DRAINAGE CRITERIA MANUAL
HAYS COUNTY, TEXAS

HAYS COUNTY
STATE OF TEXAS

September 2025

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1 Chapter 1 Introduction

1.1 Purpose

The purpose of this Drainage Criteria Manual is to establish standard criteria and procedures for the analysis, design and construction of drainage systems within Hays County in accordance with Hays County Development Regulations.

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 prior formal approval of Hays County Reviewing Authority.

1.2 Policies

1.2.1 Application

This Drainage Criteria Manual shall govern planning and design of drainage infrastructure within the unincorporated jurisdiction of Hays County. Applicant is responsible to meet any local incorporated jurisdiction amendments to these criteria.

1.2.2 General Criteria

This Drainage Criteria Manual represents the application of accepted principles of stormwater drainage engineering and is basic information obtainable from standard drainage publications on drainage design. The policy criteria in the following sections provide the underlying principles by which all drainage facilities shall be planned and designed. All plans and calculations must be signed and sealed by a Professional Engineer licensed in the State of Texas.

1.2.3 County Ordinances

In addition to this Drainage Criteria Manual, all engineering design shall reference and incorporate, as necessary, all federal and state requirements, county ordinances, and other jurisdictional entity requirements.

1.3 Related Permitting

1.3.1 Hays County Development Regulations

Hays County requires a Development Authorization for all development in the unincorporated areas. Development is defined as “all land modification activities, including the construction of or additions to buildings, roadways, dams, paved storage areas, parking lots, storm water management facilities and other impervious structures or surfaces.” All divisions of land within Hays County are subject to the Hays County Development Regulations.

1.3.2 FEMA-Designated Floodplains

Based on long experience with helping offset the costs suffered by flood victims, the Federal Emergency Management Agency (FEMA) 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 nationwide displayed on Flood Insurance Rate Maps (FIRMs). Today Digital Flood Insurance Rate Maps (DFIRMs) or the National Flood Hazard Layer (NFHL) are used to display FEMA designated flood-prone areas.

A floodplain permit system ensures compliance with Hays County Flood Damage and Prevention Ordinance Chapter 735 of the Hays County Development Regulations and FEMA regulations.

1.3.3 Storm Water Quality

The Texas Commission on Environmental Quality (TCEQ) Edwards Aquifer Protection Program, regulates activities that have the potential to pollute the Edwards Aquifer. The design engineer must check the requirements of the program for compliance. The Edwards Aquifer extents are displayed in Appendix A.

All construction sites within Hays County are required to submit their complete SWPPP, along with the weekly SWPPP inspection reports.

A secondary TCEQ site notice shall be posted at all entrances to a construction site that are not visible from the main or posted entrance.

1.3.4 Environmental

When a project to modify a natural channel is proposed, the design engineer must check the requirements of Clean Water Act Section 404, Permits for Dredged or Fill Material. If required, a permit shall be obtained from the U.S. Army Corps of Engineers (USACE) by the design Engineer.

2 Chapter 2 Hydrologic Analysis

2.1 General

Hydrology deals with estimating peak flows (discharge), 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 to consider include:

1. Rainfall amount and storm distribution
2. Drainage area size, shape, and orientation
3. Ground cover and soil type
4. Slopes of terrain and stream channel(s)
5. Antecedent moisture condition
6. Rainfall abstraction rates (Initial and constant)
7. Storage potential (floodplains, ponds, wetlands, aquifer recharge, channels, etc.)
8. Watershed development potential
9. Characteristics of the local drainage system

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

- Rational Method
- Texas Department of Transportation (TxDOT) Regression
- Statistical Analysis of Stream Gage Data
- Hydrograph Method
    - Natural Resources Conservation Service (NRCS) Unit Hydrograph Method
    - Snyder's Unit Hydrograph Method

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 summarizes the applicability of various hydrologic methods to Hays County.

Table 1.3.4-1: Constraints on Using Recommended Hydrologic Methods

Method	Size Limitations	Comments
Rational Method[1] (Section 2.2.2)	0-200 acres	Method for estimating peak flows and the design of small site or subdivision storm drainage systems.
TxDOT Regression Equations[2] (Section 2.2.3)	10 to 100 square miles	Method for estimating peak flows for rural design applications for comparison purposes only.
Statistical Analysis of Stream Gage Data[3] (Section 2.2.4)	Any size	A sufficiently large sample of annual peak stream flow data should be available
NRCS Unit Hydrograph[4] (Section 2.2.5.c)	Any size	Method for estimating peak flows and hydrographs in urbanized conditions
Snyder's Unit Hydrograph[5] (Section 2.2.5.d)	>100 acres	Method for estimating peak flows and hydrographs with approval of County.

[1] Rational method is not permitted where storage detention is required.
[2] Regression method is not permitted where storage detention is required.
[3] Statistical analysis method may not be useful when stream flow is controlled by dams
[4]This refers to the NRCS methodology included in many available programs (such as HEC-HMS).
[5] This refers to the Snyder's methodology included in many available programs (such as HEC-HMS).

Other hydrologic methods are subject to approval by the reviewing agency prior to submission and must be calibrated (tested for accuracy and reliability) to local conditions. The engineer may use local stream gage data, if available, to develop peak discharges and hydrographs. See standard hydrology textbooks for statistical procedures to estimate design flood events from stream gage data.

Note: Realize 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.

2.2 Design Procedure

2.2.1 Rainfall

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 flood event category nomenclature in this manual uses the percent annual chance exceedance (ACE) terminology and instead of the classic annual recurrence interval terminology as shown in Table 2.2.1-1 below.

Table 2.2.1-1: Flood Event Category Nomenclature

Classic Terminology	Percent Annual Chance Exceedance
2-Year Flood	50% ACE
5-Year Flood	20% ACE
10-Year Flood	10% ACE
25-Year Flood	4% ACE
50-Year Flood	2% ACE
100-Year Flood	1% ACE
250-Year Flood	0.4% ACE
500-Year Flood	0.2% ACE

On September 27, 2018, the National Oceanic and Atmospheric Administration (NOAA) published new precipitation-frequency values for Texas. This new publication, Atlas 14, is a reassessment of historical rainfall data up to 2017, adding an additional twenty years of record to the analysis. The statistical rainfall data has been obtained from the NOAA Atlas 14, Volume 11 Precipitation-Frequency Atlas of the United States, Texas. The rainfall depths vary spatially throughout Hays County and generally decrease in the northwestern portion of the county. Two rainfall data tables are provided for the northern Colorado River basin watersheds and for the southern Guadalupe-Blanco River basin watersheds (Tables 2.2.1-2 and 2.2.1-3). A map of the location can be found in Appendix A-2. Develop rainfall hyetographs for Hays County using the frequency storm methodology for the 50%, 20%, 10%, 4%, 2%, 1%, 0.4%, and 0.2% ACE events.

Table 2.2.1-2: 2018 Atlas 14 Hays County Depth-Duration-Frequency Colorado River Basins

Annual Chance Exceedance	Depth of Precipitation (inches)							
	5-min	15-min	60-min	2-hr	3-hr	6-hr	12-hr	24-hr
50%	0.52	1.05	1.94	2.40	2.68	3.15	3.62	4.11
20%	0.67	1.34	2.49	3.12	3.50	4.15	4.79	5.46
10%	0.80	1.60	2.97	3.79	4.30	5.18	5.98	6.82
4%	0.98	1.96	3.66	4.80	5.55	6.78	7.87	8.94
2%	1.13	2.25	4.20	5.66	6.64	8.24	9.58	10.80
1%	1.29	2.56	4.80	6.64	7.92	9.97	11.60	13.10
0.20%	1.71	3.39	6.55	9.53	11.70	15.10	17.60	19.70

**Point Location: Centroid of the Colorado River basins*

Table 2.2.1-3: 2018 Atlas 14 Hays County Depth-Duration-Frequency Blanco River Basins

Annual Chance Exceedance	Depth of Precipitation (inches)							
	5-min	15-min	60-min	2-hr	3-hr	6-hr	12-hr	24-hr
50%	0.53	1.06	1.97	2.45	2.74	3.23	3.70	4.20
20%	0.67	1.34	2.50	3.14	3.54	4.22	4.89	5.60
10%	0.79	1.58	2.96	3.80	4.33	5.23	6.10	7.00
4%	0.97	1.93	3.62	4.78	5.55	6.84	8.01	9.17
2%	1.11	2.21	4.15	5.63	6.65	8.31	9.74	11.10
1%	1.26	2.51	4.72	6.59	7.92	10.00	11.80	13.40
0.20%	1.63	3.23	6.28	9.25	11.40	14.90	17.70	20.10

**Point Location: Centroid of the Blanco River basins*

Table 2.2.1-4: 2018 Atlas 14 Hays County Intensity-Duration-Frequency Colorado River Basins

Annual Chance Exceedance	Intensity of Precipitation (inches per hour)							
	5-min	15-min	60-min	2-hr	3-hr	6-hr	12-hr	24-hr
50%	6.26	4.19	1.94	1.20	0.89	0.53	0.30	0.17
20%	8.04	5.36	2.49	1.56	1.17	0.69	0.40	0.23
10%	9.58	6.38	2.97	1.90	1.43	0.86	0.50	0.28
4%	11.80	7.84	3.66	2.40	1.85	1.13	0.65	0.37
2%	13.60	9.00	4.20	2.83	2.21	1.38	0.80	0.45
1%	15.50	10.20	4.80	3.32	2.64	1.66	0.96	0.55
0.20%	20.60	13.60	6.55	4.77	3.90	2.51	1.46	0.82

**Point Location: Centroid of the Colorado River basins*

Table 2.2.1-5: 2018 Atlas 14 Hays County Intensity-Duration-Frequency Blanco River Basins

Annual Chance Exceedance	Intensity of Precipitation (inches per hour)							
	5-min	15-min	60-min	2-hr	3-hr	6-hr	12-hr	24-hr
50%	6.31	4.25	1.97	1.22	0.91	0.54	0.31	0.18
20%	8.03	5.37	2.50	1.57	1.18	0.70	0.41	0.23
10%	9.50	6.34	2.96	1.90	1.44	0.87	0.51	0.29
4%	11.60	7.73	3.62	2.39	1.85	1.14	0.67	0.38
2%	13.40	8.86	4.15	2.82	2.21	1.39	0.81	0.46
1%	15.10	10.00	4.72	3.30	2.64	1.68	0.98	0.56
0.20%	19.60	12.90	6.28	4.63	3.81	2.49	1.47	0.84

**Point Location: Centroid of the Blanco River basins*

Apply a 24-hour duration storm with a temporal distribution using balanced storm method (also known as frequency storm in HEC-HMS). Apply point rainfall for basins up to a total cumulative area of approximately 10 square miles. Beyond 10 square miles, use the areal reduction curves published by the U.S. Department of Agriculture, Soil Conservation Service to reduce the rainfall totals (Figure 15, Technical Paper No. 40, 1961). For the frequency storm application, derive the areally reduced discharges for the analysis watershed using the depth-area analysis in HEC-HMS. The depth-area analysis takes into account the storm area for the depth-area reduction factor. The analysis requires the specification of analysis points for the evaluation of discharges. Junction locations with a drainage area greater than 10 square miles require evaluation points in the hydrologic model. When the analysis computes, it automatically generates frequency storms for the specified meteorological model but with the appropriate storm area for each analysis point.

2.2.2 Rational Method

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 consider these precautions:

- In determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should account for future changes in land use, as appropriate, that might occur during the service life of the proposed facility.
- The Rational Method uses a composite C and a single T_c 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 divide the basin into sub-drainage basins.
- The tables included in this section are provided 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.

Application

Use the Rational Method 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 for the Rational Method is 200 acres.

Do not use the Rational Method for storage design or any other application that requires a more detailed routing procedure.

When conveying offsite runoff through a proposed development storm infrastructure, consider the C value for the offsite basin to be fully developed based on the most applicable land use (as determined by the local review authority).

Use caution when applying the Rational Method for analysis or design of bridges, culverts, or storm drains that may act as restrictions causing storage, which can impact the peak rate of discharge.

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).

Rational Formula (Eq 2.2.2-1)

$$Q = CiA$$

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)*

Drainage areas may have more than one land use, calculate a composite runoff coefficient using the weighted method to use in the analysis. Suggested coefficients for various land use types are provided in the Table 2.2.2-1.

Table 2.2.2-1: Rational Method Runoff Coefficients

Cover Description	**Annual Chance Exceedance Events**						
Cover type and slope condition	50%	20%	10%	4%	2%	1%	0.2%
Open space (lawns, parks, golf courses, etc.)							
Poor condition (grass cover < 50%):							
Flat, 0–2% Slopes	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2–7% Slopes	0.37	0.4	0.43	0.46	0.49	0.53	0.61
Steep, over 7% Slopes	0.4	0.43	0.45	0.49	0.52	0.55	0.62
Fair condition (grass cover 50% to 75%):							
Flat, 0–2% Slopes	0.25	0.28	0.3	0.34	0.37	0.41	0.53
Average, 2–7% Slopes	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7% Slopes	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good Condition (grass cover > 75%):							
Flat, 0–2% Slopes	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2–7% Slopes	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, over 7% Slopes	0.34	0.37	0.4	0.44	0.47	0.51	0.58
Cultivated land:							
Flat, 0–2% Slopes	0.31	0.34	0.36	0.4	0.43	0.47	0.57
Average, 2–7% Slopes	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, over 7% Slopes	0.39	0.42	0.48	0.48	0.51	0.54	0.61
Pasture or range land:							
Flat, 0–2% Slopes	0.25	0.28	0.3	0.34	0.37	0.41	0.53
Average, 2–7% Slopes	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7% Slopes	0.37	0.4	0.42	0.46	0.49	0.53	0.60
Wood or forest land:							
Flat, 0–2% Slopes	0.22	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2–7% Slopes	0.31	0.34	0.36	0.4	0.43	0.47	0.56
Steep, over 7% Slopes	0.35	0.39	0.41	0.45	0.48	0.52	0.58
Developed land:							
Asphaltic	0.73	0.77	0.81	0.86	0.9	0.95	1.00
Concrete	0.75	0.8	0.83	0.88	0.92	0.97	1.00

Cover Description (continued)	**Annual Chance Exceedance Events**						
Cover type and slope condition	50%	20%	10%	4%	2%	1%	0.2%
Single family residential:							
Lots greater than 1 acre (15% impervious)							
Flat, 0–2% Slopes	0.33	0.36	0.38	0.42	0.45	0.49	0.60
Average, 2–7% Slopes	0.39	0.43	0.45	0.49	0.52	0.56	0.64
Steep, over 7% Slopes	0.43	0.46	0.48	0.52	0.55	0.60	0.66
Lots 1/4 to 1 acre (25% impervious)							
Flat, 0–2% Slopes	0.38	0.41	0.43	0.48	0.51	0.55	0.65
Average, 2–7% Slopes	0.44	0.47	0.49	0.54	0.57	0.61	0.69
Steep, over 7% Slopes	0.47	0.50	0.52	0.57	0.60	0.64	0.70
Lots 1/8 to 1/4 acre (40% impervious)							
Flat, 0–2% Slopes	0.45	0.49	0.51	0.56	0.59	0.63	0.72
Average, 2–7% Slopes	0.50	0.54	0.56	0.60	0.64	0.68	0.75
Steep, over 7% Slopes	0.52	0.56	0.58	0.63	0.66	0.71	0.76
Lots 1/8 acre or less (65% impervious)							
Flat, 0–2% Slopes	0.58	0.62	0.64	0.69	0.73	0.77	0.84
Average, 2–7% Slopes	0.60	0.65	0.67	0.72	0.76	0.80	0.85
Steep, over 7% Slopes	0.62	0.66	0.69	0.73	0.77	0.82	0.86
Multiple family residential (65% impervious):							
Flat, 0–2% Slopes	0.58	0.62	0.64	0.69	0.73	0.77	0.84
Average, 2–7% Slopes	0.60	0.65	0.67	0.72	0.76	0.80	0.85
Steep, over 7% Slopes	0.62	0.66	0.69	0.73	0.77	0.82	0.86
Commercial and General Office (80% Impervious):							
Flat, 0–2% Slopes	0.65	0.70	0.72	0.77	0.81	0.86	0.91
Average, 2–7% Slopes	0.67	0.71	0.74	0.79	0.83	0.87	0.92
Steep, over 7% Slopes	0.67	0.72	0.75	0.80	0.83	0.88	0.92
Industrial (90% Impervious):							
Flat, 0–2% Slopes	0.70	0.75	0.78	0.83	0.87	0.91	0.95
Average, 2–7% Slopes	0.71	0.76	0.79	0.83	0.87	0.92	0.96
Steep, over 7% Slopes	0.71	0.76	0.79	0.84	0.88	0.93	0.96

**Source: R.L. Rossmiller, "The Rational Formula Revisited"; City of Austin, Watershed Engineering Division*

Time of Concentration

The Rational Formula requires a time of concentration (T_c) for each design point within a drainage basin. Set the duration of rainfall equal to the time of concentration and calculate the design average rainfall intensity (I). The time of concentration consists of an overland flow time to a defined drainage feature (e.g., open channel, headwall, inlet, etc.) plus the time of flow in a closed conduit or open channel to the design point. The minimum time of concentration for any drainage area shall be five (5) minutes.

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 the computation of T_c. Calculate T_c as described below and submit calculations with flow paths to the Reviewing Authority. The procedures specified herein, are from NRCS TR-55, Urban Hydrology for Small Watersheds.

When calculating T_c, consider the following issues. First, take care to ensure that the longest time of travel chosen is characteristic of the overall drainage within the sub-basin. Second, carefully evaluate the interface between overland flow and shallow concentrated flow 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. When the computed time of concentration is less than the minimum for proposed conditions and when modeling existing built development, use the minimum five (5) minutes. Time of concentration at a design point is calculated as:

Time of Concentration (Eq. 2.2.2-2)

$$T_c = T_o + T_s + T_h$$

where:

T_c	=	*time of concentration, minutes (min)*
T_o	=	*overland flow travel time (min)*
T_s	=	*shallow concentrated flow travel time (min)*
T_h	=	*channelized flow travel time (min)*

a. Overland Flow

Compute the time of travel for the overland flow component (T_o) using Manning's kinematic equation:

Time of Concentration – Overland Flow (Eq. 2.2.2-3)

$$T_o = 0.42 \frac{(n_{ol} L)^{0.8}}{R_2^{0.5} S^{0.4}}$$

where:

T_o	=	*overland flow time of travel (min)*
n_{ol}	=	*Manning's coefficient for sheet flow*
L	=	*flow length, feet (ft)*
R_2	=	*50% (2-year) ACE, 24-hour rainfall (in), see Table 2.2.1-4*
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 2.2.2-2. Base overland flow length (*L*) on 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 2.2.2-3. Larger *L* values for undeveloped and agricultural land use are allowed for undeveloped pre-project conditions. Table 2.2.2-3 defines the maximum overland flow lengths for various land use types. These maximums are set because after that distance, the flow is generally considered shallow concentrated flow.

Table 2.2.2-2: Roughness Coefficients for Overland Flow

Surface Description		n_{ol} Value
Smooth surfaces (concrete, asphalt, gravel, or bare soil)		0.013
Fallow (no residue)		0.05
Cultivated soils:	Residue cover ≤ 20%	0.06
	Residue cover > 20%	0.17
Grass:	Short grass prairie	0.15
	Dense grasses	0.24
	Bermuda	0.41
Range (natural):		0.13
Woods:	Light underbrush	0.40
	Dense underbrush	0.80

Table 2.2.2-3: Maximum Overland Flow Lengths

Land Use	Maximum *L* (ft)
Undeveloped, agricultural	100
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

b. Shallow Concentrated Flow

Overland flow becomes shallow concentrated flow in rills, 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. Shallow concentrated flow is characterized by the soil cover as either paved or unpaved. Calculate the flow velocity using the following formula:

Time of Concentration – Unpaved Shallow Concentrated Flow (Eq 2.2.2-3)

$$T_s = \frac{L}{60 * 16.13 * S^{0.5}}$$

where:

T_s = *shallow concentrated flow travel time (min)*
L = *flow length (ft)*
S = *slope of the watercourse (ft/ft)*

Time of Concentration – Paved Shallow Concentrated Flow (Eq. 2.2.2-4)

$$T_s = \frac{L}{60 * 20.33 * S^{0.5}}$$

where:

T_s	=	*shallow concentrated flow travel time (min)*
L	=	*flow length (ft)*
S	=	*slope of the watercourse (ft/ft)*

c. Channelized Flow

Channelized flow is drainage in gutters, storm drains, channels, and streams. To better estimate the travel time, it is generally necessary to breakdown the analysis of channelized flow into a series of reaches with each reach having its own characteristics. Each reach should consist of consistent channel geometry and slope. The total time of travel of the channelized flow is the sum of the times of travel for each segment.

For natural channels, constructed channels, and roadway gutters, it is acceptable to calculate the velocity (V_h) by assuming uniform flow for bank full (or gutter full) conditions. 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 using the Manning's equation (or other method as approved by the Reviewing Authority), calculate the time of travel for channelized flow with the following equation:

Time of Concentration – Channelized Flow (Eq. 2.2.2-6)

$$T_h = \sum\left(\frac{L}{60V_h}\right)$$

where:

T_h	=	*channelized flow travel time (min)*
L	=	*flow length (ft)*
V_h	=	*average velocity of flow (fps)*

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.) After selecting a return period for design and calculating a time of concentration for the drainage area, determine the rainfall intensity from Rainfall-Intensity-Duration data given in the rainfall values in Tables 2.2.1-4 and Tables 2.2.1-5.

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 2.2.2-1 provides recommended runoff coefficients for the Rational Method.

When a drainage area has more than one land use, calculate a composite runoff coefficient to use in the analysis. It should be noted that when using composite runoff coefficient, the analysis does not consider the location of the specific land uses but sees the drainage area as a uniform land use with the composite runoff coefficient. If it is important to locate a specific land use within the drainage area, then use another hydrologic method to generate and route hydrographs through the drainage system.

Drainage Area

The specific geographic limits of any drainage area are a function of topographic features, street layout, lot grading, building structure configuration and orientation, drainage system layout and other features that are created by the urbanization process. Determine the size (acres) of the watershed for application of the Rational Method using topographic maps, as-built plans and supplemental field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow.

2.2.3 TxDOT Regression Equations

Regression equations are commonly accepted 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 gage 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. For a detailed discussion of the regional regression equations and methodologies, the engineer is referred to the TxDOT Hydraulic Design Manual.

2.2.4 Statistical Analysis of Stream Gage Data

If the analysis point is located along a stream that has a stream flow measuring gage station, then flow frequency relation can be developed by statistical analysis of the flow data. USGS published Guidelines for Determining Flood Flow Frequency, Bulletin 17C describes statistical analysis using log-Pearson type III distribution fitting procedure. For a detailed discussion of the process and its applicability, the engineer is referred to the TxDOT Hydraulic Design Manual.

2.2.5 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 sub-basin. A component may represent a surface runoff entity, a stream channel, or a pond. 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 increasing its hydraulic efficiency, reducing its surface infiltration and reducing its storage capacity. 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, roadways, drives, parking lots, and sidewalks and by constructing buildings and other facilities characteristic of urban development. Use zoning maps, future land use maps, and watershed master plans to aid in establishing the anticipated surface character following development. Base the selection of design runoff coefficients and/or percent impervious cover factors upon the appropriate degree of urbanization, which are explained in the following discussions of runoff calculation.

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 Hays County. Accordingly, the hydrologic design techniques described in this manual incorporate many of the routines contained in HEC-HMS. In situations requiring determination of a complete flood hydrograph, and not just a peak discharge, apply the unit hydrograph methods. If the engineer wishes to use an alternative design technique or a computer program, consult the Reviewing Authority prior to design.

Estimating Losses

Only a portion of the rainfall volume which falls on a watershed during a storm event ends up as stream runoff. The remainder is intercepted by infiltration, depression storage, evaporation and other mechanisms. The volume of rainfall which becomes runoff is termed the "excess" rainfall. The difference between the observed total rainfall hyetograph and the excess rainfall hyetograph is termed abstractions or losses. The most commonly used loss methods in Hays County are 1) the NRCS Method or 2) the Initial and Constant Loss Rate Method. The NRCS Method is the preferred Hays County methodology to evaluate development impacts and detention analysis.

a. NRCS Loss Method

The NRCS Method is commonly used to compute losses for unit hydrograph methods and was developed by the Soil Conservation Service (SCS), now called the Natural Resources Conservation Service (NRCS). Details of the methodology can be found in the NRCS TR-55 Urban Hydrology for Small Watersheds. Runoff factors, known as "Curve Numbers" or "CN" values are computed based on a standardized methodology.

Obtain hydrologic soil textures and types from the NRCS Soil Survey Geographic databases (SSURGO) for Hays County. A map of the Hays County soil types can be found in Appendix A. A comprehensive discussion of the derivation of NRCS "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 NRCS 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 NRCS has divided soils into four hydrologic soil groups.

Table 2.2.5-1: Hydrologic Soil Groups

Group	Description
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.

Consider the effects of urbanization on the natural hydrologic soil group. If heavy equipment will compact the soil during construction or if grading will mix the surface and subsurface soils, make appropriate changes in the soil group selection.

Runoff curve numbers vary with the antecedent soil moisture conditions. The NRCS CNs are grouped into three antecedent soil moisture conditions — Antecedent Runoff Condition (ARC) I, ARC II and ARC III. If necessary to calibrate a hydraulic model, the engineer may compute values of runoff curve numbers for all three (3) conditions following guidelines in Part 630, Chapter 10 of the National Engineering Handbook. ARC I is the dry soil condition and ARC III is the wet soil condition. ARC II is considered to be the average condition and is recommended for most hydrologic analysis. The Antecedent Runoff Condition (ARC) was previously referred to as the Antecedent Moisture Condition (AMC) in older NRCS publications. This should be considered a calibration parameter for modeling against real calibration data. Table 2.2.4-3 provides recommended curve number values for a range of different land uses.

When a drainage area has more than one land use, calculate a composite curve number to use in the analysis. It should be noted that when using composite curve numbers, the analysis does not consider the location of the specific land uses but sees the drainage area as a uniform land use with the composite curve number. Calculate composite curve numbers for a drainage area using the weighted method presented below.

Table 2.2.5-2: 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	85	68
Meadow good condition Soil Group C	0.20	71	14
		Total Weighted Curve Number	68 + 14 = **82**

The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas. The CNs provided in Table 2.2.4-3 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.

Table 2.2.5-3: NRCS Curve Numbers (assuming ARC-II conditions)

Cover Description		**Curve numbers for hydrologic soil groups**			
Cover type and hydrologic condition	*Average percent impervious area*[1]	A	B	C	D
Cultivated land:					
Bare soil		77	86	91	94
Poor condition (crop cover increases runoff)		72	81	88	91
Good condition (crop cover decreases runoff)		67	78	85	89
Pasture or range land:					
Poor condition (ground cover < 50%)		68	79	86	89
Fair condition (ground cover 50% to 75%)		49	69	79	84
Good condition (ground cover > 75%)		39	61	74	80
Meadow:					
Continuous grass, protected from grazing		30	58	71	78
Brush:					
Fair condition (ground cover 50% to 75%)		35	56	70	77
Good condition (ground cover > 75%)		30	48	65	73
Wood or forest land:					
Thin stand, poor soil cover		57	73	82	86
Good soil cover, protected from grazing		32	58	72	79
Open space (lawns, parks, golf courses, cemeteries, etc.)					
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 parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
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, business, retail, industrial, etc.	80%	89	92	94	95
Newly graded (pervious areas only, no vegetation)	0%	77	86	91	94
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

[1] 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.

b. Initial and Constant Method

The Initial and Constant loss rate method is another method to compute runoff losses. The initial assumption is that all rainfall is lost to infiltration up to the initial loss amount. After that, the uniform rate is adjusted to the calculation time step and then subtracted from each rainfall amount for that time step. The remaining precipitation is the excess rainfall. This method is a simple approximation of a typical infiltration curve, where the initial loss decays over the storm duration to a final near-constant loss rate. The initial loss and the constant rate represent physical properties of the watershed soils, land use, and the antecedent condition. If the watershed is in saturated condition, the initial loss will approach zero. The constant loss rate can be viewed as the ultimate infiltration capacity of the soils. The NRCS classified soils on the basis of this infiltration capacity as presented in Table

Table 2.2.5-4: NRCS Soil Groups Constant Infiltration Rates

Soil Group	Range of Loss Rates	
	Min. (in/hr)	Max. (in/hr)
A	0.30	0.45
B	0.15	0.30
C	0.05	0.15
D	0.00	0.05

Rainfall to Runoff Transform

After the design storm hyetograph is defined, and losses are computed and subtracted from rainfall to compute runoff volume, the time distribution and magnitude of runoff is computed with a rain-fall to runoff transform. It can be calculated using a Unit Hydrograph method. The two most common unit hydrograph methods are described below.

c. NRCS Unit Hydrograph Method

The NRCS hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The NRCS unit hydrograph 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 TxDOT Hydraulic Design Manual.

A typical application of the NRCS unit hydrograph method includes the following basic steps:

1. Determination of curve numbers that represent different land uses within the drainage area. See section 2.2.5.a.
2. Calculation of time of concentration to the study point. See section 2.2.2.d.

3. Identification of rainfall depths and distribution. See section 2.2.1
4. Calculation of runoff hydrograph for the drainage area using the NRCS unit hydrograph approach.

Use the NRCS method for both the estimation of storm water runoff peak rates and the generation of hydrographs for routing of storm water flows. The NRCS method is suitable for most design applications. But caution should be used in its application for very large drainage areas over 20 sq. mi. The runoff hydrograph from a drainage basin is the sum of the hydrographs from all the sub-areas of the basin, modified by the effects of routing.

d. Snyder's Unit Hydrograph Method

Snyder's Unit Hydrograph Method is a method utilized by the Corps of Engineers Fort Worth 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 NRCS 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.

The Snyder method requires two parameters, the Snyder standard lag and the Snyder peaking coefficient (C_p). Snyder's lag time or time to peak (T_L) is defined as the time from the centroid of the runoff-producing rainfall to the peak of the unit hydrograph. A storage coefficient (C_t) is used in estimating lag time and is a measure of the differences in slope and channel storage between drainage basins. It's value is usually in the range of 0.31 to 2.3 in Central Texas area.

The peaking coefficient (C_p) reflects the sharpness of the hydrograph and has values generally in the range of 0.4 to 0.8. High values reflect a rapidly responding basin with a sharp peaked hydrograph while Low values generally reflect a flatter, slow responding basin with a longer, flatter hydrograph.

The storage coefficient and the peaking coefficient values cannot be estimated from field observations and their values should be calibrated or estimated from similar areas.

Hydrograph 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 hydrograph/flood routing. Use flood routing to determine the effects of this storage on a flood's runoff hydrograph.

Flood routing is classified into two broad but related categories: open channel routing and reservoir routing. Use reservoir routing to determine the effectiveness of stormwater detention to reduce downstream peak flood flow rates. Open channel routing is a refinement of the description of an area's rainfall-runoff process. It modifies the time rate of runoff due to storage within the channel and its overbanks. Unit hydrograph analysis of multiple basins should consider channel routing to determine possible peak discharge attenuation.

e. Modified Puls Routing

<u>The recommended technique for both channel and reservoir routing is the Modified Puls method.</u> The Modified Puls method is based on the assumption of a discharge-storage

relationship in the routing reach of interest. The HEC-HMS program provides a routine for this flood routing technique. Obtain the storage-discharge relationships for this routing technique by using the HEC-RAS backwater program for a variety of flow conditions. Take care in developing these storage-discharge relationships with HEC-RAS as the cross-sections need to adequately define all the floodplain storage available at various water levels. For a detailed discussion of the Modified Puls routing technique and other methodologies, the engineer is referred to the TxDOT Hydraulic Design Manual.

f. Muskingum-Cunge Routing

Apply the Muskingum-Cunge technique where HEC-RAS models are not available or being developed for the project. For each Muskingum-Cunge routing reach, develop an 8-point cross-section. Derive cross-sections using topographic maps (generally using LiDAR) at a representative location for each routing reach that adequately represents each routing reach. The HEC-HMS program provides a routine for this flood routing technique. For a detailed discussion of the Muskingum-Cunge routing technique and other methodologies, the engineer is referred to the TxDOT Hydraulic Design Manual.

3 Chapter 3 Hydraulic Floodplain Analysis

3.1 General

This section summarizes the practical consideration, technical principles, and criteria necessary for proper floodplain analysis. The floodplain analysis also aids in determining other floodplain-related concerns, such as, flood depths, velocities, and elevations. All new construction and substantial improvements of buildings (structures) within a floodplain shall follow the requirements as outlined in Hays County Flood Damage and Prevention Ordinance Chapter 735 of the Hays County Development Regulations and the National Flood Insurance Program (NFIP) administered by the Federal Emergency Management Agency (FEMA). In all cases requiring floodplain delineation, the determination of the flood profile shall consider all existing property development within the contributing drainage basin up to the point of consideration. It is the responsibility of the design engineer to determine existing developed conditions within the drainage basin based upon best available data. Because of its versatility and detail, the widely used computer program HEC-RAS is recommended as the primary tool for floodplain modeling in Hays County.

It is the responsibility of the design engineer to determine the floodplain delineation based on the most accurate information available. The engineer may elect to utilize a floodplain delineation previously approved by the County, assuming the same is still applicable under present requirements and criteria. As such, the engineer needs to confirm validity of the delineation for the site.

3.1.1 County Floodplains

With funding from the Texas Water Development Board and partnerships with local communities, Lower Colorado River Authority (LCRA), Guadalupe-Blanco River Authority (GBRA), the US Army Corps of Engineers (USACE), and FEMA, Hays County has identified flood risk with updated hydrologic and hydraulic studies for over 700 stream miles throughout the County. Contact and coordinate with the Hays County Floodplain Administrator to locate and obtain the floodplains and associated models that serve as the best available data.

3.1.2 Regulatory Floodplains

FEMA develops flood hazard maps, officially known as Flood Insurance Rate Maps (FIRMs), to identify flood hazards, the risk they pose to people and property, and regulatory boundaries of floodplains and floodways. Today Digital Flood Insurance Rate Maps (DFIRMs) or the National Flood Hazard Layer (NFHL) are used to display FEMA designated flood-prone areas. Proposed development within the regulatory floodplain must obtain a floodplain permit to ensure compliance with Hays County Flood Damage and Prevention Ordinance Chapter 735 of the Hays County Development Regulations and FEMA regulations.

3.1.3 Hydraulic Procedures

In addition to the procedures outlined in Chapter 7, consider the latest version of the TxDOT Hydraulic Design Manual and FEMA Guidelines and Standards for Risk Analysis and Mapping for the analysis of floodplains in Hays County.

4 Chapter 4 Roadway Drainage

4.1.1 General

The primary purpose of roadways is transportation: to offer effective mobility for all users, and to ensure that each land parcel has reasonable access. Storm water 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 roadway (in terms of functional classification) the more important it is that storm water design does not interfere with transportation uses. Conversely, moderate interference with transportation uses is more acceptable on lower class roadways.

The design flow of water in roadways relates to the extent and frequency of interference with traffic as it relates to the roadway 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. Prevent flooding of surrounding properties by limiting the depth of flow at the curb and by containing the 1% ACE design storm flow within the roadway right-of-way or easements. Additional roadway considerations are found in the Hays County Development Regulations and Hays County Specifications for Roadway Design, Paving, and Drainage Improvements. Hays County Development Services and Transportation departments should be engaged and involved in the design. The Department requires the submission of materials documenting that the proposed development will be in compliance with this Section.

4.2 Roadway Drainage Criteria

4.2.1 Velocity of Flow

1. The maximum velocity of roadway flow shall not exceed 10 feet per second as described in Hays County Development Regulations Chapter 725. At “T” roadway intersections check flow velocity on the stem of the “T” to ensure that flow will not traverse the crown and opposing curb of the crossing roadway and enter onto private property.

2. Maintain a minimum velocity to ensure cleansing flushes at low flows by keeping the minimum gutter slope to four tenths of one percent (0.4% or 0.004 ft/ft). Flatter slopes are not allowed without specific approval of the Reviewing Authority. Flatter slopes may appear at transitions at intersections which may result is portions of gutter slopes being below 0.004 ft/ft. In this scenario, inlets should be placed to mitigate the flowrate of runoff across or through an intersection.

4.2.2 Allowable Depth of Flow

1. Limit the depth of flow to the top of curb for the design storm of the roadway and meeting spread criteria.

2. Provide design computations to prove satisfaction of this criterion.

3. Hays County Development Regulations Chapter 725 provides requirements for headwater elevation maximums and velocity of flow over the roadway.

4.2.3 Grades and Cross-slopes

1. Roadway grades shall comply with the requirements set forth in the Hays County Development Regulations, and cross-slopes shall be designed in accordance with the TxDOT Roadway Design Manual.

4.2.4 Allowable Water Spread

1. Remove storm water from the roadways by inlets or openings into adjacent drainage systems at low points and as frequently as necessary to avoid exceeding water spread requirements.

Table 4.2.4-1: Summary of Allowable Water Spread Criteria

Roadway Classification	% ACE Design Storm*	Requirements
Local Roadways	4%	Flow limited to the top of crown
Collector Roadways	4%	Flow limited so that one 12-foot wide area at center of roadway will remain clear of water
Arterial Roadways	4%	Flow limited so that one 12-foot traffic lane in each direction at the center of the roadway will remain clear of water

**Design storm headwater elevation shall not be higher than 6 inches above crown of roadway as described in* Hays County Development Regulations Chapter 725.

4.2.5 Valley Gutters

1. When the Reviewing Authority allows storm flow to cross travel lanes or through an intersection, valley gutter design must provide for smooth, uninterrupted traffic flow as stipulated by TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges.

2. Design and place inlets to intercept flow along roadways entering an intersection before the curb returns connecting the roadways based on the criteria below. In no case shall inlets be in the curb returns connecting intersecting roadways.

Table 4.2.5-1: Valley Gutter Criteria per Intersection Pair

Intersecting Roadways	Intercept	Valley Gutter Criteria
Arterial—Arterial	All drainage	No valley gutters
Arterial—Collector	All drainage	No valley gutters
Arterial—Local	All drainage	No valley gutters
Collector—Collector	All drainage	No valley gutters
Collector—Local	Collector roadway drainage	Valley gutters can parallel collector roadway to convey local roadway drainage
Local—Local	Two roadways preferred	Valley gutters acceptable

3. Convey storm drainage across roadways via an underground conduit. The Reviewing Authority may allow valley gutters cross roadways functionally classified as a local roadway. The valley gutter shall satisfy TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges .

4. At the discretion of the Reviewing Authority, additional locations requiring valley gutters where storm runoff might cross travel lanes are at transitions to super elevated sections, at the ends of long traffic islands, or at the ends of medians in super elevated sections.

5. When the Reviewing Authority allows roadways with slopes less than the minimum required, these locations shall have a concrete gutter for conveyance of low flows.

6. For valley gutters with <1% slope are required to be constructed of concrete.

4.3 Design Procedure

4.3.1 Straight Crowns

Calculate flows in roadways which have a straight crown using the following equation for triangular channels:

Flow in Triangular Channels (Eq. 4.3.1-1)

$$Q = \left(\frac{K_g}{n}\right)\left({S_X}^{5/3}\right)\left(S^{1/2}\right)\left(T^{8/3}\right)$$

where:

Q	=	*flow rate in gutter (cfs)*
K_g	=	*Coefficient = 0.56 (in English units)*
S_X	=	*transverse (or cross) slope (ft/ft)*
S	=	*longitudinal roadway (or gutter) slope (ft/ft)*
n	=	*Manning's roughness coefficient*
		n = 0.013 for concrete or asphalt pavement
T	=	*spread of flow in the curb and gutter cross-section (ft)*

Spread for a given Depth (Eq. 4.3.1-2)

$$T = \left(\frac{y}{S_X}\right)$$

where:

T	=	*spread of flow in the curb and gutter cross-section (ft)*
y	=	*depth of flow in the curb and gutter cross-section (ft)*
S_X	=	*transverse (or cross) slope (ft/ft)*

Solving Equation 4.3.1-1 for spread and using the flowrate contributing to a curb a gutter section, the engineer can calculate the spread on the roadway using Equation 4.3.1-3

Spread in Triangular Channels (Eq. 4.3.1-3)

)

$$T = \left(\frac{Qn}{K_g {S_X}^{5/3} S^{1/2}} \right)^{\frac{3}{8}}$$

where:

T	=	*spread of flow in the curb and gutter cross-section (ft)*
Q	=	*flow rate in gutter (cfs)*
n	=	*Manning's roughness coefficient*
		n = 0.013 for concrete or asphalt pavement
K_g	=	*Coefficient = 0.56 (in English units)*
S_X	=	*transverse (or cross) slope (ft/ft)*
S	=	*longitudinal roadway (or gutter) slope (ft/ft)*

4.3.2 Parabolic Crowns

Flows in roadways which have a parabolic crown become complicated and difficult to precisely solve for each design case. Engineer must use appropriate equations to determine gutter flow when roadway design includes parabolic crown sections. If planning parabolic crowns, discuss the concept during an initial meeting with the Reviewing Authority.

5 Chapter 5 Storm Drain Inlets

5.1 General

The purpose of a storm drain inlet is to intercept roadway or surface runoff and direct it into another component of the drainage system, usually an underground conduit. Typically, use curb opening type inlets for roadways and grate type for area drainage.

Standard inlets are classified into three groups: curb, grate or area, and combination inlets. These are further subdivided as follows:

- Curb Inlets
  - Curb inlets on grade without gutter depression
  - Curb inlets on grade with gutter depression

    - Curb inlets in sump
- Grate or Area Inlets
    - Inlets on grade
    - Inlets in sump
- Combination Inlets

5.2 Roadway Inlet Criteria

1. Specific configuration and exact location of inlets shall be consistent with requirements of the TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges but shall not conflict with provisions of Section 4.2.

2. Curb opening inlets shall have a minimum length of ten (10) feet.

3. Inlet locations shall conform to the requirements of Section 4.2 and shall be located as feasible to limit conflicts (caused by the inlet itself or associated storm water) with vehicle, bicycle, or pedestrian traffic.

4. Locate inlets along roadways to prevent concentrated storm water flow from crossing traffic lanes, except as outlined in Section 4.2.5.

5. With approval of the Reviewing Authority, the engineer may place recessed inlets along arterial, collector, or local roadways. Recessed inlets are horizontally displaced away from the curb line so that any depression at the mouth of the inlet occurs wholly within the limits of the gutter, with no irregularity of elevation extends into the travel lane. A diagram of a recessed inlet is illustrated in Figure 5.3.2-1.

5.2.1 Inlets in Sump

Place these inlets at low points to relieve ponding of surface water. The engineer may consider inlets having a gutter depression greater than five (5) inches on roadways with less than a one percent (1%) grade as inlets in sumps.

1. Under no circumstances shall inlets at low points in roadways allow water to exceed the criteria set forth in Table 4.2.1-1 for the project full buildout development conditions. Where computations exceed this criterion, design an overflow outlet to handle the excess flows. This can take the form of a flume draining the roadway or a swale in an adjacent drainage easement, if neither present an obstruction to non-motorized travel. Alternatively, oversize the inlet system and receiving facilities as necessary.

2. Calculate inlet in sump capacities and reduce the inlet capacities to account for clogging from debris based on table below.

Table 5.2.1-1: Clogging Reduction for Inlets in Sump

Inlet Type	Clogging Reduction
Curb	10%
Grate	25%

5.2.2 Inlets on Grade

Inlets are placed on grade to capture runoff to prevent exceeding the design criteria (either depth or crossing roadways). For purposes of design, consider inlets having a gutter depression of five (5) inches or less on roadways with greater than a one percent (1%) grade as inlets on grade.

1. Under no circumstances shall inlets at low points in roadways allow water to exceed the criteria set forth in Table 4.2.1-1 for the project buildout development conditions. Where computations exceed this criterion, design an overflow outlet to handle the excess flows. This can take the form of a flume draining the roadway or a swale in an adjacent drainage easement, if neither present an obstruction to non-motorized travel. Alternatively, oversize the inlet system and receiving facilities as necessary.

2. Inlets on grade tend to clog less than inlets in sumps. Calculate inlet on grade capacities and reduce the inlet capacities to account for debris based on the table below.

Table 5.2.2-1: Clogging Reduction for Inlets on Grade

Inlet Type	Clogging Reduction
Curb	0%
Grate	10%

5.3 Design Procedure

5.3.1 Curb Inlets on Grade

Curb Inlets without gutter depression: Use the weir equation to calculate the capacity of such inlets and reduce to account for roadway grade and cross-flow effects. The head, “y”, is the depth of flow at the upstream end of the opening using criteria stipulated in Section 4.2. Use Equations 1-3 in Table 5.3.1-1 to determine the capacities of these inlets on grade, with the value for “a” set equal to zero.

Curb Inlets with gutter depression: 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 5.3.2-2).

Use the equations in Table 5.3.1-1 to determine the necessary size of curb inlets on grade. The applicable determinates and variables are defined in the table as well as the purpose of each equation.

Table 5.3.1-1: Equations for Sizing Inlets on Grade

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 grate inlet on 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:*** 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 (ft)* T = *Total spread of water in gutter (ft)* K_c = *Coefficient = 0.6 (in English units)* L_x = *length of curb inlet required (ft)* 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 (Figure 5.3.2-1).*	

5.3.2 Curb Inlets in Sump

A curb inlet operates as a weir to depths equal to the curb opening height, and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage and base inlet capacity on the lesser of the weir and orifice capacity.

Unsubmerged Curb Inlets: Curb inlet openings that are not submerged are considered to function as a rectangular weir. Calculate the capacity of a curb opening inlet with the following equation:

Curb Opening Inlet Capacity (Weir Flow) (Eq. 5.3.2-1)

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

where:

Q	=	*inlet capacity (cfs)*
C_W	=	*weir coefficient*
		typically 3.0 for inlets without curb depression (for English units)
		typically 2.3 for depressed inlets (for English units)
L	=	*length of the opening which water enters the inlet (ft)*
y	=	*total depth of water or head on the inlet (ft)*

Submerged Curb Inlets: The equation for capacity of curb openings operating under orifice conditions is the following:

Curb Opening Inlet Capacity (Orifice Flow) (Eq. 5.3.2-2)

$$Q = C_o d_o L \sqrt{2gh}$$

where:

Q	=	*total flow reaching the inlet (cfs)*
C_o	=	*orifice coefficient = 0.67*
d_o	=	*depth of curb opening. This is the physical depth of the opening including depression depth.*
L	=	*length of curb opening inlet (ft)*
g	=	*acceleration due to gravity = 32.2 ft/sec^2*
h	=	*effective head at the center of the orifice throat.*
		h = allowable ponded depth - 0.5d_o

The depth of opening (d_o) will vary with the type of inlet. The user must input the allowable ponded depth as design criteria for curb inlet in sag calculations.

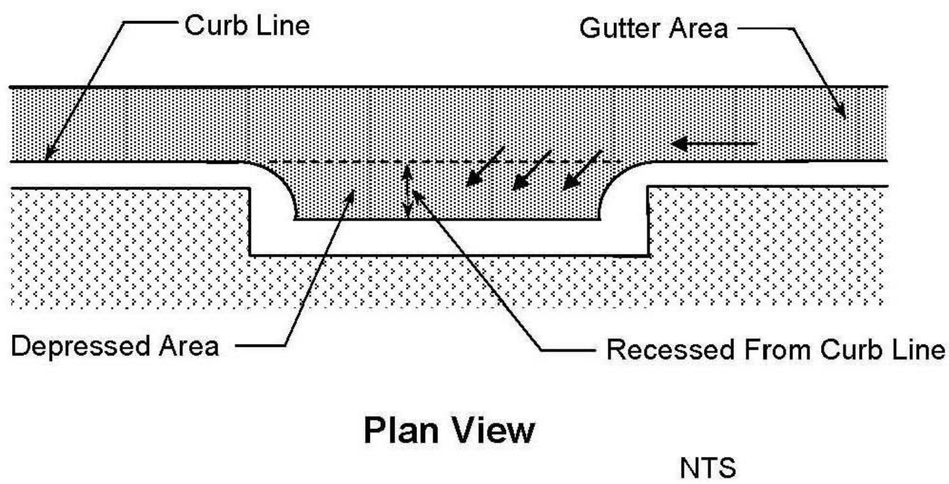


Figure 5.3.2-1: Recessed Curb Inlet Diagram

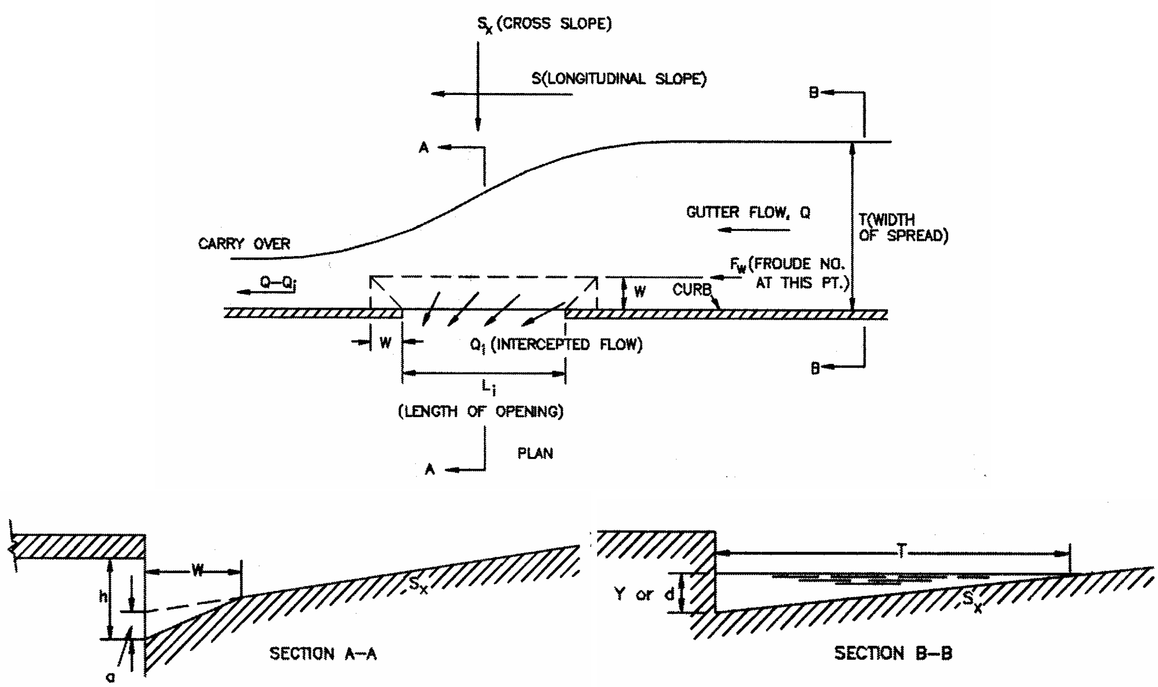


Figure 5.3.2-2: Non-Recessed Curb Inlet Diagrams

5.3.3 Grate Inlets on Grade

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 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 feet 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.

Grate Ratio of Frontal Flow (Eq. 5.3.3-1)

$$E_O = \frac{Q_W}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{\frac{8}{3}}$$

where:

E_O	=	*ratio of flow over the grate to total flow*
Q_W	=	*conveyance of the area over the grate (cfs)*
Q	=	*total conveyance in the gutter (cfs)*
W	=	*width of depressed gutter or grate (ft)*
T	=	*total spread of water (ft)*

Gutter Flow Conveyance (Eq. 5.3.3-2)

$$Q = \frac{A^{5/3}}{nP^{2/3}}$$

where:

Q	=	*total conveyance in the gutter (cfs)*
A	=	*area of cross section (sf)*
n	=	*Manning's roughness coefficient*
P	=	*wetted perimeter (ft)*

Continuity Equation (Eq. 5.3.3-3)

$$Q = AV$$

where:

Q	=	*flow (cfs)*
A	=	*cross-sectional area (ft^2)*
V	=	*velocity of flow (ft/sec)*

Frontal Flow Ratio (Eq. 5.3.3-4)

$$R_f = 1 - K_U(V - V_O)$$

where:

R_f	=	*ratio of frontal flow*
		(Note: R_f cannot exceed 1.0)
K_U	=	*coefficient = 0.295 (in English units)*
V	=	*velocity of flow in gutter (ft/sec)*
V_O	=	*gutter velocity where splash over first occurs (ft/sec)*

Splash Over Velocity (Eq. 5.3.3-5)

$$V_O = AL^N factor^{1-N}$$

where:

V_O	=	*splash over velocity*
A	=	*constant for different grate type*
N	=	*power coefficient for different grate type*
L	=	*grate length*
factor	=	*coefficient = 1 (in English units)*

Splash over velocity can be also calculated by using a chart provided in the Hydraulic Engineering Circular No. 22 Urban Drainage Design Manual by the Federal Highway Administration.

Ratio of Side Flow Intercepted (Eq. 5.3.3-6)

$$R_S = 1/\left(1 + \frac{K_U V^{1.8}}{S_X L^{2.3}}\right)$$

where:

R_S	=	*ratio of side flow intercepted to total flow*
K_U	=	*coefficient = 0.15 (in English units)*
V	=	*velocity of flow in gutter (ft/sec)*
S_X	=	*transverse (or cross) slope (ft/ft)*
L	=	*length of grate (ft)*

Grate Efficiency (Eq. 5.3.3-7)

$$E_f = \left(R_f E_O + R_S(1 - E_O)\right)$$

where:

E_f	=	*grate efficiency*
R_f	=	*ratio of frontal flow*
E_O	=	*ratio of flow over the grate to total flow*
R_S	=	*ratio of side flow intercepted to total flow*

Interception Capacity (Eq. 5.3.3-8)

$$Q_i = E_f Q$$

where:

Q_i	=	*intercepted flow (cfs)*
E_f	=	*grate efficiency*
Q	=	*total conveyance in the gutter (cfs)*

Bypass Flow (or Carry Over) (Eq. 5.3.3-9)

$$CO = Q - Q_i$$

where:

CO	=	*bypass flow (cfs)*
Q	=	*total conveyance in the gutter (cfs)*
Q_i	=	*intercepted flow (cfs)*

5.3.4 Grate Inlets in Sump

Grate inlets in sump operate as a weir at low ponding depths and as an orifice at high ponding depths. The capacity of a grate inlet in sump is based on the minimum flow calculated from weir and orifice conditions.

Use the following weir flow and orifice flow equations to calculate the capacity of a grate inlet in sump.

Grate Opening Capacity (Weir Flow) (Eq. 5.3.4-1)

$$Q = C_W P y^{1.5}$$

where:

Q	=	*inlet capacity (cfs)*
CW	=	*weir coefficient*
		typically 3.0 (for English units)
P	=	*weir perimeter (ft)*
y	=	*total depth of water or head on the inlet (ft)*

Grate Opening Capacity (Orifice Flow) (Eq. 5.3.4-2)

$$Q = C_O A_g (2gy)^{0.5}$$

where:

Q	=	*inlet capacity (cfs)*
C_O	=	*orifice coefficient = 0.67*
A_g	=	*clear opening area of grate (ft^2)*
g	=	*acceleration of gravity = 32.2 ft/sec^2*
y	=	*total depth of water or head on the inlet (ft)*

5.3.5 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. To determine the capacity of a combination inlet, calculate the capacity of each (curb and grate) and use the greater capacity of the pair for design purposes.

6 Chapter 6 Storm Drainage Systems

6.1 General

Storm drain systems are conduits for the collection and conveyance of surface water to specific discharge points. Design using the Manning equation either directly or with software programs using the equation. The following general conditions apply to the design.

All storm drains systems have an outlet discharge point. 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 conveying the storm water. The procedure for calculation of the energy grade line through a storm drainage system begins at the outfall. Therefore, consideration of outfall (or receiving system) 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.

Storm drain systems shall convey the design storm event within the roadway right-of-way or easements. Additional drainage system considerations are found in the Hays County Development Regulations and Hays County Specifications for Roadway Design, Paving, and Drainage Improvements.

6.2 Storm Drain Criteria

6.2.1 General Design Conditions

1. Design the system to accommodate all flow for the design storm at each inlet and opening that allows storm water into the system. Design the system for non-pressure flow during the 4% ACE design event with pipe capacity greater that the design runoff flow rate.

2. Design storm drains for 100% developed conditions based on current or proposed fully developed conditions for drainage areas contributing to the system. No existing system shall have flows added (or directed to it) that will exceed its theoretical design capacity.

3. Evaluate the system with associated drainage systems for the flow conditions that will result from a 4% and 1% ACE storm for project buildout development conditions. Utilize lower magnitude storm design only with specific approval of the Reviewing Authority.

4. Set the tailwater for storm drain system outlets at all creeks, rivers, tributaries, and floodplains at the design storm 4% ACE or 1% ACE elevation for the respective storms. If the 4% ACE elevation is not available, use the 1% ACE elevation or higher for all events.

5. Revise design as required to prevent the formation of any conditions that the County considers hazardous to life, property, or public infrastructure, or that creates conditions inconsistent with the requirements of these guidelines.

6.2.2 Minimum and Maximum Velocities

1. 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 3 feet per second.

2. Maximum velocities are necessary to prevent excessive erosion of the inverts. The maximum design velocity for conduits flowing full shall be 20 feet per second. There is no maximum velocity criteria storm drain laterals connecting to the main line trunk system.

3. Analyze outlet velocities for the scour potential in the receiving stream and use appropriate countermeasures to protect the receiving stream.

6.2.3 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:

Table 6.2.3-1: Storm Drain Material Roughness Coefficients (n)

Material	Roughness Coefficient (n)
Reinforced Concrete (Pipe)	0.013
Reinforced Concrete (Box)	0.015
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

Design commercial, public storm drain systems, and systems within the public right-of-way using reinforced concrete pipe (RCP). Design private systems on private property that will not become part of the public infrastructure using any of the materials listed above. The Reviewing Authority will allow the use of corrugated metal pipe for culverts within the right-of-way on individual private drive.

6.2.4 Location of Manholes and Junction Boxes

1. Provide junction boxes at all changes in conduit size and grade, and where changes in alignment are made at pipe joints. Provide manhole access as part of the design of all junction boxes unless receiving approval from the Reviewing Authority.

2. Provide manhole access points at intervals consistent with the TxDOT Hydraulic Design Manual.

Table 6.2.4-1: Access Hole Maximum Spacing

Pipe Diameter or height (in)	Maximum Distance (ft)
12-24	300
27-36	375
39-54	450
60+	900

3. Design a manhole or junction box at every vertical bend in the storm system. Pipe on pipe vertical bends are not permitted.

4. "Pipe to pipe" connections between manholes require approval by the local Reviewing Authority and must be pre-fabricated.

5. Consider inlets an access point to the storm drain system and place no more than one (1) horizontal bend from the inlet to the trunk line of the system.

6.2.5 Minimum and Maximum Grades

1. The minimum grade for conduits shall be that necessary to produce the minimum acceptable velocity per Section 6.2.1.

2. 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.

6.2.6 Minimum Pipe Diameter

1. In most instances conduit that will become an integral part of the public storm drain system shall have a diameter of 18 inches or greater.

6.2.7 Other Considerations

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

2. Place pipe on the design friction slope as much as practical.

6.3 Design Procedure

6.3.1 Flow Assumptions and Manning's Equations

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

Continuity Equation (Eq. 6.3.1-1)

$$Q = AV$$

where:

Q	=	*flow (cfs)*
A	=	*cross-sectional area (ft^2)*
V	=	*velocity of flow (fps)*

Manning's Equation (Eq. 6.3.1-2)

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

where:

Q	=	*flow (cfs)*
A	=	*cross-sectional area (ft^2)*
V	=	*velocity of flow (fps)*
n	=	*roughness coefficient of conduit*
R	=	*hydraulic radius = A/WP (ft)*
WP	=	*wetted perimeter (ft)*
S_f	=	*friction slope of conduit in (ft/ft)*

Base the capacity of a given size conduit on an assumption that it is "flowing full". Therefore, divide the cross-sectional area by the inner circumference to determine R and select an appropriate value for n and S_f.

6.3.2 Head Losses and Friction Losses

Compute head losses for all at junctions, inlets, and manholes using HEC-22 methodology consistent with the TxDOT Hydraulic Design Manual or using the following equation:

Head Loss – Junctions, Inlets, and Manholes (Eq. 6.3.2-1)

$$h_j = \frac{K_j V_1^2}{2g}$$

where:

h_j	=	*head loss at structures (ft)*
V_1	=	*velocity at upstream entrance of structure (fps)*
V_2	=	*velocity at downstream exit of structure (fps)*
k_j	=	*structure coefficient of loss (Table 6.3.2-1)*
g	=	*acceleration of gravity = 32.2 ft/sec²*

Table 6.3.2-1: 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° Bend – 85% 45° Bend – 70% 22.5° Bend – 40%

Calculate head losses due to friction for open channel flow conditions using the following equation:

Head Loss – Friction for Open Channel Flow (Eq. 6.3.2-2)

$$h_f = S_f L$$

where:

h_f = *head loss due to friction (ft)*
S_f = *friction slope; normally equal to the slope of the conduit, S_o (ft/ft)*
L = *length of conduit (ft)*

The friction slope equals the pipe slope when the pipe is flowing partially full. When the conduit is operating under pressure, calculate the friction slope of the storm drain pipe with the following equation:

Friction Slope – Storm Drain Pipe (Eq. 6.3.2-3)

$$S_f = \left[\frac{Qn}{K_Q D^{2.67}}\right]^2$$

where:

S_f = *friction slope*
Q = *flow (cfs)*
n = *roughness coefficient of conduit*
K_Q = *0.4644 (in English Units)*
D = *diameter of conduit (ft)*

6.3.3 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.

1. In areas with a high starting tailwater condition, design the storm drain system to ensure that the 4% ACE design storm theoretical hydraulic grade line shall be at least one-half foot (0.5 feet) below the inlet opening elevation, the gutter elevation, or the ground surface whichever is lowest.
2. Calculate the hydraulic grade line for the 1% ACE storm keeping the proposed development conditions in the project area within the roadway right-of-way or easement.

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

Consider the tailwater depth or elevation in the storm drain receiving system and evaluate the hydraulic grade line for a storm drainage system beginning at the system outfall with the applicable tailwater elevation. If the applicable tailwater is not above the crown of the outlet, consider the crown of the outlet to be the tailwater elevation.

6.3.4 Allowance for Surcharging

Design of the system and evaluation of hydraulic grade lines shall account for the tailwater 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 stream. Design and construct all storm systems which will operate partially or fully submerged to be fully gasketed.

6.4 Storm Drain Design Example

Design all storm drains using the Manning's Equation either directly, with software programs using the equation, or preliminary design using appropriate charts and nomographs. In the preparation of hydraulic designs, make a thorough investigation of all existing structures and their performance on the waterway in question.

A storm drainage system design example is provided below and shown on Figure 6.4-1 Computation Sheet. The design theory has been presented in the preceding sections with their corresponding tables of information.

Preliminary Design Considerations

A. Prepare a drainage map of the entire area draining to the proposed improvements. Determine drainage area maps using contour maps and supplement with field reconnaissance.

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 for current or proposed fully developed conditions (if known).

D. Indicate on each drainage area the code identification number and the direction of surface runoff by small arrows. Provide a runoff table for the 4% ACE design storm and 1% ACE storm showing area, "C" factor for each portion and composite "cA", TA, I and Q.

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 roadway.

I. Establish permissible spread of water on all roadways 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 6.4-1 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 downstream storm drain inlet point station number.
Column 3	Enter the distance (in feet) between storm drain inlet point shown in Columns 1 and 2. Column 1 stationing minus Column 2 stationing.
Column 4	Record the drainage area ID for each different drainage area corresponding to the drainage area map.
Column 5	Record the area in acres for each of the individual incremental areas of Column 4.
Column 6	Add the incremental area to the upstream total drainage area within the system to determine the total drainage area within the storm drain at the location shown in Column 1.
Column 7	Record the coefficient of runoff "C" for each incremental drainage area shown in Column 5.
Column 8	Multiply Column 5 by Column 7 for each area.
Column 9	Add the incremental "cA" to the upstream total "cA" within the system to determine the total "cA" within the storm drain at the location shown in Column 1.
Column 10	Determine and record the actual inlet time of concentration (See Section 2.2.2-d).

Column 11 Determine flow time in storm drain in minutes. The flow time in storm drain is equal to the length from Column 3 divided by 60 times the velocity of flow through the storm drain. Calculate the velocity using the Manning's or Continuity equations.

Column 12 Total time of concentration in minutes. Column 10 plus Column 11. (Do not modify times of concentration and rainfall intensities 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 (See Section 6.2.1).

Column 14 Intensity of rainfall in inches per hour corresponding to time of concentration that is the greater of Column 10 and Column 12 for the previous pipe segment. Use Rainfall Intensity-Duration-Frequency Curves as identified in Section 2.2.2.f. If the time of concentration is less than the minimum, use the intensity for the minimum.

Column 15 Design Discharge or Runoff in cfs Column 9 times Column 14.

Hydraulic Design

After the computing the quantity of storm runoff at each inlet, determine the size and gradient of pipe to carry off the design storm. Keep in mind that the quantity of flow any section of storm drain carries is not the sum of the inlet design quantities of all inlets above that section of pipe but is less than that total. This occurs because 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 following discussion provides the procedure for calculating the hydraulic grade line in the storm drain system. The engineer shall confirm that the hydraulic grade line at the downstream end of the system is equivalent to or greater than the tailwater at the outfall. Use the ground line profile in conjunction with the previous runoff calculations to establish the storm drain system and hydraulic grade line using the procedures below.

Column 16 Select and record the pipe size. Columns 16A – 16F are the pipe design data.

Column 16A Pipe cross sectional area.

Column 16B Manning's roughness coefficient for selected pipe type.

Column 16C Desired pipe slope for construction.

Column 16D Calculate pipe capacity based on Manning's equation. Ensure the calculated pipe capacity exceeds the total runoff.

Column 16E Invert elevation at upstream end of pipe.

Colum 16F Invert elevation at downstream end of pipe.

Column 17 Calculate the frictional gradient with Equation 6.3.2-3 assuming that the pipe is flowing full for the Total Runoff.

Column 18 Calculate the friction head loss (Product of Columns 3 and 17).

Column 19 With the continuity equation (Eq. 6.3.1-1), calculate the pipe velocity into the node at the location in Column 1 (should be equivalent to Column 20 on previous pipe segment).

Column 20 With the continuity equation (Eq. 6.3.1-1), calculate the pipe velocity out of the node at the location in Column 1 (current pipe segment).

Column 21 Velocity head loss for outgoing velocity (main line) at junction, inlet or manhole at design point (Column 1).

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

Column 23 Head loss coefficient "K_j", at junction, inlet or manhole at design point from Table 6.3.2-1 or using TxDOT Hydraulic Design Manual.

Column 24 Multiply Column 21 by Column 23 (0.10' minimum).

Column 25 Calculate junction head loss by subtracting Column 24 from Column 21 or by using other appropriate equation.

Column 29 Establish starting tailwater elevation from receiving stream or from Column 26 of downstream pipe.

Column 28 Set pipe downstream HGL elevation equal to the greater of Columns 29 or 33 (soffit elevation).

Column 27 Pipe upstream HGL elevation equals the sum of Columns 18 and 28.

Column 26 Upstream node HGL elevation equals the sum of Columns 25 and 27.

Column 30 Soffit Elevation at the upstream end of the pipe.

Column 31 Soffit Elevation at the downstream end of the pipe.

Column 32 Top of curb (or top of ground) elevation.

Column 33 Column 32 minus Column 30.

Column 34 Column 32 minus Column 26.

Follow the above procedure 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.

After establishing the hydraulic gradient for a 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 the engineer can lower the pipe where necessary to maintain proper cover and to minimize grade conflicts with existing utilities.

COMPUTATION SHEET
HYDRAULIC COMPUTATIONS FOR STORM DRAINS

Pipe ID	FROM	TO	Pipe Length	Drainage Area: Incremental No.	Drainage Area: Incremental Area	Drainage Area: Total Area	Runoff "c"	Incr. cA	Total cA	Time of Concentration: Inlet (calc)	Time of Concentration: Travel	Time of Concentration: Total	Design Frequency	Rain Intensity	Total Runoff	Pipe Diameter	Pipe Area	n	Pipe Slope	Pipe Capacity	Invert Elev FROM	Invert Elev TO
			feet							min.	min.	min.	years		cfs	in	sf		ft/ft	cfs	ft	ft
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	16A	16B	16C	16D	16E	16F
	LINE A																					
A1	5+42	4+00	142	A1	2.90	2.90	0.50	1.45	1.45	3.00	0.51	3.51	25	10.10	14.65	24	3.14	0.013	0.0200	32.08	805.00	802.16
A2	4+00	1+86	214	A2	1.50	4.40	0.50	0.75	2.20	4.00	0.50	4.50	25	10.10	22.22	24	3.14	0.013	0.0100	22.68	802.06	799.92
A3	1+86	1+43	43	A3	1.25	5.65	0.50	0.63	2.83	5.00	0.12	5.12	25	10.10	28.54	30	4.91	0.013	0.0200	58.16	799.42	798.56
A4	1+43	0+50	93	A4	2.50	8.15	0.50	1.25	4.08	5.00	0.19	5.19	25	10.05	40.93	30	4.91	0.013	0.0200	58.16	798.46	796.60
A5	0+50	0+00	50	A5	2.00	10.15	0.50	1.00	5.08	7.00	0.13	7.13	25	9.27	47.03	36	7.07	0.013	0.0200	94.58	796.10	795.10

Pipe ID	S_f	HEAD LOSS CALCULATIONS: H_f	V_1 (in)	V_2 (out)	$V_1^2/2G$	$V_2^2/2G$	K_j	$K_jV_1^2/2G$	Hj	U/S Node HGL	Pipe HGL U/S	Pipe HGL D/S	D/S Node HGL	Soffit Elev FROM	Soffit Elev TO	T/C ELEV	T/C - Soffit	T/C - HGL	COMMENTS
	ft/ft	ft	ft/sec	ft/sec	ft	ft		ft	ft	Elev	Elev	Elev	Elev	ft	ft	ft	ft		
	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	
A1	0.0042	0.59		4.66	0.00	0.34	1.25	0.10	0.42	809.04	808.61	808.03	808.03	807.00	804.16	810.31	3.31	1.27	Upstream End
A2	0.0096	2.04	4.66	7.07	0.34	0.78	0.50	0.17	0.61	808.03	807.42	805.37	805.37	804.06	801.92	808.66	4.60	0.63	
A3	0.0048	0.21	7.07	5.81	0.78	0.52	0.25	0.19	0.33	805.37	805.04	804.84	804.84	801.92	801.06	807.92	6.00	2.55	
A4	0.0098	0.92	5.81	8.34	0.52	1.08	0.50	0.26	0.82	804.84	804.02	803.10	803.10	800.96	799.10	806.52	5.56	1.68	
A5	0.0049	0.25	8.34	6.65	1.08	0.69	0.00	0.10	0.59	803.10	802.52	802.27	802.27	799.10	798.10	805.21	6.11		Calculated Receiving TW = 802.27 > Soffit

Figure 6.4-1. Computation Sheet

7 Chapter 7 Open Channels

7.1 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 drainage system, open channels offer significant advantages over closed conduits regarding cost, flow capacity, flood storage, recreation, and aesthetics. However, open channels require considerable right-of-way and maintenance. Give careful consideration during the design process to minimize disadvantages and maximize the benefits. When using a design approach not covered in this manual, the engineer shall obtain approval from the Reviewing Authority prior to commencing significant portions of the design effort.

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

Design of open channels involves the selection of a cross-section, surface treatment, and alignment to accommodate 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 sustainable structural components that minimize sediment loss, erosion, and maintenance requirements.

Leaving streams in their natural state offers numerous advantages, so this practice is preferred. The Reviewing Authority encourages designs that replicate the characteristics of natural streams if they meet the objectives of the provisions in these guidelines. 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. Design the channels for the 4% ACE with provisions for the 1% ACE within the right-of-way or dedicated easements.

7.2 Open Channel Criteria

The criteria outlined in this section are intended to guide the development of traditional designed/constructed open channels. Design roadside ditches as open channels per the guidelines in Section 7.3. The Reviewing Authority will consider alternate channel designs that the engineer can show meet the intent of these guidelines.

7.2.1 Physical Considerations

a. **Cross-Section Geometry**

- **Earthen Channels**: An earthen trapezoidal channel shall comply with Chapter 721 of Hays County Development Regulations. Generally, they will have a trapezoidal shape with side slopes not steeper than four feet horizontal to one foot vertical (4:1) foreslope, three feet horizontal to one foot vertical (3:1) backslope, and a channel bottom at least three (3) feet in width unless receiving approval from the Reviewing Authority.

- **Lined Channels:** A lined trapezoidal channel shall have a trapezoidal shape with side slopes not steeper than two feet horizontal to one foot vertical (2:1) and a channel bottom at least three (3) feet in width unless receiving approval from the Reviewing Authority.
- **Swales:** A swale is an earthen V-shaped channel. Its side slopes shall not be steeper than four feet horizontal to one foot vertical (4:1) foreslope, three feet horizontal to one foot vertical (3:1) backslope, and its maximum depth of three (3) feet unless receiving approval from the Reviewing Authority.

b. **Slope:** The minimum longitudinal slope shall be 0.01 ft/ft (1%) for earthen or vegetative lined channels to prevent formation of standing water. Use a reinforced concrete pilot channel if the channel slope is less than 0.01 ft/ft (1%). The pilot channel must be at least four (4) feet wide and four (4) inches deep. Any grass-lined portion of the channel bottom must have a slope of at least two percent from that portion to the concrete-lined pilot channel. However, no open channel flow line slope may be less than one-half (0.5) percent. The maximum allowable grade is a function of allowable flow velocity for channel lining materials stipulated in Table 7.2.1-1. Refer to TxDOT Hydraulic Design Manual for maximum permissible shear stress for various channel lining material. If the proposed maximum grade exceeds 70% of the calculated critical slope values for the required range of design flows, use special channel linings and energy dissipation features to compensate for the high velocities and hydraulic jumps associated with supercritical flow. Upon Reviewing Authority approval, limit designs for supercritical flow to straight sections having a minimum grade that is at least 130% of the critical slope values calculated for the required range of design flows.

Table 7.2.1-1: . 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 by the Reviewing Authority.

c. **Deflection**: 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.

d. **Protection:** Measures required for protection of earthen channels are described in Chapter 10 of these guidelines.

7.2.2 Flow Considerations

1. Design Flows:
   a. Design and construction shall account for any storm flow from future development areas contributing to the channel. Design may not add (or direct) additional flows to existing systems that will exceed its theoretical design capacity.

   b. Determine channel capacity to convey the discharge from a 4% ACE storm for project buildout development conditions that can be foreseen to discharge to the channel with provisions for the 1% ACE storm within the right-of-way or easement. Channel designs based on smaller magnitude storms must have specific approval of the Reviewing Authority.

   c. Design channels to flow subcritical for the range of discharges resulting from the 4%, and 1% ACE design storms on the project area unless receiving specific approval by the Reviewing Authority.

2. Velocity Limitations:
   a. When feasible, the average flow velocity for the design storm shall be maintained at a minimum of 3 feet per second.

   b. Maximum velocities for the design flow shall be less than the values given in Table 7.2.1-1 for the type of surface treatment(s) specified.

3. Freeboard: Design channels with a minimum freeboard equal to one (1) foot above the design depth of flow.

7.2.3 Outfall Junctures

The following guidelines apply to points of discharge into the public storm drainage system, whether from a private or public drainage facility. Where part of a storm drainage system discharges into another part of the system, 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.

Group junctures into three categories: discharge from an underground storm drain conduit into the drainage system; discharge of an open flume into the drainage system; and the confluence of two channels.

1. Design flumes that convey stormflow into a natural or designed watercourse 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.

2. 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 Chapter 10 of these guidelines.

7.3 Roadway Ditches

Where the Reviewing Authority approves the use of roadside ditches, the provisions for open channel flow as defined in this Section shall govern the design.

7.3.1 Ditch Physical Considerations

1. All culvert size, shape, material of construction, and minimum required cover should be in accordance with the TxDOT specifications.

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

3. Ditch sections shall comply with Section 7.2.1 unless receiving specific approval by the Reviewing Authority.

7.3.2 Ditch Flow Considerations

1. Ditches must, as a minimum, contain the flow from the design 4% ACE storm with a water surface elevation one (1) foot below the top of the ditch. Contain the 1% ACE storm in the roadway right-of-way or easement unless receiving specific approval by the Reviewing Authority.

7.3.3 Ditch Construction

1. Construct culverts and grading in compliance with TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges.

2. All ditches must be completely vegetated or appropriately lined in accordance with TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges.

7.4 Open Channel Design Procedure

In addition to the procedures below, the engineer may use the latest version of the TxDOT Hydraulic Design Manual for the open channel design procedures.

7.4.1 Methods of Analysis

a. **Manning's Equation** (Eq. 7.4.1-1)

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

where:

Q	=	*discharge (cfs)*
n	=	*Manning's Roughness Coefficient*
A	=	*cross-sectional area representing the depth of flow (ft^2)*
R	=	*hydraulic radius = A/WP (ft)*
WP	=	*wetted perimeter of channel section for area A (ft)*
S	=	*slope of channel bed (ft/ft)*

Apply the equation to a single cross-section and assumes a uniform channel cross-section and slope as well as steady, uniform flow in the channel. Consequently, limit its use to designed channels and suitable natural channels in the secondary drainage system.

b. The Standard-Step Procedure for analyzing natural or man-made channels.

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. Apply the method using the HEC-RAS software which is endorsed by the Hydraulic Engineering Center of the US Army Corps of Engineers, or other computer analysis programs employing the same methodology. Implementation of this process shall be in accordance with the selected software application user's manual.

7.4.2 Primary Design Parameters

a. Select cross-section(s) representative of the channel reach.

b. Selection of values for Manning's roughness coefficient (Manning's n-values) shall fall within the range of values and descriptions given in Tables 7.4.2-1 and 7.4.2-2.

c. Take the slope of the channel as the average slope along the reach.

Table 7.4.2-1: Manning's Roughness Coefficients for Channel Design

Channel Lining	n Value
Grass	0.030 (velocity check)
Grass	0.050 (capacity check)
Concrete	0.013
Gabions	0.030
Rock Riprap	0.040 (or use the USACE equation, $n = 0.038D_{90}^{1/6}$)
Grouted Riprap	0.028 (FHWA)

Table 7.4.2-2: Manning's Roughness Coefficients (n)[1]

Type of channel	n Value	
	Min.	Max.
A. Natural streams		
1. Minor streams (top width at flood stage < 100 ft)		
a. Clean, straight, full, no rifts or deep pools	0.025	0.033
b. Same as a, but more stones and weeds	0.030	0.040
c. Clean, winding, some pools and shoals	0.033	0.045
d. Same as c, but some weeds and stones	0.035	0.050
e. Same as d, lower stages, more ineffective	0.040	0.055
f. Same as d, more stones	0.045	0.060
g. Sluggish reaches, weedy, deep pools	0.050	0.080
h. Very weedy, heavy stand of timber and underbrush	0.075	0.150
i. Mountain streams with gravel and cobbles, few boulders on bottom	0.030	0.050
j. Mountain streams with cobbles and large boulders on bottom	0.040	0.070
2. Floodplains		
a. Pasture, no brush, short grass	0.025	0.035
b. Pasture, no brush, high grass	0.030	0.050
c. Cultivated areas, no crop	0.020	0.040
d. Cultivated areas, mature row crops	0.025	0.045
e. Cultivated areas, mature field crops	0.030	0.050
f. Scattered brush, heavy weeds	0.035	0.070
g. Light brush and trees in winter	0.035	0.060
h. Light brush and trees in summer	0.040	0.080
i. Medium to dense brush in winter	0.045	0.110
j. Medium to dense brush in summer	0.070	0.160
k. Trees, dense willows summer, straight	0.110	0.200
l. Trees, cleared land with tree stumps, no sprouts	0.030	0.050
m. Trees, cleared land with tree stumps, with sprouts	0.050	0.080
n. Trees, heavy stand of timber, few down trees, flood stage below branches	0.080	0.120
o. Trees, heavy stand of timber, few down trees, flood stage reaching branches	0.100	0.160
3. Major streams (top width at flood stage > 100 ft)		
a. Regular section with no boulders or brush	0.025	0.060
b. Irregular rough section	0.035	0.100

Type of channel (continued)	n Value	
	Min.	Max.
B. Excavated or dredged channels		
1. Earth, straight and uniform		
a. Clean, recently completed	0.016	0.020
b. Clean, after weathering	0.018	0.025
c. Gravel, uniform section, clean	0.022	0.030
d. With short grass, few weeds	0.022	0.033
2. Earth, winding and sluggish		
a. No vegetation	0.023	0.030
b. Grass, some weeds	0.025	0.033
c. Deep weeds or aquatic plants in deep channels	0.030	0.040
d. Earth bottom and rubble sides	0.028	0.035
e. Stony bottom and weedy banks	0.025	0.040
f. Cobble bottom and clean sides	0.030	0.050
g. Winding, sluggish, stony bottom, weedy banks	0.025	0.040
h. Dense weeds as high as flow depth	0.050	0.120
3. Dragline-excavated or dredged		
a. No vegetation	0.025	0.033
b. Light brush on banks	0.035	0.060
4. Rock cuts		
a. Smooth and uniform	0.025	0.040
b. Jagged and irregular	0.035	0.050
5. Unmaintained channels		
a. Dense weeds, high as flow depth	0.050	0.120
b. Clean bottom, brush on sides	0.040	0.080
c. Clean bottom, brush on sides, highest stage	0.045	0.110
d. Dense brush, high stage	0.080	0.140
C. Lined channels		
1. Asphalt	0.013	0.016
2. Brick (in cement mortar)	0.012	0.018
3. Concrete		
a. Trowel finish	0.011	0.015
b. Float finish	0.013	0.016
c. Unfinished	0.014	0.020
d. Gunite, regular	0.016	0.023
e. Gunite, wavy	0.018	0.025
4. Riprap (n-value depends on rock size)	0.020	0.035
5. Vegetal lining	0.030	0.500

Source: Texas Department of Transportation Hydraulic Design Manual, 2019

7.4.3 Determination of Flow Character

Keep flow within the limits of subcritical flow to Prevent formation of unplanned supercritical flow and hydraulic jumps. To do this, design flow depth must be greater than critical depth. For non-rectangular channels, find the critical depth through application of trial depths and the following relationship:

Critical Depth (Eq. 7.4.3-1)

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

where:

Q	=	*discharge (cfs)*
g	=	*acceleration of gravity = 32.2 ft/sec^2*
A_c	=	*cross-sectional area of flow at critical depth (ft^2)*
T_c	=	*top width of critical flow (ft)*

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

After determining the discharge Q, area A, and depth d, compute the slope necessary to produce these conditions in a channel from Manning's Equation.

8 Chapter 8 Culverts and Bridges

8.1 General

The general purpose of a culvert or bridge is to allow a roadway to cross a drainage way. Consequently, its primary function is to satisfy transportation purposes. As such, design objectives include the following:

1. safety of transportation users
2. safety of surrounding properties
3. long term integrity of constructed facilities
4. minimum maintenance costs
5. and integrity of the natural environment

Because roadways vary in their function and importance, related drainage parameters are varied accordingly. Additional drainage system considerations are found in the Hays County Development Regulations and Hays County Specifications for Roadway Design, Paving, and Drainage Improvements.

8.2 Culvert and Bridge Criteria

8.2.1 General Criteria

1. Design storms and maximum headwater elevations are specified in the Hays County Development Regulations Chapter 725.
2. Design and construction shall accommodate storm flow from future development areas contributing to the culvert or bridge. Do not add (or direct) flows to an existing culvert or bridge that will exceed its theoretical design capacity.
3. Structures shall include design features that can receive the discharge of roadway 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, provided a concrete apron or other suitable surfacing to receive the discharge.
4. Structures within established regulatory special flood hazard areas as defined by the Flood Damage Prevention Ordinance located in Chapter 735 of the Hays County Development Regulations 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 enforced.

8.2.2 Determination of Design Discharges

1. For structures over regulatory special flood hazard areas, determine the design discharges from the FEMA flood insurance study or best available Hays County analysis.
2. For structures over non-regulatory channels, determine the design discharges using the appropriate methods outlined in Table 2.1-1 of these guidelines.

8.2.3 Maximum Operating Headwater

1. For all discharges up to and including the 1% ACE storm, design culverts to limit upstream headwater to elevations that will not endanger their structural integrity or cause flooding to adjacent structures or properties.

2. See the Hays County Development Regulations Chapter 725 for headwater elevation requirements per roadway classification. It is recommended, subject to TxDOT confirmation, design bridges and culverts on TxDOT roadways following TxDOT Hydraulic Design Manual.

8.2.4 Allowable Over-Road Flow

Where overtopping of a roadway classified as a local roadway by flow from a 1% ACE storm is allowable due to a lesser design storm for the culvert, the engineer may convey the excess storm flow over the roadway if the following criteria are met.

1. Design roadway and storm drainage features to convey all over-road storm flow across the road and into the downstream watercourse without endangering adjacent properties or structures and ensures safety of the traveling public.

2. See the Hays County Development Regulations Chapter 725 for roadway velocity requirements per roadway classification. Use the table below to assess the roadway's safety.

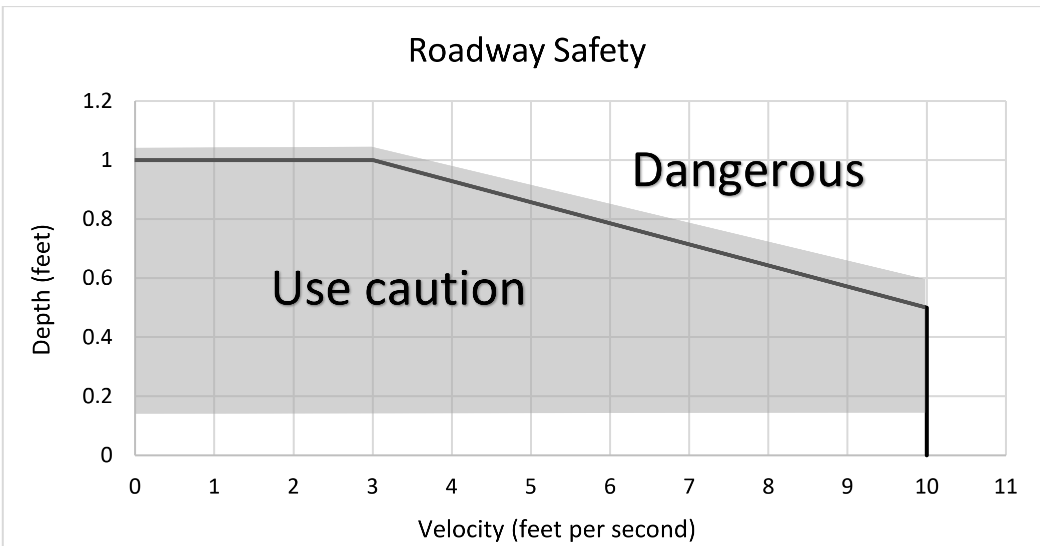


Figure 8.2.4-1: Roadway Depth versus Velocity

8.2.5 Maximum Discharge Velocities

1. Limit the velocity of discharge through a structure to channel conditions immediately downstream of the structure. Allowable flow velocity for channel lining materials are displayed in Table 7.2.1-1. For discharges from the design storm, evaluate downstream conditions to the point where the engineer can re-establish normal flow characteristics.

8.2.6 Alignment Criteria

1. 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 the engineer cannot reasonably satisfy both objectives, minimize the amount of skew in crossing a roadway. In

addition, the design must fully accommodate the hydraulic demands resulting from introducing any artificial turns in a watercourse.

2. Where driveways must cross roadside ditches, place culverts in public right-of-way, generally parallel to the roadway, and aligned with the flow line of the ditch.

8.2.7 Culvert Ends

The following guidelines shall be used in designing culvert end treatments. Figure 8.3.1-1 shows a schematic diagram illustrating terms commonly used to describe a typical culvert structure.

1. Provide concrete headwalls and end-walls to be functionally monolithic with the culvert conduit and must generally be parallel with the alignment of the crossing roadway. Orient wing walls 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. Use flared wing-walls 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. Use straight wing-walls (parallel to the flow line of a watercourse) where all the following conditions are met:

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

4. Use parallel wing-walls (parallel to the road and perpendicular to the water course) for the following conditions:

    - In areas that there is a defined channel and
    - The design needs to minimize disturbance beyond the ends of the roadway.

5. Use Safety End Treatments in accordance with TxDOT Hydraulic Design Manual and TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges where appropriate.

8.2.8 Driveway Culverts

The following guidelines shall be used in designing driveway culverts. All culvert size, shape, material of construction, and minimum required cover should be in accordance with the TxDOT specifications.

1. Design Criteria:
    - Design driveway culverts within the right-of-way to meet the criteria of the intersecting roadway.

2. Material:
    - Driveway culverts within the public right-of-way must use reinforced concrete pipe (RCP).
    - The Reviewing Authority will allow the use of corrugated metal pipe (CMP) for culverts within the right-of-way on an individual residence private driveway.

- Driveway culverts on private property that will not become part of the public infrastructure may use any approved material.

3. Size Change:
   - For residential driveway culverts, when a proposed residential driveway or driveway repair is spaced greater than 200-feet from an adjacent existing driveway, the proposed culvert shall increase to the next higher size available per TxDOT specifications unless design calculations prove otherwise or as directed by the Reviewing Authority.

4. Alignment:
   - Place culverts in public right-of-way, generally parallel to the roadway, and aligned with the flow line of the ditch

5. Culvert Ends:
   - Use Safety End Treatments or straight wing-walls and side slopes in accordance with TxDOT Hydraulic Design Manual and TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges where appropriate.

8.3 Culvert and Bridge Design

8.3.1 Analysis Methodology

Address the following items shall be addressed as part of the engineering design and analysis of crossing structures. Analyze bridges for hydraulic conditions using the HEC-RAS computer program using the guidelines and recommendations of the US Army Corps of Engineers. Unless other parameters can be substantiated to the satisfaction of the Reviewing Authority, determine discharges with the methodology in Chapter 2 of these guidelines.

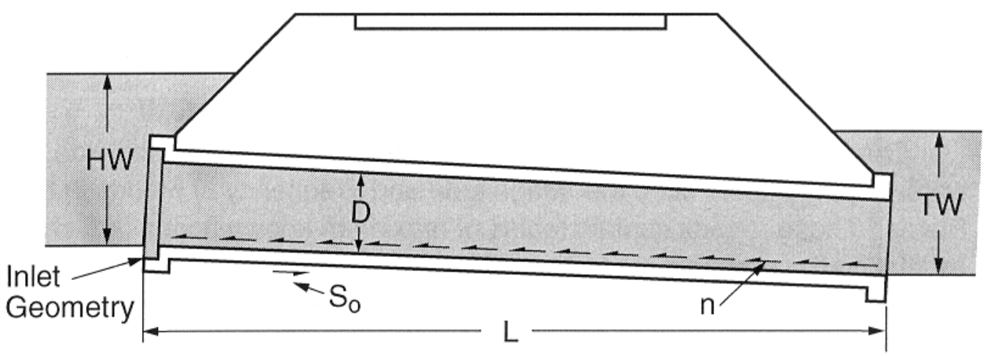


Figure 8.3.1-1: Factors Influencing Culvert Discharge

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*
S_o	=	*slope of the culvert pipe (ft/ft)*
TW	=	*tailwater depth at the culvert outlet (ft)*
K_e	=	*entrance Loss (dimensionless)*

Hays County recommends designing culverts with the widely used computer programs HY-8 or HEC-RAS as it is the primary tool for floodplain modeling in Hays County. If the culvert is not within the public right-of-way the engineer shall obtain specific approval by the Reviewing Authority. Independent of the design method, the engineer shall prove no offsite impacts. The TxDOT Hydraulic Design Manual contains a complete overview of culvert and bridge analysis and design procedures for reference. Consider the latest version of the TxDOT Hydraulic Design Manual as the standard for analysis of culverts by these guidelines.

8.3.2 Culvert Operations

Several direct factors such as slope, length, and surface roughness the rate of flow through a culvert barrel. Where conditions at the culvert entrance (inlet) prevent optimum flow, the culvert operates under "inlet control". When the flow through the barrel is less than the flow into the upstream entrance, the culvert operates under "outlet control" (sometimes referred to as "barrel control"). For each design discharge, determine the type of culvert control.

8.3.3 Headwater and Tailwater Elevations

1. Determine tailwater elevations using one of the methods described in the portion of the guidelines for open channel design (see Chapter 7).

2. Determine headwater elevations by adding the total head losses through the structure to the tailwater elevation, for the given discharge.

8.3.4 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). Compute channel losses of this type using a standard step procedure as outlined in the part of this Section dealing with open channels. Compute entry losses using the following equation:

Culvert Entrance Losses (Eq. 8.3.4-1)

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

where:

H_e	=	*entrance head loss (ft)*
V_2	=	*velocity of flow in culvert (fps)*
V_1	=	*velocity of flow in approach channel (fps)*
g	=	*acceleration of gravity = 32.2 ft/sec^2*
k_e	=	*entry loss coefficient from Table 8.3.4-1*

Table 8.3.4-1: Culvert Entrance Loss Coefficients

Type of Structure and Design of Entrance	Coefficient, k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = D/12)	0.2
Mitered to conform to fill slope	0.7
End section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D12 or beveled top edge	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

Source: Texas Department of Transportation Hydraulic Design Manual, 2019

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 k_e equals 1.0 and V_1 equals 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). Account for all friction losses in the analysis and design of crossing structures.

8.3.5 Erosion and Scour Protection

1. All culverts functioning under inlet control for the design discharge shall have an energy dissipation structure at the outlet of the culvert or shall not exceed the requirements for the downstream channel condition stipulated in Table 7.2.1-1.

8.4 Bridge Design Procedure

In addition to the procedures listed below, the latest version of the TxDOT Hydraulic Design Manual may be utilized. The following is a general bridge hydraulic design procedure:

1. Determine most efficient alignment of the proposed roadway, attempting to minimize skew at the proposed stream crossing.

2. Determine design discharge from hydrologic studies or best available data (FEMA, County, City, USACE, TxDOT, or similar sources) or determine discharges with the methodology in Chapter 2 of these guidelines.

3. If available, obtain best available hydraulic backwater model. Note: it is assumed that if a bridge is required instead of a culvert, the drainage area exceeds one square mile and could already be included in a FEMA or County study. If effective FEMA model or another model is not available, prepare a basic hydrologic model (discharges) and backwater model for the stream. The Reviewing Authority requires a backwater analysis for bridges on unstudied streams, either by hand computations or HEC-RAS.

4. Using USACE or FEMA guidelines, compute or duplicate an existing condition 1% ACE design profile. (Note: see section on exceptions.) Compute profile for design 1% ACE flood, to use as baseline for design of new bridge/roadway crossing.

5. Use the design discharge to compute an approximate opening to pass the design storm (for this preliminary sizing, use the procedures for a normal-depth design or simply estimate a 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 by comparing the proposed model with existing conditions profiles, impacts on other properties, FEMA guidelines, and city criteria.

10. Exceptions/Other Issues

    A. A Conditional Letter of Map Revision (CLOMR) and subsequent Letter of Map Revision (LOMR) will be necessary for new crossings of FEMA streams and as the Reviewing Authority requires.

    B. Comply with the intended use of applicable State and Federal permitting requirements during construction.

    C. Freeboard requirements could require an unusually expensive bridge or roadway elevation that is impractical. The Reviewing Authority will consider reasonable variance requests.

The procedures in this section are acceptable. In addition, perform backwater analysis by either hand computations or HEC-RAS modeling, for any proposed bridge, to determine accurate tailwater elevations, velocities, head losses, headwater elevations, profiles and floodplains the proposed structure affects.

8.5 Culvert Design Procedures

In addition to the procedures listed below, the engineer may utilize the latest version of the TxDOT Hydraulic Design Manual. The following paragraphs supplement the description in Section 8.3.

A culvert conveys surface water through a roadway embankment or away from the roadway 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 and does not impact adjacent properties. 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 design. 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. Define and analyze a culvert configuration for the subject discharges (Q), corresponding tailwater levels (TW), and allowable headwater level (HW_{max})The design process is trial and error modifying the 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.

Perform a backwater analysis by either hand computations or HEC-RAS modeling, to determine accurate tailwater elevation, head losses, headwater elevations and floodplains the proposed structure affects.

9 Chapter 9 Storm Water Management Ponds

9.1 General

The purpose of a storm water management facility is to store excess storm water runoff and discharge it at a predetermined controlled rate. Typically, this is done to limit flow rates from a development site to those that existed prior to any land development activities. The range of compliant design storms and design detention facilities are located in Chapter 725 Storm Water Management Standards of the Hays County Development Regulations.

Detention facilities may be on-site local facilities (receiving storm water from the project area) or regional facilities (receiving storm water from many land development project areas or sites). In regional ponds, a limiting capacity is often that of the drainage system that traverses an existing developed area. A regional facility requires a large land area for the required storage and, thus, design usually considers multiple uses compatible with its storm water purpose.

Ponds excavated to 70% capacity are required to act as sedimentation basins prior to any additional construction on the site.

Detention facilities will only be considered “Regional” at the discretion of the Reviewing Authority. The NRCS Unit Hydrograph Method is the preferred Hays County unit hydrograph methodology to evaluate development impacts and detention analysis.

9.2 Downstream Assessment

The purpose of the downstream assessment is to protect downstream properties and channels from an increase in flooding and erosion potential due to upstream development. A downstream assessment is required to determine the extent of improvements necessary for streambank protection and flood mitigation.

Convey drainage from a development to an adequate or acceptable outfall. An adequate outfall is one that does not create adverse flooding (increase in water surface elevation) or erosion conditions (increase in flow velocities) downstream and is in all cases subject to the approval of the Reviewing Authority. A ‘zone of influence’ from a proposed development extends from the development to a point downstream where the discharge from a proposed development no longer impacts the receiving stream or storm drainage system. Analyze and mitigate downstream impacts due to a development for the 4% and 1% ACE events for the entire zone of influence, with an engineer’s analysis. An engineer must define the zone of influence for any proposed development with a drainage study that determines the specific location along the drainage route where no adverse impacts from the new development exist.

The drainage study shall include the necessary hydrologic and hydraulic analyses to clearly demonstrate the limits of the zone of influence. Drainage studies must adhere to criteria in the Hays County Development Regulations and that the following conditions are met:

1. No new or increase in flooding of existing structures (homes, buildings, etc.),
2. No significant (0.1 feet) increases in flood elevations over existing roadways for the 4% and 1% ACE events.

3. No significant rise (0.1 feet or less) in 1% ACE elevations, unless the existing channel, roadway, drainage easement and/or right-of-way can contain the rise.

4. No rise (0.0 feet) in the regulatory floodway, regardless of whether the site is within the drainage easement of County right-of-way.

5. No significant increases in channel velocities for the 4% and 1% ACE events. 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 7.2.1-1, whichever is lower. Exceptions to these criteria will require certified geotechnical/geomorphologic studies that provide documentation that higher velocities will not create additional erosion.

6. The proposed development may not cause increases in downstream discharges that, in combination with existing discharges, exceed the existing capacity of the downstream storm drainage system.

7. The proposed development shall outfall in a manner that replicates existing conditions and no concentrating structures shall discharge concentrated flow directly onto arterial or collector streets.

9.3 Detention and Retention Pond Criteria

9.3.1 General Criteria

Requirements

1. Design all ponds using a hydrograph routing methodology.

2. See the Hays County Development Regulations Chapter 725 for the design storms.

3. Design ponds with 1 foot of freeboard above the 1% ACE design storm water surface elevation.

4. Provide pond side slopes of three feet horizontal to one foot vertical (3:1) or flatter or as recommended with a geotechnical report.

5. Provide a 10-foot maintenance area clear of obstructions surrounding the pond (maintenance berm) unless otherwise approved by the Reviewing Authority.

6. Protect pedestrians from the top of retaining walls with a fence or railing in accordance with Texas Department of Licensing and Regulation – Texas Accessibility Standard Usability Standards (TDLR TAS or ADA standards).

7. Provide a 12-foot wide access with four feet horizontal to one foot vertical (4:1) slopes or flatter to clean and maintain the pond.

8. Design pond bottoms with a minimum slope of 0.01 ft/ft (1%) and 0.005 ft/ft (0.5%) for concrete pilot channels.

9. Prohibit the placement of trees and bushes on pond embankments.

10. Design water quality ponds/features in accordance with the requirements of the respective authority as referenced in 1.4.3. Storm Water Quality.

Additional Considerations

1. Inform Development Services of the pond depth to determine if it is significant enough for a six (6) foot high chain link fence. The fence needs to surround the pond to limit access by unauthorized personnel.
2. Wet ponds and subsurface ponds may be used with additional guidance or specific approval from the Reviewing Authority.
3. Contact the Reviewing Authority for any additional site-specific requirements

9.3.2 Delineation of Drainage Area

1. Each detention facility shall serve a Design Drainage Area that contributes (or will contribute) runoff to the facility. Determine the Design Drainage Area and the runoff computations for existing pre-development conditions and for post-development project buildout conditions.

9.3.3 Pre-Development and Post-Development Hydrographs

1. Determine a pre-development hydrograph representing the design drainage area and land cover conditions existing prior to the proposed development. Likewise, determine a post-development hydrograph 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.
2. Determine hydrographs using the appropriate methods from Chapter 2 (Hydrology) of these guidelines.

9.3.4 Determination of Storage Volume

1. If an embankment is classified as a dam pursuant to Title 30, Part 1, Chapter 299 of the Texas Administrative Code, all design criteria of the Texas Administrative Code shall be met, as evidenced by certification by a Professional Engineer licensed in the State of Texas.
2. Storage volume shall be adequate to ensure that the peak discharges from the detention facility for the post-development hydrographs will be less than, or equal to, the peak discharges for the design storms pre-development hydrographs.
3. Design shall consider any land features, such as low areas or ponds, for the purpose of storing or detaining storm water during pre-development conditions in determining the required post-development storage volume. If the design alters or eliminates detention storage above the outfall structure of such features, then increase the required storage volume to account for their pre-development detention characteristics. Disclose the existence and effects of such features during the design review process.

9.3.5 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 existing pre-development conditions peak discharge for all design storms.

2. Design outlet structures to allow the facility to drain dry by gravity except for facilities designed to have a permanent storage component.
3. Outlet structures from storm water management ponds, parking detention, or other concentrating structures shall not discharge concentrated flow directly onto arterial or collector streets. Convey such discharges by a closed conduit to the nearest storm drainage system. If there is no existing storm drainage system nearby, the outlet design shall provide for a change in the discharge pattern from concentrated flow back to sheet flow, following as near as possible the direction of the gutter. If the outflow discharge of the detention is released to a watercourse, the proper erosion control measure shall be implemented to prevent erosion.
4. Provide an emergency overflow outlet with the capacity to carry the peak discharge from a 1% ACE storm for project buildout development conditions. Evaluate this discharge for its effect on the downstream receiving drainage system to limit and direct this discharge in a manner that will:
    a. prevent damage to adjacent properties or public infrastructure;
    b. avoid damaging the structural integrity of any element of the detention facility;
    c. present no hazardous conditions;
    d. not exceed the capacity of the downstream receiving drainage system to control and contain the storm discharge for ultimate conditions.
5. All storm water pipes discharging into a public storm drainage system shall have a minimum diameter of 18 inches for ease of maintenance and/or repair.
6. Analysis and design of outlet works shall use the methods detailed by these guidelines, namely those dealing with drainage inlets, drainage conduit, open channel flow, and culverts.
7. Maximum retention or "draw-down" time for flood detention ponds shall not exceed 24 hours from the time of peak storage to the time of complete emptying of the pond unless preapproved by the County.

9.4 Design Procedure

Apply a flow routing analysis using a hydrograph method as described in Chapter 2 for all detention pond designs. The NRCS hydrologic methods or Snyder's hydrologic methods (available in HEC-HMS) are acceptable methods. Other methods require the approval of the Reviewing Authority.

Design storm water management ponds to ensure public safety and provide ease of maintenance.

9.5 Outfall Structure Procedure

9.5.1 General

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 that may combine 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, design the control structure considering a range of design flows.

9.5.2 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
- Broad-crested weirs
- V-notch weirs
- Combination outlets

Typical designs to handle a range of flood flows include:

- a riser with different sized openings,
- an overflow at the top of a riser (drop inlet structure),
- flow over a broad crested weir,
- overflow weirs of different heights and configurations, or
- spillway through the embankment.

The engineer 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. Pays special attention 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.

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, determine the orifice discharge using the standard orifice equation below.

Standard Orifice Equation (Eq. 9.5.2-1)

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

where:

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

If the orifice discharges as a free outfall, then measure the effective head 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 orifice plate thickness is thinner than the orifice diameter, with sharp edges, a discharge coefficient (C) of 0.6 should be used. The engineer shall determine if sharp edge discharge coefficient is appropriate or if another coefficient is more appropriate. Other discharge coefficients for consideration are rounded, short tube, or borda mouthpiece (re-entrant).

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

Broad-Crested Weirs

A weir in the form of a 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 (US Bureau of Reclamation, 1997). The equation for the broad-crested weir is (Brater and King, 1976):

Broad-Crested Weir (Eq. 9.5.2-2)

$$Q = CLH^{1.5}$$

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 rounded to prevent contraction and if the crest slope is greater than or equal to the friction slope of the flow, flow will cross the weir at critical depth on 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.

V-Notch Weirs

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

V-Notch Weir (Eq. 9.5.2-3)

$$Q = 2.5 \tan\left(\frac{\theta}{2}\right) H^{2.5}$$

where:

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

Combination Outlets

Engineer can use a combination of orifices, weirs, and pipes to provide multi-stage outlet control for different control volumes within a storage facility (i.e., 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 multiple 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 discharge separately or combine 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.

9.5.3 Secondary Outlets

General

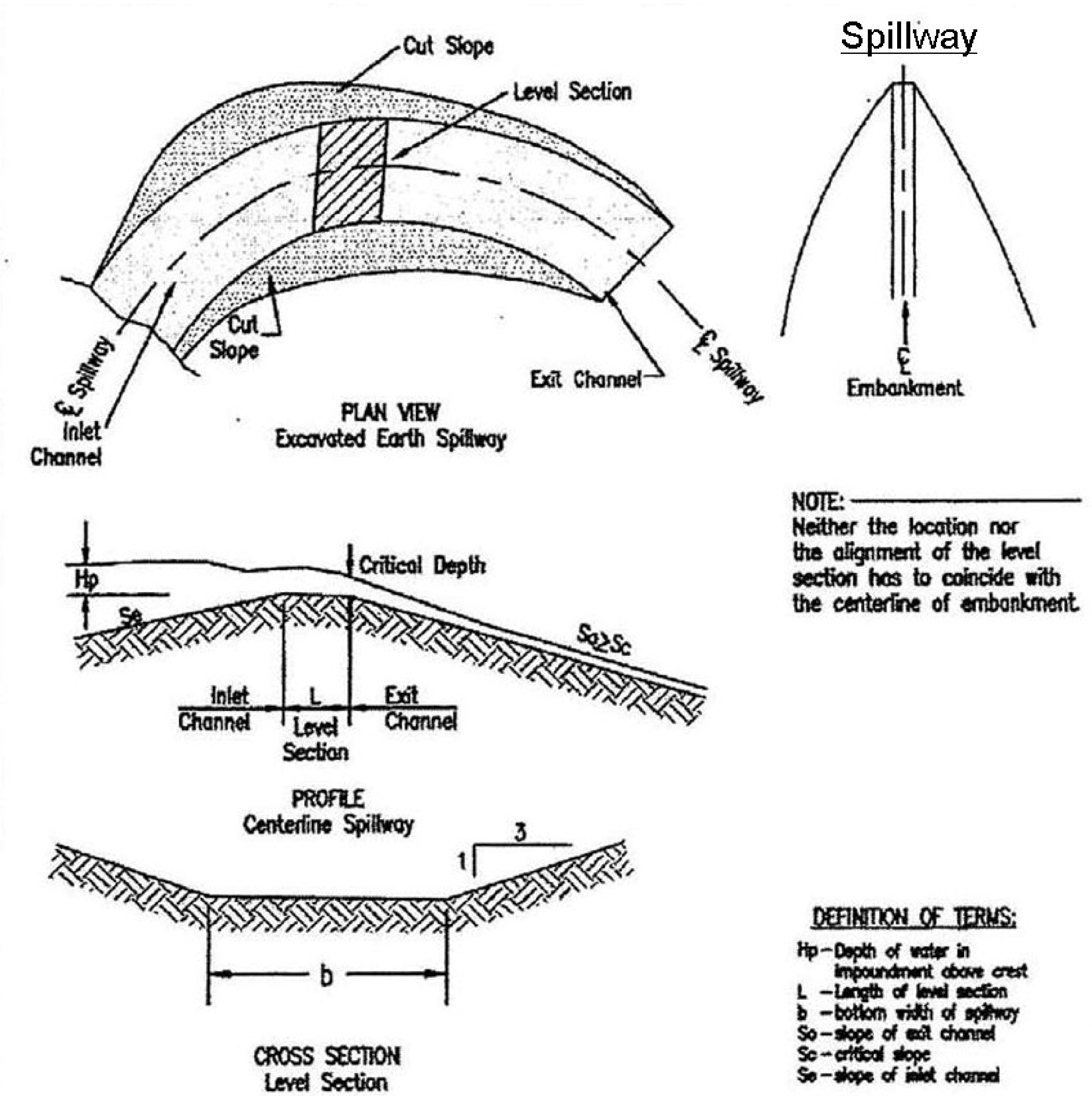
The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 9.4.3-1 shows an example of an emergency spillway.

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 of 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.

Emergency Spillway Design

Emergency spillway designs for reservoirs are open channels, usually trapezoidal in cross-section, and consist of an inlet channel, a control section, and an exit channel (see Figure 9.4.5-1). Design the emergency spillway to pass flows in excess of the design flood (typically the 1% ACE 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 relating to the passage of the probable maximum flood (refer to TCEQ Dam Safety Program). Assuming blockage of the outlet works, the emergency spillway must convey the 1% ACE discharge with freeboard as specified by local criteria. Flow in the emergency spillway is open channel flow (see Chapter

7, Open Channel Design, for more information). Normally, assume that critical depth occurs at the control section. The most common type of emergency spillway 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. An example of an excavated emergency spillway is illustrated in Figure 9.5.3-1.



Source: Virginia Department of Environmental Quality

Figure 9.5.3-1: Emergency Spillway

10 Chapter 10 Erosion and Sediment

10.1 General

The purpose of this section is to compliment the Hays County Storm Water Management Program (SWMP). Design all water quality protection measures in accordance with Hays County and other associated regulatory procedures such as the Texas Commission on Environmental Quality Edwards Aquifer Technical Guidance Manual and Lower Colorado River Authority (LCRA) Highland Lakes Watershed Ordinance.

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

1. **Non-Permanent Measures:** Design non-permanent (temporary) measures 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 storm water that traverses areas where established vegetation has been disturbed or removed, or that across loose materials, including stockpiles. The emphasis is on preventing sediment transportation and deposition, by wind, water, or actions of man, onto adjacent properties, or into drainage systems.
2. **Permanent Measures:** Design permanent measures to prevent erosion and resulting sedimentation from occurring over time, whether within earthen channels, in various facilities for managing storm water, or across unpaved land areas. Properly conceived, designed, and constructed, permanent measures can also promote the proper management of storm water.

Implement measures to prevent the movement of sediment by erosion or action of man at all areas undergoing development or construction. Take positive steps by those conducting such work to prevent the transport of sediment from all work areas onto adjacent properties or into drainage systems.

10.2 Erosion and Sediment Criteria

10.2.1 Non-Permanent Erosion Control Measures

Non-permanent methods to control or contain sediment materials generally fall into two categories: sediment basins and barriers. Use one or more methods 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.

Non-permanent erosion control measures as required by the latest regulations of the Texas Commission on Environmental Quality shall be used on all applicable land development or site projects approved for construction in the Reviewing Authority jurisdiction. Additionally, Hays County requires the use of biodegradable matting within County right-of-way. Compliance with such regulations during project construction shall be a requirement to continue operation of construction activities. Construction plans for grading, excavation, and roadway and utility construction in subdivision projects must include storm water pollution prevention plans (SW3Ps). Texas Commission on Environmental Quality Construction General Permit (CGP) TXR150000 permit requirements shall be met.

10.2.2 Permanent Erosion Control Measures

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

Land Grading

1. Vegetate the cut face of earth excavation in publicly maintained areas that is not be steeper slopes as prescribed in Chapter 7.2.1. Protect all cut slopes from erosion with approved surface treatments that that will not be vegetated.
2. Vegetate or otherwise surface exposed faces of fills to protect them from erosion.
3. 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.
4. The distance of excavations from the property line shall be approved by the County to prevent endangering adjoining property from erosion, sliding, settling, or cracking.
5. Do not place fill where it will slide or wash onto adjacent or downstream properties, including structures.
6. Do not place fill 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.

Unpaved Areas and Swales

1. All areas that are graded and stripped of natural vegetative cover shall receive at least a finish layer of topsoil at least four (4) 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.
2. Seed or sod earthen swales according to the approved plans.

Channels

1. Treat earthen channel banks and inverts with vegetative materials according to the requirements of the TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges.
2. Design velocities shall be less than the recommended maximum velocity acceptable for the proposed surface treatment as outlined in Table 7.2.1-1. Where multiple surface treatments in a length of channel in close enough proximity have interactive effects, use the limiting velocity among those representing the proposed surface treatment types.
3. Channels designed to function with supercritical flow require lining and energy dissipation features adequate to handle the resulting velocities and hydraulic jumps.
4. Protect the integrity of channel linings and cross-sections at all locations where storm water enters a channel from other storm water facilities. See “Outfall Junctures” in Section 7.2.3 of these guidelines.

Drainage Buffers

Buffer zones are undisturbed strips of natural vegetation or established suitable plantings that will provide a living filter to reduce soil erosion and runoff velocities. Drainage buffers protect channels, swales and ditches and may be incorporated into natural landscaping of an area. They can provide critical habitat adjacent to streams and wetlands, as well as assist in controlling erosion, especially on unstable steep slopes.

The buffer zone may be an area of vegetation that is left undisturbed or newly planted during construction. If preserving buffer zones, existing vegetation, good planning, and site management need to prevent disturbances such as grade changes, excavation, damage from equipment, and other activities. Buffer zones are one of the approved water quality protection features, when designed and constructed in accordance with Hays County Development Regulations Chapter 761, shall quality for the economic incentives from the County. Drainage buffers are defined in accordance with the following table.

Table 10.2.2-1: Drainage Buffers

Drainage Classification	Drainage Area	Buffer Offset (on either side of centerline)
FEMA Floodway	NA	Floodway Boundary
Major	> 640 acres	200 feet
Intermediate	300 – 640 acres	100 feet
Minor	64 – 300 acres	50 feet

1. Surface development shall not occur in the drainage buffer zones unless the surface development is in compliance with the Hays County Development Regulations Chapter 735 or with authorization of the Reviewing Authority.
2. Treat new or impacted buffer zones with vegetative materials according to the requirements of the TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges. The slope cannot exceed 12%.
3. Where there is a “high bank” on one or both sides of the stream, consisting of a canyon, cliff, bluff or other similar feature, the engineer may reduce the width of the buffer zone to accommodate this condition upon Reviewing Authority approval. The buffer zone offset adjustment on that “high bank” must extend at least 25 feet horizontally beyond the edge of the “high bank.”

Energy Dissipation

Energy dissipation features are required at any point where the design indicates stormflow design velocities will exceed the surface erosion characteristics of the receiving facility or the criteria established elsewhere in these guidelines.

1. Design velocities shall be less than the recommended maximum velocity acceptable for the proposed surface treatment as outlined in Table 7.3.1-1.
2. Acceptable configurations for energy dissipation structures at outfall structures and channels are in TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, but the Reviewing Authority will consider other special designs and encourages designs suitable to specific situations. Design the reinforcing steel to resist the anticipated hydraulic, hydrostatic, dead, and live loads for the structures.
3. Design energy dissipation features 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.

10.3 Erosion and Sediment Procedure

A number of agencies and researchers have studied and developed procedures for design erosion and sediment control measures. Consider the latest version of the TxDOT Hydraulic Design Manual the standard for erosion and sediment procedures.

10.3.1 Stone Riprap Design

The TxDOT Hydraulic Design Manual (Chapter 7 Channels, Section 3 Roadside Channel Design) shall be used as the stone riprap design procedure.

10.3.2 Channel Lining Design

The TxDOT Hydraulic Design Manual (Chapter 7 Channels, Section 3 Roadside Channel Design) shall be used as the channel lining design procedure.

10.3.3 Gabion Design

The TxDOT Hydraulic Design Manual (Chapter 7 Channels, Section 4 Revetments) shall be used as the gabion design procedure.

10.3.4 Energy Dissipation Design

The TxDOT Hydraulic Design Manual (Chapter 8 Culverts, Section 5 Velocity Control Devices) shall be used as the energy dissipation design procedure.

Appendix A – Maps

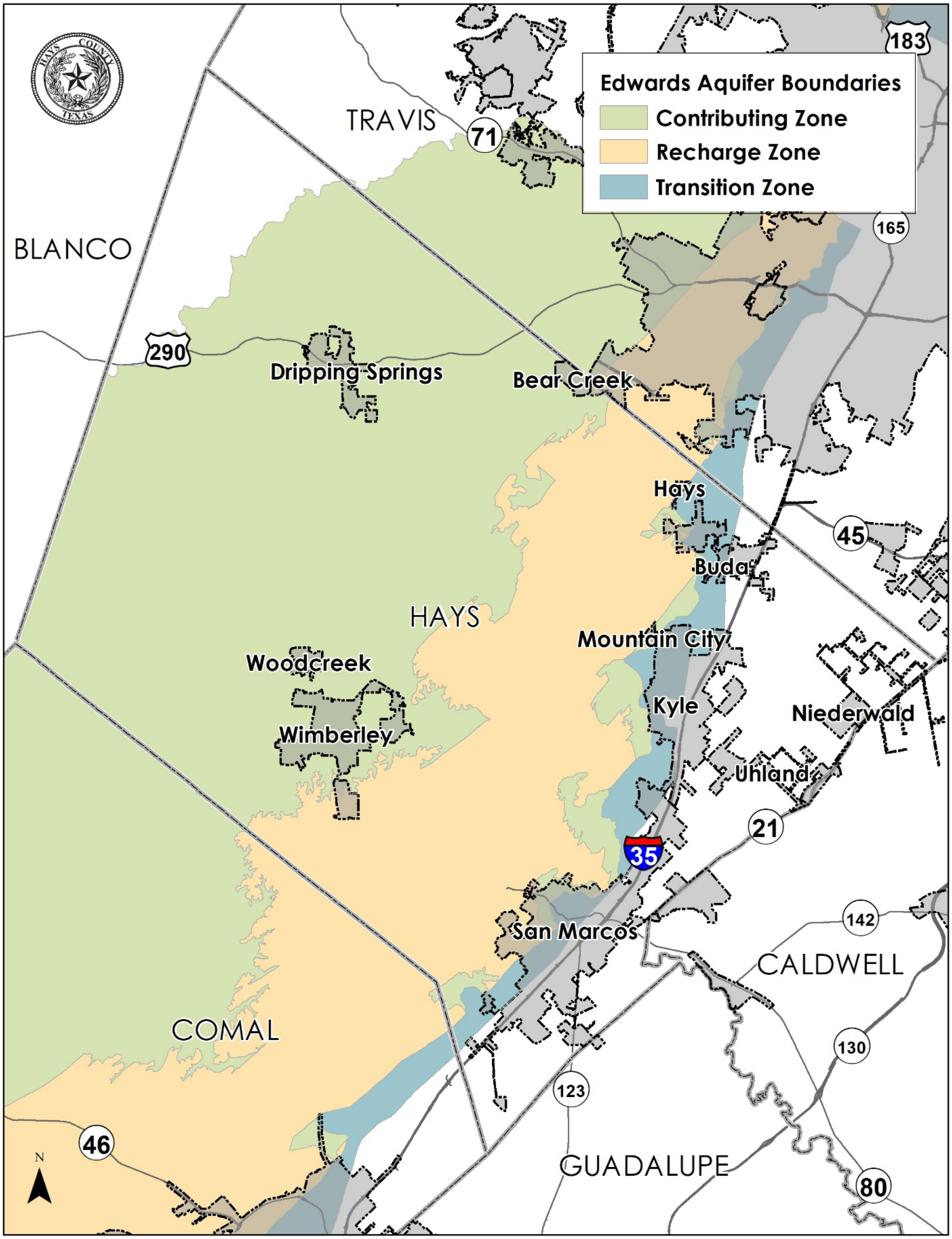


Figure A-1. Edwards Aquifer Boundaries in Hays County

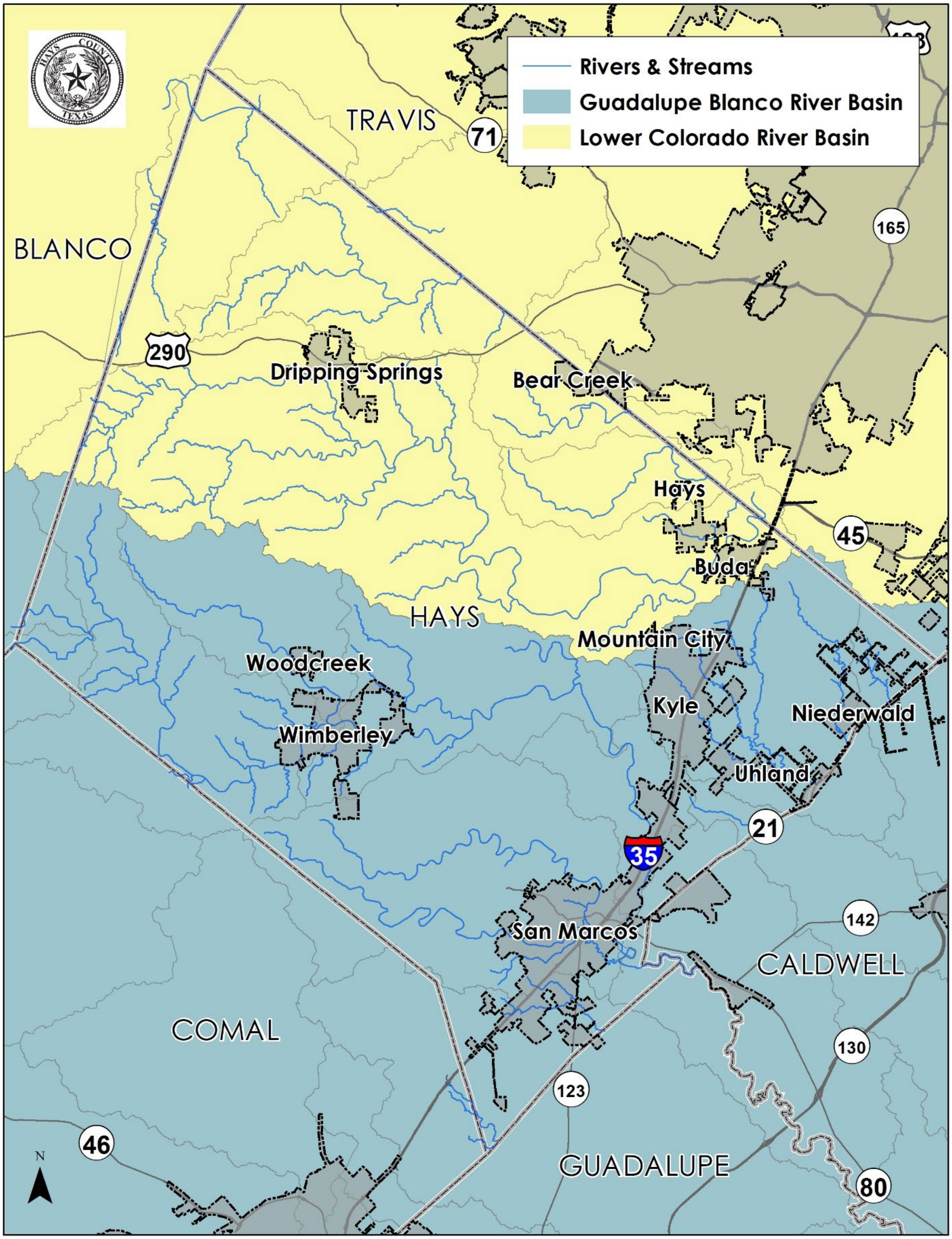


Figure A-2. Hays County River Basins

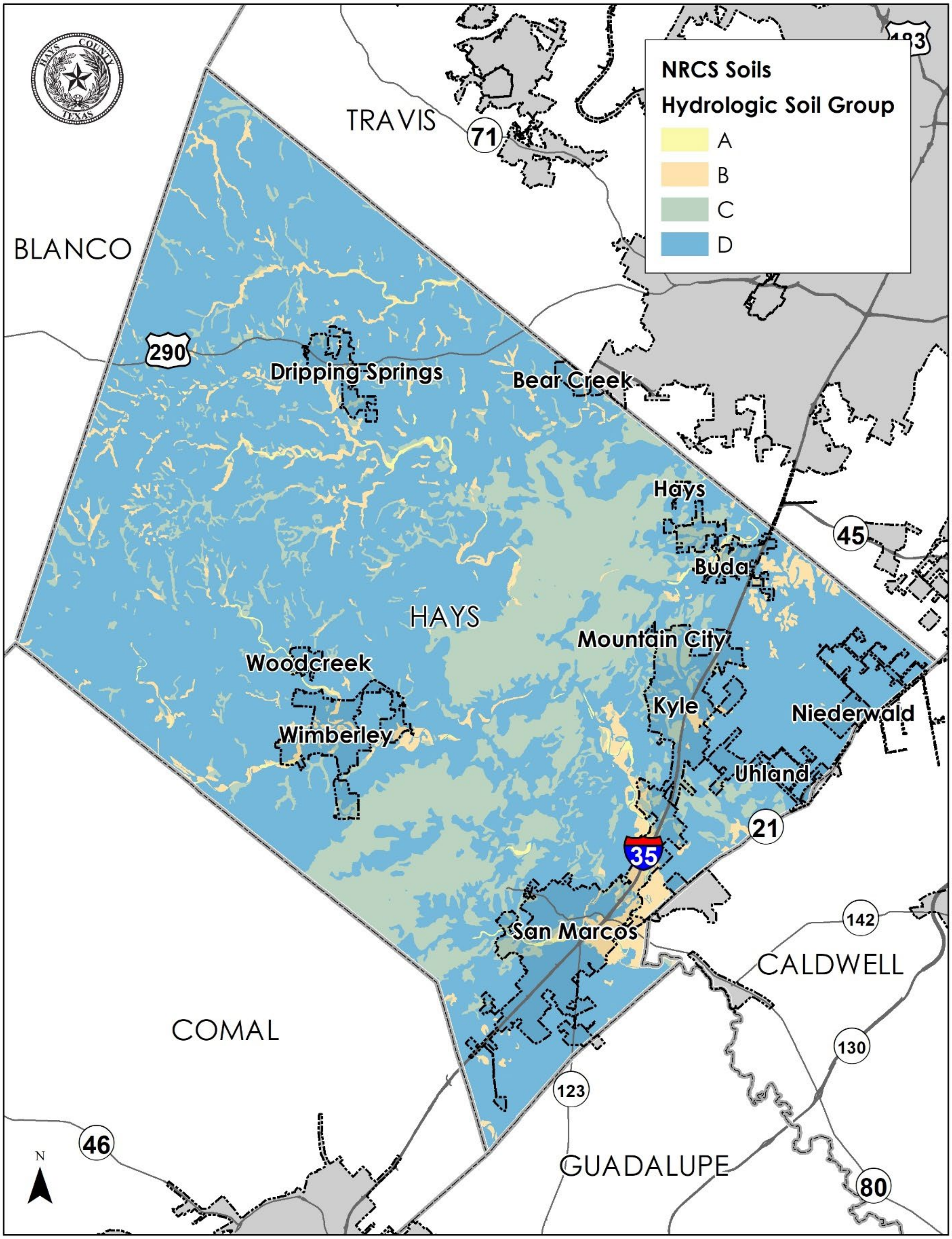


Figure A-3 Hays County Hydrologic Soil Groups