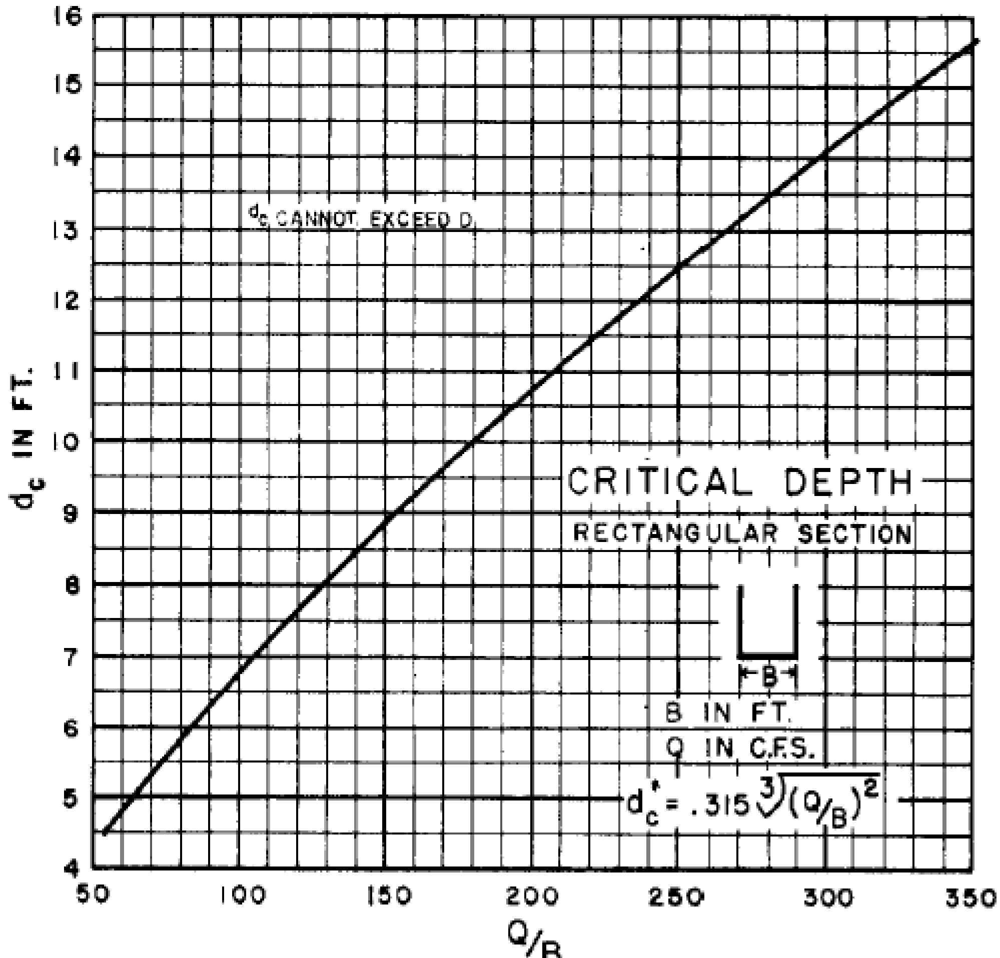
CHART 9B



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° & 45°
WITH BEVELED EDGE AT TOP OF INLET

CHART 14B

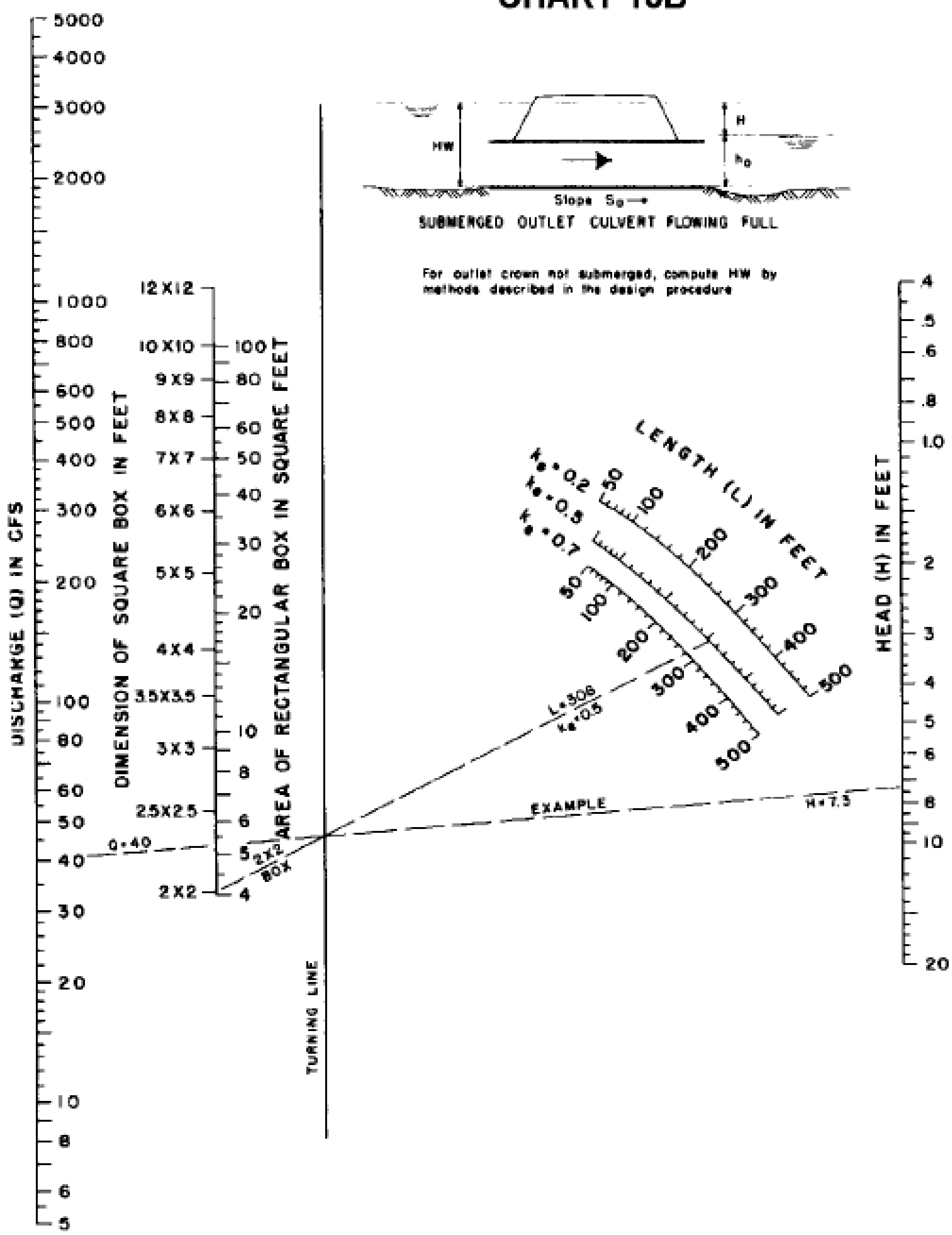




BUREAU OF PUBLIC ROADS JAN 1963

CRITICAL DEPTH
RECTANGULAR SECTION

CHART 15B



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

AU OF PUBLIC ROADS JAN. 1963

CHECKLISTS AND VARIANCE PROCEDURES

ENGINEER'S CHECKLIST FOR PRELIMINARY STORM WATER MANAGEMENT PLAN

Please attach additional sheets as necessary for comments and descriptions.

---

1. Project Information

A. *Name of Development:* ____________________ B. *Date:* ____________________

C. *Location of Development:* ____________________

D. *Type of Development:* ____________________ E. *Total area (acres):* ____________________

F. *Proposed Land Uses:* ____________________

G. *Anticipated project schedule:* ____________________

H. *Name of Owner:* ____________________ I. *Telephone No.:* ____________________

J. *Owner Contact Name:* ____________________ K. *FAX No.:* ____________________

L. *Owner Address:* ____________________ M. *Email Address:* ____________________

N. *Engineer's Name:* ____________________ O. *Email Address:* ____________________

P. *Engineering Firm:* ____________________ Q. *Telephone No.:* ____________________

R. *Engineer Address:* ____________________

Attachments:

_____ **Preliminary Plat or Site Plan**

_____ **Preliminary Storm Water Management Plan** (Checklist and Exhibits)

_____ **Preliminary Project Layout Map**

_____ **Preliminary Drainage Area Map**

2. Project Layout Map(s) shows the following information on or adjacent to the development site:

	Yes	No	N/A	Comments and Descriptions
A. *Digital ortho-photography showing project boundaries*	___	___	___	____________________
B. *Existing topography (normally 2-foot contours)*	___	___	___	____________________
C. *Preliminary street and lot layout*	___	___	___	____________________
D. *Benchmarks used for site control*	___	___	___	____________________
E. *Construction phasing plan, if applicable*	___	___	___	____________________

F. *Limits of proposed clearing and grading* ____ __ __ ________________

G. *Proposed dams > 6' high (attach Dam Safety Checklist)* ____ __ __ ________________

H. *Proposed FEMA floodplains with flood study reference info* ____ __ __ ________________

I. *Proposed ponds subject to TCEQ water rights permits* ____ __ __ ________________

J. *If yes, has water rights permit been applied for?* ____ __ __ ________________

3. Drainage Area Map(s) shows the following information for the development site:

A. *Preliminary street and lot layout (scale 1"=200')* ____ __ __ ________________

B. *All off-site drainage areas with topography (reduced scale)* ____ __ __ ________________

C. *Delineation of watershed boundaries with flow arrows* ____ __ __ ________________

D. *Proposed modifications to watershed boundaries* ____ __ __ ________________

E. *File numbers for existing developments & drainage facilities* ____ __ __ ________________

F. *Zoning or Comp Plan info to document off-site land use* ____ __ __ ________________

G. *Preliminary hydrology with supporting data & calculations for on-site existing & proposed, & off-site ultimate conditions* ____ __ __ ________________

H. *Proposed detention ponds or other storm water controls, with summary hydrology for all applicable design storms* ____ __ __ ________________

I. *Delineate zone of influence for all outfalls* ____ __ __ ________________

J. *Downstream constrictions, flooding, or erosion locations* ____ __ __ ________________

K. *Proposed facilities with private maintenance (Maintenance Agreement and Maintenance Plan required for final)* ____ __ __ ________________

4. **Description of Downstream Assessment and Zones of Influence:** Describe and provide supporting methodology:

5. **Additional Study Attachments (include if applicable)**

   A. Dam Safety Checklist

   B. Storm Water Pollution Prevention Plan (SWPPP)

   C. *Executed Maintenance Agreement (with Maintenance Plan)*

   D. *Landscaping Plan (for Storm Water controls)*

   E. *Copy of approved Waiver Request*

6. **Applicable Local, State and Federal Permits** (Indicate acquired or application pending)

   A. CLOMR, LOMR or LOMA

   B. TCEQ water rights permit

   C. 404 permit

   D. Other:

Other:

7. Description of Any Proposed Variance Requests: (for informational purposes only; all Variance Requests must follow published procedures)

8. Other Comments:

ENGINEER'S CHECKLIST FOR
FINAL STORM WATER MANAGEMENT PLAN

Please attach additional sheets as necessary for comments and descriptions.

---

1. Project Information (for Items 1.C to 1.Q, N/C = No Change from Preliminary SWM Plan)

A. *Name of Development:* ______________________________ B. *Date:*______________________

C. *Location of Development:* ____

D. *Type of Development:*______________________________ E. *Total area (acres):* ______________

F. *Proposed Land Uses:* ____

G. *Anticipated project schedule:* ____

H. *Name of Owner:* ______________________________ I. *Telephone No.:*______________________

J. *Owner Contact Name:*______________________________ K. *FAX No.:*______________________

L. *Owner Address:*______________________________ M. *Email Address:*______________________

N. *Engineer's Name:* ______________________________ O. *Email Address:*______________________

P. *Engineering Firm:* ______________________________ Q. *Telephone No.:*______________________

R. *Engineer Address:*____

Attachments: _____ **Final Plat or Site Plan**

_____ **Concept Storm Water Management Plan** (Checklist and Exhibits)

_____ **Preliminary Storm Water Management Plan** (Checklist and Exhibits)

_____ **Additional Attachments as Specified Below**

2. Changes or Modifications to Preliminary Storm Water Management Plan (May be reprinted with changes tracked or highlighted)

____

____

____

____

____

____

Culvert Hydraulics Documentation Checklist – Form

Project:		Date:	
Road:	Watershed:	Stream:	
Type of work:			
FEMA considerations (Detailed or Approx. Study?):			
Culvert location:			
Culvert size & shape:			
Culvert material:	**Fill height:**	**Skew angle:**	
Hydrologic method used: Hydrograph USGS Station ____________________ Other (specify) ___________________________________			
Design frequency (yrs):		**Drainage area:**	
Channel analysis:	**Channel slope (m/m):**	**N values (channel):**	
100 Yr Proposed discharge (cfs):		**100 Year Ultimate discharge - Q_{100} (cfs):**	
100 Yr Proposed tailwater (ft):		**100 Year Ultimate tailwater (ft):**	
100 YR Proposed headwater (ft):		**100 Year Ultimate headwater (ft):**	
Allowable highwater (ft):			
100 Yr Proposed velocity thru bridge (fps):		**100 Year Ultimate velocity thru bridge (fps):**	
Design unconstricted velocity (fps)		**100 Year unconstricted velocity (fps)**	
% Flow overtopping road for Q_{100}:		**Height of water over road for Q_{100} (ft):**	
Est. overtopping frequency (years):			
Headwater computation method: THYSYS-CULVERT HEC-RAS* HEC 2 Other ______________________________ *Required by DRAINAGE REVIEW AUTHORITY			
Comparison with existing hydraulic condition:			
Meets FEMA requirements___________Yes_____________No______N/A			
Outlet velocity excessive_______Yes_______No			
Outlet protection/control:			
Safety end treatment:			
Comments:			

Bridge Hydraulics Documentation Checklist – Form

Project:		Date:
Road:	Watershed:	Stream:
Type of work:		
FEMA considerations (Detailed or Approx. Study?):		
Bridge Length:	Pier Configuration:	
Bridge Width:	Bridge Low Chord and Roadbed Elev.:	

Hydrologic Method Used: Hydrograph Only
Gaged - USGS Station ____________________
Other___________________________________

Design Frequency (yrs):*		**Drainage Area:**
Channel Dimensions:	**Channel slope(ft/ft):**	**N value:**

	DESIGN PROPOSED			100 YR EXISTING			100 YR PROPOSED			100 YR ULTIMATE		
STATION	Q (cfs)	V (fps)	WSEL (ft)	Q (cfs)	V (fps)	WSEL (ft)	Q (cfs)	V (fps)	WSEL (ft)	Q (cfs)	V (fps)	WSEL (ft)
EXIT												
FULL V												
BRIDGE												
APPR (CONSTR)												
APPR (UNCONS)												

Headwater computation method: HEC-RAS_______________ OTHER_________________

Bridge/Roadway overtopping: _____Yes _____No	Overtopping Frequency(years):
% Flow overtopping road:	**Height of water over road(ft):**
Existing Bridge Length(ft):	Meets FEMA requirements: _______Yes_______No______N/A
Type of Bridge Rail:	Skew:
Abutment protection (rock riprap, etc):	
Comments:	

***Complete for cases where "design frequency" (such as TxDOT structures) may be different than 100-year.**

Variance Procedure – Wharton County Drainage Design Manual

Good engineering practice and practical considerations are necessary when developing storm water management plans and preparing construction drawings for specific projects. The criteria in this manual cannot cover every possibility.

The closer the criteria are followed, the more likely the plan or drawing will be approved and the construction accepted. For those situations where varying from the criteria is warranted, a variance process is described below.

Submit variance request in writing on the Request for Variance from Wharton County – Storm Water Form (Wharton County) as early as possible. The variance request must include the following:

- The specific criteria that you want to vary.
- Why the criteria needs to be varied.
- How the basis for the criteria will still be satisfied or why the criteria is not applicable.
- Indicate if there are no criteria for the proposed analysis, design, or feature in this manual.
- Appropriate technical information supporting the variance request, such as calculations, excerpts from the drainage or design plan, and/or construction drawings.

Note: Submittals with insufficient technical information to support the variance request will be returned without review.

The DRAINAGE REVIEW AUTHORITY will either approve or reject the variance in writing on the variance request form. If it is rejected, a written explanation will be provided.

REQUEST FOR VARIANCE FROM WHARTON COUNTY – DRAINAGE REQUIREMENTS

Submitted by:____________________________ Phone:____________________ Email:______________

Company:____Date:________________

Proposed Project Description

Name:____

Type:____

Location:____(include map)

Existing Condition (show information on map or drawing)

DRAINAGE REGULATION ENTITY Maintained Facilities:____
____

Existing Right-of-Way for DRAINAGE REGULATION ENTITY facility:______________________________
____

Topography:____

Other Pertinent Data Related to Variance Request:

____
____

Variance Request

Specific criteria you want to vary:____
____

Explain why the criteria needs to be varied or is not applicable:_________________________________
____

Explain how the basis for the criteria will be satisfied:______________________________________
____

List attachments supporting variance request (preliminary design report excerpt, construction drawings, calculations, photographs, map, etc.):

DRAINAGE REGULATION ENTITY to fill in this area *DEV ID#* ____________

Date	Reviewer	Dept./Section	Action Taken

Justification of Decision:____
____

Approval of Final Decision:_______________________________ Date:________________

PRELIMINARY AND FINAL
DAM MAINTENANCE AND EMERGENCY ACTION PLAN
FORM *

Please attach additional sheets as necessary for comments and descriptions.
Fold all sheets to 8½" x 11" or 9" x 12" and bind with a clip

. * Some requirements are scheduled to be revised in 2009. (See Appendix C)

---

1. Project Information

A. Name of Development: ________________ *B. Case No.:* ________________

C. Dam Name, Number or Tributary: ________________ *D. Date:* ________________

E. Name of Owner: ________________ *F. Telephone No.:* ________________

G. Owner Contact Name: ________________ *H. E-mail:* ________________

I. Owner Address: ________________________________

J. Engineer's Name: ________________ *K. Texas P.E. No.:* ________________

L. Engineering Firm: ________________ *M. Telephone No.:* ________________

N. Engineer Address: ________________ *O. E-mail:* ________________

2. Dam Summary Information (Item H not required for Preliminary Submittal)

A dam with a height of six (6) feet or greater, measured from the crest of the dam to the bottom of the outfall channel immediately below the dam, must be registered with the TCEQ, and have a breach analysis, hazard assessment, and emergency action plan per 30 TAC §299.

A. Dam height (feet): ________________ *B. Impoundment surface area (acres):* ________________

C. Watershed size (acres): ________________ *D. Approx. impoundment volume (acre-feet):* ________________

E. Who will own and maintain dam (HOA, City park, etc.)? ________________________________

F. Was dam previously registered and/or inspected by TCEQ? When? ________________________________

G. TCEQ Impoundment size classification (30 TAC §299.12): ____ *Exempt (<6' high)* ____ *Small* ____ *Intermediate* ____ *Large*

H. Hazard Assessment (from 6.B. below per 30 TAC §299.13): ____ *N/A (<6' high)* ____ *Low* ____ *Significant* ____ *High*

3. Attachments:

_____ **Dam and Pond Site Map(s)**

_____ **Water Rights Permit (where applicable)**

_____ **Breach Analysis (where applicable)**

_____ **Emergency Action Plan (final submittal)**

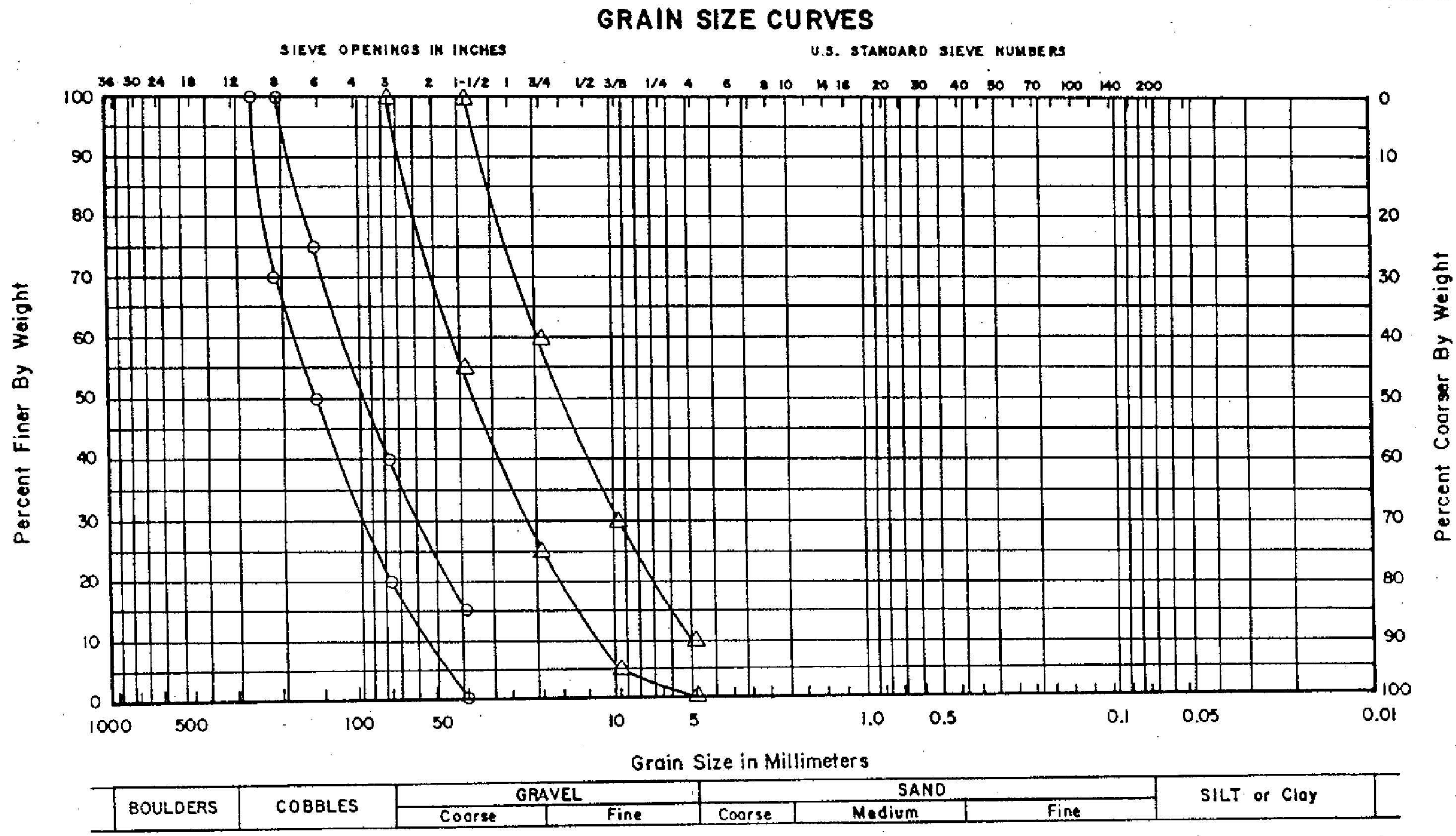
For Drainage Review Authority Use: Reviewer: ______________ *Date:* _______________
Accepted *Not Accepted* *Case No.:* ____________________

	Yes	No	N/A	Comments and Descriptions
4. State Water Rights				
In accordance with Texas Water Code §11, all surface impoundments not used for domestic or livestock purposes must obtain a water rights permit from the TCEQ.				
Has water rights permit been obtained or applied for?	__	___		________________
5. Dam and Pond Site Map(s), showing:				
A. Proposed and existing contours, with recent aerial	__	___	___	________________
B. Existing and proposed FEMA floodplain limits	__	___	___	________________
C. Street and lot layout around dam and inundation area	__	___	___	________________
D. Contributing watershed (reduced scale if necessary)	__	___	___	________________
E. Hydrologic calculations for Q100 and PMF	__	___	___	________________
F. Location, size and capacity of proposed spillway	__	___	___	________________
G. Conceptual or final spillway and erosion protection design	__	___	___	________________
6. Dam Breach Analysis – Attach and Include: (Required for Final Submittal only, for dams at least 6’ in height)				
A. Breach analysis for “sunny day”, “barely overtopping” or Q100, and Probable Maximum Flood (PMF) conditions	__	___	___	________________

B. Hazard Assessment based on potential for loss of life or property damage in breach/non-breach comparison __ ___ ___ ____

C. Emergency Action Plan per current Drainage Regulation Entity standards __ ___ ___ ____

Appendix F
Miscellaneous Details and Specifications



PROJECT STANDARD GRADATIONS

DESCRIPTION STONE RIPRAP AND BEDDING CURVES

8 INCH THICK RIPRAP AND 6 INCH THICK BEDDING

DATE MAY 1987

LEGEND

—o——o— RIPRAP

—△——△— BEDDING

Freese and Nichols Inc. Consulting Engineers

Plate 1

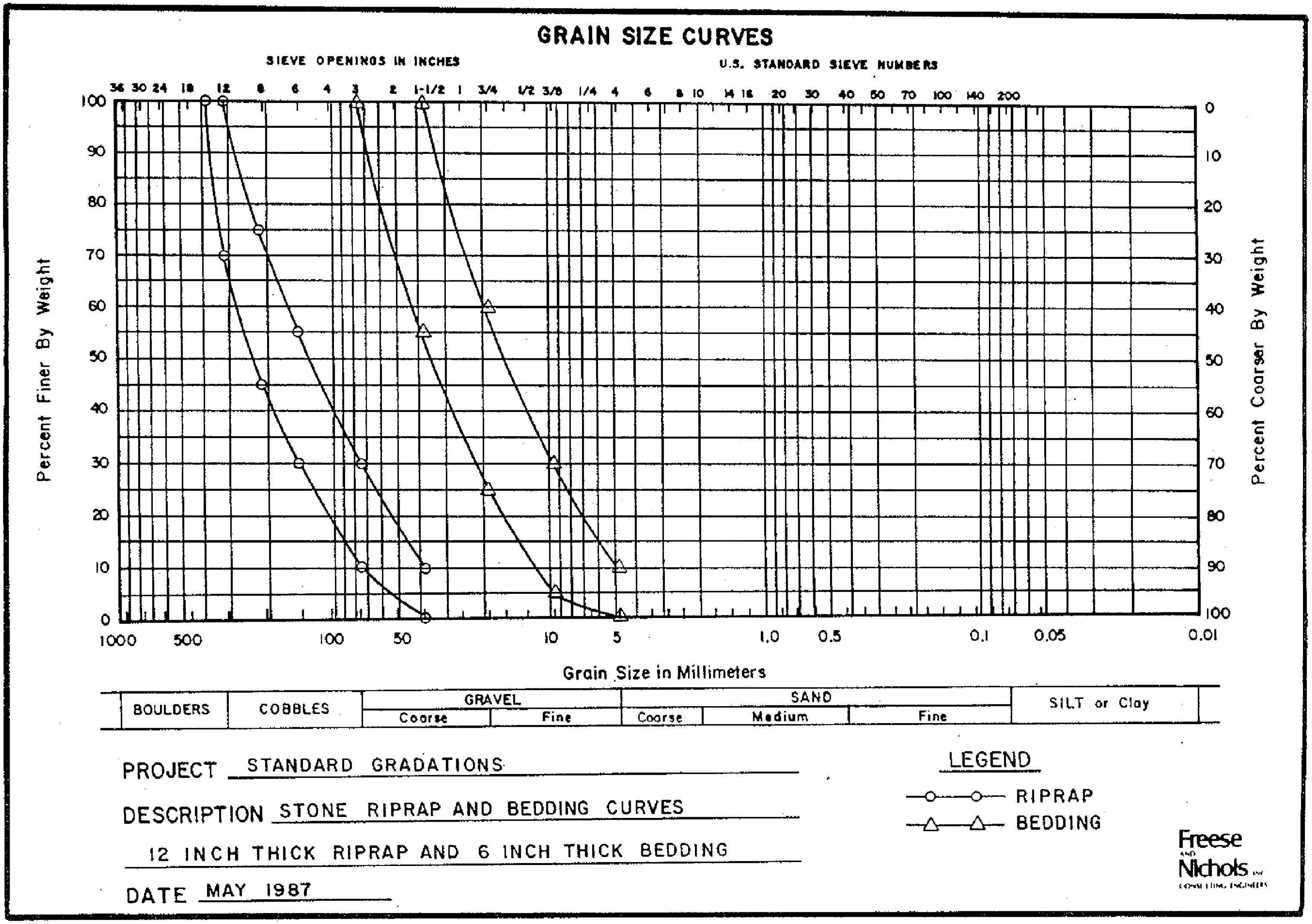

GRAIN SIZE CURVES
SIEVE OPENINGS IN INCHES
U.S. STANDARD SIEVE NUMBERS
36 30 24 18 12 8 6 4 3 2 1-1/2 1 3/4 1/2 3/8 1/4 4 6 8 10 14 16 20 30 40 50 70 100 140 200
Percent Finer By Weight
Percent Coarser By Weight
1000 500 100 50 10 5 1.0 0.5 0.1 0.05 0.01
Grain Size in Millimeters
BOULDERS
COBBLES
GRAVEL
Coarse
Fine
SAND
Coarse
Medium
Fine
SILT or Clay
PROJECT STANDARD GRADATIONS
DESCRIPTION STONE RIPRAP AND BEDDING CURVES
12 INCH THICK RIPRAP AND 6 INCH THICK BEDDING
DATE MAY 1987
LEGEND
RIPRAP
BEDDING
Freese and Nichols
CONSULTING ENGINEERS

Plate 2

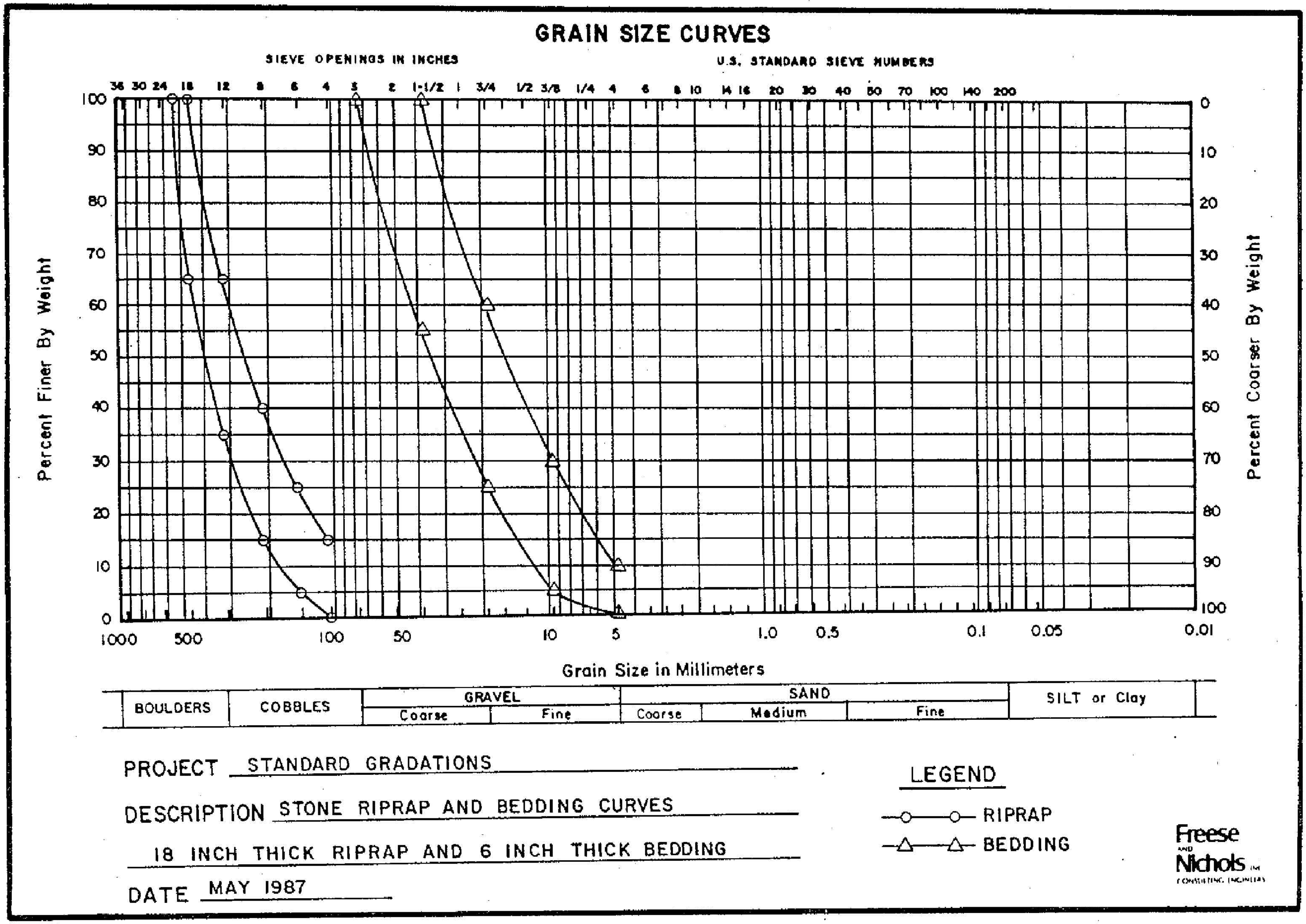
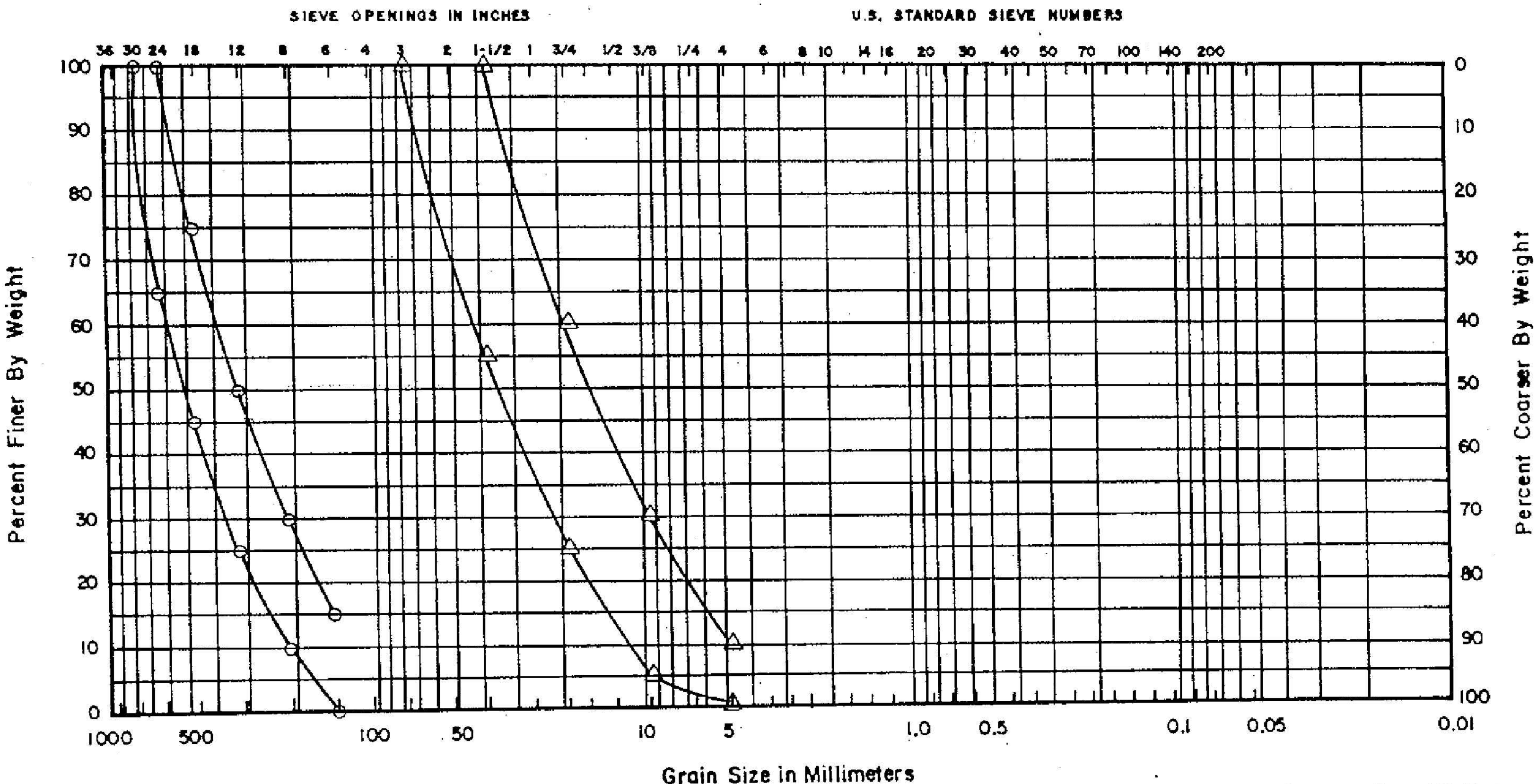

GRAIN SIZE CURVES
SIEVE OPENINGS IN INCHES
U.S. STANDARD SIEVE NUMBERS
Percent Finer By Weight
Percent Coarser By Weight
Grain Size in Millimeters
BOULDERS
COBBLES
GRAVEL
Coarse
Fine
SAND
Coarse
Medium
Fine
SILT or Clay
PROJECT STANDARD GRADATIONS
DESCRIPTION STONE RIPRAP AND BEDDING CURVES
18 INCH THICK RIPRAP AND 6 INCH THICK BEDDING
DATE MAY 1987
LEGEND
RIPRAP
BEDDING
Freese and Nichols

Plate 3

GRAIN SIZE CURVES



BOULDERS	COBBLES	GRAVEL		SAND			SILT or Clay
		Coarse	Fine	Coarse	Medium	Fine	

PROJECT STANDARD GRADATIONS

DESCRIPTION STONE RIPRAP AND BEDDING CURVES

24 INCH THICK RIPRAP AND 6 INCH THICK BEDDING

DATE MAY 1987

LEGEND

—o——o— RIPRAP

—△——△— BEDDING

Freese and Nichols Inc.
CONSULTING ENGINEERS

Plate 4

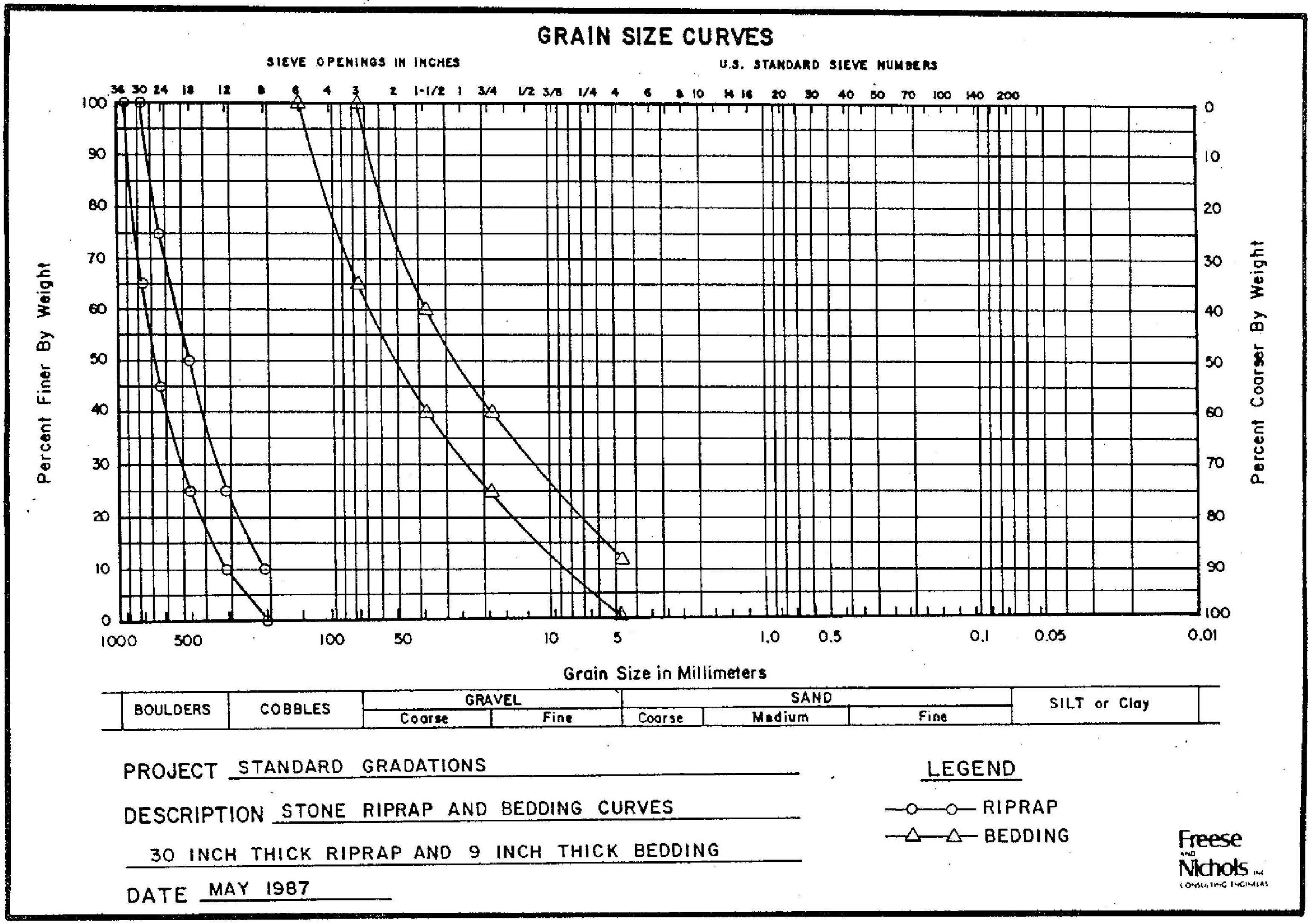
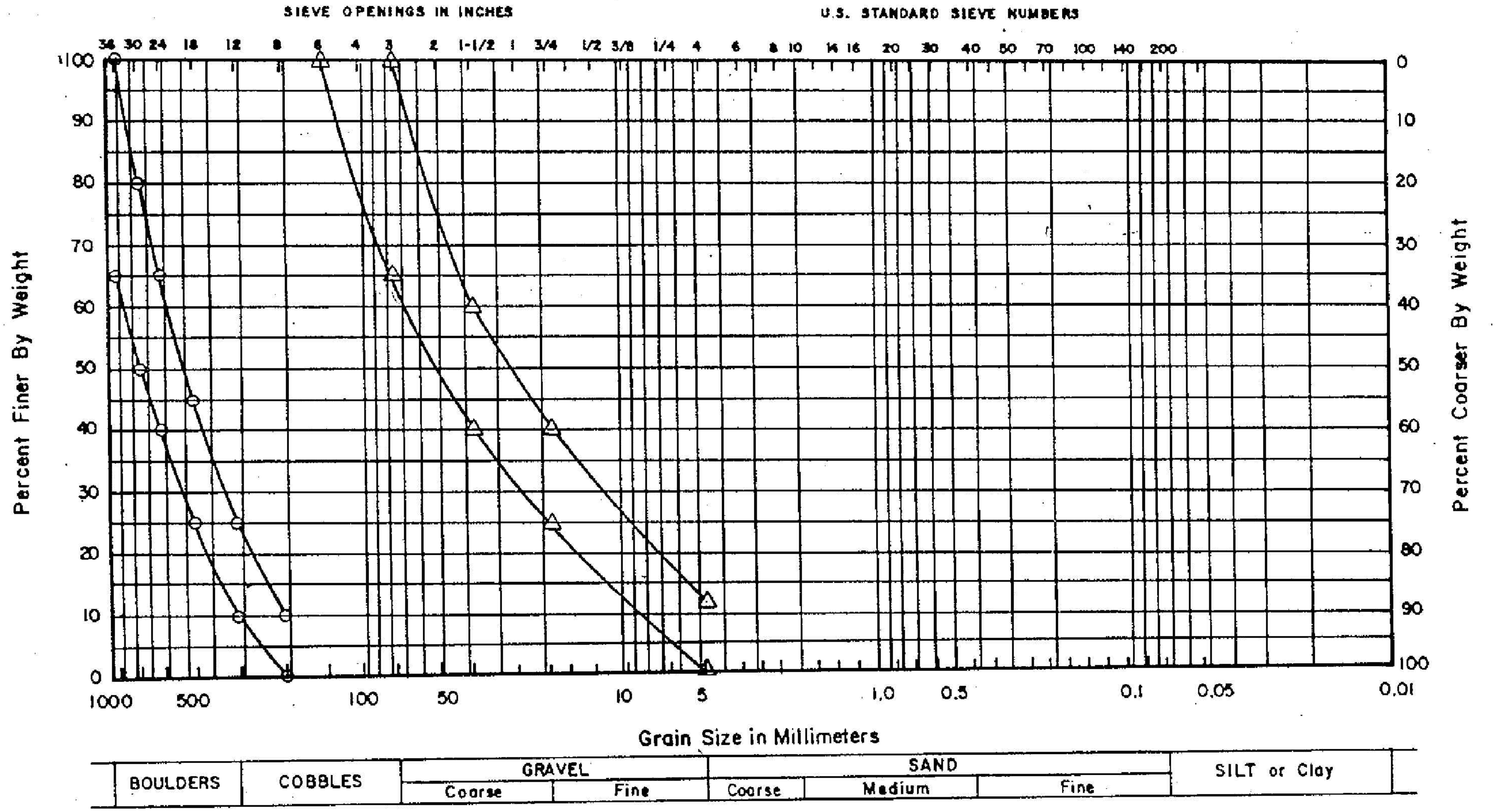

GRAIN SIZE CURVES
SIEVE OPENINGS IN INCHES
U.S. STANDARD SIEVE NUMBERS
36 30 24 18 12 8 6 4 3 2 1-1/2 1 3/4 1/2 3/8 1/4 4 6 8 10 14 16 20 30 40 50 70 100 140 200
Percent Finer By Weight
100 90 80 70 60 50 40 30 20 10 0
Percent Coarser By Weight
0 10 20 30 40 50 60 70 80 90 100
1000 500 100 50 10 5 1.0 0.5 0.1 0.05 0.01
Grain Size in Millimeters
BOULDERS
COBBLES
GRAVEL
Coarse
Fine
SAND
Coarse
Medium
Fine
SILT or Clay
PROJECT STANDARD GRADATIONS
DESCRIPTION STONE RIPRAP AND BEDDING CURVES
30 INCH THICK RIPRAP AND 9 INCH THICK BEDDING
DATE MAY 1987
LEGEND
RIPRAP
BEDDING
Freese and Nichols Inc
CONSULTING ENGINEERS

Plate 5

GRAIN SIZE CURVES



PROJECT STANDARD GRADATIONS

DESCRIPTION STONE RIPRAP AND BEDDING CURVES

36 INCH THICK RIPRAP AND 9 INCH THICK BEDDING

DATE MAY 1987

LEGEND

—o——o— RIPRAP

—△——△— BEDDING

Freese and Nichols, Inc.
Consulting Engineers

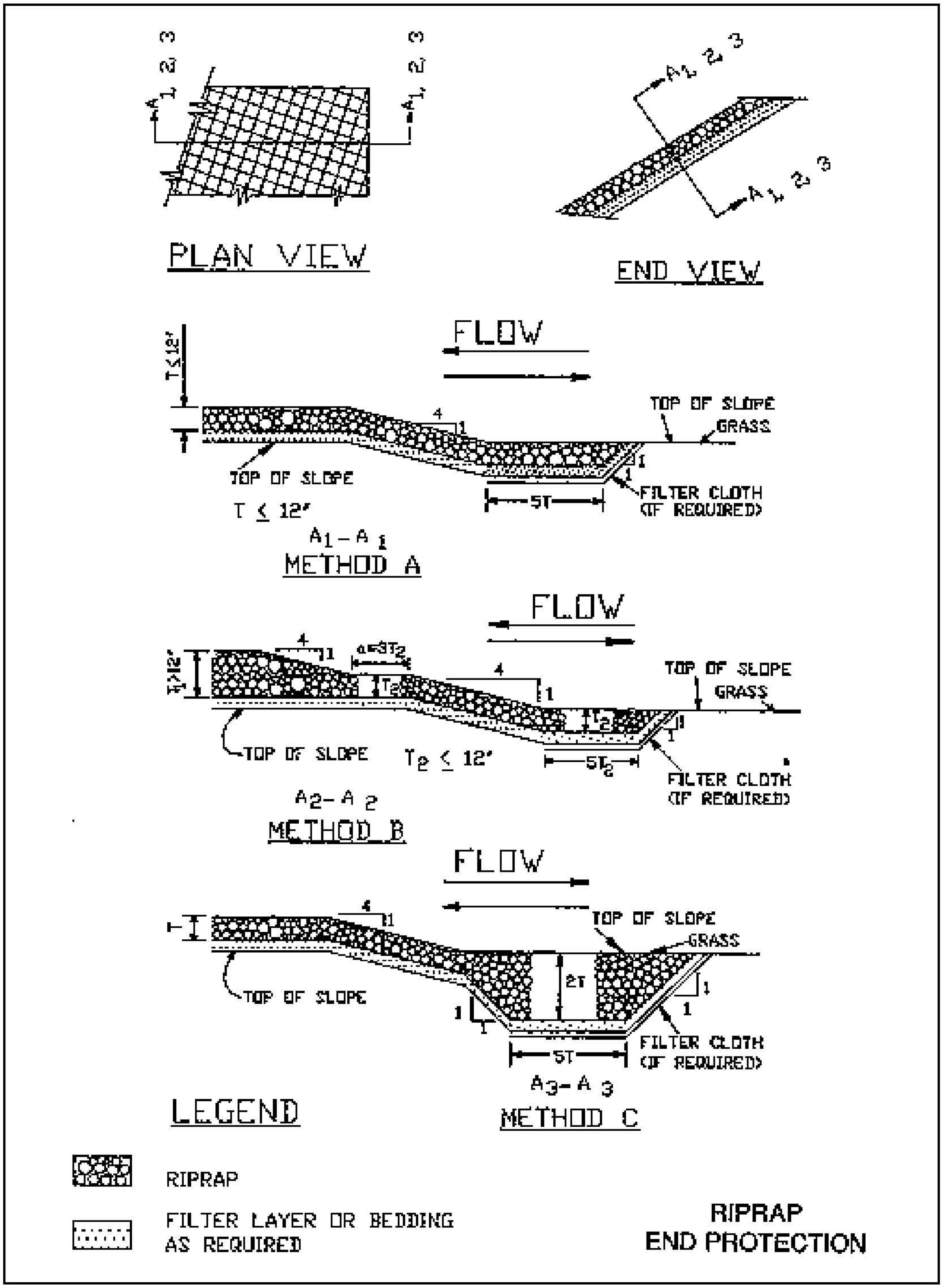
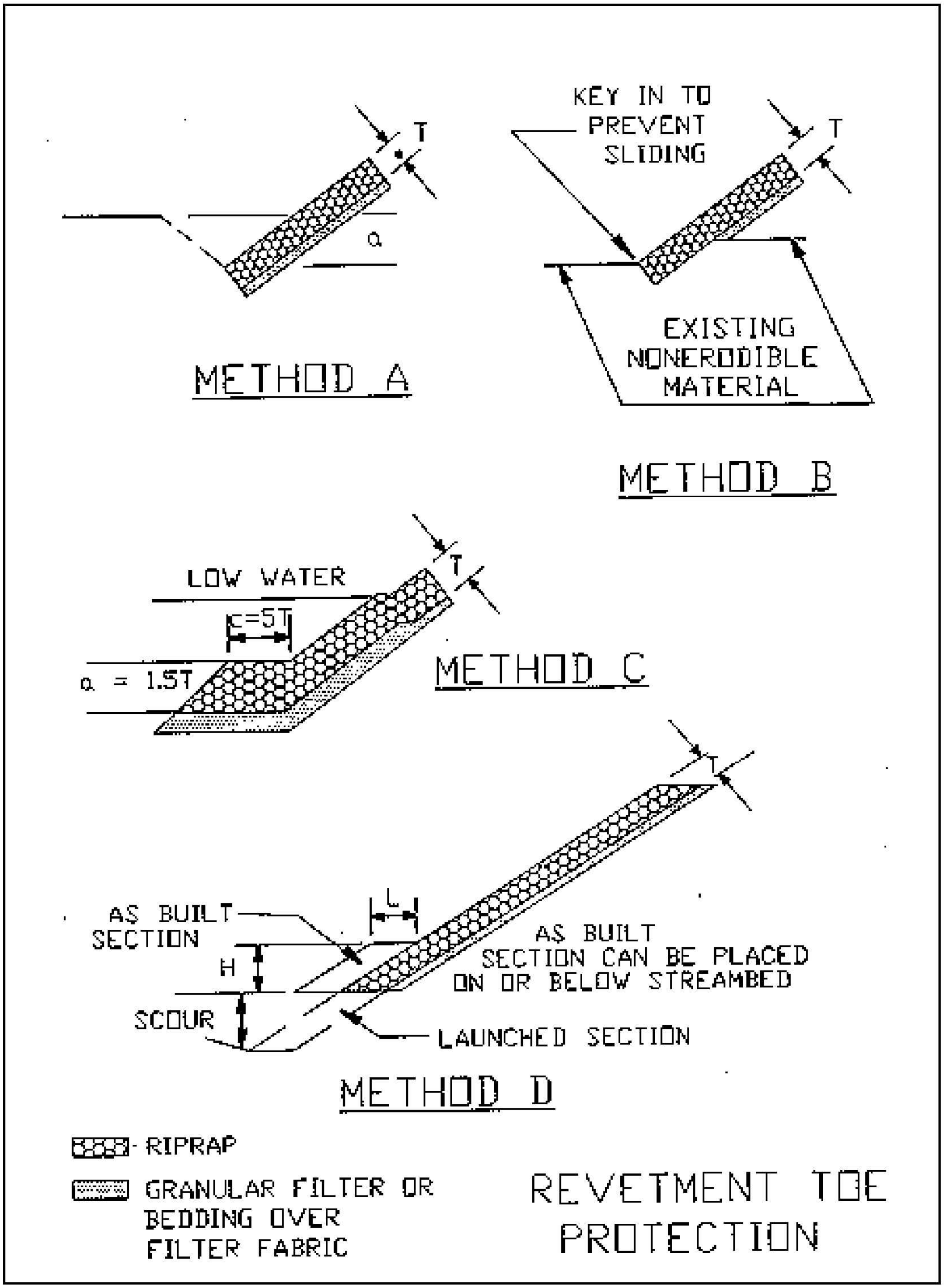

A1, 2, 3
A1, 2, 3
PLAN VIEW
A1, 2, 3
A1, 2, 3
END VIEW
FLOW
T≤12"
4
1
TOP OF SLOPE
GRASS
1
1
TOP OF SLOPE
FILTER CLOTH
(IF REQUIRED)
5T
T ≤ 12"
A1 – A 1
METHOD A
FLOW
T>12"
4
1
4=3T2
T2
4
1
TOP OF SLOPE
GRASS
T2
1
1
TOP OF SLOPE
T2 ≤ 12"
5T2
FILTER CLOTH
(IF REQUIRED)
A2– A 2
METHOD B
FLOW
T
4
1
TOP OF SLOPE
GRASS
2T
1
1
1
1
TOP OF SLOPE
FILTER CLOTH
(IF REQUIRED)
5T
A3– A 3
METHOD C
LEGEND
RIPRAP
FILTER LAYER OR BEDDING
AS REQUIRED
RIPRAP
END PROTECTION

Plate B-41



T
a
METHOD A
KEY IN TO
PREVENT
SLIDING
T
EXISTING
NONERODIBLE
MATERIAL
METHOD B
T
LOW WATER
c=5T
a = 1.5T
METHOD C
T
L
AS BUILT
SECTION
H
AS BUILT
SECTION CAN BE PLACED
ON OR BELOW STREAMBED
SCOUR
LAUNCHED SECTION
METHOD D
RIPRAP
GRANULAR FILTER OR
BEDDING OVER
FILTER FABRIC
REVETMENT TOE
PROTECTION

Plate B-43

9.3 Straight Drop Spillways

Overview 9.3.1

The three parts of a straight drop spillway (see Exhibit 9-1) are:

- **Upstream draw down reach**
- **Drop opening**
- **Downstream hydraulic jump reach**

The drop is usually constructed of steel sheet piling. Reinforced concrete lining and riprap are placed upstream and downstream of the drop structure for erosion and scour protection.

Design Criteria 9.3.2

Design criteria for straight drop spillways are:

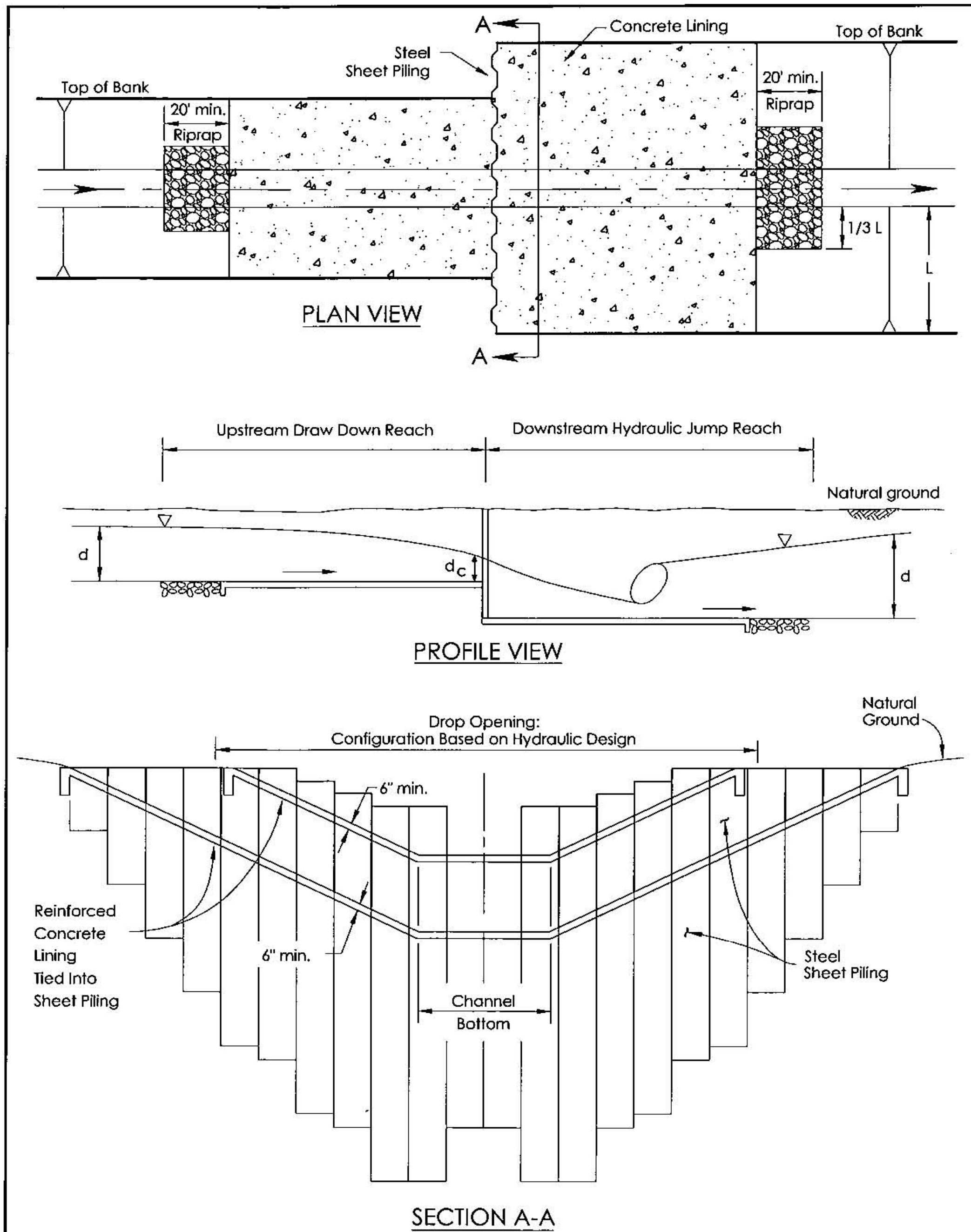
- **Comply with general design criteria for all transition control structures in Section 9.2.1, General Design Criteria.**
- **Design steel sheet piling to prevent bending or rotating.**
- **Coat steel sheet piling in accordance with industry standards to reduce rusting and scaling.**
- **Use concrete lining on the entire cross-section upstream and downstream of the drop.**
- **Tie the concrete lining to the steel sheet piling drop structure.**
- **Use a minimum 6-inch thick slab on the downstream concrete lining due to the impact load and potential sever turbulence.**
- **Determine length of concrete lining upstream and downstream of the drop.**
- **Include 20 feet of riprap at the ends of the concrete slope paving to decrease flow velocities and protect the concrete toe from scour (see Section 4.4.8 Stone Riprap Design)**

General Design Criteria

9.2.1

General design criteria for transition control structures are:

- **Design for a range of flows and tailwater conditions up to and including the 1% exceedance event.**
- **Conduct a geotechnical investigation to assist with design of the structure.**
- **Locate transition control structures where flow is straight. Avoid channel bends and high turbulence areas, if possible.**
- **Provide structural erosion protection where maximum velocities are exceeded upstream and downstream of the transition control structure and where the hydraulic jump occurs.**
- **For drop structures in lateral channels at the confluence with the receiving channel:**
  - **Locate the drop just inside the ultimate right-of-way of the receiving channel.**
  - **Design the hydraulic jump to occur before it enters the receiving channel.**



PLAN VIEW

PROFILE VIEW

SECTION A-A

Harris County Flood Control District

POLICY, CRITERIA, & PROCEDURE MANUAL

TYPICAL STRAIGHT DROP

DATE: 10/5/04	EXHIBIT 9-1

Baffled Chutes

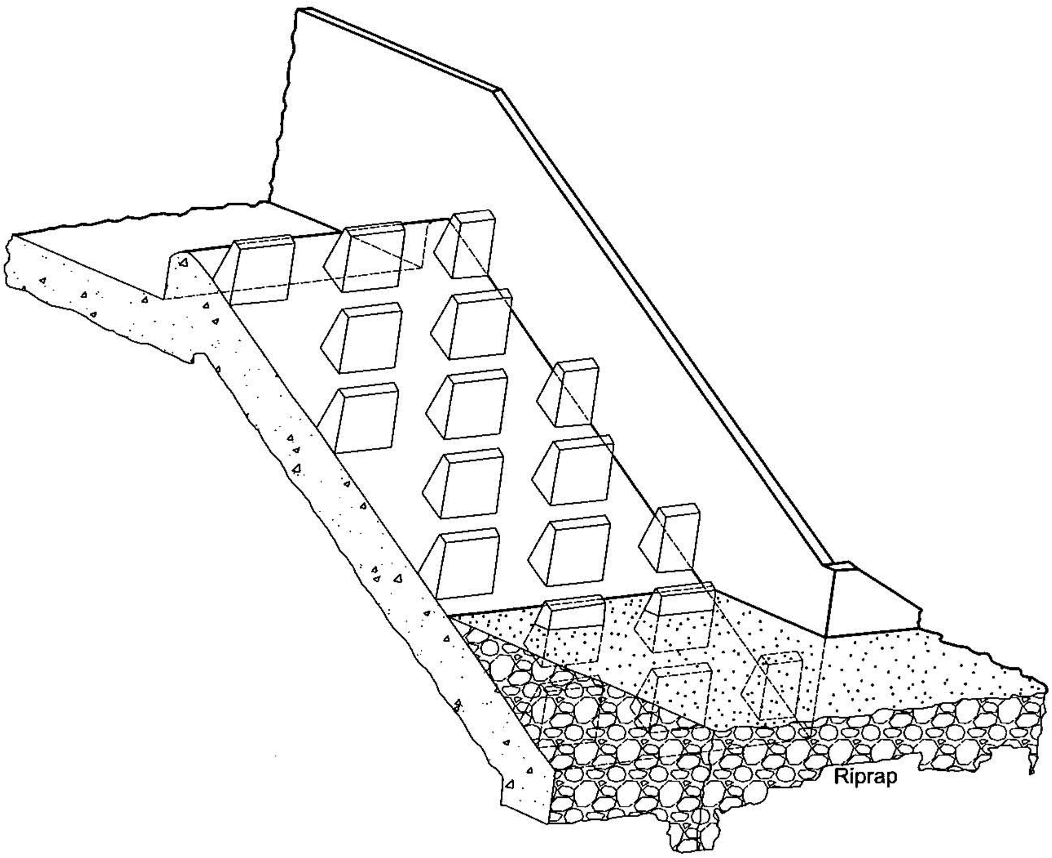
Overview

Baffled chutes are used to dissipate energy at abrupt changes in channel flowline and require no tailwater to be effective. They are generally selected over straight drop spillways for larger drop heights and where lateral channels drop into main channels. Baffle blocks prevent undue acceleration of the flow as it passes down the chute. Since the flow velocities entering the downstream channel are low, no stilling basin is needed. A generic baffled chute is shown in Exhibit 9.3.

Design Criteria

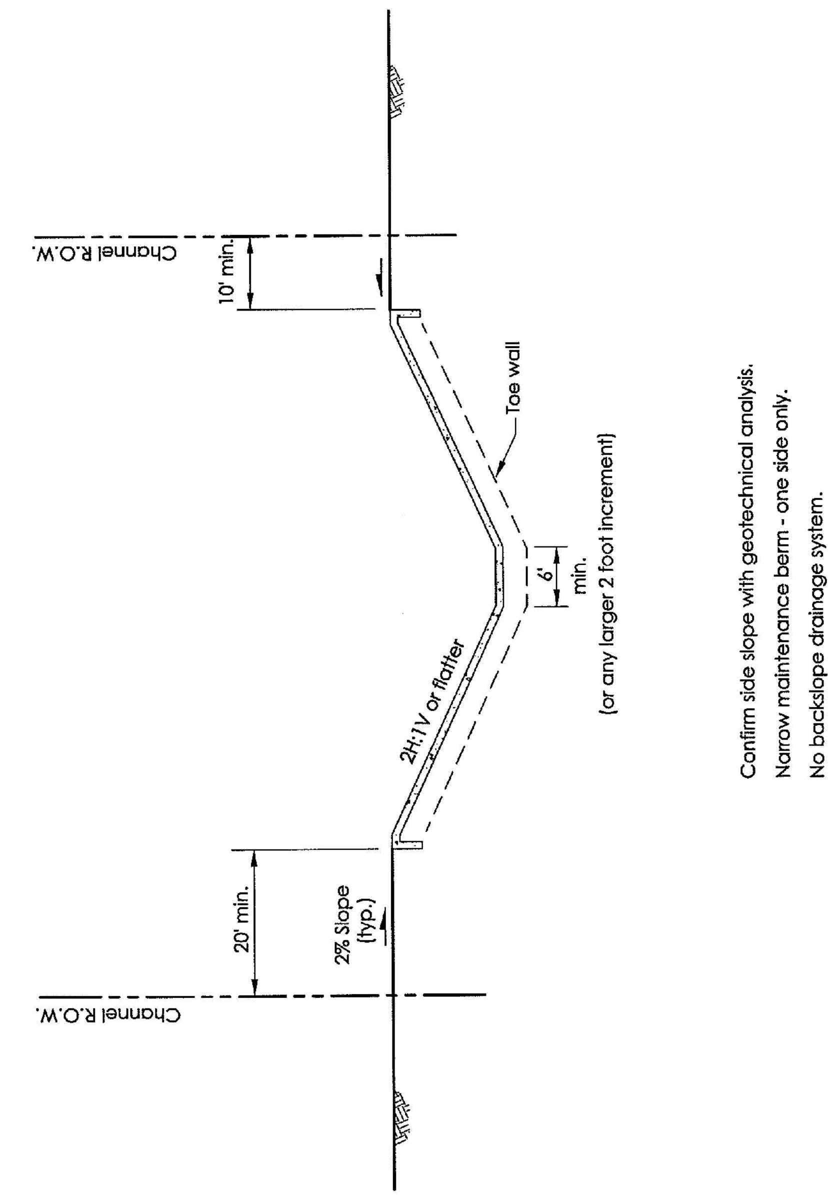
Design criteria for baffled chutes:

- Comply with minimum design criteria for all transition control structures in Section 9.2.1, General Design Criteria.
- Use concrete lining on the entire cross section for the structure.
- Include 20 feet of riprap at the upstream end of the concrete lining to decrease flow velocities and protect the concrete toe from scour (see Section 4.6.4 Stone Riprap Design).
- Use an applicable structural and hydraulic design methodology for baffled chutes.
- Use ultimate watershed conditions for establishing the design flow rate to avoid rebuilding the baffled chute as the watershed develops.

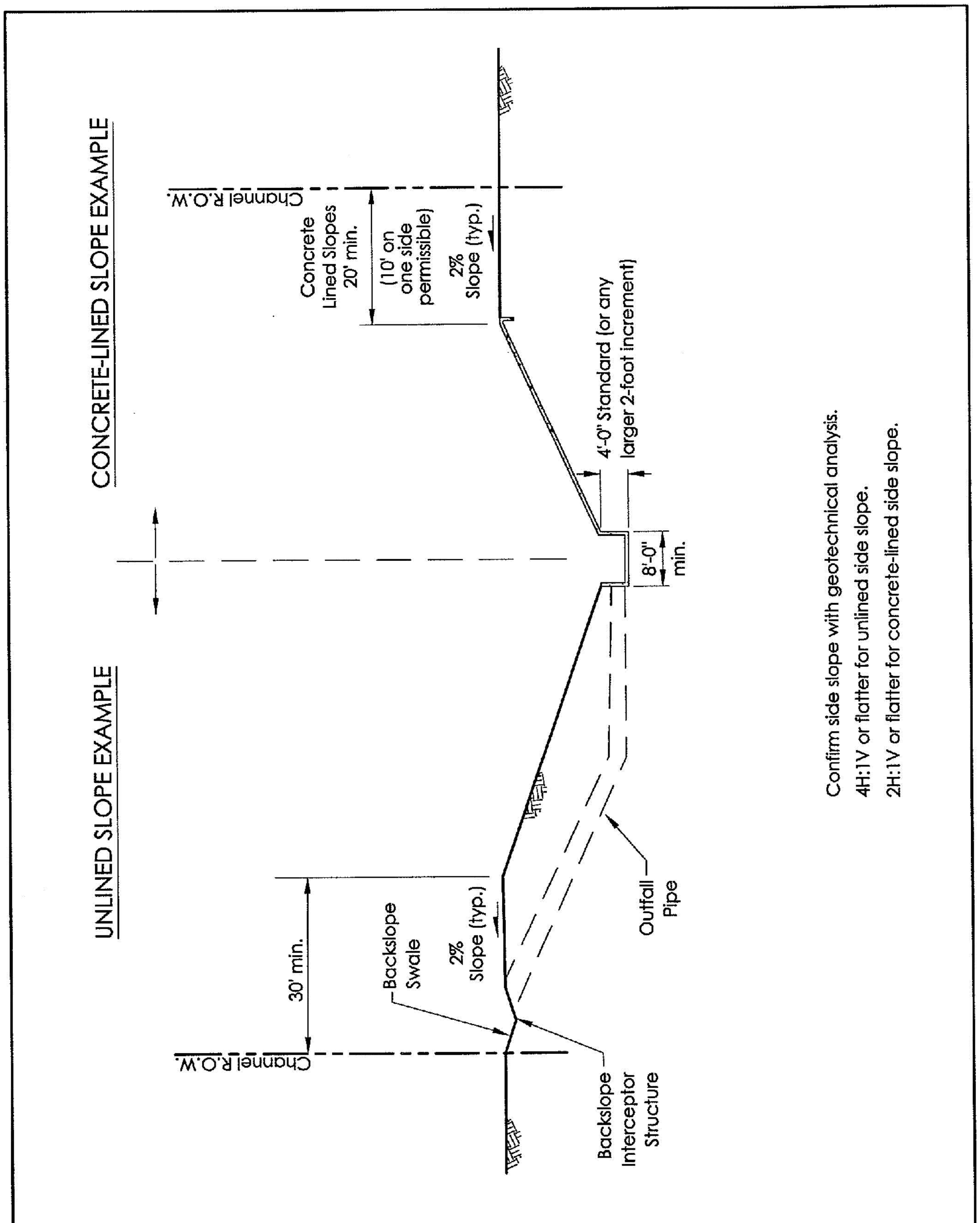


See "Hydraulic Design of Stilling Basins and Energy Dissipators," Engineering Monograph No. 25, U.S. Department of the Interior, Bureau of Reclamation, 1984.

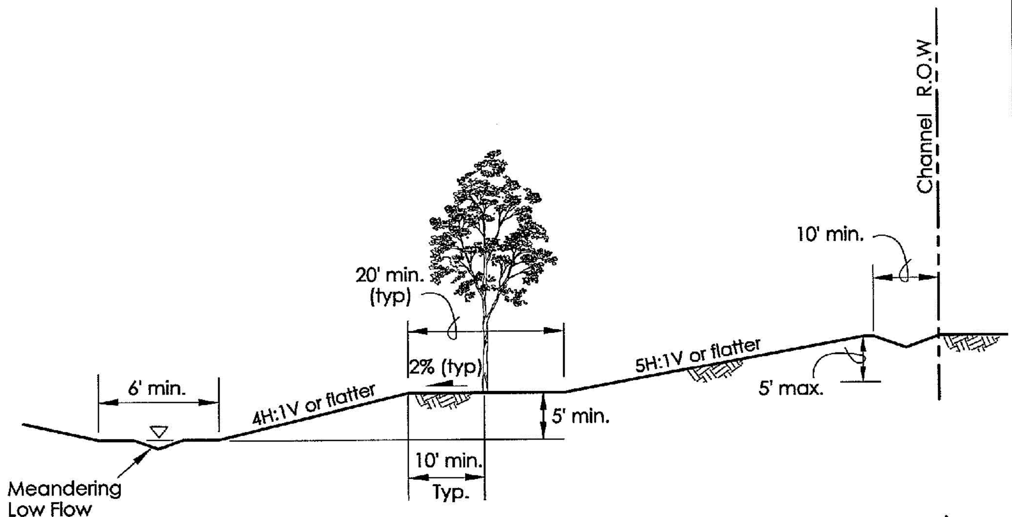
Harris County Flood Control District	POLICY, CRITERIA, & PROCEDURE MANUAL	BAFFLE BLOCK DROP
		DATE: 10/5/04 — EXHIBIT 9-3



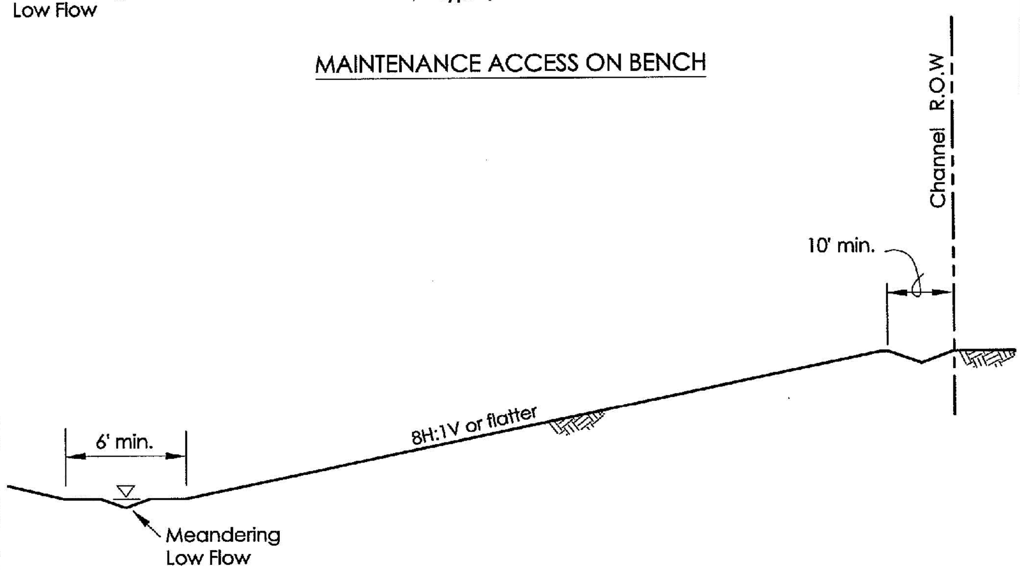
Harris County Flood Control District	POLICY, CRITERIA, & PROCEDURE MANUAL	**TYPICAL CONCRETE-LINED TRAPEZOIDAL CHANNEL SECTION**
		DATE: 10/5/04 — EXHIBIT 5-2



Harris County Flood Control District — POLICY, CRITERIA, & PROCEDURE MANUAL	TYPICAL RECTANGULAR CONCRETE-LINED CHANNEL SECTION	
	DATE: 10/5/04	EXHIBIT 5-4

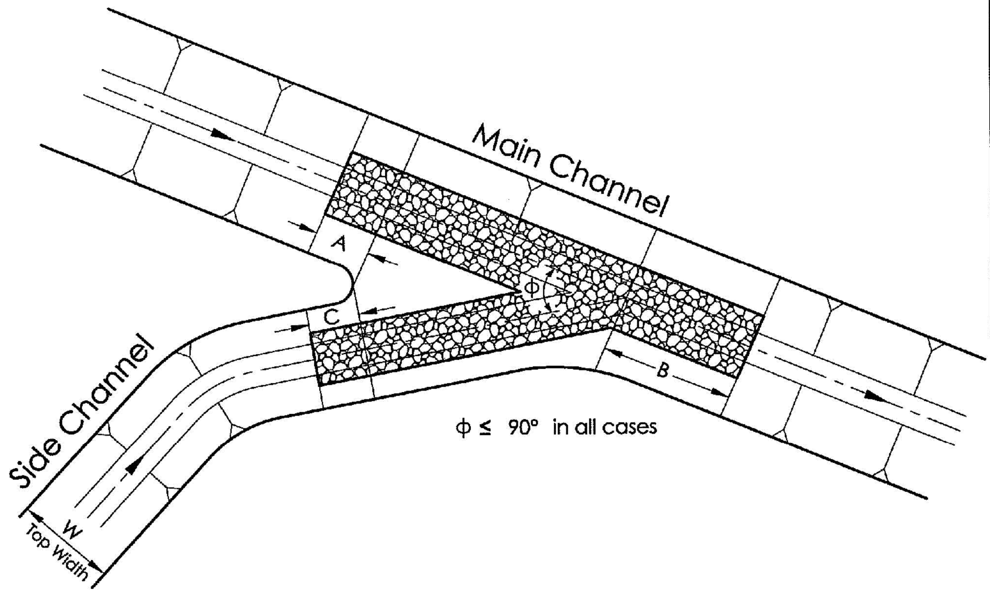


MAINTENANCE ACCESS ON BENCH



MAINTENANCE ACCESS ON SIDE SLOPE

Harris County Flood Control District	**POLICY, CRITERIA, & PROCEDURE MANUAL**	**GRASS-LINED CHANNEL SECTIONS- MAINTENANCE ACCESS ALTERNATIVES**
	DATE: 10/5/04	**EXHIBIT 5-5**



Φ ≤ 90° in all cases

MINIMUM EXTENT OF EROSION PROTECTION

Location	Distance (ft.)
A	20'
B	Larger of 50' or 0.75 X W ÷TAN Φ
C	20'

Extend erosion protection across bottom and at least one-third up the side slopes.

1% Exceedance Velocity * In Side Channel (ft. per sec.)	Angle of Intersection, Φ	
	30°- 45°	45°- 90°
5 or more	Protection	Protection
3 - 5	No Protection	Protection
3 or less	No Protection	No Protection

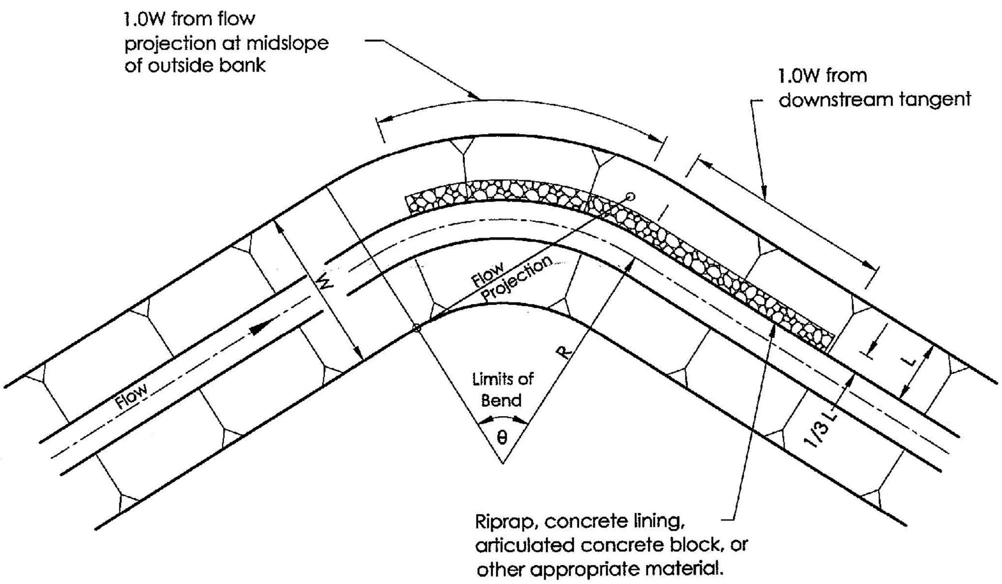
* Assume no backwater from main channel

Harris County Flood Control District

POLICY, CRITERIA, & PROCEDURE MANUAL

EROSION PROTECTION AT CHANNEL CONFLUENCES

DATE: 10/5/04 | **EXHIBIT 5-7**



θ = Bend Angle

R = Radius of curvature

W = Ultimate channel top width

L = Length of side slope

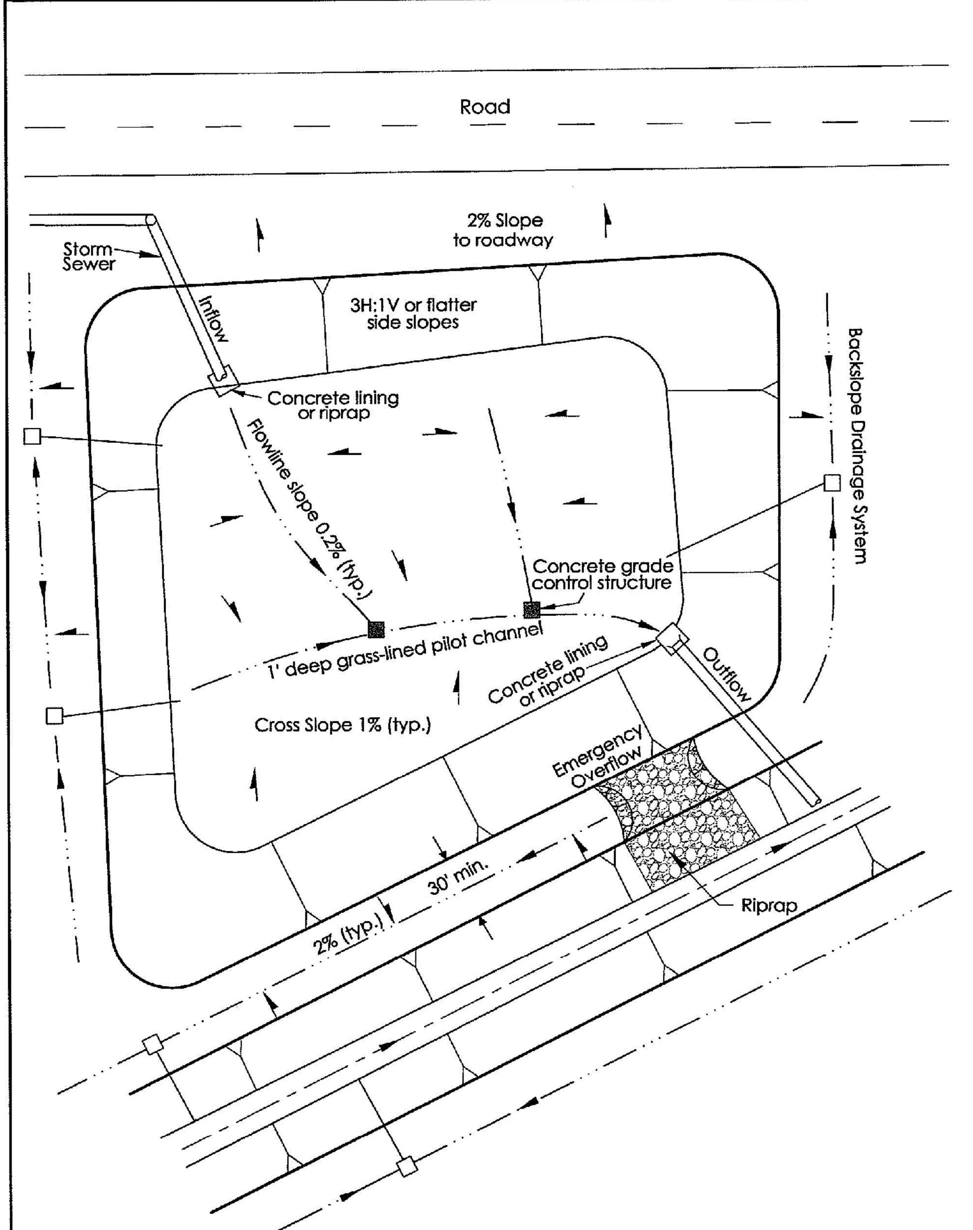
Recommended bend design: $R \geq 3\ W$, $\theta \leq 90°$

Erosion protection required when:

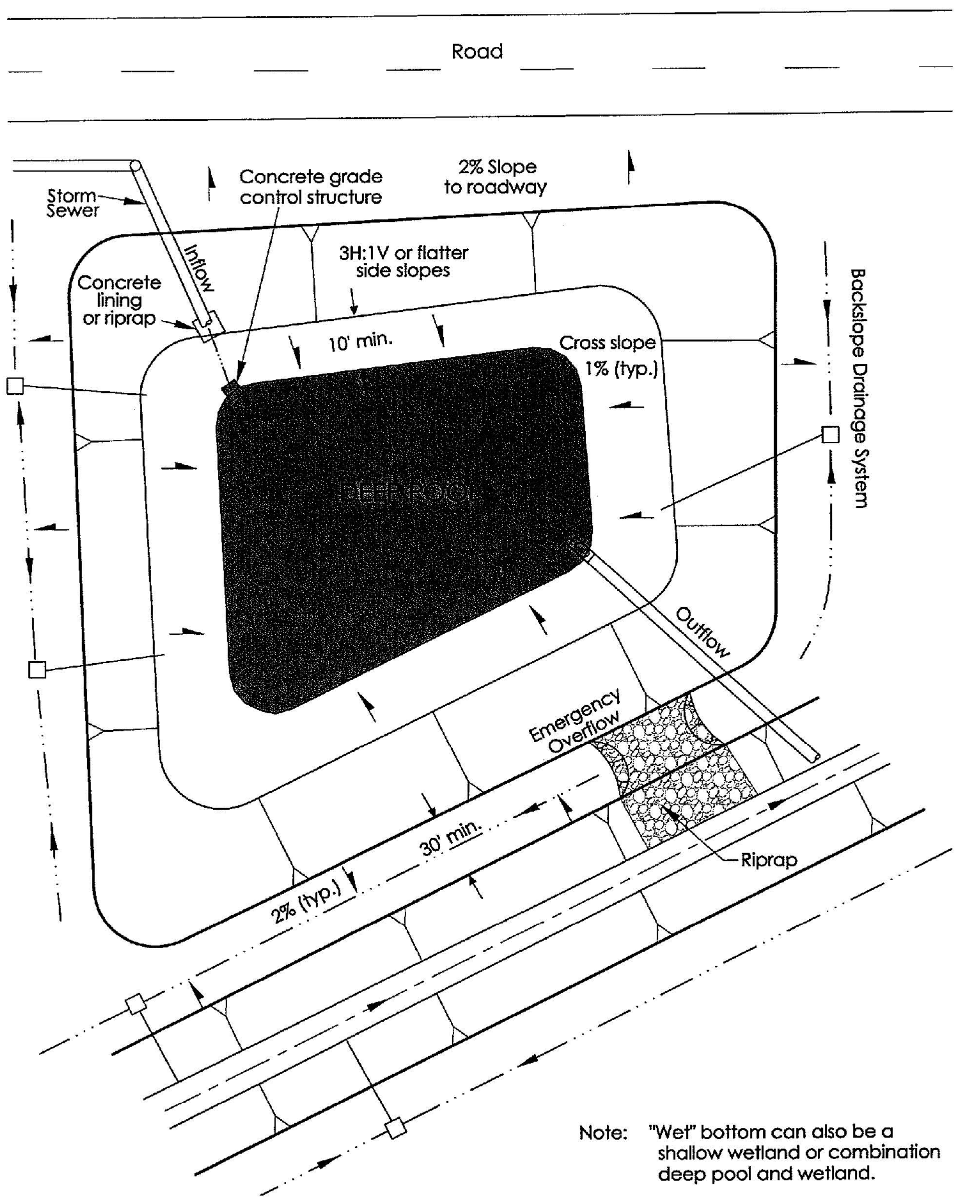
- $R < 3\ W$ and 1% exceedance velocity > 3 feet per second
- Soil type, channel geometry, sinuosity or velocity indicate a potential problem
- Recommended minimum R = W.

Erosion protection in the channel bottom is not shown, but it may be needed.

Harris County Flood Control District	POLICY, CRITERIA, & PROCEDURE MANUAL	EROSION PROTECTION AT CHANNEL BEND	
		DATE: 10/5/04	EXHIBIT 5-8

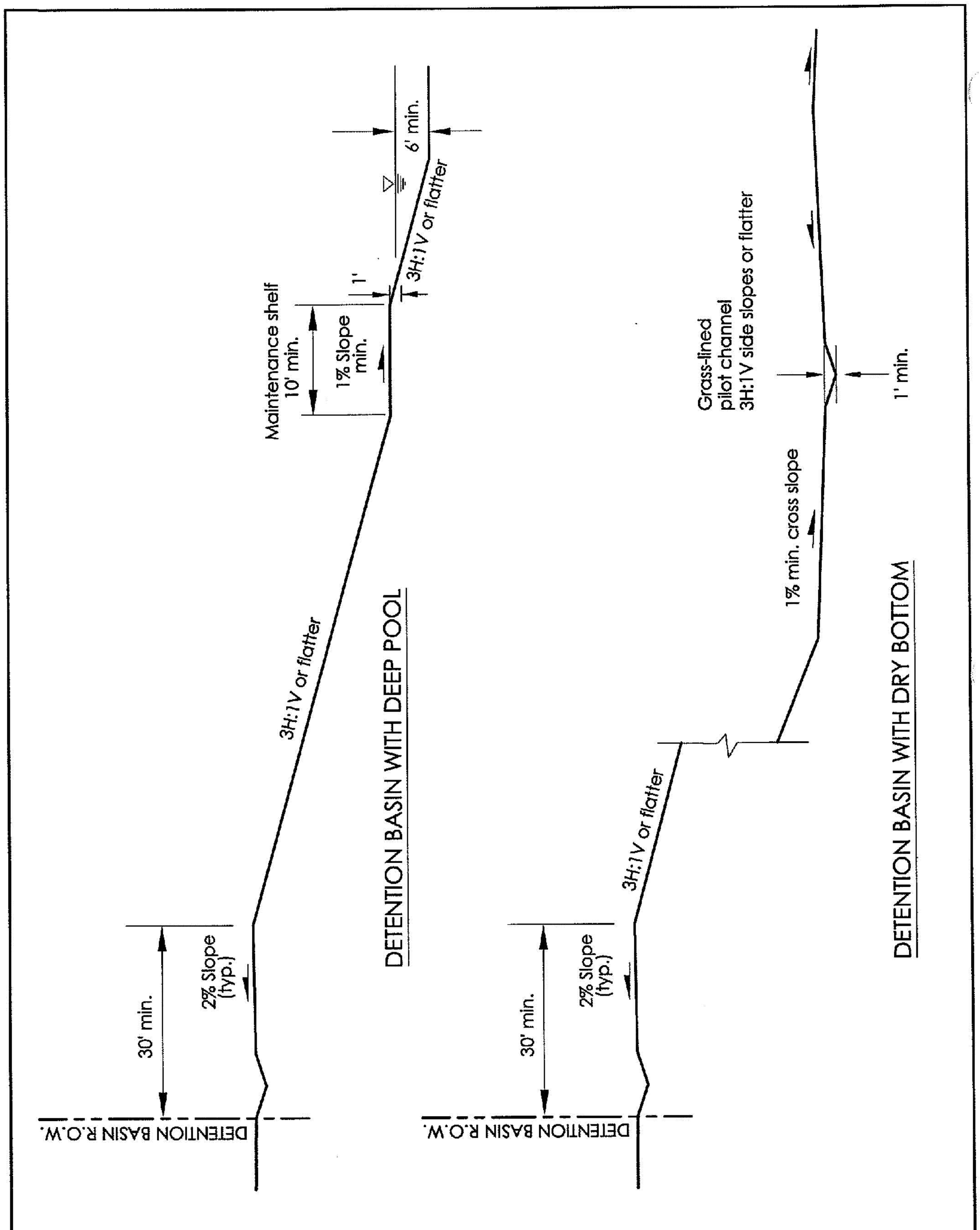


Harris County Flood Control District **POLICY, CRITERIA, & PROCEDURE MANUAL**	**WELL-GRADED, "DRY" DETENTION BASIN**	
	DATE: 10/5/04	**EXHIBIT 6-2**



Note: "Wet" bottom can also be a shallow wetland or combination deep pool and wetland.

Harris County Flood Control District — POLICY, CRITERIA, & PROCEDURE MANUAL	"WET" BOTTOM DETENTION BASIN	
	DATE: 10/5/04	EXHIBIT 6-3



Harris County Flood Control District **POLICY, CRITERIA, & PROCEDURE MANUAL**	**DETENTION BASIN SECTIONS**	
	DATE: 10/5/04	**EXHIBIT 6-4**

Appendix G
Glossary

Abbreviated Drainage Plan

A brief written plan stating and schematically showing how a small proposed **land development project** will satisfy stormwater management requirements of these **Guidelines**. Generally this is applicable only to projects that will be devoid of detention facilities and public stormwater infrastructure of any kind. This may be accomplished with a site plan showing vertical dimensional controls or a site grading plan.

Above-Project Area

Land area(s) adjoining or near a proposed **land development project** that contributes stormwater runoff to, or through, the project at the time of hydrologic analysis or in the future. Above-project areas are included in the **drainage study area**.

Anticipated Development

Full potential urbanization of a **basin** or **watershed** area in compliance with the Comprehensive Plan. Such an area may include one or more subdivisions, one or multiple property holdings, wholly undeveloped land or both developed and undeveloped land areas.

Area Engineer

The Yoakum District Office of the Texas Department of Transportation (TxDOT) operates several Area Offices, each of which has responsibility for several counties. The engineer in charge of each Area Office has the title of **Area Engineer**.

Areas (Hydrologic)

For uniformity of meaning within these **Guidelines** land areas are defined according to the general hierarchy listed below. Specific definitions of each are included in the Glossary.

- **Watershed** (area)
  - **Basin** (area)
    - **Drainage Study Area**
      - **Project Area**
        - **Above-Project Area**
          - **Pathway Area**
            - **Design Drainage Area**

Base Flood

The flood having a one percent chance of being equaled or exceeded in any given year, also known as “100 year” flood.

Basin

A land area making up a portion of a **watershed**. A basin can be thought of as the entire area contributing storm flow to a **watercourse** serving as a tributary to a **primary stream**. Several basins usually comprise a **watershed**.

Buildout Condition

Full completion of any land development project in all of its phases, if any, representing the entire contiguously owned tract(s), whether proposed for near-term or possible future development. This refers to: completion of any single-lot site project; the final completion of any multi-stage project entailing a site project staged over time; or final completion of multiple subdivision projects collectively making up a parent tract (or preliminary plat submittal) representing ownership of an un-platted parcel of land regardless of size.

BFE

Base Flood Elevation

The high water surface elevation(s) along a **watercourse** resulting from the **base flood** passing down that **watercourse**.

CFS

A measure of water flow in cubic feet per second.

CLOMR

Conditional Letter of Map Revision as related to **FEMA** requirements for managing **FEMA**-designated flood prone areas.

Comprehensive Plan

An urban general plan officially adopted by a **City.**

Conveyance Pathway

An identifiable route by which concentrated (non-sheet flow) stormwater will travel within and from a **project area** to a discharge point at a main channel of the Primary Drainage System.

County Engineer

The principal person in Wharton County government who has responsibility for engineering decisions.

Conveyance Pathway Area

See "**Pathway Area**"

Datum

Any level surface to which elevations are referred (for example, mean sea level); is also referred to as datum plane, although it is not actually a plane

Design Drainage Area

The surface area contributing stormwater runoff to any particular point of design in a stormwater management system of any kind. Examples can range in size from the area contributing to a single curb inlet, to that contributing to a flood control facility astride a major stream. Depending on the point of design, the design drainage area can equal an entire **watershed**, an entire **basin**, a **drainage study area**, an **off-project ar**ea, a **project area** or portion(s) of any of these areas.

Detention

Temporary storage and metered release of stormwater

Detention Facility

A permanent facility designed for the temporary storage and metered release of stormwater without creating a permanent pool of water.

Discharge

Stormwater out flow from an area of any kind, or from a storm water feature such as a conduit or a detention facility.

Drainage Development Permit

A permit issued by the **Drainage Regulation Entity** that allows the start of clearing, grubbing, or earthwork as the early stage(s) of a land development project, based on an approved **drainage plan** or an approved **abbreviated drainage plan**.

Drainage Easement

An interest in land granted to the **Drainage Regulation Entity** for the maintenance of a **drainage facility**, on which certain uses are prohibited; and providing for the entry and operation of machinery and vehicles for maintenance purposes.

Drainage Facilities

All elements (public and private) necessary to manage and convey stormwater runoff from its initial contact with earth to its disposition in a **watercourse** making up the primary drainage system of the **Drainage Regulation Entity**. These may include but are not limited to storm sewers, improved channels, unimproved drainage ways, areas within **drainage easements** or **drainage right-of-way** providing concentrated or overland sheet flow, and all appurtenances to the foregoing, such as inlets, manholes, junction boxes, headwalls, culverts, etc.

Drainage Plan

A detailed representation of how stormwater will be managed as part of a proposed **land development project** (site or subdivision). Usually accompanied by (or incorporated into) an engineering report, it is to be based on an approved **preliminary drainage plan.**

Drainage Report

A report, prepared by a Registered Professional Engineer, that presents the **drainage plan** for a **land development project** (site or subdivision) in compliance with the provisions of these **Guidelines**. It must document the hydrologic and hydraulic analyses accomplished to address the **project area**, **above-project area**(s) and **pathway area**(s), and any watercourse conveying stormwater to or from the **project area**.

Drainage Regulation Entity

The unit of local government (county or city) with legal authority to regulate engineering criteria.

Drainage Review Authority

The principle person in local government (county or city) who has responsibility for engineering decisions.

Drainage Study

See "**Drainage Report**".

Drainage Study Area

The full extent of land area that must be analyzed for the effects of stomrwater runoff, whether part of a project, upland of the project, or contributing stormwater runoff to the **conveyance pathway** downstream of the project. The drainage study area is equal in size to the sum of the **project area,** the **above-project area**, if any, and the **pathway area**, if any.

Drainage Right-of-Way

An area of land dedicated to the **Drainage Regulation Entity** for the purposes of conveying and containing stormwater flow, constructing drainage facilities, and/or allowing entry and/or operation of equipment for maintaining such drainage features and facilities.

Elevation

The vertical distance from a datum, to a point or object. For example, if the elevation of point “A” is 802.46 feet, point “A” is 802.46 feet above some datum.

Encroachment

Existing or proposed buildings, foundations, drainage structures, streets (including bridges and culverts), utilities, or earthwork of any kind which is situated in **floodplain**, or **flood fringe** areas, the geographic limits of which are defined on the official **Flood Insurance Rate Maps** of the **Wharton County**.

Equal Encroachment

Equitable **encroachment** into **floodplain** or **flood fringe** areas along a significant reach of both sides of a **watercourse**, as a function of “low side” and “high side” hydrologically proportionate areas.

Engineer

A Registered Professional Engineer duly authorized and licensed, under provisions of the Texas Engineering Practice Act, to practice the profession of engineering.

Erosion

The process whereby the surface of the earth is loosened and carried away by the action of wind, water, gravity, ice, or a combination thereof.

Existing Condition

The hydrologic condition of the **project area** or the **drainage study area** that exists (or existed) prior to any proposed land development work and at the time for which a hydrologic analysis is conducted. Where man-made topographic features predate adoption of these **Guidelines**, such features shall be considered “exiting condition”.

Extraterrritorial Jurisdiction (ETJ)

Within the terms of the Texas Municipal Annexation Act, means the unincorporated area, not a part of any other city, which is contiguous to the Corporate Limits of a **City**, the outer boundaries of which are measured from the extremities of the corporate limits of the **City** outward for such distances as may be stipulated in the Texas Municipal Annexation Act, in which area, within the terms of the act, the **City** may enjoin the violation of its subdivision control ordinance.

FEMA

Federal Emergency Management Agency of the US Government.

Flood Insurance Rate Map (FIRM)

Any of a series of maps published by **FEMA** that depicts the geographic limits of flood prone areas along the principal **watercourse**s of the **Drainage Regulation Entity**, for the purpose of identifying those areas in which property owners are eligible to participate in the National Flood Insurance Program.

Floodplain

Overbank areas along a **watercourse** that are subject to inundation by stormflow due to unusually larger storms events.

Flood Study

The official study, or collection of studies, that defines the **flood plains, flood fringe,** and **floodways** of the primary drainage system and tributaries thereof as required in connection with the National Flood Insurance Program sponsored by **FEMA**.

Floodway

The channel and adjacent overbank areas of a river or other **watercourse** that may not be filled or hydraulically altered if such fill or alterations will cause a cumulatively increase in the **base flood elevation** of a specified amount.

Freeboard

That portion of a channel bank, detention embankment, or other stormwater management facility that is above the water surface elevation expected to be generated by the design storm for which the facility is designed.

Guidelines

The design guidelines referenced in this document: "Wharton County Drainage Criteria Manual."

Hydraulics

A branch of science that deals with practical applications (such as the transmission of energy or the efforts of flow) of liquid (such as water) in motion

Hydrology

A science dealing with the properties, distribution, and circulation of water on the surface of the land, in the soil and underlying rocks, and in the atmosphere

Land Development Project

Any proposed site development or subdivision project requiring building permit(s) or platting under provisions of **Drainage Regulation Entity** ordinances.

Legal Lot

A parcel of land having been divided from a parent tract via a plat duly processed and approved by the **Drainage Regulation Entity**, and filed of record in county records under the platting provisions of Texas State Law.

LOMA Letter of Map Amendment as related to **FEMA** requirements for managing **FEMA**-designated flood prone areas

LOMR

Letter of Map Revision as related to **FEMA** requirements for managing **FEMA**-designated flood prone areas

Lowest Floor

The lowest floor, or the lowest enclosed area (including basement), of a structure. An unfinished or flood resistant enclosure, usable solely for the parking of vehicles, building access or storage, in an area other than a basement area, is not considered a building's lowest floor, provided that such enclosure is not built so as to render the structure in violation of the applicable non-elevation design requirements of **Drainage Regulation Entity** ordinances.

Master Drainage Plan

An official plan of the **Drainage Regulation Entity** for comprehensive management of stormwater runoff in an entire **basin** or **watershed,** or in specific **reaches** thereof.

Mean Sea Level (MSL)

The average height of the surface of the sea for all stages of the tide taken over a 19-year period.

Natural Land

The cover and topography of land before any man-made changes that would substantively affect the path or intensity of stormwater runoff.

Natural Watercourse

A stream, waterway, or channel more or less in the alignment created by natural forces, with our without man-made alteration of its surfacing and configuration at limited locations.

Pathway Area

Land area(s) that drain to the **conveyance pathway** of a project, but that are not included in the **project area** or **above-project** area(s). See **conveyance pathway area**.

Primary Streams (Watercourses)

Major water courses or streams in the Wharton County region as listed in Appendix B.

Preliminary Drainage Plan

See "**Preliminary Drainage Report**"

Preliminary Drainage Report

A report showing a schematic representation of how stormwater will be managed as part of a proposed land development project. It will document pertinent topographic, hydrologic, and land ownership characteristics of all land areas contributing stormflow to a **project area**, as well as all hydrologic parameters proposed for analysis of design stormflow throughout the project.

Project Area

The entire land area of a proposed site development or subdivision project, at **buildout condition**, into which buildings, structures, and/or street and utility facilities are to be constructed. This area(s), together with any **above-project area**(s) and **pathway area**(s) make up the **drainage study area** that must be considered in developing plans for stormwater management facilities for the project.

Project Site

See "**Project Area**"

Reach

A length or portion of a **watercourse**, whether wholly natural or influenced by man-made improvements or alterations.

Regional Detention

A flood control facility approved by the **Drainage Regulation Entity** as a mechanism for managing stormwater runoff form a large land area comprised of one or more subdivisions, one or multiple property holdings, developed and undeveloped land areas, or any combination of such areas.

Retention Facility

A facility that provides for the storage of stormwater flows by means of a permanent pool of water or a permanent pool in conjunction with a temporary storage component.

Right-of-Way

Land set aside for street and storm drain facilities or utilities, or exclusively for stormwater management purposes.

Rural Subdivision

An area of land divided by platting into lots none of which are smaller than one (1) acre, and which is served by roadways having a rural cross section (one characterized by presence of roadside ditches and no curb and gutter). See also **Urban Estates.**

Sedimentation

Deposits of detached soil particles or rock fragments after being transported from their site or origin by runoff water.

Site

See "**Site Project**".

Site Project

A land area consisting of a single platted lot or two or more contiguous platted lots upon which a building project is planned, consisting of building structures, parking, and other facilities and exclusive of public streets. A site project may or may not include public utilities situated in easements, or stormwater management facilities situated in drainage right-of-way. See "**Site**"

Special Design

Any stormwater management facility or technique the design of which is not specifically addressed by these **Guidelines** or the TxDOT Technical Specifications.

Standard Specifications for Construction

See **Technical Specifications**

Structure

A walled and roofed building that is principally above the ground, as well as a manufactured home.

Study Limits

Associated with a drainage study for a **drainage report**, this is the geographic limits of the hydrologic and hydraulic analyses that are required for the study.

Subdivision Project

A land development project involving the division of land into lots and ROW for public streets and utilities or the dividing of land into individual lots for near term construction or planned long term construction of **site projects**.

Surveyor

A Registered Public Surveyor or Registered Land Surveyor as licensed by the State of Texas.

Swale

A shallow drainage way characterized as having a "V" shape the sides of which have flat slopes, generally on the order of sides 3 horizontal to 1 vertical (3:1) or flatter.

Technical Design Summary

A drainage report format that may be used in lieu of a traditional prose report. Following a question/answer process, it is to use the forms provided in Appendix D, with attachments as needed.

Tributaries

Waterways, watercourses, streams, or creeks that directly flow into the Primary Watercourses of the Wharton County region. Some may be referred to by a name on maps or other reference.

TxDOT Texas Department of Transportation.

Ultimate Development

This term generally relates to the extent to which impervious materials and plant growth will, at some future time, cover land contributing stormwater runoff to one or more design points in a stormwater management system. Of necessity this requires some plan or a series of assumptions about future characteristics of undeveloped areas. See **Anticipated Development**

Urban Estates

A class of zoning resulting in single family homes on relatively large lots, generally one acre or larger. See **Rural Subdivision**.

Watercourse

Any depression, channel, storm sewer, or culvert serving to give direction to a current of stormwater.