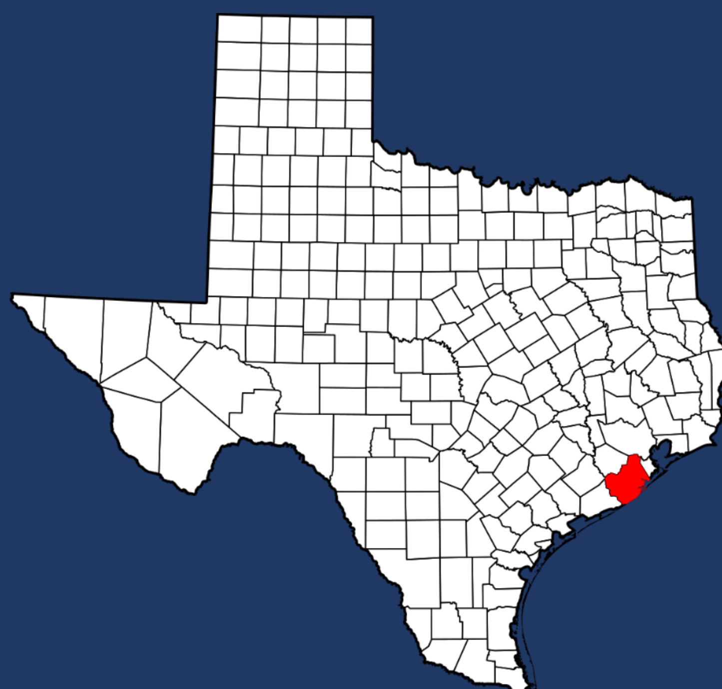
Brazoria County Drainage Criteria Manual



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Brazoria County Drainage Criteria Manual

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1 Introduction

1.1 Background

Brazoria County, along with various incorporated cities, drainage districts, municipal utility districts, and other special purpose districts, have authority over the regulation, funding, and maintenance of public drainage facilities. It is to the advantage of each of these political entities that adequate drainage is maintained throughout Brazoria County.

The Drainage Regulatory Entity cannot enforce total regulatory and funding control over a given watershed. However, the Entity can help shape uniform and consistent watershed management throughout Brazoria County. This will allow entire watersheds to be analyzed consistently and further provide regulation where there is no existing governmental entity currently providing such regulation. Finally, this manual will help further coordinate and disseminate drainage information on a county-wide basis.

1.2 Purpose

The purpose of this drainage manual is to establish standards and practices for the design and construction of drainage systems within Brazoria County. The design factors, formulas, graphs, and procedures are intended for use as minimum engineering requirements in the formation of solutions to drainage problems involving determination of the quantity, rate of flow, extents of inundation, method of collection, storage, and conveyance of storm water.

In those areas currently regulated by a governmental entity, such as a city or a drainage district with established drainage criteria, the criteria set forth herein may be superseded by the entity's work, plans, or specifications. However, the criteria set forth herein may be mutually beneficial to all the entities and stakeholders involved.

Methods of evaluation, analysis, or design other than those indicated herein may be considered in cases where experience clearly indicates that they are preferable. Variations from the practices established herein are to be allowed by the Drainage Regulatory Entity when justified.

1.3 Scope

This manual presents various applications of accepted principles of drainage engineering and is a working supplement to basic information obtainable from standard water resources handbooks and other publications on drainage. It is written for users with knowledge and experience in the application of standard engineering principles and practice of stormwater analysis, design, and management. It is presented in a format that gives logical development of solutions to the problems of storm drainage. This manual is not intended to serve as a textbook. For definitive technical guidance, appropriate texts and documentation should be consulted.

1.4 Drainage Policy

Properties can be impacted by overflow from diverted water, ponding due to impounding, or overflow events occurring more frequently. One of the primary objectives of any drainage regulation is to see that no person, corporation, or governmental entity diverts or impounds the natural flow of surface waters in the relevant jurisdiction in a manner that damages another property. Absolute caution shall be exercised to protect and safeguard the downstream reaches of each watershed by the diversion and/or modification of waters by any person or entity upstream as mandated by the Texas Commission on Environmental Quality (TCEQ) and the State Water Code.

It is a further objective of the Drainage Regulatory Entity to ensure construction and maintenance of facilities are intended to minimize the threat of flooding to all areas of the county and comply with the requirements of the National Flood Insurance Program. It is Brazoria County's goal to prevent additional flooding to existing property so as not to increase the limits of the floodplains as shown on the flood insurance rate maps for Brazoria County.

The net effect of the Drainage Regulatory Entity's policies will be to ensure that ponding will be minimized in the street system of new developments, and that, at a minimum, new habitable structure floor elevations are set at least 24 inches above the maximum ponding levels associated with a 100-year storm, or 24 inches above the original natural ground, whichever is higher. "Original natural ground" is defined as the ground elevation before any fill is placed.

1.5 Plat and Plan Approval Process and Drainage Acceptance Procedures

All developments shall be required to submit a Preliminary and Final Drainage Plan to the Drainage Regulatory Entity prior to development.

The approval process for the drainage plan is outlined in the following sections.

1.5.1 Preliminary Plan Review

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

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

1. Name, address, and phone number of the owner and the engineer preparing the plan
2. Submittal and re-submittal dates
3. A scaled drawing of any existing drainage facilities and any proposed improvements on an Architectural D Size sheet, at a minimum scale of 1" = 200'
4. Vicinity map and legend
5. A primary benchmark referenced to a N.A.V.D. benchmark with elevation, datum, year of adjustment, and description that is adjusted to the datum of the current FIRM
6. North arrow on all sheets oriented upward or to the right, where feasible
7. Location and dimensions of all lot lines, property lines, rights-of-way lines, existing and proposed drainage easements, reserves, fee strips, and easement lines
8. Contour lines at half-foot intervals covering the entire development including offsite elevations 100 feet around perimeter
9. Cross-section of existing and/or proposed detention facility, swales, and ditches
10. Drainage area boundaries for the project area, including off-site areas
11. Location of all drainage conveyance adjacent to or crossing the development as determined by recent (within past year) ground survey
12. Stream alignment shall be shown continuously to 200 feet upstream and downstream of development
13. Detention tabulations, including the detention volume required and the detention volume provided (detention calculations shall be completed as outlined in Section 6.4)
14. Limits of the floodway and the 100-year floodplain, scaled from the current FIRM, if applicable
15. Location of all planned drainage improvements
16. Location of existing pipelines and/or any other underground features and structures
17. Hydrology and hydraulic calculations to demonstrate no adverse impacts

1.5.2 Final Plan Review

The second step in the review and approval process for a proposed development is to submit a Final Drainage Plan to the Drainage Regulatory Entity demonstrating that the analysis and design are consistent and that adverse drainage or flooding conditions will not be created as a result of the development. The Final Drainage Plan shall be filed and approved prior to commencement of construction.

The Final Drainage Plan shall include all the items in the Preliminary Drainage Plan as well as the following:

1. The specific deliverables required under each section of this manual
2. Seal of a Registered Professional Engineer registered in the State of Texas on all plans and technical reports
3. Approval of the drainage district(s) in which the project property is located

1.5.3 Time Limit for Approvals

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

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

1.5.4 Acceptance Procedures

Prior to the Drainage Regulatory Entity's approval of any drainage facilities servicing a development, the project engineer shall submit a letter to the Drainage Regulatory Entity certifying that the drainage facilities were constructed in substantial conformance with the approved plans and specifications and shall specifically certify the following:

1. The elevations and grades were taken by an on-site survey on a certain date.
2. All features and appurtenances are constructed to the grade shown on the record drawings and in compliance with specifications.

3. All pipes, structures, etc. are of the size and dimensions shown on the record drawings.
4. All improvements are capable of performance as designed by the engineer and approved by the Drainage Regulatory Entity.

The approval process will be accomplished by meeting the criteria of the Drainage Regulatory Entity, as well as the criteria stated herein.

2 Hydrology

2.1 General

An accurate estimation of storm runoff is critical for the planning, design, and construction of drainage facilities. Continuous, long-term records of rainfall and the resultant storm runoff in an area are the best data source for storm drain design and flood control systems for that area. However, it is not possible to obtain such records for all locations. Therefore, the accepted practice is to relate storm runoff to rainfall, thereby providing a means of estimating the rates, timing, and volume of runoff expected within local watersheds at various recurrence intervals.

Because of its versatility and accuracy, the widely-used computer program HEC-HMS is recommended as the primary tool for modeling storm runoff hydrographs in Brazoria County. HEC-HMS can be downloaded at no cost from the USACE website. The use of other computer programs shall be approved in writing by the Drainage Regulatory Entity.

The principal routines used for describing runoff in the County as presented in this section are based on the Clark Unit Hydrograph technique, design storms, and rainfall loss rates. To derive the parameters used to compute the Clark Unit Hydrograph, Brazoria County adopted the Basin Development Factor (BDF) method for determining the time of concentration (TC) and the residence time or Clark storage coefficient (R). The concept of BDF was developed by the U.S. Geological Survey and was presented in the U.S. Geological Survey Water-Supply Paper 2207 in 1983. Section 2.3.2.3 offers instructions on the application of BDF to the development of Clark's TC and R. Regional rainfall-runoff data and standard unit hydrograph techniques are appropriate for a wide range of drainage area sizes and are the preferred methods in analyzing watersheds greater than 640 acres. For areas less than 640 acres and greater than 100 acres, area-discharge curves have been developed to determine peak discharge (Figure 2-2 to Figure 2-6). For watersheds 100 acres or less, the Rational Method is the accepted methodology for determining peak discharges in Brazoria County. (See Section 2.2.1.)

For areas smaller than 640 acres that require the determination of a complete flood hydrograph and not just a peak discharge, the Malcom Small Watershed Method shall be utilized. (See Section 2.3.1.) If the engineer wishes to use an alternative design technique, the appropriate Drainage Regulatory Entity shall provide written acceptance of the alternative methodology.

2.2 Peak Flow Determination

Under certain circumstances, only a peak flow will be required to perform drainage calculations. In this case, the Rational Method can be used for areas of development less than 100 acres. The Drainage-Area Discharge Curves developed for Brazoria County can be used for areas of development ranging from 100 acres to 640 acres in size. For drainage areas 100 acres to 200 acres in size, Rational Method calculations may be considered acceptable if written acceptance is provided by the Drainage Regulatory Entity.

For proposed development with contributing drainage areas larger than 640 acres, a full hydrologic analysis using hydrograph methodology shall be required. (See Section 2.3.2.)

2.2.1 Rational Method (less than 100 acres)

The Rational Method represents an accepted method for determining peak storm runoff rates for small watersheds that have a drainage system unaffected by complex hydrologic situations including ponded areas, storage basins, and watershed transfers (overflows) of storm runoff. This widely-used method provides satisfactory results if understood and applied correctly. In Brazoria County, the Rational Method shall be used only for contributing drainage areas less than 100 acres. For drainage areas between 100 and 200 acres, the Rational Method may be utilized upon prior written approval from the Drainage Regulatory Entity.

The Rational Method is based on a direct relationship between rainfall and runoff, and is expressed by Equation (2-1):

$$\boldsymbol{Q = CiA} \tag{2-1}$$

Where,

Q	=	Peak rate of runoff (cfs);
C	=	Dimensionless coefficient of runoff representing the ratio of peak discharge per acre to rainfall intensity;
i	=	Average intensity of rainfall for a period of time equal to the longest time of concentration from the upstream end of the drainage area to the point of interest (in/hr);
A	=	Area contributing runoff to the point of interest during the critical time of concentration (ac)

Basic assumptions associated with the Rational Method are:

1. The computed peak rate of runoff at the design point is a function of the average rainfall rate during the time of concentration to that point.
2. The frequency or recurrence interval of the peak discharge is equal to the frequency of the average, uniform rainfall intensity associated with the critical time of concentration (duration).
3. The storm duration is equal to the critical time of concentration.
4. The ratio of runoff to rainfall, C, is constant for the entire storm duration.
5. Rainfall intensity is constant for the entire storm duration.

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

2.2.1.1 Runoff Coefficient (C)

In relating peak rainfall rates to peak discharges, the runoff coefficient "C" in the Rational Formula is dependent on the characteristics of the drainage area's surface.

Coefficients for specific surface types shall be used to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage area. Table 2-1 presents values for the runoff coefficient "C" for land use types in Brazoria County.

Table 2-1 Typical Average Values for Impervious Cover and Runoff Coefficient

Land-use Type	Impervious (%)	Runoff Coefficient
Water Body	100	1.00
Commercial/Multifamily	85	0.75
Industrial	72	0.65
Detention (Wet or Dry)	85	0.75
Major Thoroughfares	90	0.80
Open Space-Row crops	0	0.30
Open Space-Pastureland	0	0.15
Undeveloped-Wooded/forested	0	0.15
Open Space-Parks/green space	5	0.20
Cemeteries	0	0.15
Churches (up to 5-acre parcel)	40	0.45
Residential – 1/9 Acre*	60	0.60
Residential – 1/8 Acre*	60	0.60
Residential – 1/7 Acre*	60	0.60
Residential – 1/6 Acre*	60	0.60
Residential – 1/5 Acre*	60	0.60
Residential – 1/4 Acre*	50	0.55
Residential – 1/3 Acre*	45	0.50
Residential – 1/2 Acre*	38	0.45
Residential – 1 Acre*	22	0.35
Residential ≥ 5 Acre*	5	0.20
Schools**	45	0.50
Concrete, Asphalt, and Roofs	100	0.85
Open Space-Lawns	0	0.20

Note: * Local streets are included in impervious cover % for Residential Land Use
** For elementary schools, higher values may be considered.

2.2.1.2 Rainfall Intensity (i)

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

A direct method for determining intensity data is through an equation that is based on the time of concentration. The following Equation (2-2) is provided for this method.

$$\boldsymbol{i} = \frac{\boldsymbol{b}}{(\boldsymbol{T_c} + \boldsymbol{d})^{\boldsymbol{e}}} \qquad (2\text{-}2)$$

Where,

i	=	Rainfall intensity (in/hr);
T_c	=	Total time of concentration of the watershed (min);
e, b, d	=	coefficients based on return interval

Table 2-2 presents the e, b, d parameters for Region 1 and Region 2 of Brazoria County (see Figure 2-8). These coefficients are valid when the time of concentration is shorter than 3.0 hours.

Table 2-2 Intensity-Duration-Frequency Coefficients

Return Period	Region 1			Region 2		
	e	b	d	e	b	d
2-Year	0.754	57.440	11.511	0.761	59.876	11.894
5-Year	0.712	58.019	9.236	0.710	57.332	9.519
10-Year	0.676	57.515	7.777	0.658	51.858	6.792
25-Year	0.618	52.780	5.022	0.613	50.414	4.832
50-Year	0.574	49.157	3.081	0.572	47.465	2.941
100-Year	0.533	46.316	1.555	0.545	47.790	2.105
500-Year	0.474	47.179	0.322	0.485	47.597	0.562

Alternatively, total rainfall depths for various storm events and durations, based on Atlas 14 rainfall statistics, are provided in Table 2-5 and Table 2-6. To obtain intensity values from these tables, the appropriate depth value for a given time shall be converted into inches per hour: divide the selected value by the corresponding time of concentration in minutes and multiply by 60 minutes per hour.

2.2.1.2.1 Time of Concentration

The time of concentration is the time it takes for a drop of water to travel from the hydraulically most distant point in the drainage area to the point of interest, assuming that the entire drainage area is contributing to the runoff. The time of concentration flow path and supporting calculations shall be submitted to support the selected values.

The time of concentration depends on the slope of the flow paths as runoff moves through the watershed, the roughness of the surface, attenuation effects caused by storage or ponded areas, and increased conveyance efficiency caused by storm sewers or channels. It is recommended that the calculation of the time of concentration be divided into the time of travel of overland flow and the time of travel in well-defined channels or storm sewers as shown in Equation (2-3).

$$\boldsymbol{T_c = T_{ov} + T_{ch}} \tag{2-3}$$

Where,

T_c	=	Total time of concentration of the watershed (min);
T_{ov}	=	Time of travel in the form of overland or sheet flow (min);
T_{ch}	=	Time of travel in well-defined channels (i.e., channels with identifiable streambed and banks) or storm sewers (min)

Time of Concentration for Overland Flow

The time of concentration for the overland flow portion, T_{ov}, can be approximated by the Kerby Equation, Equation (2-4):

$$\boldsymbol{T_{ov} = 0.828(L * N)^{0.467} S^{-0.235}} \tag{2-4}$$

Where,

L	=	Overland flow length (ft);
S	=	Slope of the travel path (ft/ft);
N	=	Dimensionless retardance coefficient from Table 2-3

Table 2-3 Retardance Coefficient, N

Land Cover	"N" Value
Pavement	0.02
Smooth, bare, packed soil	0.10
Poor grass, cultivated row crops, or moderately rough packed surfaces	0.20
Pasture, average grass	0.40
Deciduous forest	0.60
Dense grass, coniferous forest, or deciduous forest with deep litter	0.80

Time of Concentration for Channel Flow

The time of concentration for the channel flow or pipe can be calculated using Equation (2-5).

$$T_{ch} = \frac{D_F}{60V} \tag{2-5}$$

Where,

T_{ch} = Overland flow time (min);
D_F = Flow distance (ft);
V = Average velocity of runoff flow (ft/sec)

The velocity term of Equation (2-5) can determined either using Figure 2-1 or the following equations.

Paved Areas	$V = 20.328\sqrt{S}$	(2-6)
Grassed Waterway	$V = 16.135\sqrt{S}$	(2-7)
Nearly Bare Ground	$V = 9.965\sqrt{S}$	(2-8)
Cultivated Straight Row Crops	$V = 8.762\sqrt{S}$	(2-9)
Short Grass Pasture and Lawns	$V = 6.962\sqrt{S}$	(2-10)
Fallow or Minimum Tillage Cultivation	$V = 5.032\sqrt{S}$	(2-11)
Forest with Heavy Ground Litter and Meadow	$V = 2.516\sqrt{S}$	(2-12)

Where,

V	=	Velocity (ft/sec);
S	=	Conduit slope (ft/ft)

Time of Concentration for Storm Drain Analysis

The time of concentration to any point in a storm drainage system is a combination of the “inlet time” and the “time of flow in the conduit”. The inlet time is the time for water to flow over the surface to the storm sewer inlet. Inlet time decreases as the slope and the imperviousness of the surface increases. Inlet time increases as the watercourse distance increases and as retention by the contact surfaces increases. Where Rational Method computations are used solely for storm drain analysis in an urban setting, overland flow time shall be replaced with inlet time of concentration. A minimum inlet time of 15 minutes shall be assumed.

The time of flow in a conduit is the quotient of the length of the conduit and the velocity of flow as computed using the hydraulic characteristics of the conduit, using Manning’s Equation.

$$\boldsymbol{V = \frac{1.49}{n} R^{2/3} \sqrt{S}} \tag{2-13}$$

Where,

V	=	Velocity (ft/sec);
n	=	Manning’s Coefficient (dimensionless);
R	=	Hydraulic Radius (ft);
S	=	Conduit Slope (ft/ft)

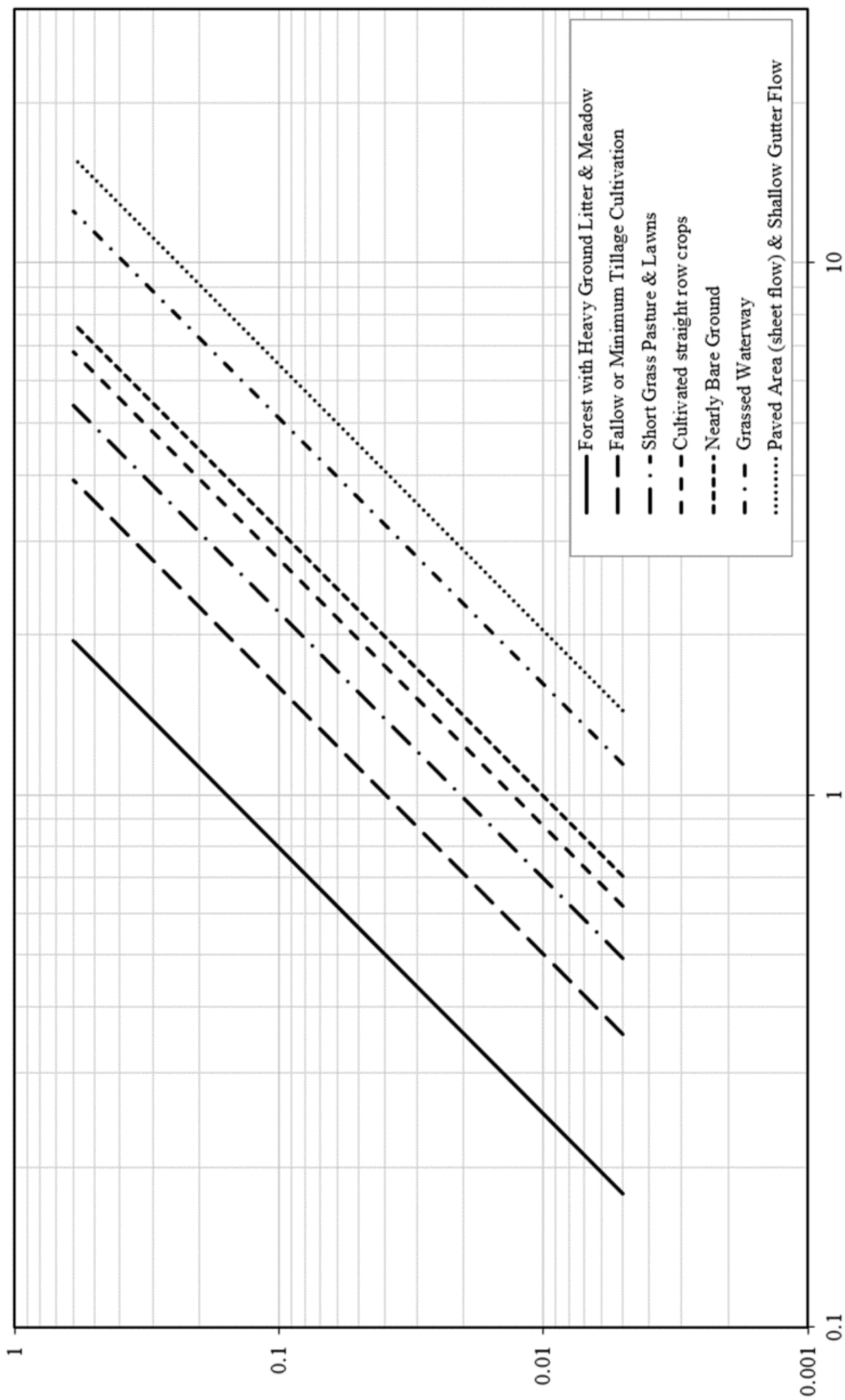


Figure 2-1 Average Velocities for Estimating Travel Time for Channel Flow

2.2.1.3 Drainage Area (A)

The size and shape of the drainage area shall be determined using topographic maps and supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. A drainage area map shall be provided for each project. The entire drainage area contributing to the drainage system and the drainage subarea contributing to each point of interest shall be identified. Drainage area maps shall contain topographic information far enough outside the study area to confirm that project drainage divides are shown correctly.

2.2.2 Drainage Area–Discharge Curves (100 to 640 acres)

Drainage area-discharge curves represent a simplified method for the determination of the peak discharge for small- to moderate-sized watersheds. Usage of this type of analysis requires that the watershed and its physical characteristics be relatively uniform and not contain complex hydrologic features such as ponding areas, storage basins, or watershed overflows.

The curves developed for this manual for the 2-year, 5-year, 10-year, 25-year, and 100-year rainfall events (for both Regions 1 and 2) are shown in Figure 2-2 through Figure 2-6, and are applicable to drainage areas between 100 and 640 acres. Use of these curves for drainage areas larger than 640 acres will require written acceptance from the Drainage Regulatory Entity prior to submittal of the project for review. For areas with more than 85% impervious cover, use the 85% impervious curve.

These curves were developed for a typical watershed assuming adequate conveyance capacity and uniformly-spaced development with an average impervious cover of 40% over the developed portions. Since there is a large variation in the physical characteristics of partially-developed watersheds and conveyance capacity (i.e., floodplain storage), these curves are not applicable for watersheds that vary widely from the assumed conditions. Applicable flow rates for existing conditions in the design of detention facilities shall be determined on a case-by-case basis, working closely with the appropriate Drainage Regulatory Entity (see Chapter 6). For development in a watershed between 100 and 640 acres where the development requires the determination of a complete flood hydrograph and not just a peak discharge, Malcom's Small Watershed Method shall be used as described in Section 2.3.1.

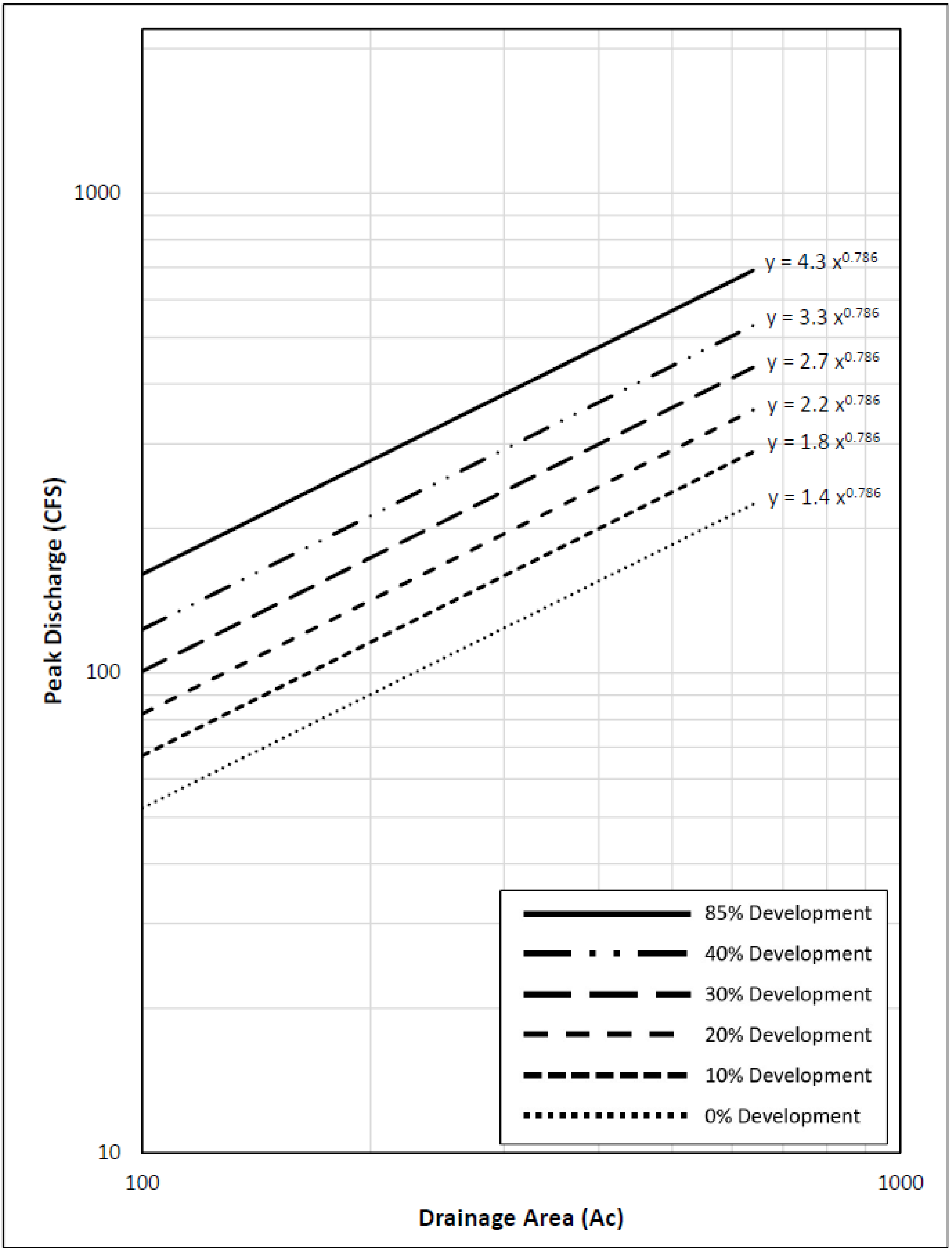


Figure 2-2, 2-Year Drainage Area-Discharge Curves for Brazoria County, TX

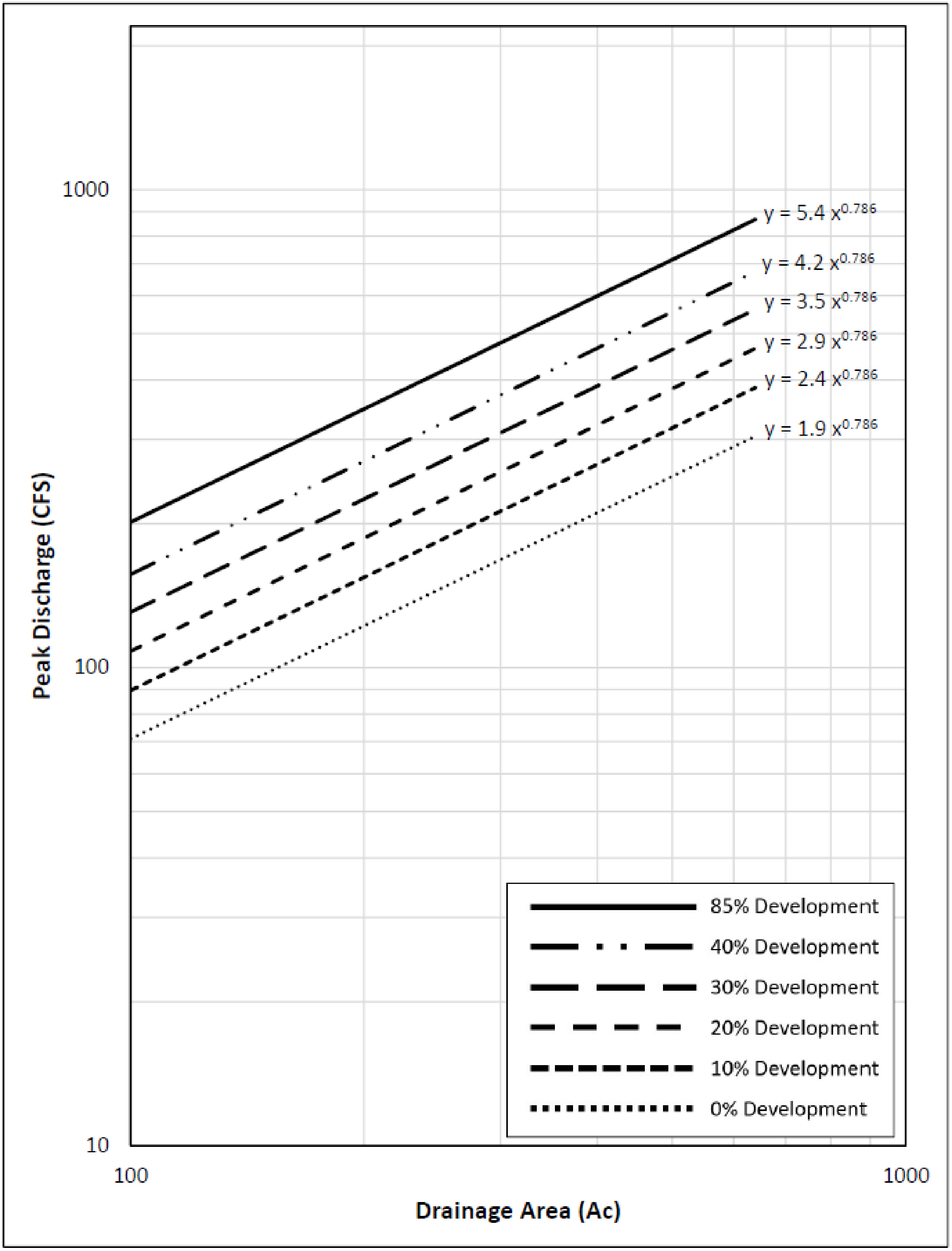


Figure 2-3, 5-Year Drainage Area-Discharge Curves for Brazoria County, TX

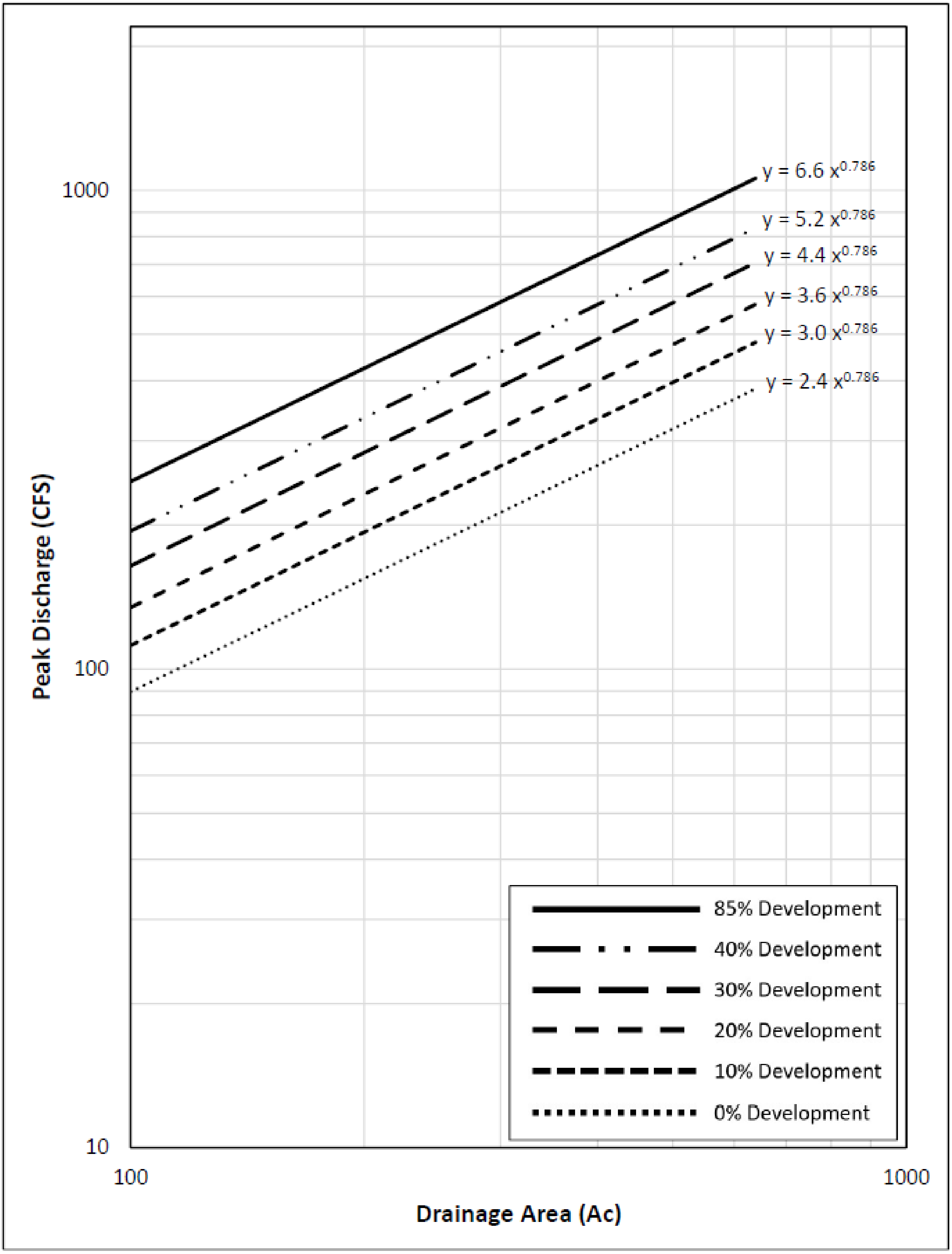


Figure 2-4, 10-Year Drainage Area-Discharge Curves for Brazoria County, TX

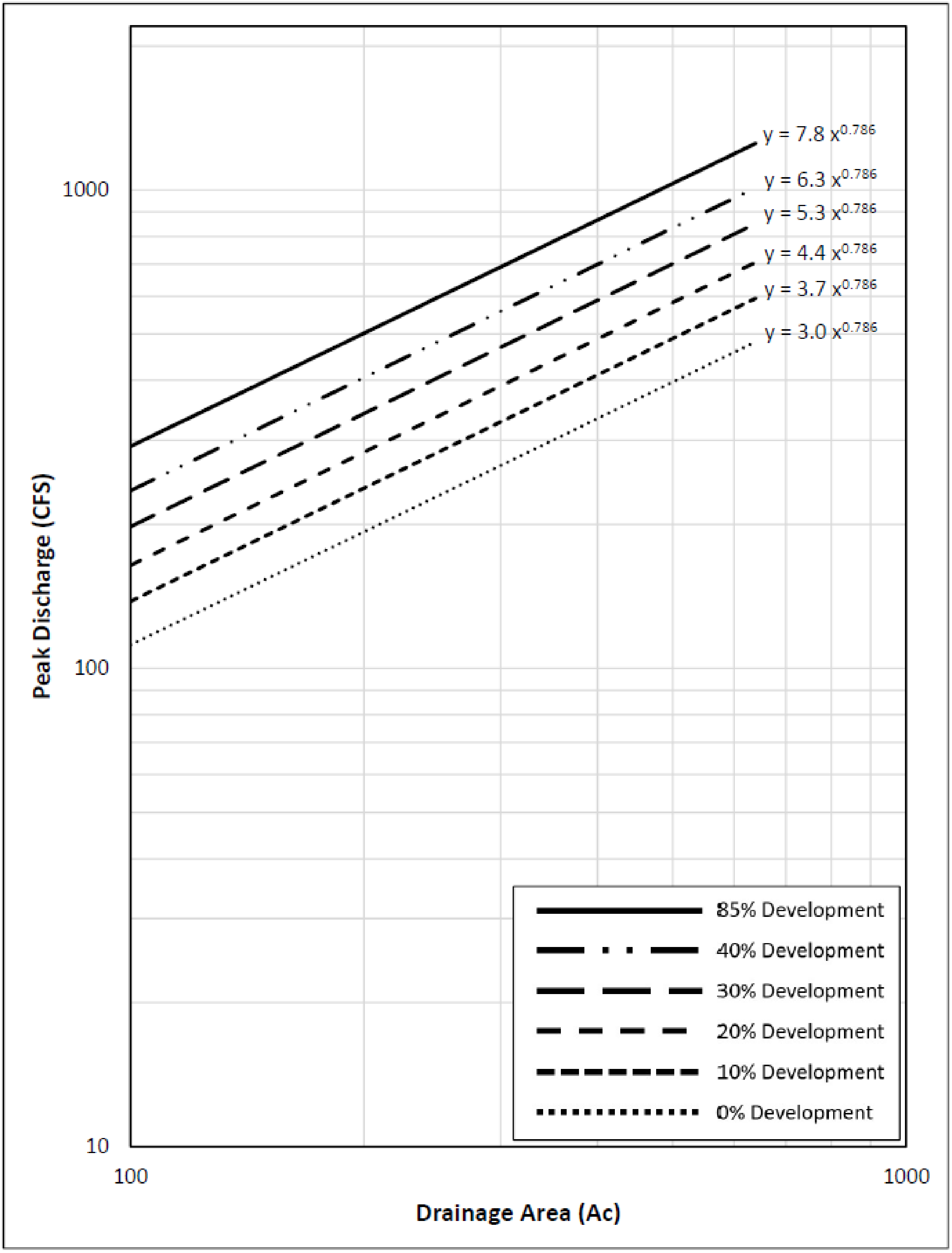


Figure 2-5, 25-Year Drainage Area-Discharge Curves for Brazoria County, TX

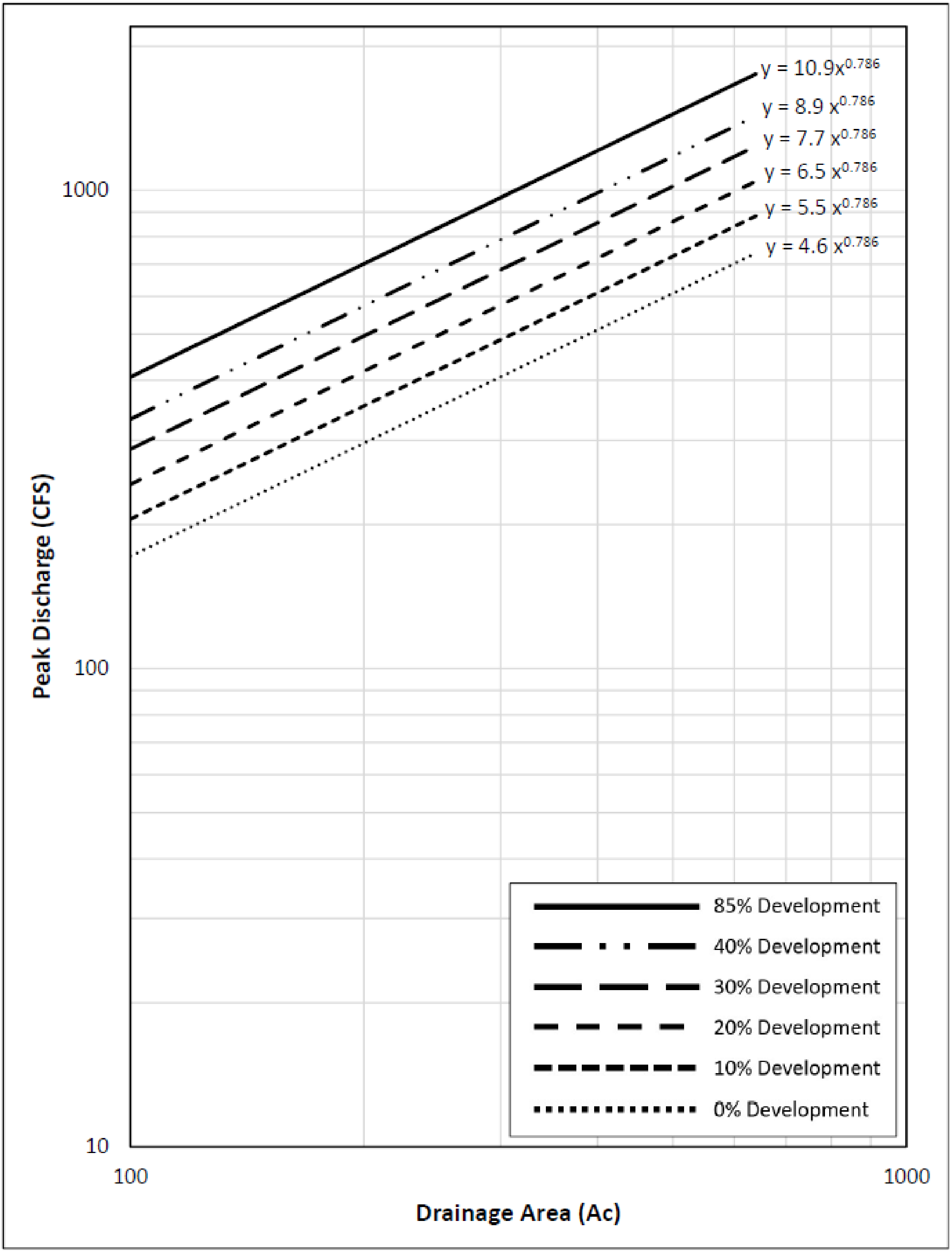


Figure 2-6, 100-Year Drainage Area-Discharge Curves for Brazoria County, TX

2.3 Hydrograph Development

In some circumstances, a full hydrograph will need to be developed during the hydrologic analysis. For watersheds up to and including 640 acres in size, Malcom's Method may be used in coordination with the appropriate peak flow determination method described in Section 2.3.1. Otherwise, the Clark Unit Hydrograph method shall be used.

For areas larger than 640 acres, rainfall-runoff computations will be required using the Clark Unit Hydrograph method described in Section 2.3.2.

2.3.1 Malcom's Method (up to and including 640 acres)

A technique for hydrograph development which is useful in the design of detention facilities serving relatively small watersheds (up to approximately 640 acres) was presented by H.R. Malcom. In Brazoria County, the method is acceptable for developments with contributing drainage areas up to and including 640 acres. This procedure can be used in conjunction with peak flows developed from either the drainage area-discharge curves or the Rational Method. Malcolm's methodology utilizes a pattern hydrograph to obtain a curvilinear design hydrograph which peaks at the design flow rate and which contains a runoff volume consistent with the design rainfall. The pattern hydrograph is a step function approximation to the dimensionless hydrograph proposed by the Bureau of Reclamation and the Soil Conservation Service.

The Malcom's Method consists of the following equations:

$$T_P = \frac{V}{1.39Q_p} \tag{2-14}$$

$$q_i = \frac{Q_p}{2}\left[1 - cos\left(\frac{\pi ti}{T_p}\right)\right] \qquad for\ t_i < 1.25T_p \tag{2-15}$$

$$q_i = 4.34Q_p e^{-1.30\frac{t_i}{T_p}} \qquad for\ t_i > 1.25T_p \tag{2-16}$$

*Cosine function expressed in radians (not degrees)

Where,

Q_p	=	Peak design flow rate in cubic feet per second (cfs);
T_p	=	Time to Q_p (sec);
V	=	Total volume of runoff for the design storm in (ft^3);
T_i	=	Time which determines the shape of the hydrograph (sec);
q_i	=	Flow rate which determines the shape of the hydrograph (cfs)

A plot of a hydrograph illustrating this equation set and parameters is included in Figure 2-7.

The volume of the hydrograph is determined by the total volume of runoff produced by the drainage area and is dependent on the level of development of the area (i.e., percent of impervious cover). Loss rate values for the 5-year, 10-year, 25-year, and 100-year, 24-hour rainfall events are included in Table 2-4.

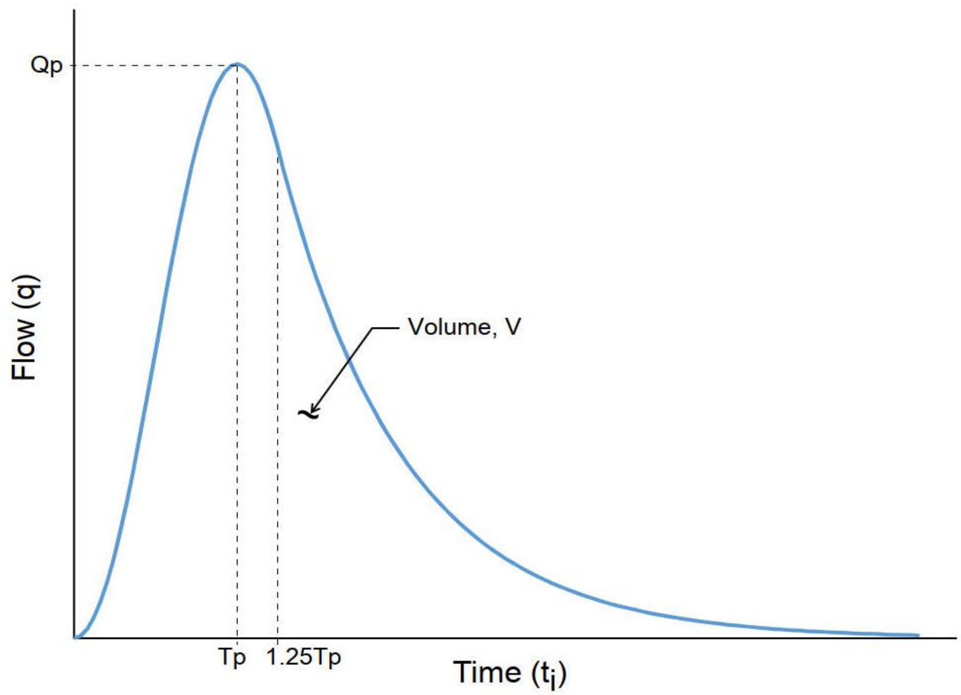


Figure 2-7 Malcom Method for Hydrograph Development

Source: Policy, Criteria, and Procedure Manual for the Approval and Acceptance of Infrastructure. Harris County Flood Control District, October 2018

It should be noted that the underlying assumptions of rainfall patterns and timing used in the Malcom method differ from those in analyses of larger areas using HEC-HMS. Therefore, the resulting hydrograph from the Malcom method shall not be used as an input for a small area flowing into a larger stream system analyzed with HEC-HMS.

Table 2-4 Excess Rainfall for Computing Runoff Volumes

Percent Imper-vious	2-YR, 24-HR Rainfall			5-YR, 24-HR Rainfall			10-YR, 24-HR Rainfall			25-YR, 24-HR Rainfall			100-YR, 24-HR Rainfall		
	Total	Losses	Excess	Total	Losses	Excess	Total	Losses	Excess	Total	Losses	Excess	Total	Losses	Excess
	in	in	in	in	in	in	in	in	in	in	in	in	in	in	in
Region 1															
0	5.27	2.06	3.21	7.05	2.29	4.76	8.83	2.43	6.40	11.60	2.58	9.02	17.00	2.70	14.30
20	5.27	1.67	3.60	7.05	1.85	5.20	8.83	1.96	6.87	11.60	2.08	9.52	17.00	2.18	14.82
40	5.27	1.28	3.99	7.05	1.41	5.64	8.83	1.50	7.33	11.60	1.59	10.01	17.00	1.66	15.34
60	5.27	0.89	4.38	7.05	0.98	6.07	8.83	1.03	7.80	11.60	1.09	10.51	17.00	1.14	15.86
80	5.27	0.49	4.78	7.05	0.54	6.51	8.83	0.57	8.26	11.60	0.60	11.00	17.00	0.62	16.38
Region 2															
0	5.20	2.02	3.18	6.93	2.25	4.68	8.61	2.38	6.23	11.20	2.53	8.67	16.00	2.66	13.34
20	5.20	1.64	3.56	6.93	1.82	5.11	8.61	1.93	6.68	11.20	2.05	9.15	16.00	2.15	13.85
40	5.20	1.25	3.95	6.93	1.39	5.54	8.61	1.47	7.14	11.20	1.56	9.64	16.00	1.64	14.36
60	5.20	0.87	4.33	6.93	0.96	5.97	8.61	1.01	7.60	11.20	1.07	10.13	16.00	1.12	14.88
80	5.20	0.48	4.72	6.93	0.53	6.40	8.61	0.56	8.05	11.20	0.59	10.61	16.00	0.61	15.39

2.3.2 Clark Unit Hydrograph Method for HEC-HMS (greater than 640 acres)

A stream network model that simulates the runoff response of a river basin to applied rainfall can be developed utilizing the HEC-HMS computer program with the appropriate combination of hydrograph and routing computations. The following sections describe the elements required to develop a HEC-HMS computer model. For Brazoria County, the Clark Unit Hydrograph Method is the default methodology. The Drainage Regulatory Entity shall provide written approval prior to a drainage study submittal for any other unit hydrograph method.

2.3.2.1 Design Storm Rainfall

A point rainfall depth and intensity distribution need to be determined to develop a hydrograph. A design hydrograph is developed from a point rainfall depth based on the precipitation frequency analyses for a given area.

Design storm rainfall can be described in terms of frequency, duration, areal extent, and distribution of intensity with time. A design storm's rainfall distribution in the HEC-HMS program offsets the intensity position of the hyetograph by 67%. In most cases, the design shall be based on a 24-hour duration storm event.

In September of 2018, the National Oceanic and Atmospheric Administration (NOAA) released updated rainfall values for Texas in the form of "NOAA Atlas 14, Volume 11 Precipitation-Frequency Atlas of the United States, Texas". The publication updated, detailed, and localized rainfall values for the State of Texas. In the southeastern part of the state particularly, this new research yielded increased rainfall intensities from those previously determined using Technical Paper No. 40 (TP-40, U.S. Department of Commerce, Weather Bureau) and Technical Memorandum NWS Hydro-35.

In Brazoria County, data from TP-40 and NWS Hydro-35 are superseded by values from Atlas 14. TP-40 and NWS Hydro-35 shall no longer be used for new development.

Table 2-5 and Table 2-6 were developed using NOAA Atlas 14 data. The design storm rainfall for areas located in Drainage Districts 1, 2, 3, 4, 5, and 8 (see Figure 2-8, Region 1) can be obtained from Table 2-5. For areas located in District 11 and portions of Brazoria County outside of a Brazoria County drainage district (see Figure 2-8, Region 2) rainfall can be obtained from Table 2-6. The tables give depth vs. duration data for a variety of storm frequencies, from the 1-year to the 1000-year storm. The tabular data in Table 2-5 and Table 2-6 is Partial Duration Series data, as opposed to Annual Maximum Series data. Partial Duration Series shall be used for all storms analyzed in Brazoria County.

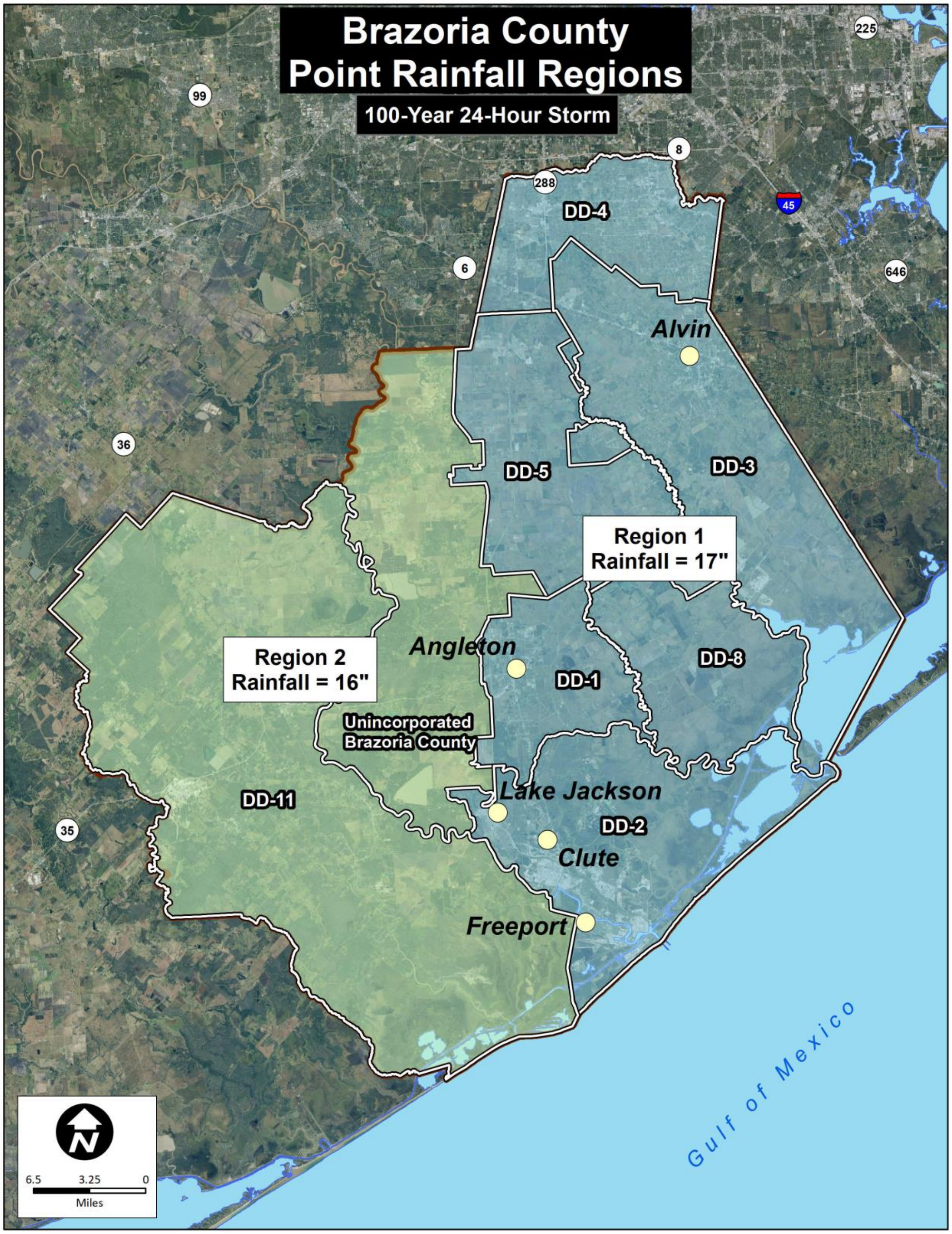


Figure 2-8 Brazoria County Point Rainfall Regions

Table 2-5 Partial Duration Precipitation Frequency Estimates by Annual Exceedance Probability for Region 1

	100%	50%	20%	10%	4%	2%	1%	0.2%	0.1%
	1-Year	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year	1000-Year
5-min	0.51	0.60	0.74	0.86	1.04	1.17	1.31	1.65	1.80
10-min	0.81	0.95	1.18	1.37	1.65	1.87	2.09	2.59	2.80
15-min	1.03	1.21	1.49	1.73	2.07	2.34	2.61	3.27	3.56
30-min	1.48	1.73	2.13	2.47	2.94	3.30	3.68	4.66	5.11
60-min	1.96	2.31	2.86	3.33	3.99	4.50	5.05	6.56	7.29
2-hr	2.39	2.90	3.63	4.32	5.36	6.24	7.23	9.91	11.20
3-hr	2.62	3.25	4.11	4.97	6.29	7.48	8.84	12.50	14.30
6-hr	3.04	3.88	4.99	6.15	7.97	9.66	11.60	17.00	19.60
12-hr	3.49	4.55	5.98	7.43	9.72	11.80	14.30	21.20	24.80
24-hr	3.96	5.27	7.05	8.83	11.60	14.10	17.00	25.30	29.60
2-day	4.44	6.01	8.18	10.30	13.60	16.60	20.00	29.00	33.40
3-day	4.81	6.53	8.92	11.30	14.90	18.10	21.70	30.90	35.20
4-day	5.15	6.95	9.47	11.90	15.60	19.00	22.70	31.90	36.30

Source: NOAA Atlas 14 Volume 11 Version 2, September 2018, Partial Duration Series Rainfall Depths (Districts 1, 2, 3, 4 5, 8; Latitude: 29.3667°, Longitude: -95.3597°)

Table 2-6 Partial Duration Precipitation Frequency Estimates by Annual Exceedance Probability for Region 2

	100%	50%	20%	10%	4%	2%	1%	0.2%	0.1%
	1-Year	**2-Year**	**5-Year**	**10-Year**	**25-Year**	**50-Year**	**100-Year**	**500-Year**	**1000-Year**
5-min	0.52	0.604	0.733	0.847	1.01	1.14	1.28	1.60	1.74
10-min	0.82	0.96	1.16	1.35	1.61	1.83	2.04	2.52	2.71
15-min	1.05	1.22	1.47	1.70	2.02	2.28	2.56	3.18	3.44
30-min	1.52	1.75	2.11	2.42	2.87	3.23	3.60	4.52	4.93
60-min	2.01	2.33	2.83	3.26	3.89	4.40	4.94	6.34	7.00
2-hr	2.44	2.92	3.62	4.28	5.26	6.10	7.02	9.43	10.60
3-hr	2.67	3.27	4.12	4.94	6.19	7.30	8.54	11.80	13.30
6-hr	3.08	3.89	5.01	6.11	7.83	9.38	11.10	15.80	18.10
12-hr	3.52	4.53	5.95	7.33	9.48	11.40	13.60	19.70	22.80
24-hr	3.97	5.20	6.93	8.61	11.20	13.40	16.00	23.40	27.10
2-day	4.40	5.85	7.91	9.89	12.90	15.60	18.60	26.70	30.70
3-day	4.74	6.32	8.57	10.70	14.00	16.90	20.10	28.40	32.30
4-day	5.09	6.73	9.11	11.30	14.70	17.70	21.00	29.30	33.10

Source: NOAA Atlas 14 Volume 11 Version 2, September 2018, Partial Duration Series Rainfall Depths (District 11 and Brazoria County, Texas, USA; Latitude: 29.2334°, Longitude: -95.694°)

2.3.2.2 Hydrologic Losses

The volume of rainfall that becomes runoff is the “excess” rainfall. The differences between the observed total rainfall hyetograph and the excess rainfall hyetograph are termed “abstractions” or “losses”. These losses represent interception, depression storage, and most significantly, soil infiltration. The calculated loss values will be utilized in determining the design storm runoff described in Section 2.3.2.3.

The recommended procedure for calculating abstractions in Brazoria County is the Green and Ampt Loss Method combined with the Simple Canopy Method. A tabulation of the required Green and Ampt parameter values to be used in Brazoria County are shown in Table 2-7. Any alternative methodology shall be approved in writing by the appropriate Drainage Regulatory Entity prior to submittal.

Because the Green and Ampt method is based upon the physical characteristics of the soil, it is not as reliant on the need for estimation and gage calibration as the previously used Exponential Loss Method. However, if adequate gage data is available for calibration, it should be considered. The HEC-HMS user's manual contains a detailed discussion of this method, and descriptions of the necessary input parameters are included below:

- Initial Canopy Storage: the percentage of the canopy that is full of water at the beginning of the simulation.
- Max Canopy Storage: the maximum amount of water that can be stored in the canopy, held on leaves before fall-through to the surface begins; expressed as an effective depth of water in inches.
- Crop Coefficient: a ratio applied to the potential evapotranspiration when computing the amount of water to extract from the soil.
- Initial Content: initial saturation of the soil at the beginning of the simulation; specified in terms of a volume ratio.
- Saturated Content: the maximum holding capacity of the soil; expressed as a volume ratio. This parameter is a function of the soil texture and is assumed to be the total porosity of the soil.
- Suction or Wetting Front Suction: a function of the soil texture and is expressed in inches.
- Hydraulic Conductivity: the volume of water that will flow through a unit of soil in a given time; expressed in inches per hour. This parameter is determined based on the soil texture with consideration of the hydrologic soil group.
- Impervious %: the percentage of the sub-basin which is impervious area.

Table 2-7 Green and Ampt Loss Parameters in Brazoria County

Initial Canopy Storage (%)	Max Canopy Storage (in)	Crop Coefficient	Initial Content (in)	Saturated Content	Suction (in)	Conductivity (in/hr)
0	0.1	1	0.075	0.46	12.45	0.024

Values for the percentage of impervious cover corresponding to various types of development in Brazoria County are given in Table 2-1. These values shall be used to calculate a composite percentage impervious cover for the proposed project.

2.3.2.3 Hydrograph Model

Using the design storm excess rainfall described in Section 2.3.2.2, the storm runoff hydrograph can be determined at the point of interest using a hydrograph model.

The Clark Unit Hydrograph model simulates the process of converting excess rainfall into a runoff hydrograph. This method reflects two processes: translation of excess runoff from its source to the outlet and attenuation of the excess rainfall due to surface storage within the drainage area. The translation and attenuation for a drainage basin, given its hydraulic conveyance characteristics, is reflected by the time of concentration (TC) and storage coefficient (R) parameters.

The Basin Development Factor (BDF) method is recommended to determine TC and R values for a watershed in Brazoria County. These two parameters influence the shape of the hydrograph. When coupled with the BDF, TC and R reflect the runoff's response to drainage conveyance characteristics of the basin.

2.3.2.3.1 Basin Development Factor

The BDF for a drainage area is a measure of the level of improvements made to a basin's drainage system and in turn, the basin's conveyance and runoff routing efficiency. Improvements to a basin's drainage system typically increase the intensity of the runoff hydrograph and the basin's peak outlet flow rate. The BDF is particularly helpful in identifying changes in the runoff response for a basin due to changes to the drainage conveyance characteristics.

The BDF is composed of two main factors: 1) the main conveyance system (major drainage channels and principal tributaries) for the basin, and 2) the collector system for sub-areas of the basin. These two factors are summed together to determine the final BDF values for a basin. BDF ranges from 0 (representing basins with no improved conveyance systems) to 12 (representing areas with fully effective drainage systems). BDF reflects improvements in the drainage system itself and does not directly account for impervious cover. The following factors are considered when determining BDF:

- Natural Channel (N): This categorization includes the main drainage channel and principal tributaries that remain in a natural state (see Figure 2-9). This may also apply to channels that were once modified but have not been maintained and are taking on characteristics of a natural channel.

Figure 2-9 Natural Creek (N) Example

- Channel Improvements (I): This categorization includes natural creeks that have received man-made improvements (i.e., channelization) including bank grading, deepening, widening, and straightening as well as the implementation of maintenance programs to maintain grass-lined banks. These improvements are frequently completed in connection with urbanization to provide additional conveyance capacity to reduce flooding and sufficient depth for storm sewer outfalls (Figure 2-10).

Figure 2-10 Improved Channel (I) Example

- Channel Linings (C): This category includes main drainage channels and principal tributaries that are lined with an impervious material. Channel linings normally consist of concrete as shown in Figure 2-11. However, riprap, gabions, and other materials may also be used and should be considered in this category.

Figure 2-11 Lined Channel (C) Example

- Undeveloped (U): Undeveloped areas include areas in their undisturbed and natural conditions. This would include wooded areas, rangeland, and other areas that are undeveloped and not graded to drain (see Figure 2-12).

Figure 2-12 Undeveloped Area (U) Example

- Open Space (OS): Open space areas are either undeveloped or minimally developed areas that have been cleared of any trees and have been graded in a way that allows for positive drainage to occur throughout the site. Golf courses, city parks, and regional detention basins would be considered open space (see Figure 2-13). Detention basins within residential or commercial development should be considered part of that development's drainage characterization (impervious cover is calculated separately).

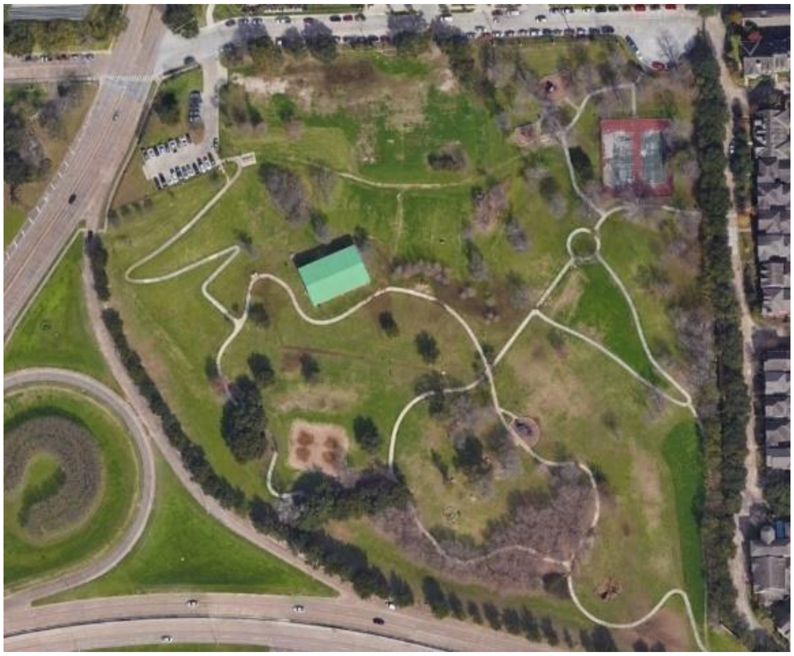


Figure 2-13 Open Space (OS) Example

- Roadside Ditches (R): This category includes areas that are served by roadside ditches as the primary drainage collection and conveyance system (see Figure 2-14). These tend to be industrial or large-acre, rural residential developments.

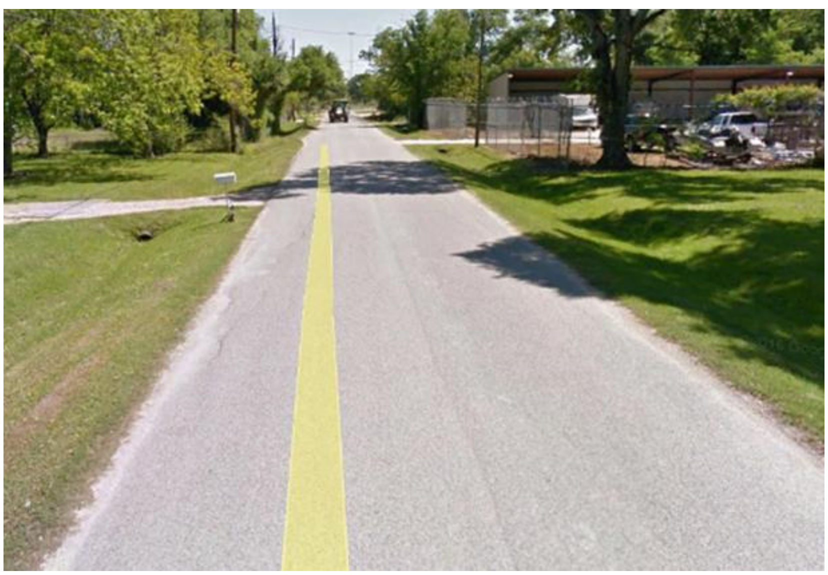


Figure 2-14 Roadside Ditch (R) Example

- Storm Sewers pre-1992 (SS_{pre}-1992): This category includes areas that are served by curb and gutter streets paired with storm sewer as the primary drainage collection and conveyance system but built prior to 1992 (see Figure 2-15). Storm sewer systems designed and built prior to 1992 are assumed to be developed with less stringent drainage criteria and therefore do not have as hydraulically efficient curb and gutter and storm sewer systems as developments designed and built after 1992.

Figure 2-15 Pre-1992 Development (Google Earth 1992 Aerial Imagery)

- Storm Sewers post-1992 (SS_{post}-1992): This category includes areas that are served by curb and gutter streets paired with storm sewer as the primary drainage collection and conveyance system and built after 1992 (see Figure 2-16). Storm sewer systems designed and built after 1992 are assumed to be developed with more stringent drainage criteria and therefore have more hydraulically efficient curb and gutter and storm sewer systems than developments designed and built prior to 1992. Additionally, during high rain events, developments constructed after 1992 were designed to convey extreme event runoff either through the storm sewer or overland by street cascading.

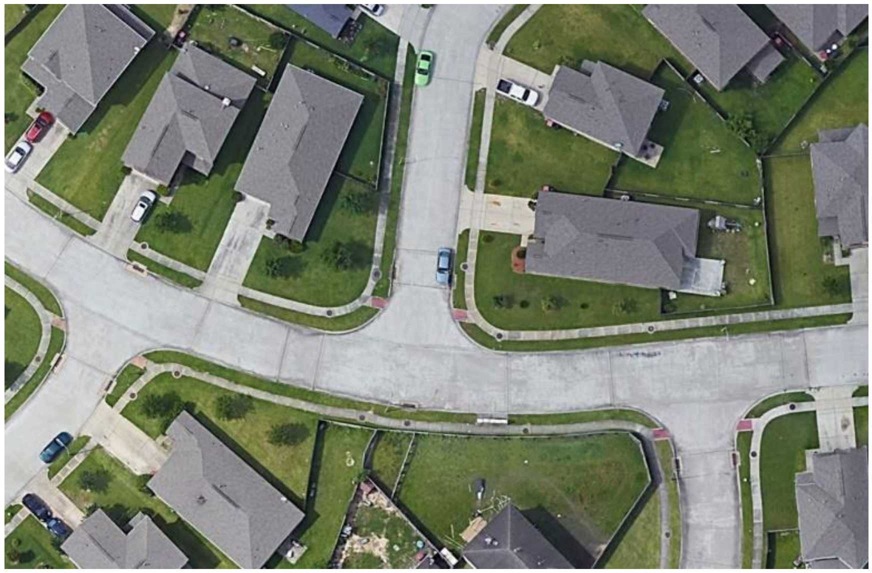


Figure 2-16 Post-1992 Development

To simplify the quantification of these factors, the Brazoria County method of determining BDF uses ratios of the above factors. This allows for the use of GIS applications to produce easily reviewed inputs for the BDF equation. The equation is broken into two parts: the ratio of improved and lined channel to the total channel length, and the ratio of improved drainage system (graded, roadside ditches, and storm sewers) to the total area. If desired, the channel lengths can be estimated as percentages of the total channel, instead of measuring lengths in a GIS application.

$$\mathbf{BDF} = \frac{(\mathbf{I}*\mathbf{3})+(\mathbf{C}*\mathbf{6})}{\mathbf{N}+\mathbf{I}+\mathbf{C}} + \frac{(\mathbf{OS}*\mathbf{1})+(\mathbf{R}*\mathbf{1.5})+\left(\mathbf{SS}_{\mathbf{pre-1992}}*\mathbf{3}\right)+\left(\mathbf{SS}_{\mathbf{post-1992}}*\mathbf{6}\right)}{\mathbf{U}+\mathbf{OS}+\mathbf{R}+\mathbf{SS}_{\mathbf{pre-1992}}+\mathbf{SS}_{\mathbf{post-1992}}} \quad (2\text{-}17)$$

Where,

N	=	Length of natural channel (ft or %);
I	=	Length of improved channel (ft or %);
C	=	Length of concrete channel (ft or %);
U	=	Undeveloped area (ac or %);
OS	=	Open space graded to drain area (ac or %);
R	=	Developed area served by roadside ditch (ac or %);
$SS_{pre\text{-}1992}$	=	Pre-1992 developed area served by storm sewer (ac or %);
$SS_{post\text{-}1992}$	=	Post-1992 developed area served by storm sewer (ac or %)

The result of Equation (2-17) is then used in the following equations to determine base TC and R values to be used in the Clark Unit Hydrograph model.

$$\mathbf{T_r = 10^{[(-0.05228*BDF)+0.4028\log_{10}(A)+0.3926]}} \qquad (2\text{-}18)$$

$$\mathbf{TC_{BDF} = T_r + \frac{\sqrt{A}}{2}} \qquad (2\text{-}19)$$

$$\mathbf{R_{BDF} = 8.271e^{-0.1167*BDF} \times A^{0.3856}} \qquad (2\text{-}20)$$

Where,

BDF	=	Basin Development Factor (0 to 12, dimensionless);
A	=	Drainage area to point of interest (sq mi);
T_r	=	Lag time (hr);
TC_{BDF}	=	Time of concentration based on BDF (hr);
R_{BDF}	=	Clark storage coefficient or residence time based on BDF (hr)

Slope Adjustments

The TC and R values can be significantly dependent on the slope of the watershed. To account for this dependency, slopes in the watershed need to be evaluated and used to develop slope factors to apply to the base TC and R values. The slopes in question are the channel slope and the overland slope, both of which are measured in feet per mile. Measurement of the channel slope over the course of a watershed is straightforward. To measure overland slopes, the engineer shall average the slopes of several representative perpendicular slopes or divide the watershed up into large grid cells (typical grid cells of 10 acres or 660 feet by 660 feet) and average the overland slopes of these cells. The engineer should carefully consider the level of detail required to achieve an accurate representation of the overall watershed slopes, as this factor can have a great effect on the final TC and R values.

$$\mathbf{K_S = -0.162\ln(S \times S_o) + 1.5232} \qquad (2\text{-}21)$$

Where,

S	=	Channel slope measured along the entire watercourse, excluding drops in flowline such as control structures (ft/mi);
S_o	=	Overland slope, average of multiple representative “perpendicular” slopes (ft/mi);
$S \times S_o$	=	Maximum of 26 and the product of both numbers (i.e., if $S \times S_o < 26$, use 26);
K_S	=	Slope factor to be applied to the time of concentration and Clark storage coefficient and should be less than 1.0

Detention Adjustments

The base TC and R values shall also be adjusted to account for detention within the watershed that is not within the 100-year floodplain (basins within the 100-year floodplain will be accounted for within the hydraulic calculations discussed in the next section of this manual). Care should be taken to confirm the accuracy of whatever data is being used to calculate the detention. If there are developments within the watershed that have been constructed since the most recent LiDAR data has been released, the engineer will need to utilize other methods of accounting for these developments, such as as-built construction plans or field survey data.

The Detention Rate (DR) used when calculating the detention correction factor is not the same as the typical detention rate used in development. The engineer shall be careful not to mix these two parameters. Regional detention basins which are analyzed as part of the hydraulic modeling of the floodplain can be omitted from the DR calculation since they will be analyzed separately in the hydrologic model. Large storage areas such as sand or gravel pits shall not be included in these calculations as they are unlikely to produce runoff. If the value calculated for DR is less than 10 for a watershed or subwatershed, this adjustment factor can be disregarded. As with the slope adjustment above, the same correction factor is applied to both TC and R.

$$\mathbf{C_f = 3 \times 10^{-5} \times DR^2 - 0.00095 \times DR + 1} \qquad (2\text{-}22)$$

Where,

DR	=	Detention rate for watershed or subwatershed (ac-ft/sq. mi);
C_f	=	Correction factor for detention (for DR $\geq$ 10) to be applied to both TC and R

Ponding Adjustments

Increased levels of ponding can also affect the base TC and R values. The ponding adjustment has typically been applied to subwatersheds with agricultural applications that use small levees to pond water within a given area, particularly rice farming. However, this adjustment has also been used for other applications where not all runoff for a subwatershed makes it to the main watercourse. The adjustment is based on the size of the area draining through a ponded area and not just the ponded area itself. Therefore, when performing ponding adjustments, the engineer shall be careful to include all areas of the watershed affected by ponding. As a result, an area of ponding at the upstream end of a watershed may have a small effect on the watershed overall, but a ponded area at the downstream end may affect a large portion of the upstream watershed area.

Ponding adjustments vary by return period, as shown in the equations listed below. Only ponding that affects 20% or more of the watershed is generally considered; watersheds with less than 20% of the area affected by ponding do not require this adjustment. However, this adjustment factor can be used for calibration purposes for watersheds with less than 20% of the area affected by ponding, as needed.

Areas affected by ponding may be delineated for agricultural fields where terracing or levees is evident in the latest LiDAR dataset. The USDA 2018 Cropland Data Layer can also be referenced to identify active and fallow rice fields. Areas encompassed by large berms or levees and large sand pits can also be identified from these sources.

$$\mathbf{RM_2 = 1.33 \times DPP^{0.242}} \quad (2\text{-}23)$$

$$\mathbf{RM_5 = 1.31 \times DPP^{0.214}} \quad (2\text{-}24)$$

$$\mathbf{RM_{10} = 1.28 \times DPP^{0.199}} \quad (2\text{-}25)$$

$$\mathbf{RM_{25} = 1.25 \times DPP^{0.171}} \quad (2\text{-}26)$$

$$\mathbf{RM_{50} = 1.23 \times DPP^{0.153}} \quad (2\text{-}27)$$

$$\mathbf{RM_{100} = 1.21 \times DPP^{0.132}} \quad (2\text{-}28)$$

$$\mathbf{RM_{200} = 1.19 \times DPP^{0.113}} \quad (2\text{-}29)$$

$$\boldsymbol{RM_{500} = 1.17 \times DPP^{0.086}} \quad (2\text{-}30)$$

Where,

DPP = Percentage of the watershed affected by ponding (between 0 to 100);

RM_x = Adjustment factor for the x-year frequency storm.

Final TC and R Values

To determine the final TC and R values to be used in the Clark Unit Hydrograph model, the following two equations may be used, with the parameters calculated above:

$$\boldsymbol{TC = K_S \times C_f \times \left(T_r + \frac{\sqrt{A}}{2}\right)} \quad (2\text{-}31)$$

$$R = K_S \times C_f \times RM_n \times \left(8.271e^{-0.1167 \times BDF} \times A^{0.3856}\right) \qquad (2\text{-}32)$$

Where,

K_S	=	Slope factor to be applied to both TC and R;
C_f	=	Correction factor for detention to be applied to both TC and R for DR > 10;
T_r	=	Lag time (hr);
A	=	Watershed area to point of interest (sq. mi.);
RM_n	=	Ponding factor applied to Clark storage coefficient (n=return period) for DPP ≥ 20%;
TC	=	Adjusted time of concentration (hr);
R	=	Adjusted Clark storage coefficient or residence time (hr)

Using HEC-HMS, or a comparable hydrologic modeling software, these parameters may then be combined with the Clark Unit Hydrograph model to develop the runoff hydrograph at the point of interest. The time-area curve is handled internally by HEC-HMS unless the engineer specifies a particular time-area relationship. An example of the step-by-step procedure for the development of a design runoff hydrograph is presented in Section 2.3.2.3.2. A spreadsheet for calculating TC and R by the above method is available from the Brazoria County Engineering Department.

2.3.2.3.2 Procedure for Developing a Design Runoff Hydrograph

The following general procedure (and example) shall be followed in developing design runoff hydrographs in Brazoria County:

1. The design storm specified by the Drainage Regulatory Entity is the 100-year, 24-hour storm.
2. Develop the design storm hyetograph. This process can be carried out internally by HEC-HMS as discussed in Section 2.3.2.1. HEC-HMS requires depth-duration data, which shall be used as presented in Table 2-5 and Table 2-6.
3. Determine hydrologic losses. Calculate the composite impervious cover for the watershed. Combine the composite impervious cover percentage with the Green and Ampt parameters in HEC-HMS. For all watershed calculations in Brazoria County, the Green and Ampt parameter values presented in Section 2.3.2.2 of the manual are required input for a HEC-HMS model.
4. Determine the Basin Development Factor for the watershed, as described in Section 2.3.2.3.

Example 2.2.1 Determine the Basin Development Factor for a Watershed

Given: The BDF parameters of the watershed, which are illustrated in Figure 2-17, are:

N (natural channel) = 1553 ft

I (improved channel) = 7988 ft

C (concrete lined channel) = 0 ft

U (undeveloped) = 78 ac

OS (open space graded to drain) = 377 ac

R (area drained by roadside ditches) = 43 ac

$SS_{pre\text{-}1992}$ (pre-1992 storm sewer development) = 33 ac

$SS_{post\text{-}1992}$ (post-1992 storm sewer development) = 160 ac

Solution:

$$BDF = \frac{(I \times 3) + (C \times 6)}{N + I + C} + \frac{(OS \times 1) + (R \times 1.5) + \left(SS_{pre-1992} \times 3\right) + \left(SS_{post-1992} \times 6\right)}{U + OS + R + SS_{pre-1992} + SS_{post-1992}}$$

$$= \frac{(7988 \times 3) + (0 \times 6)}{1553 + 7988 + 0} + \frac{(377 \times 1) + (43 \times 1.5) + (33 \times 3) + (160 \times 6)}{78 + 377 + 43 + 33 + 160}$$

$$BDF = 4.68$$

5. Determine base TC and R values using BDF. The Clark parameters are determined by solving Equation (2-31) and Equation (2-32).

Example 2.2.2 Determine the base TC and R values using BDF

Given:

BDF = 4.68
A = 1.08 mi^2

Solution:

$$T_r = 10^{[(-0.05228 \times BDF)+0.4028 \log_{10} A+0.3926]}$$
$$= 10^{[(-0.05228 \times 4.68)+0.4028 \log_{10} 1.08+0.3926]}$$
$$T_r = 1.45\ hours$$

$$TC_{BDF} = T_r + \frac{\sqrt{A}}{2}$$
$$= 1.45 + \frac{\sqrt{1.08}}{2}$$
$$TC_{BDF} = 1.97\ hours$$

$$R_{BDF} = 8.271e^{-0.1167 \times BDF} \times A^{0.3856}$$
$$= 8.271e^{-0.1167 \times 4.68} \times 1.08^{0.3856}$$
$$R_{BDF} = 4.93\ hours$$

6. Determine slope adjustment factor using Equation (2-21)

Example 2.2.3 Determine the slope adjustment factor

Given: In this example watershed, the channel slope (S) is 3 feet per mile and the overland slope (S_o) estimated as the average of the slopes of several overland flow paths perpendicular to the channel is 7 feet per mile. The product SxS_o is 3x7= 21, which is less than 26. Then, a value of 26 is entered as SxS_o in Equation (2-21).

Solution:

$$K_S = -0.162 \ln(S \times S_o) + 1.5232$$
$$= -0.162 \ln(26) + 1.5232$$
$$K_S = 1.00$$

7. Determine detention adjustment factor using Equation (2-22)

Example 2.2.4 Determine the detention adjustment factor
Given: The subwatershed is determined to have a detention volume that is not modeled directly of 103 acre-feet. The total area of the subwatershed is 1.08 square miles, yielding a DR value of 95.4 acre-feet per square mile. **Solution:** $C_f = 3 \times 10^{-5} \times DR^2 - 0.00095 \times DR + 1$ $= 3 \times 10^{-5} \times 95.4^2 - 0.00095 \times 95.4 + 1$ $C_f = 1.18$

8. Determine ponding adjustments using Equation (2-28)

Example 2.2.5 Determine the ponding adjustments for a 100-year storm
Given: Based on aerial photography and LiDAR data, it was determined that 31% of the watershed is affected by ponding. **Solution:** $RM_{100} = 1.21 \times DPP^{0.132}$ $= 1.21 \times 31^{0.132}$ $\mathrm{RM}_{100} = 1.904$

9. Determine Final TC and R values using Equation (2-31) and (2-32) and input into HEC-HMS.

Example 2.2.6 Determine the final TC and R
Given: $K_S = 1.00$ $C_f = 1.18$ $T_r = 1.45$ hours $A = 1.08$ mi^2 $RM_{100} = 1.904$ BDF = 4.68 **Solution:** $TC = K_S \times C_f \times \left(T_r + \frac{\sqrt{A}}{2}\right)$ $= 1.00 \times 1.18 \times \left(1.45 + \frac{\sqrt{1.08}}{2}\right)$ $TC = 1.00 \times 1.18 \times \left(1.45 + \frac{\sqrt{1.08}}{2}\right) = 2.33\ hours$ $R = K_S \times C_f \times RM_n \times (8.271e^{-0.1167 \times BDF} \times A^{0.3856})$ $= 1.00 \times 1.18 \times 1.904$ $\times (8.271e^{-0.1167 \times 4.68} \times 1.08^{0.3856})$ $R = 11.09\ hours$

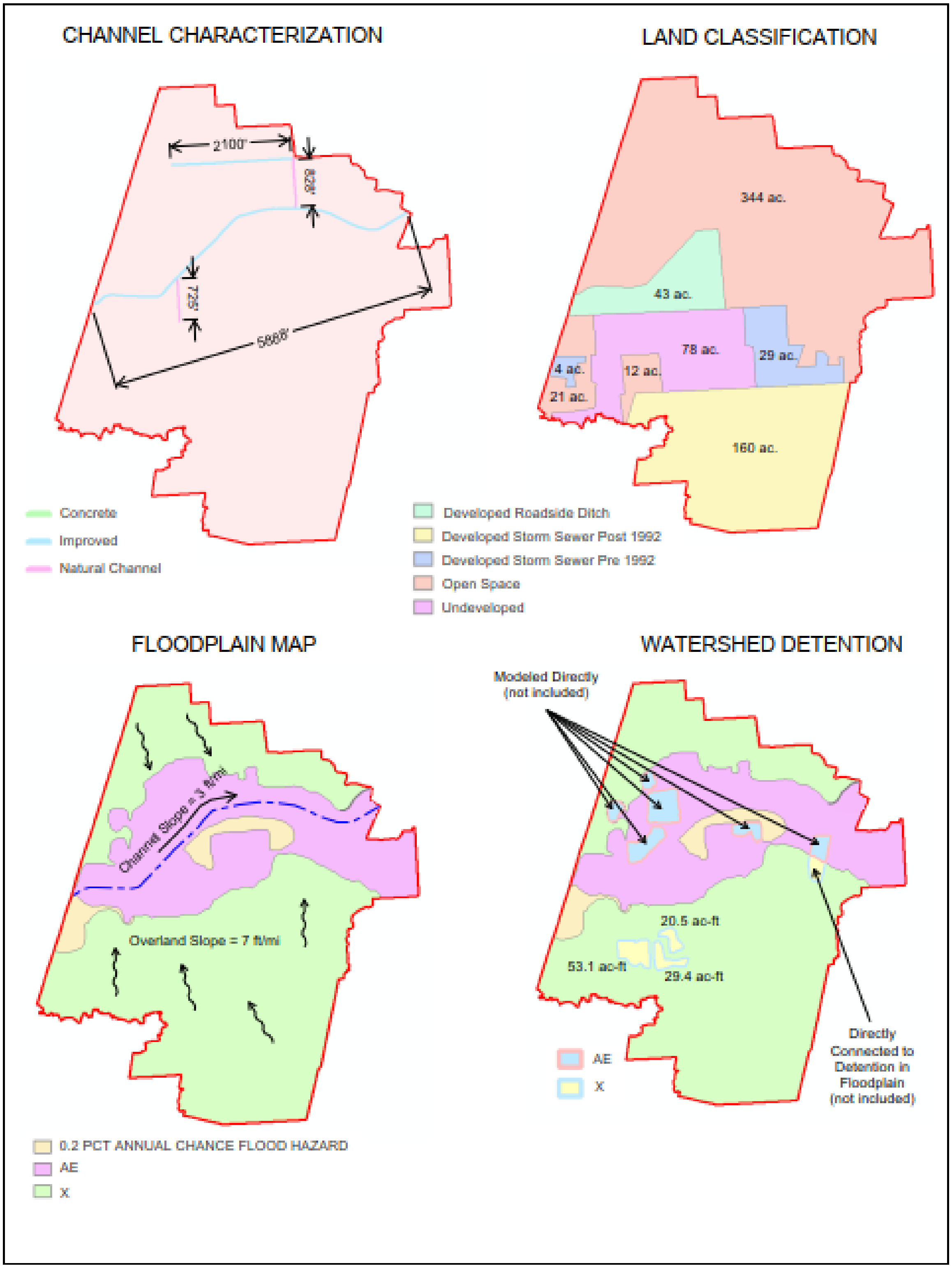


Figure 2-17 Example Watershed for Application of Basin Development Factor (BDF)

2.3.2.4 Flood Routing

As a flood wave passes downstream through a floodplain or detention facility, its shape is altered because of storage. The procedure for determining how the shape of the flood hydrograph changes is called flood routing. Flood routing can be used to determine the effects of storage on a flood's runoff pattern or hydrograph.

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

For channel routing in Brazoria County, only the Modified Puls method and the Muskingum method are acceptable. The method to be used depends on the information available to calculate channel and floodplain storage. For Reservoir Routing, the Modified Puls is recommended. If a HEC-RAS steady flow model is available for the stream being studied, the recommended method is also Modified Puls using the technique outlined in the HCFCD's *Hydrology and Hydraulics Guidance Manual*. If a HEC-RAS unsteady flow model is available, it is suggested to complete the routing using Muskingum method in HEC-HMS with the parameters obtained from the unsteady-flow HEC-RAS model.

2.3.2.4.1 Modified Puls Method

The Modified Puls routing method, known as storage routing or level-pool routing, assumes a non-variable discharge-storage relationship and a constant level pool in the storage reach or reservoir of interest. This Modified Puls technique is available in the HEC-HMS software. However, to derive the storage-discharge relationship required for channel routing, the HEC-RAS backwater program can be used. The model needs to be run for different flow conditions (low to high range of flow). It is recommended to use a percentage of the 100-year flow to develop the rating data (e.g., 10%, 20%, 30%, …, 120%). The cross-sections need to define the entire floodplain storage available at various water levels. However, only the effective flow area of the cross-section is used to establish the proper discharge-water level relationship.

The required storage-elevation-discharge relationships for this routing technique can be obtained from a HEC-RAS model, developed by elevation and storage data estimated or measured from topographic data, by elevation and discharge data developed through the use of structure/spillway options in HEC-HMS, or a rating curve developed outside of HEC-HMS which represents the hydraulic properties of the structure/spillway.

For detailed information and discussion of the Modified Puls routing technique and other methodologies, the engineer is referred to the *Handbook of Applied Hydrology*, by Ven Te Chow, 1964.

2.3.2.4.2 Muskingum Method

Muskingum routing is based on an assumed linear relationship between a channel's storage and inflow and outflow discharge; and consequently, it accounts for prism (related to the flow of relatively uniform depth below the wedge) and wedge storage (related to the sloping water surface). The Muskingum method conforms to the standard mass balance equation used in routing, with coefficients of K and x added to represent the prism and wedge storage components, respectively. K and x are assumed to be constant throughout the full range of flows. The value of K is typically related to the travel time (in hours) of the flood wave through the reach. The value of x is a weighting factor that varies between 0.0 (uniform flow with no backwater) and 0.5 (maximum wedge storage typical of a steep channel entering a reservoir).

The required K and x values for this routing technique can be estimated using least-squares regression analysis. The HEC-RAS unsteady flow model can be used to develop hydrographs to calibrate K and x values.

For a discussion of the Muskingum routing technique and other methodologies, refer to *Applied Hydrology*, by V.T. Chow, D.R. Maidment, and L.W. Mays, 1988.

3 Open Channel Flow

3.1 General

This section summarizes the practical considerations, technical principles, and criteria necessary for proper design of open channels. When a design approach is to be used that is not covered in this manual, it shall be reviewed and discussed with the Drainage Regulatory Entity prior to commencing significant portions of the design effort.

3.2 Channel Design

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

All open channels in Brazoria County inland from the coastal region (see Chapter 8) shall be designed to adhere to the design requirements below.

3.2.1 Design Considerations

The path taken by an existing, naturally-carved channel often represents the logical pathway of flow. For runoff rates associated with undeveloped conditions, the natural channel is largely stable against erosion and is topographically efficient in draining adjacent land. Therefore, the engineer should take advantage of naturally-carved drainage paths when locating and designing open channels, when possible.

Where development occurs adjacent to existing drainage channels, the developer shall provide right of way or an easement for the ultimate channel section, as determined by the Master Drainage Plan (if available) or other appropriate engineering study, at no cost to the County or Drainage District.

The following design characteristics shall be utilized where possible when designing channels. As much as the design of the open channel will allow, the engineer shall:

1. Maintain creek overbank storage;
2. Follow the natural drainage course;
3. Avoid crossing drainage divides;
4. Avoid tight channel bends;
5. Minimize conflicts with existing buildings, homes, pipelines, and contaminated sites; and

6. Minimize the number of property owners affected.

When the use of these features is not possible, sufficient documentation shall be provided to justify their infeasibility.

3.2.2 Minimum Requirements

For the purposes of this drainage criteria, all channels fall under one of the following categories: roadside ditches; small channels; large channels; or large, rectangular channels. All channels except the large, rectangular channel are assumed to be trapezoidal and grass-lined. For additional guidance concerning trapezoidal, concrete lined channels, see Section 3.3.2.2.

A summary of minimum requirements for each of the channel types is provided in Table 3-1. Further detail for each channel is provided in the relevant sections.

Table 3-1 Minimum Requirements for Channel Types

Channel Type	Roadside Ditch	Small Channel	Large Channel
Description	• Shares a common edge with a roadway • Typically located within the road ROW	• Does not share a common edge with a roadway • Has a service area of less than 100 acres • Typically located within a drainage easement	• Does not share a common edge with a roadway • Has a service area greater than 100 acres • Typically located within a drainage easement
Design Storm	5-Year	100-Year	100-Year
Min. Bottom Width	2 feet	4 feet	6 feet
Min. Depth	18 inches	18 inches	N/A
Max. Depth	4 feet	4 feet	N/A
Min. Invert Longitudinal Slope	0.05%	0.05%	0.05%
Steepest Allowable Side Slope (H:V)	4:1	4:1	4:1
Max. Velocity	See Section 3.3.1.1	See Section 3.3.1.1	See Section 3.3.1.1
Bottom Cross Slope	Not required	Not required	6-inch gradient or 3% (Depends on top width. See Section 3.2.2.4)
Berm Width	2 feet (along road)	15 feet (along one edge)	See Section 3.2.2.4
Freeboard	6 inches	6 inches	12 inches
Bench Section Width	Not allowed	Not allowed	10-foot minimum

3.2.2.1 General Criteria

1. Earthen channel slopes shall be re-vegetated immediately after construction to minimize erosion.

2. The minimum invert slope for an open channel shall be 0.05 percent. Slope of the channel invert is generally governed by topography and the energy head required for flow. Since invert slope directly affects channel velocities, channels shall have sufficient grade to prevent significant siltation, but grades shall not be so large as to create erosion problems. Topographic conditions may necessitate a flatter slope than the minimum requirement in certain areas. In these instances, discussion with the Drainage Regulatory Entity is required.

3. The maximum allowable velocities for grass-lined channels are 4 feet per second for sandy soils and 5 feet per second for clay soils. For expected velocities higher than these maxima, refer to Section 3.3.1.1 for additional erosion protection measures.

4. The values in Table 3-2 of the Manning's roughness coefficient shall be used in man-made channels. Alternative values shall be discussed with the Drainage Regulatory Entity.

Table 3-2 Manning's Roughness Coefficient for Channels

Channel Cover	Manning's "n" Value
Grass-lined	0.04
Concrete-lined	0.015
Riprap-lined	0.04
Natural existing channels	0.04 – 0.08

5. In highly developed areas or in areas where grass-lined channels are expected to receive overland sheet flow, back slope interceptors shall be provided.

3.2.2.2 Roadside Ditch

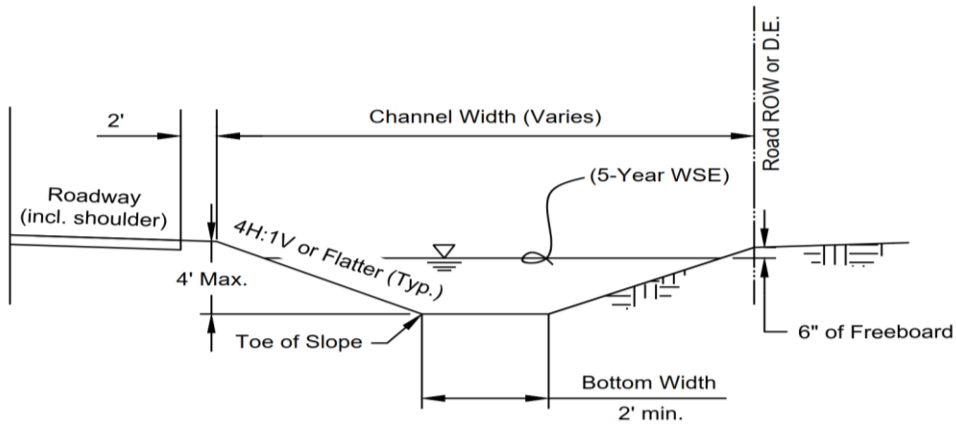


Figure 3-1 Typical Grass-Lined Trapezoidal Roadside Channel Section

A roadside ditch is defined as a trapezoidal channel typically located in a road right-of-way and shares a common edge with a roadway.

Roadside ditches shall be contained completely within the road right-of-way and/or drainage easement.

Roadside ditches may not accept flow from upstream small channels or large channels.

<u>Design Frequency</u>
Roadside ditches shall be designed to convey the fully developed peak flow rate from the 5-year storm event.

<u>Bottom Width</u>
The minimum bottom width of a roadside ditch is 2 feet.

<u>Depth</u>
The maximum depth of a roadside ditch shall be 4 feet. Depths greater than 4 feet shall require a guard rail. The minimum depth shall be 18 inches.

<u>Side Slopes</u>
Side slopes for roadside ditches shall be no steeper than 4 to 1. If design constraints dictate the need for 3 to 1 side slopes, approval from the Drainage Regulatory Entity shall be obtained prior to beginning of design.

<u>Bottom Cross Slope</u>
There is no minimum bottom cross slope for roadside ditches.

Berm Width
Berms are not required for roadside ditch access. Two feet of separation is required between the edge of pavement and the top of bank.

Freeboard
A minimum freeboard of 6 inches to top of bank is required at the maximum 5-year water surface elevation in the roadside ditch.

3.2.2.3 Small Channels

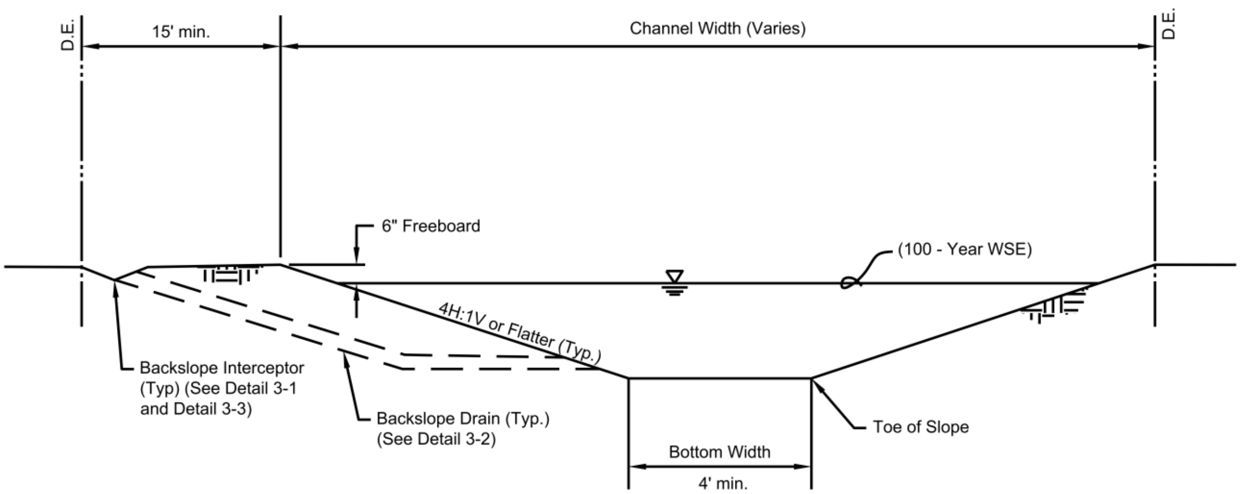


Figure 3-2 Typical Grass-Lined Trapezoidal Small Channel Section

A small channel is defined as a trapezoidal channel that does not share an edge with a roadway and has a service area less than 100 acres. These types of channels typically act as collectors for small storm sewer systems, roadside ditch networks, or route a development's runoff to the outlet channel or detention pond.

Design Frequency
Small channels shall be designed to convey the fully developed peak flow rate from the 100-year storm event.

Bottom Width
The minimum bottom width of a small channel is 4 feet. The maximum bottom width of a small channel is 6 feet. A channel bottom width greater than 6 feet is considered a large channel.

Depth
The maximum depth of a small channel shall be 4 feet and the minimum depth shall be 18 inches. A channel with a depth greater than 4 feet is considered a large channel.

Side Slope
Side slopes for small channels shall be no steeper than 4:1.

Slopes flatter than 4:1 may be necessary in some areas due to local soil conditions. If steeper side slopes are needed for design, a concrete lining may be considered. For specific criteria on concrete-lined channels, see Section 3.3.2.2.

Bottom Cross Slope
There is no minimum bottom cross slope for small channels.

Berm Width
A minimum 15-foot maintenance berm is required on one side of the channel. The elevation of the top of the berm shall be at natural ground along the channel reach.

An additional 15-foot minimum maintenance berm is required the opposite side of the channel if back slope swales are required to prevent sheet flow erosion on that side of the channel.

Freeboard
A minimum freeboard of 6 inches to top of bank is required at the maximum 100-year design storm water surface elevation in the channel.

Ensure that the channel design has sufficient freeboard to drain lateral storm sewers during the 25-year storm.

Geotechnical Investigation
Unless waived by the Drainage Regulatory Entity, a geotechnical investigation and report shall be provided.

3.2.2.4 Large Channels

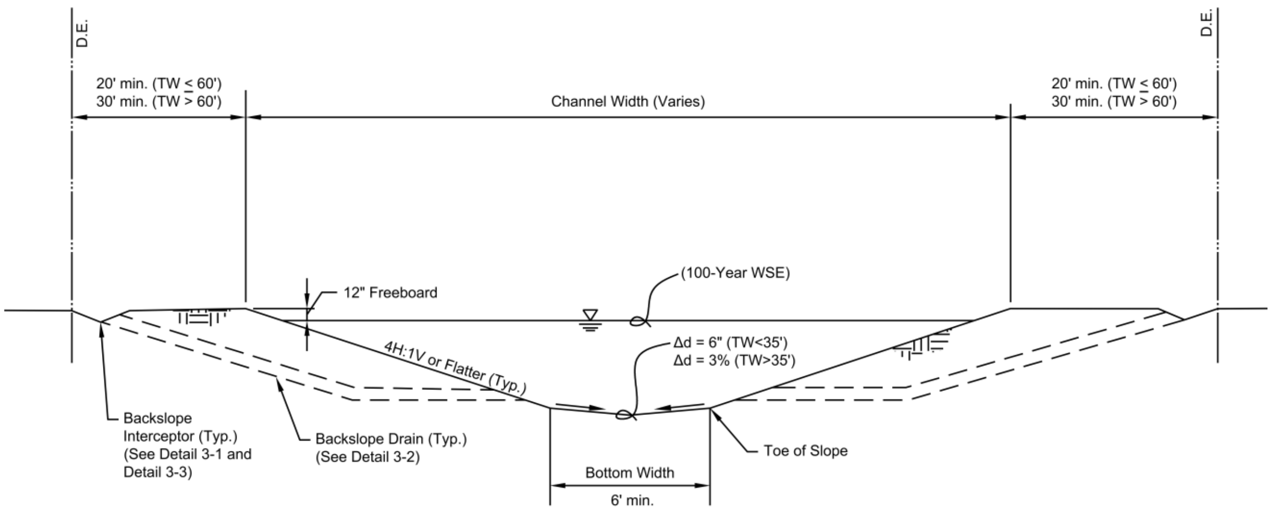


Figure 3-3 Typical Grass-Lined Trapezoidal Large Channel Section

A large channel is defined as a trapezoidal channel that does not share an edge with a roadway and has a service area greater than 100 acres. These types of channels typically act as the main conveyance system for a development and receive flow from small channels, storm sewer systems, detention ponds, etc. These channels are also used to improve existing natural creeks.

In circumstances where the large channel is designed to improve existing creeks in order to control increased flows from a proposed development, proposed water surface profiles shall not exceed the 100-year flood profile under existing conditions.

<u>Design Frequency</u>
Large channels shall be designed to convey the fully developed peak flow rate from the 100-year storm event.

<u>Bottom Width</u>
The minimum bottom width of a large channel is 6 feet.

<u>Side Slope</u>
Side slopes for large channels shall be no steeper than 4 to 1.

Slopes flatter than 4:1 may be necessary in some areas due to local soil conditions. If steeper side slopes are needed for design, a concrete lining may be considered with approval from the Drainage Regulatory Entity. For specific criteria on concrete-lined channels, see Section 3.3.2.2.

<u>Bottom Cross Slope</u>
For channels with a top width of less than 35 feet, the channel bottom shall have a cross slope provided by a 6-inch change in elevation. The minimum bottom cross slope for channels with a top width greater or equal to 35 feet is 3 percent.

Berm Width

A maintenance berm is required on both sides of the channel. For channels with a top width of 60 feet or less, 20-foot maintenance berms are required on each side of the channel. For channels with a top width of 60 feet or greater, 30-foot minimum berms are required on each side of the channel. The elevation of the top of the berm shall be at natural ground along the channel reach. See Table 3-3.

Large grass-lined channels with side slopes of 8:1 or flatter do not require a maintenance berm. See Table 3-3.

Table 3-3 Maintenance Berm Widths for Large Channels

Side Slope	Top Width of Channel	Berm Width (Each side)
Less than 8:1	T ≤ 60 feet	20 feet
	T > 60 feet	30 feet
8:1 or Greater	T = all	None required

Freeboard

A minimum freeboard of 12 inches to top of bank is required at the maximum 100-year design storm water surface elevation in the channel.

Ensure that the channel design has sufficient freeboard to drain lateral storm sewers during the 25-year storm.

Geotechnical Investigation

Unless waived by the Drainage Regulatory Entity, a geotechnical investigation and report shall be provided.

Bench Section

To improve safety or aesthetics of the channel, a bench may be provided. The minimum allowable width for a bench section is 10 feet. The bench shall be placed at least 5 feet above the normal water level. The minimum cross slope toward the channel is 2 percent. Figure 3-4 shows a typical bench section.

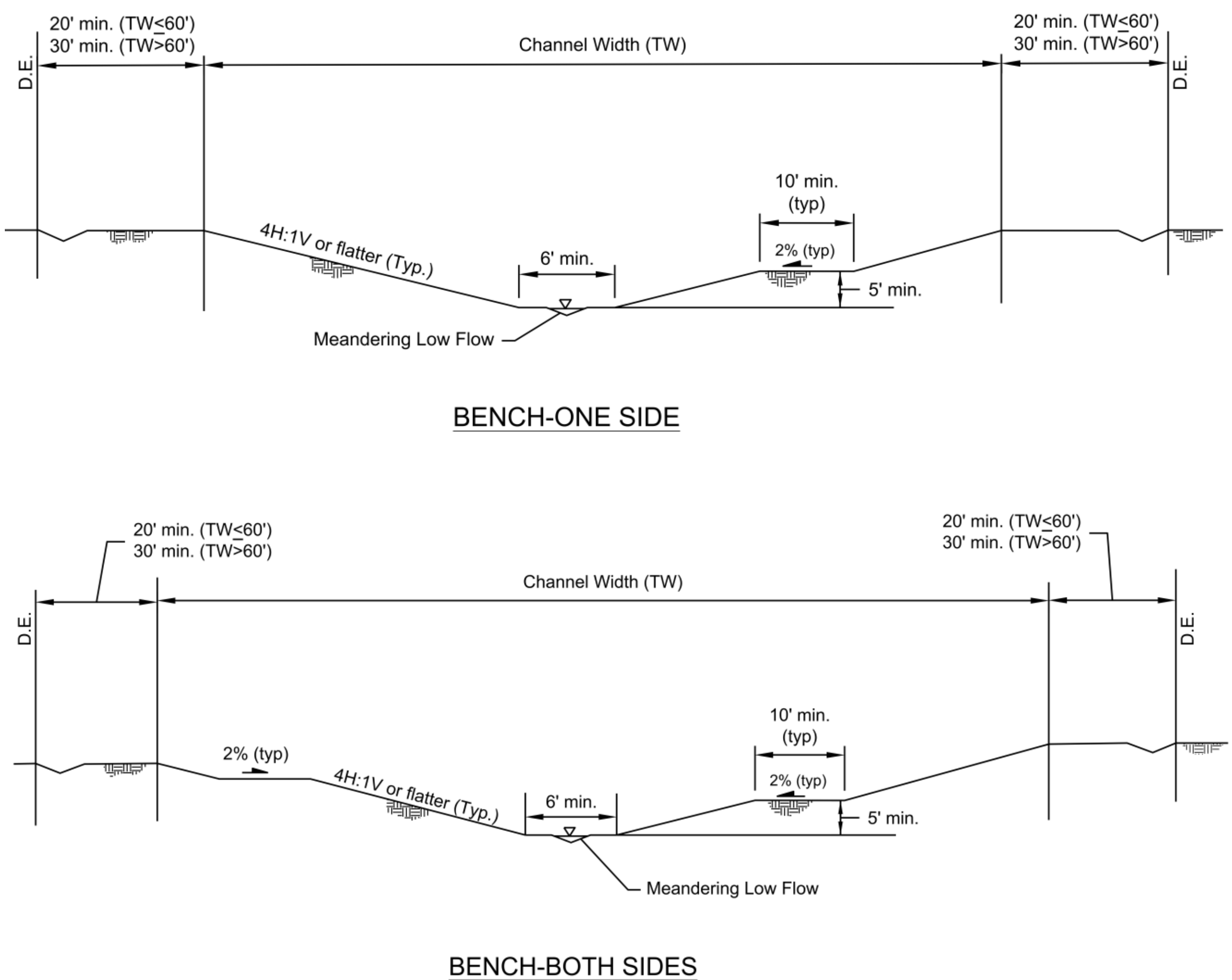


Figure 3-4 Typical Bench Section

3.2.2.5 Large, Rectangular Channels

In some areas, it may be necessary to use a vertical-walled, rectangular section. Approval from the Drainage Regulatory Entity shall be obtained prior to the design of any concrete rectangular channel. In such cases, the following sub-sections apply.

Bottom Width
The minimum bottom width is 8 feet.

Bottom Cross Slope
The minimum cross slope for bottom widths greater than 12 feet is 0.5 in/ft.

Berm Width

A maintenance berm is required on both sides of the channel. For channels with a top width of 60 feet or less, 20-foot berms are required. For channels with a top width of 60 feet or greater, 30-foot berms are required.

The elevation of the top of the maintenance berm shall be at natural ground along the channel reach. See Table 3-3.

Freeboard

The channel shall be designed to have 12 inches of freeboard for the fully developed 100-year storm event.

Ensure that the channel design has sufficient freeboard to drain lateral storm sewers during the 25-year storm.

Velocity

The maximum design velocity in the channel shall be 12 feet per second for the 100-year event. For further information, refer to Section 3.3.1.1.

Geotechnical Investigation

Unless waived by the Drainage Regulatory Entity, a geotechnical investigation and report shall be provided.

Concrete Channel Lining

All concrete shall be Class A concrete unless noted otherwise.

Reinforcement

The structural steel design for the concrete reinforcement shall be ASTM A615, Grade 60 steel.

Wall Height

Minimum height of vertical walls shall be 4 feet. Heights above this shall be in 2-foot increments. Exceptions shall be reviewed at the discretion of the Drainage Regulatory Entity.

For pilot channels with grass side slopes above the 25-year water surface elevation, the top of the vertical wall shall be constructed to allow for future adjoining concrete slope paving.

Weep Holes

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

Escape Stairways

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

Seal Slab

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

Pilot Channel

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

Control joints shall be provided at approximately 25 feet on center. The use of a sealing agent shall be utilized to prevent moisture infiltration.

3.3 Erosion Control

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

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

1. Areas of channel curvature, especially where the radius of the curve is less than three times the design flow top width.
2. Around bridges where channel transitions create increased flow velocities.
3. When the channel invert is steep enough to cause excessive flow velocities.
4. Along grassed channel side slopes where significant sheet flow enters the channel laterally.
5. At channel confluences
6. At the outfall of backslope drains, roadside ditches, and storm sewers.
7. In areas where the soil is particularly prone to erosion.

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

3.3.1 Minimum Erosion Protection Requirements

Minimum erosion protection requirements are detailed in the following sections.

3.3.1.1 Velocity

High velocities can cause erosion and may pose a threat to safety. Velocities which are too low may allow sediment deposition and subsequent channel clogging. Table 3-4 provides average and maximum allowable velocities based on the channel lining type.

The use of riprap and concrete lining shall be approved by the Drainage Regulatory Entity prior to design.

Table 3-4 Allowable Velocities for Channel Design Storm

Channel Description	Average Velocity (Feet per Second)	Maximum Velocity (Feet per Second)
Grass Lined		
Predominantly Clay Soil	3.0	5.0
Predominantly Sandy Soil	2.0	4.0
Riprap Lined*		
Gradation #1	5.0	7.0
Gradation #2	5.0	9.0
Concrete Lined*	6.0	12.0

*The use of riprap and concrete lining shall be approved by the Drainage Regulatory Entity.

3.3.1.2 Confluences

Figure 3-5 presents the minimum requirements of erosion protection or channel lining for small and large channels when given the angle of the confluence. The top edge of the erosion lining shall extend to the 25-year water surface elevation. A healthy cover of grass shall be established above the top edge of the erosion lining (see Section 3.3.2) extending to the top of the bank. The angle of intersection between the tributary and main channels shall be between 15° and 45°. Angles in excess of 45° are permissible but discouraged. Angles in excess of 90° are not permitted.

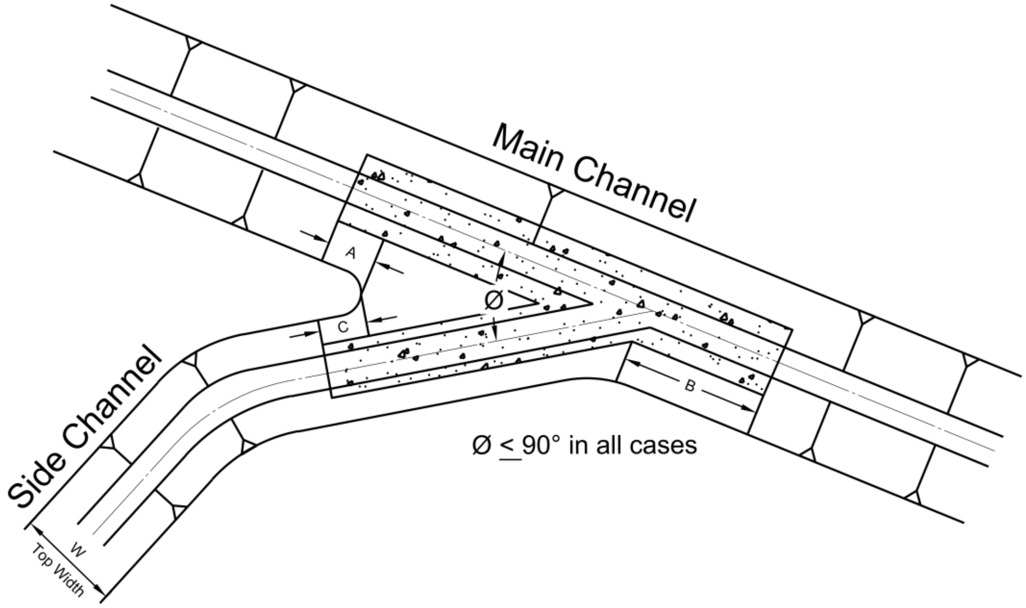


Figure 3-5 Erosion Protection at Confluences

Table 3-5 25-Year Erosion Protection Velocities for Confluences

25-Year Velocity inside Channel[1]	Angle of Intersection Φ	
	15° - 45°	45° – 90°
Velocity ≥ 4 fps	Protection	Protection
2 fps < Velocity < 4 fps	No Protection	Protection
Velocity ≤ 2 fps	No Protection	No Protection

1. 25-year velocity assumes no downstream backwater from the receiving channel.
2. Riprap in lieu of concrete slope paving shall be approved by the Drainage Regulatory Entity.

3.3.1.3 Bends

The following characteristics shall be implemented when designing any bend in open channels.

- Curve centerlines shall be as gradual as possible given the constraints of the design.
- Curves shall have a minimum radius of three times the top width of the design flow unless erosion protection is provided.
- The radius of any centerline curve on an open channel shall not be less than 100 feet.
- The maximum curvature for any man-made channel shall be 90°.

When required, erosion protection shall extend along the outside bank of the bend. Erosion protection shall extend the length of the channel width from the downstream tangent, at a minimum. The top edge of the lining shall extend to the 25-year water surface elevation. Additional protection on the channel bottom, inside bank, or beyond the designated length downstream will be required if maximum allowable velocities are exceeded. For further velocity information, see Section 3.3.1.1. An example of the required protection is shown in Figure 3-6 below.

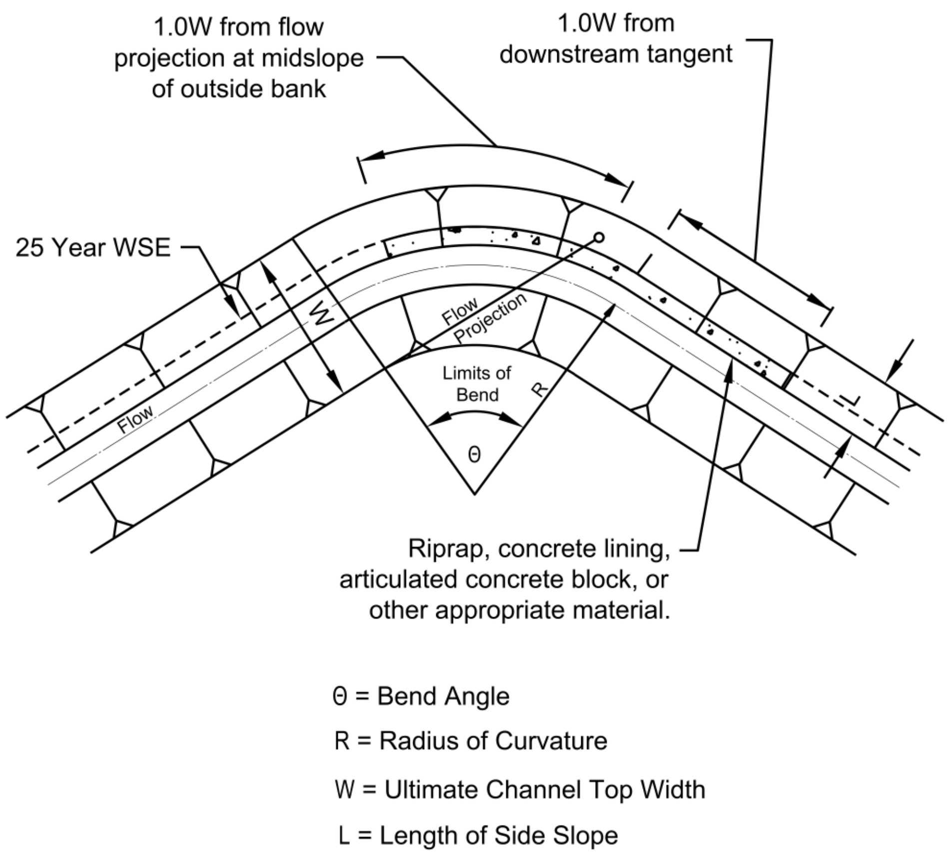


Figure 3-6 Erosion Protection in Channel Bend

Riprap in lieu of concrete slope paving shall be approved by the Drainage Regulatory Entity.

Erosion protection is required when:

- R < 3W and 1% exceedance velocity > 3 feet per second
- Soil type, channel geometry, sinuosity or velocity indicate a potential problem

3.3.1.4 Channel Transitions

The following design applications shall be used when designing channel transitions:

1. Expansions and contractions shall be designed to create minimal flow disturbance and thus minimal energy loss.
2. Transition angles shall be less than 12°.
3. When connecting rectangular and trapezoidal channels, a warped or wedge transition is recommended.

3.3.1.5 Culverts

In areas where culvert outlet velocities exceed 5 feet per second when transitioning to a grass-lined channel, a rigid channel lining or an energy dissipation structure shall be required.

3.3.1.6 Outfalls

Erosion protection will be necessary in areas of high turbulence or velocity as typically found at the outfall of backslope drains, roadside ditches, and storm sewers into the main channel. See Detail 3-1 through Detail 3-5 and Detail 5-1 for typical pipe and storm sewer outfall details.

3.3.2 Structural Erosion Controls

When flow velocities exceed those allowed in Table 3-4 or when soils are deemed excessively erosive by a geotechnical engineer, acceptable structural erosion control shall be provided. The slope protection shall extend up the channel bank to a minimum height of the design 5-year water surface elevation for roadside ditches and the design 25-year water surface elevation for small and large channels.

3.3.2.1 Riprap

Riprap is defined as broken concrete rubble or well-rounded stone. Riprap provides erosion protection and energy dissipation. The use of riprap is discouraged, and concrete slope paving is the preferred erosion control measure. Any use of riprap shall require approval from the Drainage Regulatory Entity.

All minimum requirements concerning the selection and installation of riprap shall be in accordance with the latest version of the Harris County Flood Control specifications.

3.3.2.2 Concrete Slope Paving

Lining a channel with concrete may be required due to high velocities, soil propensity to erode, or other factors. In these cases, further requirements apply to the channel design, as detailed below.

1. Approval from the Drainage Regulatory Entity shall be obtained prior to the design of any concrete-lined channel such that the embankment of the channel is lined with concrete so as to achieve a steeper side slope than those minimum requirements specified for grass-lined channels.
2. All concrete shall be Class A concrete.
3. Concrete slope protection placed on 3:1 side slope shall have a minimum thickness of 4 inches and a minimum reinforcement of #3 rebar at 18 inches o.c. each way.
4. Concrete slope protection placed on 2:1 side slope shall have a minimum thickness of 4 inches and minimum a minimum reinforcement of #3 rebar at 15 inches o.c. each way.
5. Cast-in-place concrete side slopes shall not be steeper than 2:1.
6. All slope paving shall include a minimum 24-inch toe wall at the top and sides of the channel and a 36-inch toe wall across or along the channel bottom.
7. Partially-lined channels will require backslope drainage structures. For fully-lined channels, backslope drainage structures may not be required at the discretion of the Drainage Regulatory Entity.
8. Weep holes shall be used to relieve hydrostatic head behind lined channel sections. The specific type, spacing, and construction method for the weep holes will be based on the recommendations of the geotechnical report.
9. Where construction is to take place under conditions of mud and/or standing water, a seal slab of Class C concrete shall be placed in the channel bottom prior to placement of concrete slope paving.
10. Control joints shall be constructed at approximately 25 feet on center. The use of a sealing agent shall be utilized to prevent moisture infiltration.

3.3.2.3 Backslope Drainage Systems

In highly developed areas or in areas where small and large grass-lined channels are expected to receive overland sheet flow, back slope interceptors are required. Subject to the approval of the Drainage Regulatory Entity, backslope drains and swales may not be required in areas expected to remain sparsely developed.

The design engineer shall account for the drainage area to be intercepted by such systems, particularly if the channel passes through large areas of undeveloped acreage where natural sheet flow could overload the backslope swale and drainage system. In these areas, drain spacing and backslope drainage pipe requirements may have to be greater than the parameters discussed below. Refer to Detail 3-1 through Detail 3-5 at the end of this chapter for backslope swale and structure details.

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

General requirements for backslope drains and swales are as follows:

1. Minimum backslope drainpipes shall be 24 inches in diameter.
2. Maximum spacing is 800 feet for dispersive soils with a minimum of 400 feet.
3. The drain structure and swale centerline shall be 5.5 feet inside the channel drainage easement line.
4. Minimum design depth in the swale is 0.5 feet.
5. Maximum design depth in the swale is 2.0 feet.
6. Minimum gradient for swale invert is 0.2 percent.
7. The swale shall have a maximum side slope of 2 to 1.

3.3.2.4 Sloped Drops

Sloped drop structures are recommended when the required drop elevation is small, generally 1 to 4 feet.

1. Sloped drops shall be no steeper than 3:1 and no flatter than 4:1.
2. Sloped drops shall be used for channels with a bottom width of 10 feet or less.
3. Sloped drops shall be designed to convey the fully developed 100-year storm event flow.
4. The structure shall be located at a uniform and straight location.
5. Sloped drops shall be constructed of concrete slope paving.
6. Sloped drop structures, when located near a culvert, shall be placed immediately upstream of the culvert, and monolithically connected to the upstream headwall of the culvert.

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

Downstream of the drop, the required length for protection is dependent on the length of the hydraulic jump. The length of the hydraulic jump can be estimated with the equations below.

$$q = \frac{Q}{T} \tag{3-1}$$

$$L_j = \frac{q}{2} \tag{3-2}$$

Where,

Q	=	Flow in channel (cfs);
T	=	Width of channel (ft);
q	=	Design flow per unit width (cfs/ft);
L_j	=	Length of hydraulic jump (ft)

The use of riprap or a combination of riprap and concrete slope paving is recommended downstream of the drop to force the jump closer to the drop. The height of the riprap shall reach the 25-year water surface elevation. A minimum of 20 feet of riprap is required both upstream and downstream of any slope paving used at a drop structure to help reduce velocities and protect the concrete toe. The minimum recommended apron length is 40 feet. Figure 3-7 below exhibits the requirements for a sloped drop.

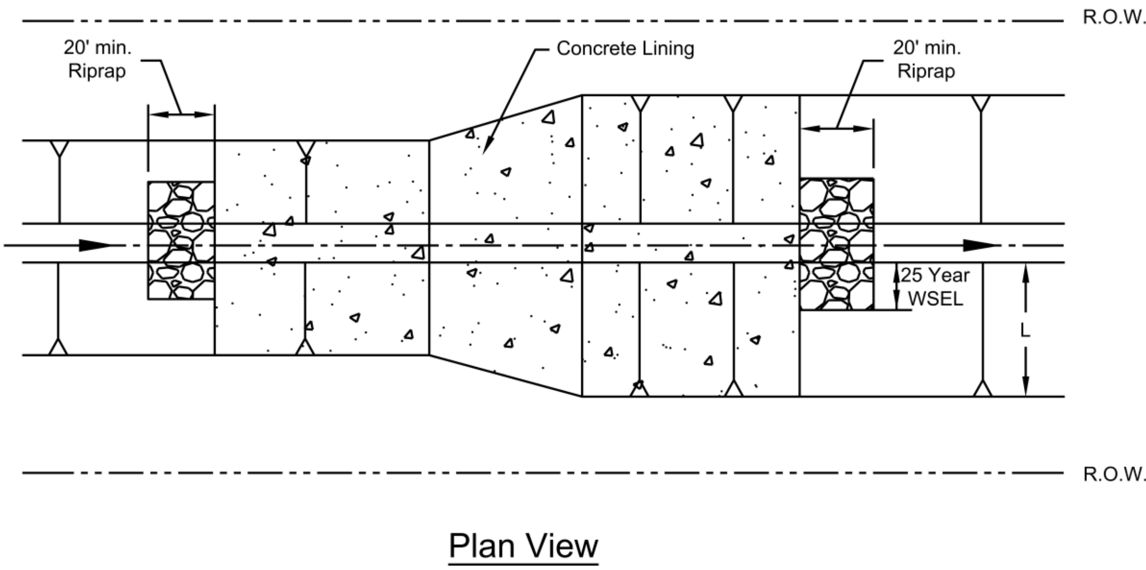


Figure 3-7 Sloped Drop Requirements

3.4 Hydraulic Analysis

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

Channel hydraulic analyses are not only necessary to adequately size proposed channels to convey runoff from the design storm but are also required to demonstrate new construction results in no adverse impact to flood risks, in particular if the project is in designated flood area either by FEMA or by the Drainage Regulatory Entity.

In general, the latest version of HEC-RAS shall be used for any newly developed 1D and 2D models. Software programs other than HEC-RAS will be considered on a case-by-case basis, upon approval by the Drainage Regulatory Entity.

3.4.1 Uniform Channel Calculations

This method shall only be used for proposed and natural, uniform channels including roadside ditches. A channel shall be considered uniform where the following conditions apply:

- New channel or existing natural channel has a suitably consistent cross-section shape.
- Little-to-no variation is seen between channel cross-sections for the portion of the channel being analyzed.
- Channel slope is constant for the portion of the channel being analyzed.
- Flow is completely contained within the inner banks of the channel.
- There are no structures located within the channel. Head losses across culvert structures may be computed separately and applied to the water surface profile for the channel.

For channels with uniform channel conditions, the manning's equation, Equation (3-3), can be used to size a single cross section for the channel.

Manning's N Normal Depth Equation

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

Where,

Q	=	Peak design discharge in the channel (cfs);
n	=	Manning's coefficient of roughness;
A	=	Cross-sectional area of the flow (ft^2);
R	=	Hydraulic radius of the flow (ft);
S	=	Channel longitudinal slope (ft/ft)

Head losses at transitions and bends need to be taken into consideration when performing uniform channel depth calculations. Head losses can cause localized incongruencies in the water surface elevation. These changes in the flow profile will eventually trend back to the normal depth profile; however, these minor losses must be accounted for during channel design efforts.

Incorporate head losses into hydraulic profile computations for channel bends when the:

- Radius of curvature is less than three times the channel top width,
- Average channel velocity is greater than 4 feet per second for the 100-year storm event.

Equation (3-4) can be used to calculate head loss across transitions and Equation (3-5) can be used to calculate head loss through a bend.

Transition Head Loss Equation

$$h_T = C\frac{\left|{V_2}^2 - {V_1}^2\right|}{2g} \tag{3-4}$$

Where,

h_T	=	Head loss across the transition (ft);
C	=	Empirical expansion or contraction coefficient, see Table 3-6;
V_2, V_1	=	Average channel velocity of the downstream and upstream sections, respectively (ft/s);
g	=	Acceleration of gravity (32.2 ft/s^2)

Table 3-6 Contraction and Expansion Coefficients

Transition	Coefficient	
	Contraction	Expansion
Gradual or warped	0.1	0.3
Bridge sections; wedge; straight-lined	0.3	0.5
Abrupt or square-edged	0.6	0.8

When the term $({V_2}^2 - {V_1}^2)$ is positive, the contraction coefficient should be used. When the term is negative, the expansion coefficient should be used.

Bend Head Loss Equation

$$h_B = c_f\left(\frac{V^2}{2g}\right) \tag{3-5}$$

Where,

h_B	=	Head loss (ft);
c_f	=	Coefficient of resistance, see Table 3-7;
V	=	Average channel velocity (ft/s);
g	=	Acceleration due to gravity (32.2 ft/s^2)

Table 3-7 Coefficient of Resistance

Radius of Curvature Divided by Channel Top Width	C_f
Between 1.5 and 3.0	0.2
Between 1.0 and 1.5	0.3

3.4.2 Steady-State 1D Modeling

A channel hydraulic analysis shall use steady state 1D backwater modeling when any of the following conditions apply:

- Analyzing existing, non-uniform streams in which channel cross section shape is highly irregular and the channel slope varies.
- There are several culvert structures crossing the channel in close proximity to each other.
- There are bridge structures crossing the channel.
- There are multiple confluences and bends in the channel.
- Flow profile is expected to enter a super-critical flow regime.

For non-uniform channels with steady flow, applicable software may be used to complete the analysis. HEC-RAS is the most common and preferred modeling software, but other computer programs employing the same methodology may be used with approval of the Drainage Regulating Entity. The hydraulic analysis shall be in accordance with the recommendations provided within the Harris County Flood Control District's *Hydrology and Hydraulic Guidance Manual*.

3.4.3 Unsteady-State 1D Modeling

Unsteady-state hydraulic modeling calculates the full flow hydrograph or any change in flow over time through the creek or channel. As a result, flood wave attenuation is accounted for and resulting stage hydrographs can be produced. Unsteady-state 1D modeling shall be used if any of the following conditions apply:

- Flow rate in the channel changes over time (hydrograph).
- The overbank topography is flat and provides significant floodplain storage, unless the storage can be adequately accounted for using the routing methods outlined in Section 2.3.2.4.
- Varying tailwater conditions are present such as near the coast.
- Offline storage basins are being designed along the creek and are utilizing inflows from the creek for mitigation.
- It is desired to determine hydrograph routing along the channel or creek in lieu of the flood routing methods detailed in Section 2.3.2.4.

Unsteady 1D hydraulic modeling will also be necessary to obtain the stage hydrograph for the channel or natural creek, if needed.

HEC-RAS or another applicable software, with approval of the Drainage Regulating Entity, may be used to complete this analysis. The hydraulic analysis shall be in accordance with the recommendations provided within the Harris County Flood Control District *Hydrology and Hydraulic Guidance Manual*.

3.4.4 Unsteady-State 2D Modeling

In areas where there is expected to be significant 2D flow patterns, dynamic 2D modeling may be required for a proper hydraulic analysis. Unsteady 2D modeling shall be used if any of the following conditions apply:

- The channel is located in close proximity to the coast and tidal and storm surge influences are of interest
- Significant 2D flow patterns exist, especially behind levees, roads, and other bermed structures
- The topography naturally falls away from the creek inner banks such that flow is directed away from the creek once flow breaches the banks.
- There is significant sharing of flow between watersheds.
- Areas with significant storage but low conveyance (ineffective areas) are activated during any design storm.

HEC-RAS or another applicable software, with approval of the Drainage Regulating Entity, may be used to complete this analysis. The analysis shall be in accordance with the recommendations within the document provided by Harris County Flood Control District, *Two-Dimensional Modeling Guidelines*.

3.4.5 Manning's N

The Manning's roughness coefficient values in Table 3-8 through Table 3-9 shall be used in the 1D hydraulic analysis of channels. Any alternative values shall be discussed with the Drainage Regulatory Entity.

Table 3-8 1D Manning's N Values

Type of Channel and Description	Minimum	Normal	Maximum
A. Lined or Built-up Channels			
A1. Corrugated Metal	0.021	0.025	0.030
A2. Nonmetal			
a) Concrete			
1) Trowel finish	0.011	0.013	0.015
2) Float finish	0.013	0.015	0.016
3) Finished, with gravel on bottom	0.015	0.017	0.020
4) Unfinished	0.014	0.017	0.020
5) On good excavated rock	0.017	0.020	-
6) On irregular excavated rock	0.022	0.027	-
b) Concrete bottom float finished with sides of			
1) Dressed stone in mortar	0.015	0.017	0.020
2) Random stone in mortar	0.017	0.020	0.024
3) Cement rubble masonry, plastered	0.016	0.020	0.024
4) Cement rubble masonry	0.020	0.025	0.030
5) Dry rubble or riprap	0.020	0.030	0.035
a) Asphalt			
1) Smooth	0.013	0.013	-
2) Rough	0.016	0.016	-
B. Excavated or Dredged			
a) Earth; straight, uniform			
1) Clean, recently completed	0.016	0.018	0.020
2) Clean, after weathering	0.018	0.022	0.025
3) Gravel, uniform section, clean	0.022	0.025	0.030
4) With short grass, few weeds	0.022	0.027	0.033
b) Earth, winding and sluggish			
1) No vegetation	0.023	0.025	0.040
2) Grass, some weeds	0.025	0.030	0.035
3) Dense woods or aquatic plants in deep channels	0.030	0.035	0.040
4) Earth bottom, rubble sides	0.028	0.030	0.035
5) Stony bottom, weedy banks	0.025	0.035	0.040
6) Cobble bottom, clean sides	0.030	0.040	0.050
c) Dragline – excavated or dredged			
1) No vegetation	0.025	0.028	0.033
2) Light brush or banks	0.035	0.050	0.060
d) Rock cuts			
1) Smooth and uniform	0.025	0.035	0.040
2) Jagged and irregular	0.035	0.040	0.050
e) Channels not maintained, weeds and brush uncut			
1) Dense weeds, high as flow depth	0.050	0.080	0.120
2) Clean bottom, brush on sides	0.040	0.050	0.080

Type of Channel and Description	Minimum	Normal	Maximum
3) Dense brush, high stage	0.080	0.100	0.140
C. Natural Streams			
C1. Minor Streams (top width at flood stage < 100ft)			
a) Streams on plain			
1) Clean, straight, full stage, no rifts or deep pools.	0.025	0.030	0.033
2) Clean, winding, some pools and shoals	0.033	0.040	0.045
C2. Floodplains			
a) Pasture, no brush			
1) Short grass	0.025	0.030	0.035
2) High grass	0.030	0.035	0.050
b) Cultivated areas			
1) No crop	0.020	0.030	0.040
2) Mature row crops	0.025	0.035	0.045
3) Mature field crops	0.030	0.040	0.050
c) Brush	0.035	0.050	0.070
d) Trees			
1) Dense willows, summer, straight	0.110	0.150	0.200
2) Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
C3. Major streams (top width at flood stage > 100 ft)*			
a) Regular section with no boulders or brush	0.025	-	0.060
b) Irregular and rough section	0.035	-	0.100

*The n value is less than minor streams of similar description because banks offer less effective resistance.
Source: Open-Channel Hydraulics by Ven Te Chow, 1959.

Table 3-9 Computation of Composite Roughness Coefficient for Excavated and Natural Channels

$n = (n_0 + n_1\ n_2 + n3 + n_4)$

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

3.5 Geotechnical Investigation

Before initiating final design of a small or large channel, a detailed soils investigation by a professional geotechnical engineer, licensed in the State of Texas, shall be undertaken. The following minimum requirements shall be addressed:

1. Stability of the channel side slopes for short-term, long-term, and rapid drawdown conditions; if channel depth ≤ 5 feet, a slope stability analysis is not required, however, a geotechnical report is still required to address the other issues.
2. Stability of the side slopes.
3. Evaluation of bottom instability due to excess hydrostatic pressure
4. Control of groundwater

5. Identification of dispersive soils
6. Potential erosion problems
7. Constructability issues
8. Evaluation of inflow structures
9. Investigation into the potential for structural movement on areas adjacent to the channel may be required. This is mainly due to the induced loads from exiting or proposed structures and methods of controlling it.

3.6 Supercritical Transitions

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

3.7 Drainage Easements

The amount of easement required for open channels shall be based on ultimate development of the watershed. Easement needed is dependent on the channel top width and channel type (earthen or lined) required to accommodate the discharge resulting from the design storm event, as mentioned in previous sections. Minimum easement requirements include the channel from bank-to-bank as well as the maintenance berms on both sides.

The easement shall be dedicated at the time of platting of the adjacent property. If additional easement is required to serve upstream development prior to the downstream platting, enough easement shall be dedicated to accommodating the improved channel along with adequate maintenance berms. Prior to design of open channels, the appropriate Drainage Regulatory Entity shall be consulted for information regarding the ultimate channel cross-section and easement.

The Drainage Regulatory Entity may require dedication of enough easement for runoff produced by presently planned developments and other improvements. If a master drainage plan is available, the amount of land specified in the ultimate easement shall be set aside. The reserved land shall be shown as drainage reserves on any plat.

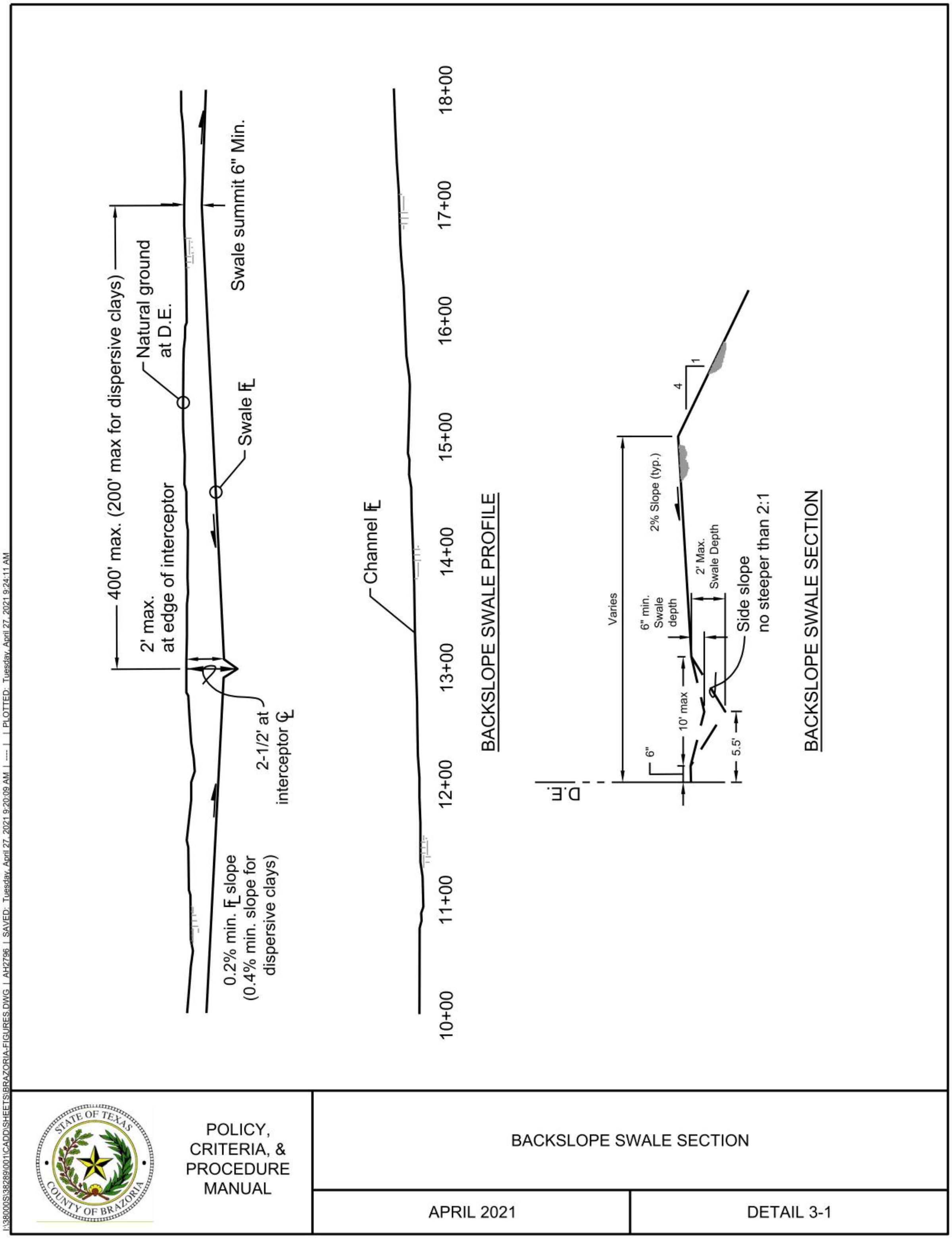
3.8 Utility Line Crossings

Approval by the Drainage Regulatory Entity shall be obtained for all future utility lines crossing the agency's flood control facilities. All utility manholes shall be located outside the drainage right-of-way. All utility lines shall be located 5 feet below the ultimate lowest flowline unless more stringent criteria is required by the Drainage Regulatory Entity.

3.9 Required Deliverables

The following information shall be submitted to the Drainage Regulatory Entity for the design of open channels.

1. A vicinity map that identifies the full scope of improvements
2. A detailed map of the area and proposed channels with all pertinent physiographic information including topographic information, right-of-way, drainage facilities, and floodplain information
3. A watershed map showing the existing and proposed drainage area boundary along with all subarea delineations and all areas of existing or proposed development
4. Discharge calculations specifying methodology and key assumptions used including discharges at key locations
5. Hydraulic calculations specifying methodology used; all assumptions and calculated values of the design parameters shall be clearly stated.
6. A profile of the subject reach which includes the following:
    a. All pertinent water surface profiles; this will minimally include the design storm event. For large channels, this will also include the 100-year frequency floods for both existing and proposed channel conditions.
    b. All existing and proposed bridge, culvert, and pipeline crossings
    c. The location of all tributary and drainage confluences
    d. The location of all hydraulic structures (e. g. dams, weirs, drop structures, etc.)
    e. Existing and proposed flowlines with flow direction arrows and proposed longitudinal slope
    f. Natural ground elevations at the right-of-way
7. A map delineating existing and proposed design flood extents
8. A map delineating existing and proposed easements
9. Typical existing and proposed cross-sections
10. The relevant surveying benchmark, elevation, datum and year of adjustment
11. A soils report which addresses ground water, erosion, and slope stability when applicable
12. All hydraulic modeling data


400' max. (200' max for dispersive clays)
Natural ground at D.E.
Swale summit 6" Min.
2' max. at edge of interceptor
Swale FL
2-1/2' at interceptor CL
0.2% min. FL slope (0.4% min. slope for dispersive clays)
Channel FL
10+00
11+00
12+00
13+00
14+00
15+00
16+00
17+00
18+00
BACKSLOPE SWALE PROFILE
D.E.
6"
10' max
5.5'
Varies
6" min. Swale depth
2' Max. Swale Depth
2% Slope (typ.)
Side slope no steeper than 2:1
4
1
BACKSLOPE SWALE SECTION
POLICY, CRITERIA, & PROCEDURE MANUAL
BACKSLOPE SWALE SECTION
APRIL 2021
DETAIL 3-1

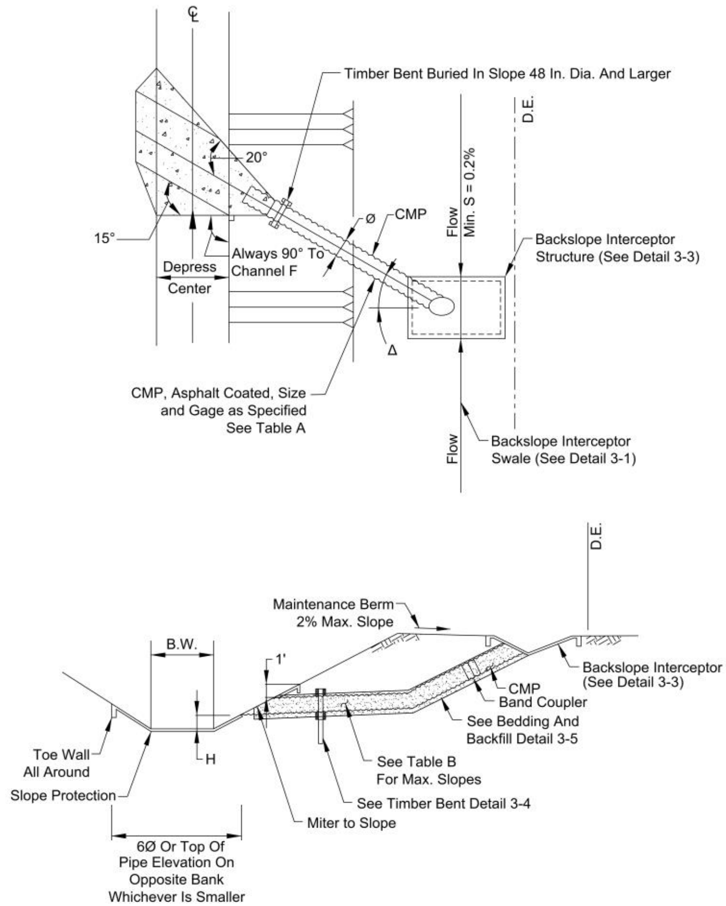


TABLE A

Size $2\frac{2}{3}$" - $\frac{1}{2}$" Corrugation	Pipe Gauge	Band Coupler Gage
24"	16	16
30"	16	16
36"	16	16
42"	14	16
48"	14	16
54"	12	14
60"	12	14
66"	10	12
78"	10	12
84"	8	10
96"	8	10

Δ : Prop. 24" To 42" Δ15°
Prop. 48" And Larger Δ = 30°

H : For Pipe Sizes 24" to 42"
H=3' Max. And 1' Min.
For Pipe Sizes 48" And Larger
H=1' Max. And Min.
For All Pipe Sizes Outfalling Into
Static WSE H=6" Max. and Min.

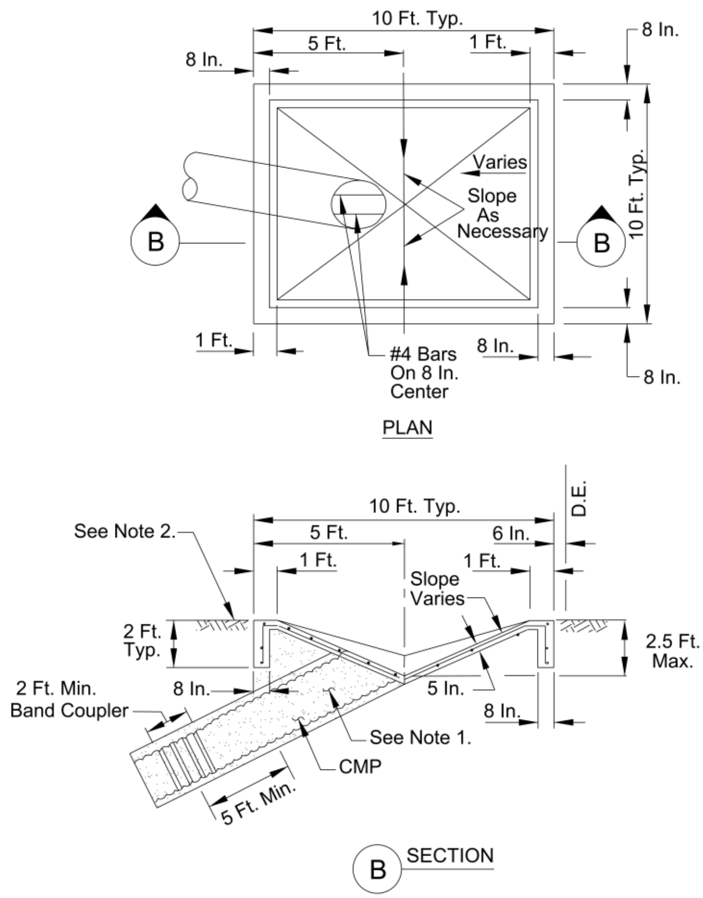
TABLE B

Pipe Diameter ø	Slope	Velocity
24"	0.6%	3.25 f.p.s
36"	0.3%	3.00 f.p.s
42"	0.2%	2.75 f.p.s
48"	0.2%	3.00 f.p.s
54"	0.2%	3.25 f.p.s

POLICY, CRITERIA, & PROCEDURE MANUAL

TYPICAL BACKSLOPE DRAIN DETAIL

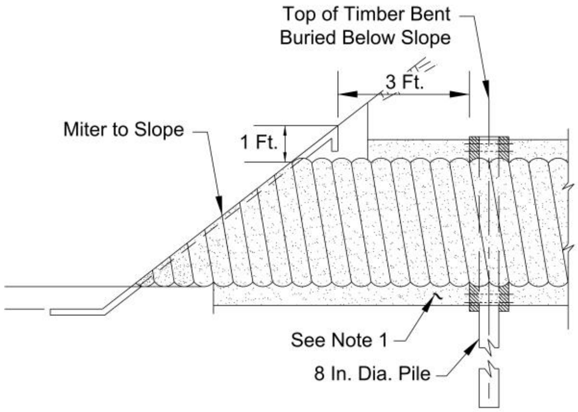
APRIL 2021 | DETAIL 3-2



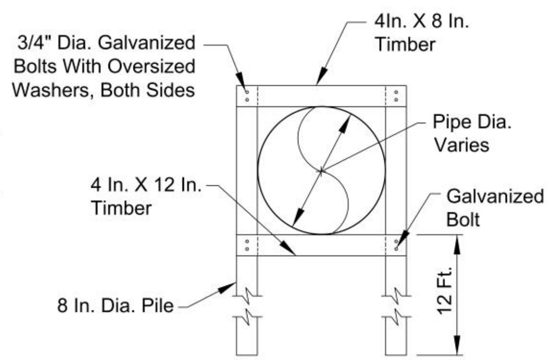
NOTES

1. Provide And Place Cement Stabilized Sand
2. Interceptor Structures:
   - 2.a. Adjust Length And Width In The Field As Necessary.
   - 2.b. 2-Feet Deep X 8-Inch Wide Tow All Around The Structure.
   - 2.c. Steel Reinforcing #4 Bars (Grade 60) At 12 Inches On Center Each Way
   - 2.d. Any Interceptor Outfall Pipe Larger Than Maximum Size Indicated Requires A Separate Detail.
   - 2.e. Match Top Of Concrete With Natural Ground.

POLICY, CRITERIA, & PROCEDURE MANUAL	BACKSLOPE INTERCEPTOR STRUCTURE	
	APRIL 2021	DETAIL 3-3



TYPICAL SECTION

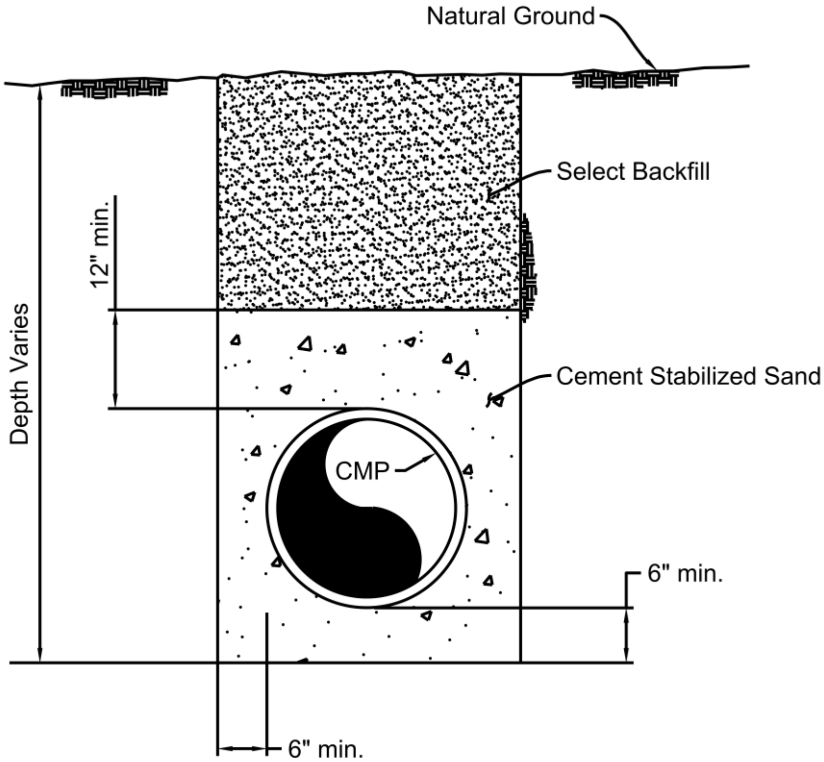


ELEVATION

NOTES

1. Provide And Place Cement Stabilized Sand
2. Piling and Timber to be Pressure-Treated with on Approved Pressure

POLICY, CRITERIA, & PROCEDURE MANUAL	TIMBER BENT DETAIL FOR 48-INCH CMP OR LARGER OUTFALLS	
	APRIL 2021	DETAIL 3-4



POLICY, CRITERIA, & PROCEDURE MANUAL	TYPICAL BEDDING AND BACKFILL DETAIL FOR CMP OUTFALL FOR BRAZORIA COUNTY, TEXAS	
	APRIL 2021	DETAIL 3-5

4 Culverts and Bridges

4.1 General

For small drainage areas, the most economical means of moving open channel flow beneath a road or railroad is usually with the use of culverts. Discussion in this section addresses procedures for determining the culvert size and shape given a design discharge and allowable headwater elevation.

In addition, this section will include a brief discussion of the hydraulic considerations pertinent to bridge design.

This section considers all design to be completed for the ultimate planned development. Where appropriate, the actual construction of a crossing may be phased as development occurs. In this case, both the ultimate and the interim phase shall be shown on the construction plans. Calculations for both phases shall be submitted for approval. The ultimate right-of-way is required for interim phases of construction.

4.2 Culverts

4.2.1 Design Frequency

1. Driveway culverts crossing roadside ditches shall be designed to convey runoff for the 5-year storm event when flowing full. (Friction slope is equal to the pipe slope.)
2. For culverts providing cross-drainage for residential roadways with roadside ditch drainage systems, the culverts shall be designed to convey runoff for the 25-year storm event for fully developed conditions without causing water surface profiles to be higher than the edge of road pavement.
3. All other culverts shall be designed to convey runoff for the 100-year storm event for fully developed conditions without causing water surface profiles to be higher than the edge of road pavement.
4. In all cases, the design of the culvert shall ensure that the maximum water surface elevation for the 100-year storm event does not exceed an elevation greater than two feet below the lowest structure finished floor elevation.
5. In all cases, the culvert shall be able to pass flow without causing flood level impacts to neighboring properties for the design storm event and the 100-year storm event.
6. Every attempt shall be made to design major thoroughfares so that they are passable during the 100-year storm event.

7. Culverts located within FEMA-regulated Special Flood Hazard Areas shall adhere to the minimum regulations set forth in the Brazoria County floodplain regulations.

4.2.2 Culvert Alignment

Culvert alignment shall adhere to the following criteria:

1. Culverts shall be aligned with the centerline of the existing/natural channels, ditches, swales, and other low areas to ensure maximum hydraulic efficiency and minimal erosion.
2. In areas where a change in alignment is necessary, the change shall be made in the channel upstream of the culvert and appropriate erosion protection shall be provided.
3. When possible, culverts shall be aligned with the direction of flow in the channel downstream of the culvert.
4. Culverts shall be aligned to avoid severe (greater than 90°) channel bends.
5. Culverts shall be aligned to avoid areas with high erosion potential.
6. Culvert locations and alignments shall be designed to allow adequate access for maintenance.
7. Culverts shall be aligned to avoid conflicts with existing buildings, homes, pipelines, and contaminated sites.
8. Culverts shall be aligned to minimize the number of property owners affected.
9. Culvert intersections shall be 90° or less, measured from the upstream outfall.
10. The maximum skew angle for box culverts is 45°.

4.2.3 Minimum Culvert Sizes

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

4.2.4 Culvert Length

Culverts shall be designed to span the road or railroad embankment.

4.2.5 Manning's "n" Values

The minimum Manning's "n" value to be used in circular concrete culverts shall be 0.013. For reinforced concrete boxes (RCB), the minimum Manning's "n" value shall be 0.015. For corrugated metal pipes, the "n" values shall be determined based on the type of corrugation as provided in Table 4-1.

Table 4-1 Typical Manning's "n" Values for Corrugated Metal

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

4.2.6 Headwalls and End Treatments

End treatments serve several different purposes but typically act as a retaining wall to keep the roadway embankment material out of the culvert opening. Secondary characteristics of end treatments include hydraulic improvements, traffic safety, debris interception, flood protection, and prevention of piping (flow through the embankment outside of the culvert). General design requirements for the use of headwalls and end treatments are outlined below.

1. Headwalls and endwalls shall be utilized for culverts greater than 36 inches in diameter to control erosion and scour, anchor the culvert against lateral pressures, and ensure bank stability.
2. All headwalls shall be constructed using reinforced concrete and may be straight or parallel to the channel; flared or warped; and with or without aprons as required by site and hydraulic conditions.
3. Headwalls shall be functionally monolithic with the culvert conduit and shall generally be parallel with the alignment of the crossing roadway.
4. Protective handrails and guardrails are required along culvert headwalls if the crossing depth is greater than 4 feet.

Table 4-2 provides some general design requirements for choosing an appropriate headwall.

Table 4-2 Headwall Design Requirements

Parallel Headwall and Endwall
1. Approach velocities are less than 6 feet per second (fps). 2. Backwater pools may be permitted. 3. Approach channel is undefined. 4. Ample right-of-way or easement is available. 5. Downstream channel protection is not required.
Flared Headwall and Endwall
1. Channel is well defined. 2. Approach velocities are greater than 6 fps. 3. Medium amounts of debris exist. The wings of flared walls shall be located with respect to the direction of the approaching flow instead of the culvert axis.
Warped Headwall and Endwall
1. Channel is well defined and concrete lined. 2. Approach velocities are greater than 8 fps. 3. Medium amounts of debris exist. These headwalls are effective with drop down aprons to accelerate flow through culvert and are effective headwalls for transitioning flow from closed conduit flow to open channel flow. This type of headwall shall be used only where the drainage structure is large, and right-of-way or easement is limited.

Source: Drainage Criteria Manual, City of Austin, Texas

For roadway culverts less than 36 inches in diameter, a slope end treatment is required.

End treatments are not required for driveway culverts.

4.2.7 Structural Requirements

The structural requirements for culvert design are as follows:

1. All corrugated metal pipes shall meet the requirements of TxDOT Standard Specification 2014, Item 460 “Corrugated Metal Pipes”.
2. All precast reinforced concrete pipes and joint sealing material for precast concrete culverts shall comply with the requirements of TxDOT Standard Specification 2014, Item 464 “Reinforced Concrete Pipes”.
3. HL-93 Loading shall be used for all culverts.
4. Two sack per ton cement stabilized sand shall be used for backfill around culverts.
5. Requirements for precast box culvert are as follows:
   a. All precast reinforced concrete pipes and joint sealing material for precast concrete culverts shall comply with the requirements of TxDOT Standard Specification 2014, Item 464 “Reinforced Concrete Pipe”.
   b. Provide Grade 60 reinforcing steel.
   c. Provide Class C concrete (f’c = 3,600 pounds per square inch) for the closures.
   d. Provide cement stabilized backfill meeting the requirements of TxDOT Standard Specification 2014, Item 400 “Excavation and Backfill for Structures”.
   e. Chamfer the bottom edge of the top slab closure 3 inches at culvert closure ends.
   f. All precast reinforced concrete box culverts with less than 2 feet of cover shall be ASTM 850-7 9.
6. Requirements for cast-in-place box culverts are as follows:
   a. Provide Grade 60 reinforcing steel.
   b. Provide Class C concrete (f'c = 3,600 pounds per square inch) with these exceptions:
      1. For top slabs of culverts with overlay of 1-to-2 course surface treatment, Class S concrete (f'c = 4,000 pounds per square inch) shall be provided instead of Class C concrete
      2. For culverts with the top slab as the final riding surface, Class S concrete (f'c = 4,000 pounds per square inch) shall be provided instead of Class C concrete.
   c. Chamfer the bottom edge of the top slab closure 3 inches at culvert closure ends.

4.2.8 Erosion

Because of their hydraulic characteristics, culverts generally increase the velocity of flow compared to flow in the natural and designed channel. For this reason, the tendency for erosion should be addressed, especially at the outlet.

Structural erosion protection shall be added downstream of the culvert per Section 3.3.1.1 based on the culvert exit velocity. Erosion protection shall continue downstream to the point where the engineer can re-establish normal flow characteristics.

In general, culverts beneath roadways shall be placed at a slope that matches the slope of the serviced watercourse as closely as possible. Steep slopes along the culvert profile shall be avoided. In instances where the culvert is serving a natural watercourse that has a steep profile, the culvert shall be placed at the minimum channel slope and a sloped drop structure shall be placed immediately upstream of the culvert headwall. See Section 3.3.2.4 for sloped drop structure criteria.

4.2.9 Traffic Safety

Guardrails are suggested at all roadway culvert crossings and are required for any culvert crossings with a depth greater than 4 feet. The approach ends of the guardrail shall be properly terminated from the roadway and anchored per TxDOT specifications.

Safety end treatment (SET) of a culvert provides a method of mitigating a less safe condition without interfering with the hydraulic function of the culvert. SETs such as those used with small diameter culverts may be more hydraulically efficient by providing tapered wingwalls and a beveled edge instead of using a mitered section. SETs for larger culverts that are not protected by a railing or guard fence shall use pipe runners arranged either horizontally or vertically.

Shielding by metal beam guard fence is a traditional protection method and has proven to be very effective in terms of safety. However, metal beam guard fence also can be more expensive than safety end treatment.

Generally, if clear zone requirements can be met, neither safety end treatment nor protection such as a guard fence is necessary. However, some site conditions may still warrant such measures. See the Design Clear Zone Requirements in the TxDOT Roadway Design Manual for more information.

4.2.10 Hydraulic Analysis

Several methods and equations are available for computing head losses through a culvert. Many are based on the Federal Highway Administration publication: Hydraulic Design of Culverts.

Peak discharge for the culvert design shall be developed following the hydrology criteria outlined in Chapter 2.

4.2.10.1 Driveway Culverts

Driveway culverts shall be sized assuming full flow conditions using Manning's formula (Equation (4-1).

$$Q = \frac{1.49}{n} A R^{2/3} S_o^{1/2} \tag{4-1}$$

Where,

Q	=	Discharge in the culvert (cfs);
n	=	Manning's 'n' value for the pipe;
A	=	Cross-sectional area of the conduit (sf);
R	=	Hydraulic Radius of the conduit (ft);
S	=	Slope of the pipe (ft / ft)

4.2.10.2 Culvert Hydraulic Analysis Software

A steady-state culvert hydraulic analysis software such as HY-8 is acceptable for sizing and analyzing culverts if the objective of the hydraulic analysis is to only determine the head losses through the culvert. This type of analysis is typically acceptable for designing new culverts within a proposed development or proposed roadside ditch system where effects of back water can be easily determined and managed. This type of analysis is acceptable if the following criteria are met:

1. The culvert is in line with a roadside ditch, or a small channel as defined in Chapter 3.
2. The culvert is not located within a FEMA-designated Special Flood Hazard Area.
3. The upstream backwater effects due to the culvert head losses can be easily determined and shown to not cause adverse impacts or structural flooding.
4. Tailwater conditions such as the cross-sectional area of the downstream channel can be adequately represented in the software program and other tailwater conditions such as backwater from downstream channel confluences, coastal influences, or other hydraulic structures are not present.
5. The barrels share the same upstream and downstream flowlines and are of the same shape and material.

Tailwater conditions can cause a significant effect on the resulting headwater elevation for the culvert. Careful consideration shall be made to the potential tailwater conditions for the specific location of the culvert. If potential tailwater conditions other than downstream channel geometry is present, then a separate analysis shall be performed for each tailwater type with the most conservative tailwater condition controlling. Otherwise, a backwater modeling software shall be used to more accurately determine tailwater conditions.

4.2.10.3 Backwater Modeling Software

A steady state back water hydraulic modeling software such as HEC-RAS shall be utilized for analyzing or sizing culverts for the following conditions:

1. The culvert is in line with a large channel per Chapter 3.
2. The culvert is in line with a natural creek with an irregular geometry.
3. The culvert is located within a FEMA-designated Special Flood Hazard Area.
4. Any type of “no-impact” analysis is required to demonstrate that the culvert has no adverse impact on flooding levels.
5. The upstream backwater effects due to head losses through the culvert are a critical component of the design including sensitive infrastructure, neighboring properties, and low-lying structures being located nearby.
6. Other tailwater conditions exist such as backwater from a downstream channel confluence, coastal influences, or other hydraulic structures are present.
7. The barrels have different upstream and downstream flowlines or vary in shape and material.

Tailwater conditions can have a significant effect on the resulting headwater elevation for the culvert. Careful consideration should be made to the potential tailwater conditions for the specific location of the culvert. The hydraulic model shall extend far enough downstream so that the water surface profile at the culvert is controlled entirely by normal depth and is not sensitive to the assumed downstream boundary condition. The model shall also include any confluences or structures that may have an impact on the tail water conditions for the culvert.

The latest version of HEC-RAS shall be used to complete the hydraulic analysis. The use of other software programs shall be approved by the Drainage Regulating Entity. The analysis shall be in accordance with the recommendations provided within the Harris County Flood Control District’s *Hydrology and Hydraulic Guidance Manual.*

4.3 Bridges

4.3.1 Bridge Design Considerations

Bridges shall be designed per current AASHTO LRFD Bridge Design Specifications with Interims. HL-93 Loading shall be used for bridges.

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

Existing vegetation shall be incorporated into the overall bridge plan. Where practicable, trees and shrubs shall be left intact even within the right-of-way. Minimizing vegetation removal also tends to control turbulence of the flow into, through, and out of the bridge and mitigate erosion potential.

4.3.2 Bridge Location and Orientation Requirements

Newly constructed bridges shall be designed to completely span the existing or proposed channel such that the channel will pass under the bridge without modification. Energy losses due to flow transitions shall be minimized. In addition, provision shall be made for future channel enlargements should they become necessary. Coordination with the Drainage Regulating Entity is required when considering the ultimate channel width for a bridge span.

The bridge shall be centered on the main channel portion of the floodplain. This may cause an eccentricity in location with respect to the stream cross section but allows for better accommodation of the usual and low flows of the stream. Consider including either relief openings or guide banks if the intrusion of either or both roadway headers into the stream floodplains is more than about 800 feet. For some configurations, roadway approaches may need to accommodate overflow. Such overflow approaches allow floods that exceed the design flow to overtop the roadway, thereby reducing the threat to the bridge structure itself. Protection of the approaches from overflow damage shall be considered.

The bridge waterway opening shall be designed to provide a flow area sufficient to maintain the through-bridge velocity for the design discharge no greater than the allowable through-bridge velocity. The headers and interior bents shall be oriented to conform to the streamlines at flood stage.

The piers and the toe the header slope shall be located away from deep channels, cuts, and high velocity areas to avoid scour problems or interference with low flows in the stream.

Bridge appurtenances shall not obstruct maintenance access to adjoining drainage facilities.

4.3.3 Freeboard

At a minimum, bridges shall be designed to pass the fully developed 100-year design flow without causing backwater problems, structural damage, or erosion.

The lowest point on the low chord of all bridges shall be located at least one foot above the existing 100-year flood elevation, or at the level of natural ground, whichever is higher. More freeboard may be appropriate for bridges over streams that are prone to heavy debris loads, such as large tree limbs. Rivers with upstream areas that are heavily wooded are prime examples of such streams.

4.3.4 Flood Damage Protection

Flood-related damage results from a variety of factors including the following:

- Scour around piers and abutments
- Erosion along toe of highway embankment due to flow parallel to embankment
- Erosion of embankment due to overtopping flow
- Long-term vertical degradation of stream bed
- Horizontal migration of stream banks
- Force of debris impact on structures
- Clogging debris causing redirection of flow

These adverse effects shall be minimized when designing bridge crossings.

A scour analysis utilizing HEC-RAS or similar software as approved by the Drainage Regulating Entity is required for new bridges, replacements, and widenings. Where a scour analysis indicates high depths of potential contraction scour, measures to withstand the scour shall be taken. There is no substitute for analysis and cost estimation to determine whether building larger structures or providing suitable foundation and armoring is more cost-effective. A multi-disciplined team shall assess the validity of calculated scour depths.

Stream stability issues, such as potential vertical and horizontal degradation, may warrant accommodations in the bridge design. If the channel is vertically degrading, it is likely that as the channel deepens the banks will slough, resulting in channel widening. Meanders tend to migrate downstream and increase in amplitude, potentially causing significant horizontal degradation. Structural options to accommodate either of these cases may include:

- structures of increased length with deep enough foundations to accommodate anticipated degradation or
- sufficiently deep foundations with the abutment foundations designed to act as interior bents to allow future lengthening of the bridge.

4.3.4.1 Pier Foundation

The following items shall be factored into design to reduce the potential for pier scour:

- Reduce number of piers by increasing span lengths, especially where large debris loads are expected.
- Use bullet-nosed or circular-shaped piers.
- Use drilled-shaft foundations.
- Align bents with flood flow as much as is practical.
- Increase bridge length to reduce through-bridge velocities, where possible.

Where there is a chance of submergence, a superstructure that is as slender as possible with open rails and no curb shall be used.

Because of uncertainties in scour predictions, use extreme conservatism in foundation design. The capital costs of providing a foundation secure against scour are usually small when compared to the risk costs of scour-related failure.

4.3.4.2 Approach Embankments

Embankments that encroach on floodplains are most commonly subjected to scour and erosion damage by overflow and flow directed along the embankment to the waterway openings. Erosion can also occur on the downstream embankment due to turbulence and eddying as flow expands from the openings to the floodplain and due to overtopping flow.

The potential for erosion along the toe of an approach embankment can be minimized by avoiding extensive clearing of vegetation and avoiding the use of borrow areas in the adjacent floodplain. Embankment protection may be required such as:

- stone protection / riprap,
- stable vegetation on the embankment,
- pervious dikes of timber, or
- spaced finger dikes of earthen material, placed normal to the approach fill

All of these methods will impede flow along the embankment, providing some erosion protection around the structure.
Preventative methods may also be required at some crossings to protect against wave action, especially at reservoirs. Riprap of durable, hard rock shall be used at such locations. The top elevation of the rock required depends on storage and flood elevations in the reservoir and wave height computed using wind velocities and the reservoir fetch.

4.3.4.3 Abutments

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

Protective measures are required at each of the following instances:

- Header slopes
- Deep toe walls
- Vertical abutment walls
- Sheet pile toe walls
- Deep foundations of piles or drilled shafts

To prevent embankment failure from undermining by contraction scour, a toe wall shall be extended below the level of expected scour. Stone is generally preferred to concrete for erosion protection near abutments.

4.3.5 Hydraulic Analysis

A hydrologic and hydraulic analysis is required for designing all new bridges over waterways, bridge widening, bridge replacement, and roadway profile modifications that may adversely impact the floodplain even if no structural modifications are necessary. The analysis should extend far enough downstream to account for any potential backwater from downstream structures and channel and upstream far enough to evaluate potential impacts of the crossing.

Peak flow rates or hydrographs for the design storm event shall be determined using the hydrologic methodologies outlined in Chapter 2.

The hydraulic analysis shall include the following:

- Determination of the backwater associated with each alternative profile and waterway opening(s)
- Determination of the effects on flow distribution and velocities

- Existing and proposed condition water surface profiles for design and check flood conditions
- A scour analysis

Tailwater conditions can have a significant effect on the resulting headwater elevation for the bridge. Careful consideration should be made to the potential tailwater conditions for the specific location of the bridge. The hydraulic model shall extend far enough downstream so that the water surface profile at the bridge is controlled entirely by normal depth and is not sensitive to the assumed downstream boundary condition. The model shall also include any confluences or structures that may have an impact on the tail water conditions for the bridge.

The latest version of HEC-RAS shall be used to complete the hydraulic analysis. The use of other software programs shall be approved by the Drainage Regulating Entity. The analysis shall be in accordance with the recommendations provided within the Harris County Flood Control District's *Hydrology and Hydraulic Guidance Manual.*

4.4 Required Deliverables

The following information shall be submitted to the Drainage Regulatory Entity for the design of bridges and roadway culverts.

1. A vicinity map that identifies the full scope of improvements
2. A detailed map of the area and proposed culverts and bridges with all pertinent physiographic information including topographic information, right-of-way, existing drainage easements, proposed drainage easements, drainage facilities, and floodplain information
3. Discharge calculations specifying methodology and key assumptions used including discharges at key locations
4. Hydraulic calculations specifying methodology used; all assumptions and calculated values of the design parameters shall be clearly stated
5. A profile of the reach at the proposed structure which includes the following:
    a. All pertinent water surface profiles; this will include the design storm event and the 100-year storm event, at a minimum
    b. All existing and proposed bridge, culvert, and pipeline crossings
    c. The location of all tributary and drainage confluences
    d. The location of all hydraulic structures (e. g. dams, weirs, drop structures, etc.)
    e. Existing and proposed culvert or creek flowlines with flow direction arrows

6. A map delineating existing and proposed rights-of-way
7. Typical existing and proposed cross-sections at the proposed bridge or culvert
8. The relevant surveying benchmark, elevation, datum and year of adjustment
9. A soils report which addresses erosion and slope stability
10. All hydraulic modeling data

5 Storm Sewers and Overland Flow

5.1 General

This section focuses on the design of curb-and-gutter streets with underground, closed conduit storm sewers and the analysis of extreme event overland routing.

Prior to design of storm sewers and any overland flow, the Design Engineer shall consult the Drainage Regulatory Entity to determine if right-of-way for sheet flow will be accepted by the appropriate authority.

5.2 Runoff Analysis

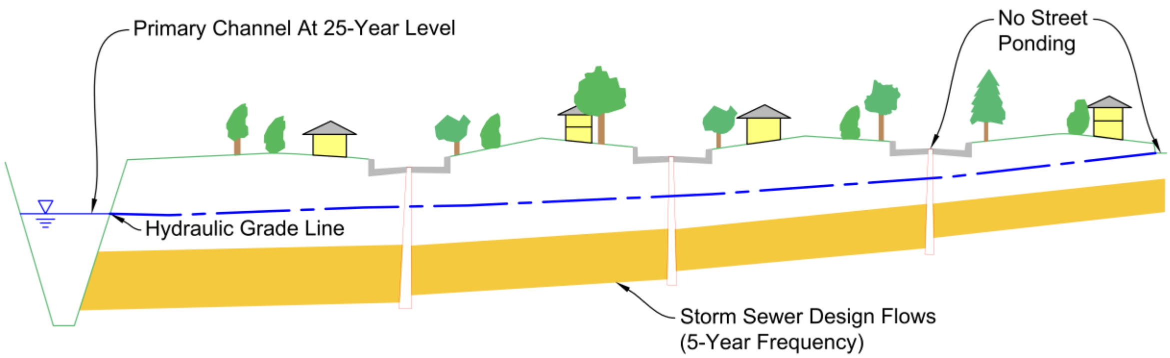
5.2.1 Frequency Considerations

Flooding in Brazoria County is generally associated with one of two types of severe rainfall events. The first type is a localized, high-intensity rainfall of short duration which floods a small, localized area causing ponding of water and interruption of traffic flow. The second type is a more generalized rainfall of longer duration which can cause more widespread flooding and can result in severe damage and loss of life.

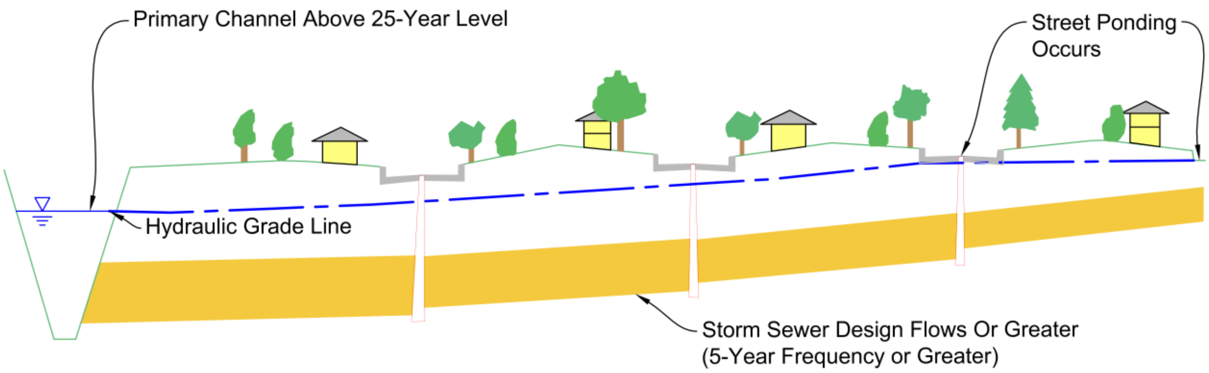
Underground storm sewer systems are typically designed to convey runoff from the localized, high-intensity, short-duration rainfall events. However, storm sewer systems are typically connected to regional conveyance systems which are designed for long duration storm events. Therefore, the design of these combined systems shall take into consideration both systems and both types of storm events.

Figure 5-1 illustrates the effect on the hydraulic grade line of a storm sewer for three outlet conditions, assuming the outlet channel is at the design 25-year water level.

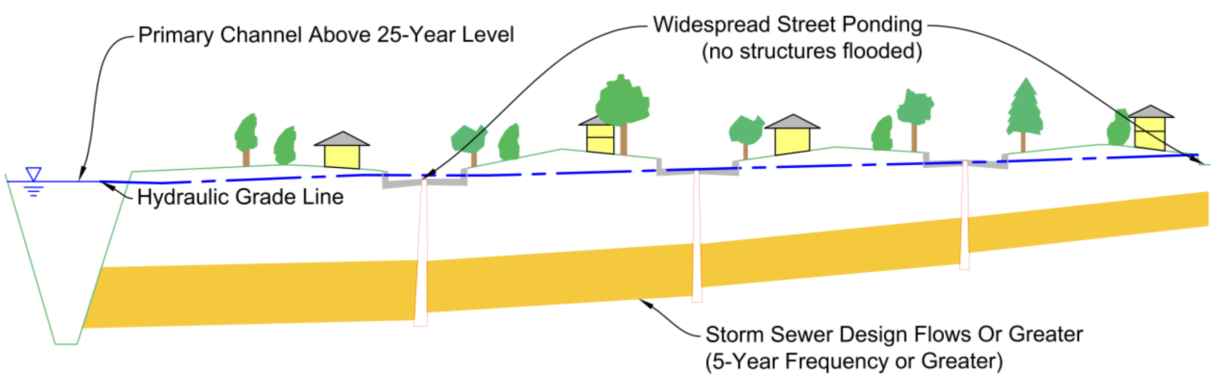
5 – Storm Sewers and Overland Flow



A) Standard Storm Sewer Design Considerations



B) Street Ponding Due to Tailwater Higher Than 25-year level or Rainfall in Excess of 5-Year Event in Storm Sewer



C) System Operating at Maximum Capacity

Figure 5-1 Storm Sewer - Channel Interaction for Brazoria County, Texas

Part A shows the hydraulic grade line for the standard design condition at or below the gutter level at the furthest inlet. For this condition, there is no street ponding and the storm sewers are functioning at or below their design capacity.

Parts B and C of the figure show the case where the tailwater condition is above the design level. For this condition, street ponding begins to occur throughout the drainage system as the storm sewer is unable to operate at the design capacity. This local flooding situation could also occur when the tailwater is below design conditions if local rainfall is in excess of that used in the design of the storm sewer system. As this widespread street ponding starts to occur, provisions shall be made to limit the depth of ponding, so no significant property damage occurs. In general, 100-year flood elevations shall be considered unacceptable when they exceed the lowest of the following:

1. One foot over natural ground
2. One foot over top of curb
3. Two feet below the lowest slab elevation
4. The natural ground elevation at the right-of-way lines

5.2.2 Design Flow Frequency Criteria

The recommended design flow frequency criteria to be used for continuous closed-conduit systems located inland of the coastal region are given below:

1. Peak flow rates for the design storm event and extreme storm event shall be determined using the hydrologic methodologies outlined in Chapter 2.
2. For the portions of the storm sewer system with a service area less than 100 acres, the design storm shall be the 5-year frequency event.
3. For the portions of the storm sewer system with a service area greater than 100 acres but less than 200 acres, the design storm shall be the 25-year storm event.
4. For the portions of the storm sewer system with a service area greater than 200 acres, the design storm shall be the 100-year storm event.
5. For all service areas and portions of the storm sewer system, overland flow shall be considered for the 100-year storm event as discussed in Section 5.5. Every attempt should be made to design major thoroughfares so that they are passable during the 100-year storm event.
6. Closed conduit systems receiving runoff from an upstream open channel shall be designed for the 100-year ultimate discharge from the channel.

5.2.3 Design Tailwater Conditions

The following tailwater conditions shall be used for determining the hydraulic grade line for the closed conduit system discharging into a detention pond:

1. For the 5-year design storm event, the starting tailwater elevation for the storm sewer system shall be the 2-year water surface elevation of the receiving detention pond or the top of outfall pipe, whichever is greater.
2. For the 25-year design storm event, the starting tailwater elevation for the storm sewer system shall be the 10-year water surface elevation of the receiving detention pond or the top of outfall pipe, whichever is greater.
3. For the 100-year storm event, the starting tailwater elevation for the storm sewer system shall be the 25-year water surface elevation of the receiving detention pond or the top of outfall pipe, whichever is greater.

For storm sewer discharging into natural creeks, channels, or larger storm trunk lines, the starting tailwater shall be determined per Section 6.3.4.

5.3 Closed Conduit

5.3.1 Design Criteria

Unless superseded by specific requirements of the appropriate Drainage Regulatory Entity, the following specific criteria and requirements shall apply to the design and construction of storm sewer systems in Brazoria County.

1. Calculation of the hydraulic grade line for design conditions in a specific branch of storm sewer shall proceed upstream from the water surface level in the outfall channel as specified in Section 5.2.3.
2. The storm sewer system shall be designed to convey runoff from the design storm event without causing the hydraulic grade line to exceed the gutter flow line in the street.
3. The minimum diameter of a storm sewer pipe shall be 24 inches.
4. Pipe sizes shall not decrease in the downstream direction, regardless of additional capacity developed by increased pipe slope.
5. Pipe soffit (inside top) elevations shall match whenever practical.
6. The minimum velocity to be allowed in a section of storm sewer flowing full shall be 3 feet per second. The maximum velocity shall be 10 feet per second. Refer to Section 3.3 for storm sewer outfall velocities and erosion protection.
7. All storm sewers and appurtenant construction shall conform to the Texas Department of Highway and Public Transportation Specifications and all subsequent revisions or approved equal.
8. All storm sewer, excluding outfalls, shall be constructed with reinforced concrete pipe or approved equal. HDPE or any comparable plastic pipe is not an accepted pipe material type.

9. Corrugated Metal Pipe (CMP) used at storm sewer outfalls shall conform to ASTM C76. See Detail 5-1 for storm sewer outlet design and Detail 3-5 for CMP bedding and backfill detail.
10. All cast-in-place concrete storm sewers shall follow the alignment of the right-of-way or easement.
11. All precast concrete pipe storm sewers shall be typically designed in a straight line or shall conform to the Texas Department of Highways and Public Transportation Specifications and all subsequent revisions or approved equal.
12. In most cases where easements are restricted to storm sewers, the storm sewer shall be centered within the limits of the easement.
13. For all storm sewers with a cross-sectional area equivalent to a 42-inch diameter pipe or larger, soil borings with logs shall be made along the alignment of the storm sewer at intervals not to exceed 500 feet and to a depth not less than 3 feet below the flowline of the sewer. The required bedding of the storm sewer as determined from these soil borings shall be shown in the profile of each respective storm sewer. The design engineer shall inspect the open trench and may authorize changes in the bedding indicated on the plans. Such changes shall be shown on the record drawings and, along with soil boring logs, submitted to the Drainage Regulatory Entity. All bedding and subsequent revisions shall be constructed as specified in the Texas Department of Highways and Public Transportation Specifications or approved equal.
14. All storm sewer inlet leads shall be designed in a straight-line alignment.
15. All storm sewers shall be located in public street right-of-way or in easements that will not prohibit future maintenance access.

5.3.2 Computation of the Hydraulic Grade Line

In order to adequately design the storm sewer system, the hydraulic grade line shall be computed in order to determine whether the storm sewer meets the criteria set forth under Section 5.3.1 and Section 5.3.2. The following steps are recommended for computing the hydraulic grade line computations.

1. Determine the peak design flow rates for all segments of the storm sewer system.
2. Determine the conduit sizes needed to convey the calculated design flow assuming full flow conditions using Manning's Equation.

$$Q = \frac{1.49}{n} A R^{2/3} S_o^{1/2} \tag{5-1}$$

Where,

Q	=	Discharge in the conduit (cfs);
n	=	Manning's 'n' value for the pipe;
A	=	Cross-sectional area of the conduit (sf);
R	=	Hydraulic Radius of the conduit (ft);
S	=	Slope of the pipe (ft / ft), this is equal to the friction slope

3. Calculate the friction and minor losses for all portions of the storm sewer system per Section 5.3.3 and Section 5.3.4.
4. Determine the appropriate starting water surface elevation at the outlet pipe per Section 5.2.3.
5. Beginning with the downstream starting water surface elevation and proceeding upstream, add the friction losses and minor losses for each conduit and junction to determine the resulting hydraulic grade line elevation at each junction throughout the system.
6. Adjust the conduit sizing until a desired hydraulic grade elevation is achieved.

5.3.3 Friction Losses

The main source of head loss in a storm sewer system is the friction loss in the conduit. The following two equations were derived from the Manning's Equation and can be used to determine the friction loss in the conduit. Equation 5-2 is the friction loss for a circular pipe and Equation 5-3 is the friction loss of any other type of conduit of a known cross-sectional area and hydraulic radius.

$$\boldsymbol{h_f = L\left[\frac{Qn}{0.4644D^{8/3}}\right]^2} \tag{5-2}$$

$$\boldsymbol{h_f = L\left[\frac{Qn}{1.496R^{2/3}A}\right]^2} \tag{5-3}$$

Where,

h_f	=	Head loss due to friction along the length of the conduit (ft);
Q	=	Discharge in the conduit (cfs);
L	=	Length of the conduit (ft);
n	=	Manning's 'n' value for the pipe;
D	=	Diameter of the pipe (ft);
R	=	Hydraulic radius of the conduit (ft);
A	=	Cross-sectional area of the conduit (sf)

The following table, Table 5-1, provides Manning's "n" values for typical closed-conduit pipe materials in Brazoria County.

Table 5-1 Manning's "n" for Typical Pipe Materials

Material	Roughness Coefficient (n)
Reinforced Concrete Pipe	0.013
Reinforced Concrete Box	0.015
Corrugated Metal	0.024

5.3.4 Minor Head Losses

Minor head losses shall be determined at all bends, junctions, and entrances. While the head losses at each structure may be minor, the cumulative effect of the combined minor losses to the hydraulic grade line throughout the entire system can be significant.

5.3.4.1 Head Loss at Bends

For flow through a junction (such as a manhole) where no additional flows are entering into the system and where there is no change in pipe size across the junction, Equation 5-4 along with the loss coefficients provided in Table 5-2 may be used to determine the head loss across the structure.

$$h_m = K\frac{(V)^2}{2g} \qquad (5\text{-}4)$$

Where,

h_m = Minor head loss across the junction (ft);
K = Loss coefficient;
V = Velocity in the conduit (ft/sec);
g = Acceleration due to gravity (ft/sec^2)

Table 5-2 Loss Coefficients for Bends

Type of Junction	Coefficient (K_j)
Straight through Manhole	0.05
22.5-Degree Bend	0.20
45-Degree Bend	0.35
60-Degree Bend	0.43
90-Degree Bend	0.50

5.3.4.2 Head Loss at Junctions

For junctions where a pipe change occurs, where lateral pipes join the junction, or where additional flow is introduced through an inlet, Equation 5-5 along with the loss coefficients provided in Table 5-3 may be used to determine the head loss across the structure:

$$h_m = \frac{(V_2)^2}{2g} - K\frac{(V_1)^2}{2g} \qquad (5\text{-}5)$$

Where,

h_m = Minor head loss across the junction (ft);
K = Loss coefficient;
V_1 = Velocity in the main line upstream of the junction (ft/sec);
V_2 = Velocity in the main line upstream of the junction (ft/sec);
g = Acceleration due to gravity (ft/sec^2)

Table 5-3 Loss Coefficients for Junctions

Type of Junction	Coefficient (K_j)
Inlet on main line	0.50
Inlet on main line with a lateral branch	0.25
Junction or manhole on main line with a 22.5-degree lateral branch	0.75
Junction or manhole on main Line with a 45-degree lateral branch	0.50
Junction or Manhole on main line with 60-degree lateral branch	0.35
Junction or manhole on main line with 90-degree lateral branch	0.25

5.3.4.3 Entrance Losses

For entrances, Equation 5-4 along with the loss coefficients provided in Table 5-4 may be used to determine the head loss at the entrance:

Table 5-4 Loss Coefficients for Entrances

Type of Entrance	Coefficient (K_j)
Inlet Entrance	1.25
Conduit Projecting from Fill, Socket End (Groove End)	0.20
Projecting from Fill, Square Cut End	0.50
Headwall and Wingwalls	
Socket End of Pipe (Groove-End)	0.20
Square-Edge	0.50
Rounded (radius = 1/12D)	0.20
Mitered to Conform to Fill Slope	0.70

5.3.5 Manholes

Manholes shall be placed at the following locations:

1. At the location of all changes in storm sewer size or cross section
2. At storm sewer intersections or P.I.'s
3. At storm sewer slope changes
4. At street intersections
5. At all inlet lead intersections with the storm sewer where precast concrete storm sewers are proposed
6. At maximum intervals measured along the centerline of the storm sewer per Table 5-5 below

Table 5-5 Manhole Placement Intervals

Pipe Diameter or Height (in)	Maximum Distance (ft)
24	300
30-36	375
42-54	450
60+	900

7. Not in the wheel travel lane of the street
8. Not immediately adjacent to a driveway

5.4 Inlets

5.4.1 Design Criteria

Unless superseded by specific requirements of the appropriate Drainage Regulatory Entity, the following specific criteria and requirements shall apply to the design and construction of inlets.

1. Two types of curb inlets are recommended for use in Brazoria County: Type "BB" Inlet and the Type "C-1" Inlet (with or without wings). All inlets shall be constructed per the City of Houston latest specifications.
2. All inlets shall be designed to convey the peak flow rate for the design storm event and meet the following limitations:
    a. Depth of ponding shall not exceed the top of curb.

b. For residential streets, the flow spread shall not exceed the center crown of the roadway.

c. For a roadway with two or more lanes in each direction, the flow spread shall not exceed the inside edge of the outside travel lane.

3. Inlets shall be spaced so that the maximum travel distance of water in the gutter will not exceed 600 feet one way for residential streets and 300 feet one way on major thoroughfares and streets within commercial developments.
4. Curb inlets shall be located on intersecting side streets to major thoroughfares for all original designs or developments to prevent concentrated storm water flow from crossing traffic lanes. Special conditions warranting other locations of inlets shall be determined on a case-by-case basis.
5. All storm sewer inlet leads shall be designed in a straight-line alignment.
6. All inlets shall be located in public street right-of-way or in easements that will not prohibit future maintenance access.

5.4.2 Sag Inlet Hydraulic Calculations

Inlets placed in the trough of the road profile are in a sag configuration. In a sag configuration, the flow velocity across an inlet is assumed to be zero. In this case, the standard weir and orifice equations are applicable for determining flow capture capacity.

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 inlet capacity is based on the lesser of the weir and orifice capacities.

For determining the capacity of a curb inlet located in a sag that is submerged, the standard orifice equation, Equation 5-6, may be used.

$$Q = C_o d_o L \sqrt{2gh} \tag{5-6}$$

Q	=	Total flow reaching the inlet (cfs);
C_o	=	Orifice coefficient (typically 0.67);
d_o	=	Physical depth of curb opening, including depression depth (ft);
L	=	Length of curb opening inlet (ft);
g	=	Acceleration due to gravity (32.2 ft/s^2);
h	=	Effective head at center of the orifice throat (ft)

For determining the capacity of a curb inlet located in a sag that is not submerged, the standard weir equation, Equation 5-7, may be used.

$$\mathbf{Q = C_w(L + 1.8W)y^{1.5}} \quad (5\text{-}7)$$

Where,

Q	=	Inlet capacity (cfs);
C_w	=	Weir coefficient (typically 3.0 for inlets without curb depression and typically 2.3 for depressed inlets when using English units);
L	=	Length of the opening which water enters the inlet (ft);
y	=	Total depth of water or head on the inlet (ft);
W	=	Gutter depression depth (ft)

5.4.3 On-Grade Inlet Hydraulic Calculations

Inlets placed on the sloped portion of a road profile are in an on-grade configuration. Inlets are typically placed on-grade in order to reduce the amount of flow in the gutter and, in turn, the flow spread in the roadway prior to reaching the trough in the road profile.

For determining the length of inlet required to capture the total flow in the gutter, Equation 5-8 may be used.

$$\boldsymbol{L_r = K_c Q^{0.42} S^{0.3} \left(\frac{1}{nS_e}\right)^{0.6}} \quad (5\text{-}8)$$

Where,

L_r	=	Length of opening required to intercept total flow in the gutter (ft);
K_c	=	Coefficient = 0.6 (English units);
Q	=	total flow in the gutter (cfs);
S	=	Longitudinal slope of gutter (ft/ft);
n	=	Manning's roughness coefficient;
S_e	=	Equivalent cross slope (ft/ft)

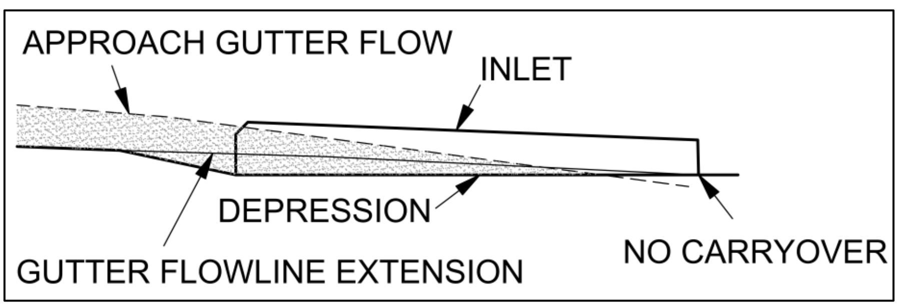


Figure 5-2 On-Grade Inlet Capacity Diagram

If the total required length of inlet cannot be provided, then the carryover flow shall be computed and applied to the next downstream gutter and inlet hydraulic calculations (see Figure 5-2). This carryover flow may be calculated using Equation 5-9.

$$Q_{co} = \mathbf{Q}(1 - \frac{L_a}{L_r})^{1.8} \tag{5-9}$$

Where,

Q_{co}	=	Carryover flow (cfs);
Q	=	Total flow in the gutter (cfs);
L_a	=	Design length of the curb opening (ft);
L_r	=	Length of opening required to intercept total flow in the gutter (ft)

Typically, on-grade inlets are inefficient unless the inlet is configured with a depression (see Figure 5-3). Adding a depression to the curb will adjust the equivalent cross slope parameter, S_e, in Equation 5-10. To determine the equivalent cross slope, use Equations 5-11 through 5-16.

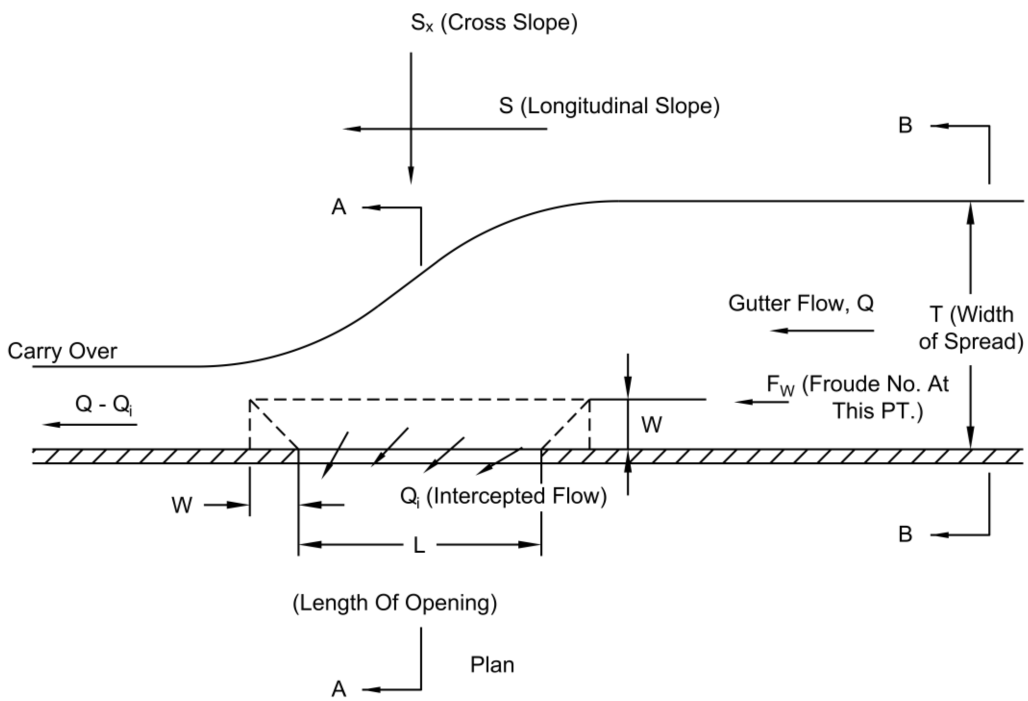


Figure 5-3 Gutter Carry Over Diagram

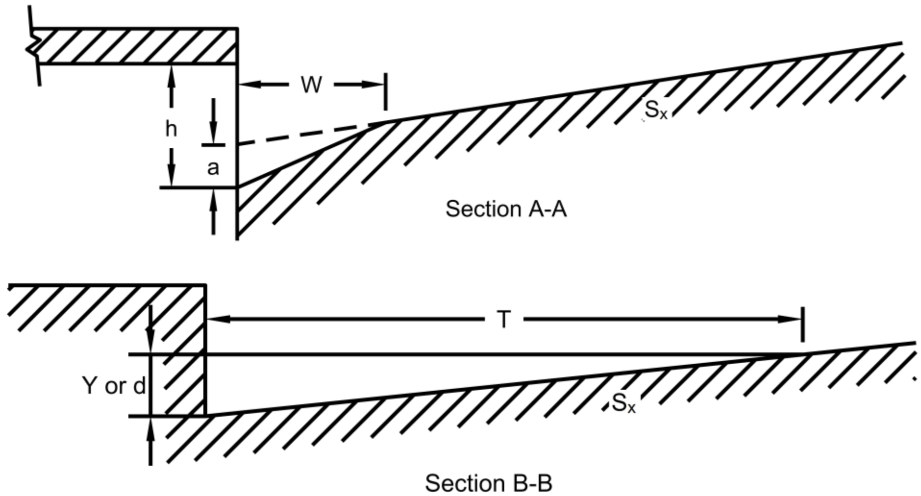


Figure 5-4 Gutter Flow Cross Section

$$S_e = S_x + \frac{a}{W} E_o \quad (5\text{-}10)$$

Where,

S_e	=	Equivalent cross slope (ft/ft);
S_x	=	Cross slope of the road (ft/ft);
a	=	Gutter depression depth (ft);
W	=	Gutter depression width (ft);
E_o	=	Ratio of depression flow to total flow

$$E_o = \frac{K_w}{K_w + K_o} \quad (5\text{-}11)$$

Where,

E_o	=	Ratio of depression flow to total flow;
K_w	=	Conveyance of depressed gutter section (cfs);
K_o	=	Conveyance of gutter section beyond the depression (cfs)

$$K_{(w/o)} = \frac{1.486A^{5/3}}{nP^{2/3}} \quad (5\text{-}12)$$

Where,

$K_{(w/o)}$	=	Conveyance of the cross-section (cfs);
A	=	Area of the cross-section (cfs);
n	=	Manning's roughness coefficient;
P	=	Wetted Perimeter (cfs)

$$A_w = WS_x\left(T - \frac{W}{2}\right) + \frac{1}{2}aW \quad (5\text{-}13)$$

Where,

A_w	=	Area of the depressed gutter section (ft^2);
W	=	Depression width (ft);
S_x	=	Cross slope (ft/ft);
T		Calculated ponded width (ft);
a	=	Curb opening depression depth (ft)

$$P_w = \sqrt{(WS_x + a)^2 + W^2} \quad (5\text{-}14)$$

Where,

P_w	=	Wetted perimeter of the depressed gutter section (ft);
W	=	Depression width (ft);
S_x	=	Cross slope (ft/ft);
a	=	Curb opening depression depth (ft)

$$A_o = \frac{S_x}{2}(T - W)^2 \quad (5\text{-}15)$$

Where,

A_0	=	Area of the depressed gutter section (sf);
W	=	Depression width (ft);
S_x	=	Cross slope (ft/ft);
T	=	Calculated ponded width (ft)

$$P_o = \mathbf{T} - \mathbf{W} \tag{5-16}$$

Where,

P_0	=	Wetted perimeter of the depressed gutter section;
W	=	Depression width (ft);
T	=	Calculated ponded width (ft)

5.4.4 Gutter Hydraulic Calculations

More frequent inlet spacing, especially for on-grade inlets, may be required to maintain a spread of gutter flow in the roadway less than the maximum allowed ponding widths specified in Section 5.4.1. Equation 5-17 may be used to determine the depth of flow in the gutter and Equation 5-18 may be used to determine the flow spread width in the roadway.

$$y = z\left(\frac{QnS_x}{S^{1/2}}\right)^{3/8} \tag{5-17}$$

Where,

y	=	Depth of water in the curb and gutter cross section (ft);
z	=	1.24 for English measurements or 1.443 for metric;
Q	=	Gutter flow rate (cfs);
n	=	Manning's roughness coefficient;
S_x	=	Transverse Slope (ft/ft);
S	=	Longitudinal slope (ft/ft)

$$T = \frac{y}{S_x} \tag{5-18}$$

Where,

T	=	Ponded width of flow (ft);
y	=	Depth of standing water or head on the inlet (ft);
S_x	=	Transverse slope (1/z) (ft/ft)

Table 5-6 Manning's n-Values for Street Pavement Gutters

Type of Gutter or Pavement		Roughness Coefficient (n)
Asphalt Pavement	Smooth Texture	0.013
	Rough Texture	0.016
Concrete Gutter with Asphalt Pavement	Smooth Texture	0.013
	Rough Texture	0.015
Concrete Pavement	Float Finish	0.014
	Broom Finish	0.016

5.5 Extreme Event Analysis

When the capacity of the underground system is exceeded and street ponding begins to occur, careful planning can reduce or eliminate the flood hazard for adjacent properties. Street layout and pavement grades are the key components in developing a successful system which can convey the storm sewer overflows to the outfall channel designed to carry the 100-year storm runoff.

5.5.1 Land Plan and Street Layout

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

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

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

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

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

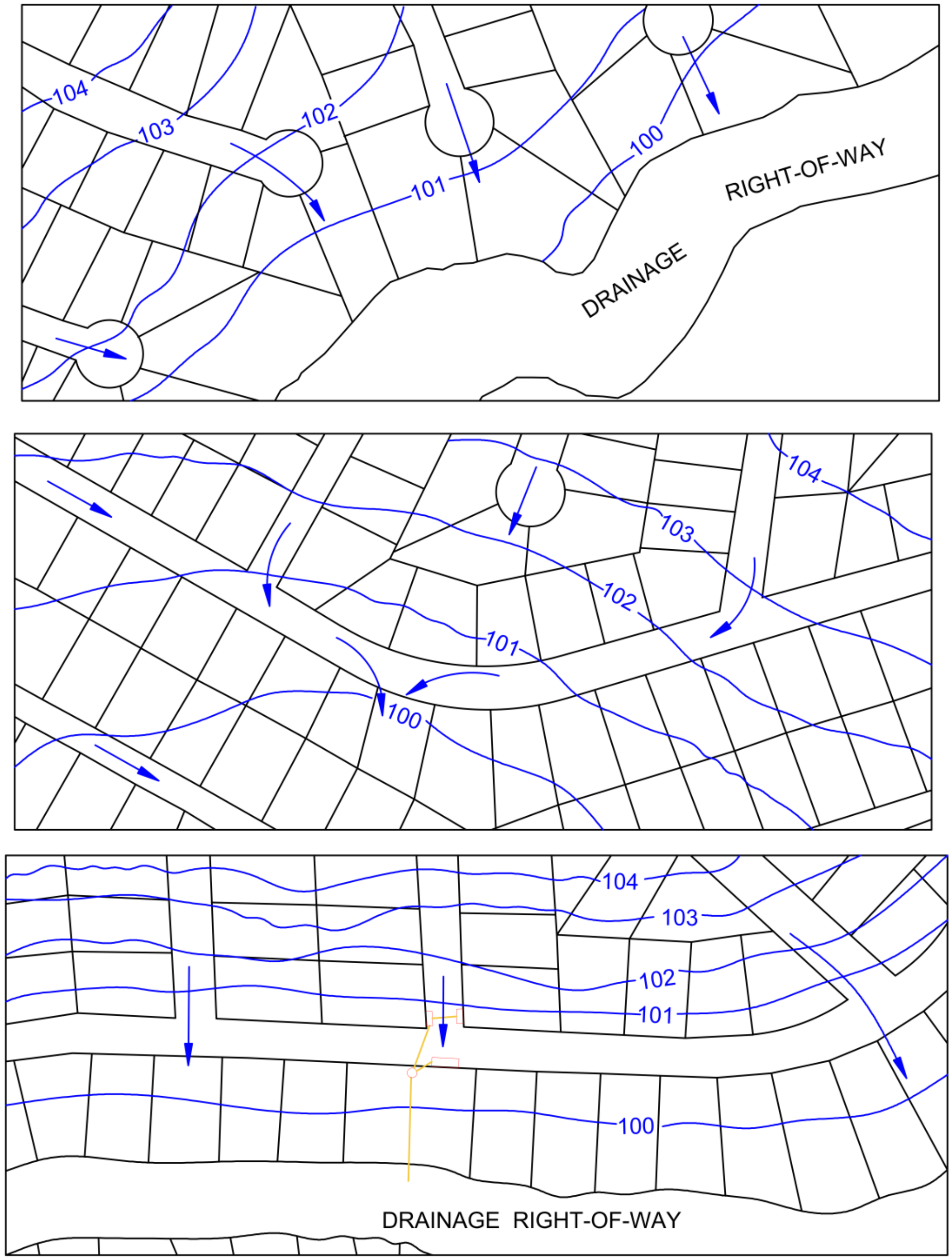


Figure 5-5 Undesirable Sheet Flow Patterns

Proper engineering foresight in the design of items such as emergency relief swales or underground systems can solve these potential problems. Some examples of acceptable sheet flow patterns are shown in Figure 5-6.

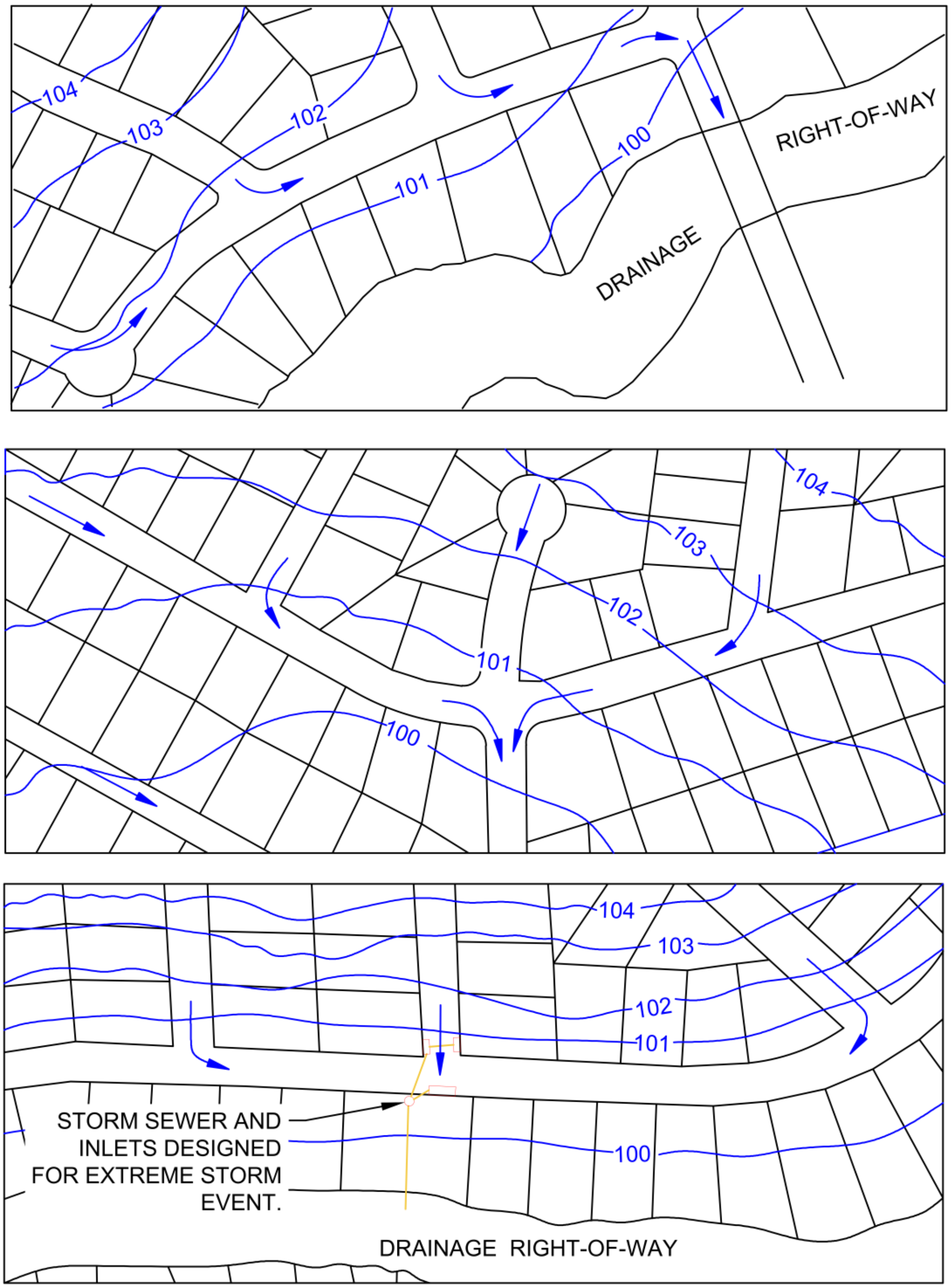


Figure 5-6 Desirable Sheet Flow Patterns

The maximum allowable ponding level in a street during a 100-year storm event is the lowest of the following:

1. One foot above top of curb at low points
2. Six inches above the top of curb at high points
3. Two feet below the lowest finished floor elevation
4. Natural ground at the street right-of-way

The design engineer shall verify that the storm sewer system and inlets can convey flows from a 100-year storm event without ponding water in the street at levels that exceed the maximum allowable levels listed previously.

If the maximum allowable water surface elevation is exceeded under current conditions, the engineer shall analyze the route along the street system that will convey the overflows to the major drainage channel or detention pond. The engineer shall verify again that the overflows will not exceed the maximum allowable ponding level. In making this analysis, the engineer should account for the flows that would be carried by the sewer system in addition to the street system.

Provisions shall be made to route the overflows from the street collection system into the appropriate drainage channel or detention pond.

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

Since a surface swale system would act only under emergency conditions and would not operate under normal circumstances, all precautions shall be taken to ensure that the relief system is regularly maintained and will function when needed. The recommended design procedure for sizing storm sewers for sheet flow conveyance is presented in Section 5.5.2.

The design engineer shall submit supporting calculations, exhibits, and drawings which define the conveyance capacity of the roadway, the flow paths of overland sheet flow, and the ponding depths of overland sheet flow.

5.5.2 Extreme Event Routing Analysis

To determine whether adequate conveyance capacity has been provided to route extreme event flows to the outlet location, three analysis methods as presented in *Technical Paper No. 101, Simplified 100-year Event Analyses of Storm Sewers and Resultant Water Surface Elevations for Improvement Projects in the City of Houston, Harris County, Texas Region* will be acceptable to the Drainage Regulatory Entity.

1. Method 1: Hydraulic Grade Line Analysis. This method is a simplified approach to analyze and control the 100-year water surface elevation by designing the storm sewer system for the 5-year frequency rainfall event; imposing a 100-year frequency storm event on the proposed design; calculating the hydraulic grade for the 100-year frequency event for the proposed design; and adjusting the position of the HGL to not exceed the critical elevation by increasing the size of the proposed storm sewer for selective reaches as well as the size of the inlets.

2. Method 2: Overland Analysis. This method utilizes overland flood routing to control the 100-year water surface elevation by designing the storm sewer for the 5-year frequency rainfall and designing the overland drainage system (either street gutters or emergency swales) to route the excess flows not conveyed by the storm sewer system and inlets to the outlet point. Typically, the street crests should be cascaded to allow flood waters to pass over the road crest before reaching critical ponding depths. In cases where this is not possible due to topographic restraints, overland channel systems will be required to convey the excess runoff away from the streets and to the outlet point.

$$\mathbf{Q_t = Q_o + Q_c} \qquad (5\text{-}19)$$

Where,

Q_t = Total flow conveyed (cfs);
Q_o = Overland flow component (cfs);
Q_c = Calculated flow in the conduit for the 5-year design event (cfs)

The overland flow component (Q_o) is computed by applying Manning's Equation to calculate the flow across the critical street cross-section, along the R.O.W., or over the overflow emergency swale. The conduit flow component is computed following the procedure outlined in Section 5.3.2.

This method accounts for flow in the storm sewer and overland flow across the street crest, but it does not account for street ponding or storage.

3. Method 3: Storage Analysis. This method is similar to Method 2 but adds a storage component to account for the available floodwater storage in the overland drainage system.

$$\boldsymbol{Q_t = Q_o + Q_c + \Delta S/T} \qquad (5\text{-}20)$$

Where,

Q_t	=	Total flow conveyed (cfs);
Q_o	=	Overland flow component (cfs);
Q_c	=	Calculated flow in conduit for the 2-year design event (cfs);
$\Delta S/T$	=	Change in storage volume relative to time provided in the streets and adjacent areas upstream of the point of interest being analyzed (cfs)

This method uses a volumetric calculation based on a 100-year frequency storm event with a duration of 3-hours for developments less than 200 acres and 6-hours duration for developments over 200 acres. The Soil Conservation Service, TR-20 method is used to set a peak triangular hydrograph shape. This method accounts for flow in the storm sewer, overland flow across the street crest, and storage within the street and adjacent area.

5.6 Submittals

Before a storm sewer system design will be reviewed, the following items shall be presented to the Drainage Regulatory Entity.

1. A contour and drainage area map showing all pertinent subareas, including contributing off-site areas
2. A listing of all relevant hydrologic design flow calculations including:
    a. Contributing drainage area (i.e., service area) for each closed conduit and inlet
    b. All pertinent hydrologic parameters for the hydrologic methodology chosen in accordance with Section 2
    c. Peak flow rates for each closed conduit and inlet
3. A listing of all relevant hydraulic design calculations including:
    a. Maximum velocity for each closed conduit assuming full flow conditions
    b. Minimum velocity for each closed conduit assuming full flow conditions
    c. Calculations for determining the hydraulic grade line including friction losses and minor losses
    d. Hydraulic capacity for each inlet
    e. Maximum depth of flow and spread in the road in front of both sag inlets and on-grade inlets
4. A plan showing the location of all manholes and inlets, and the alignment of all storm sewers in the right-of-way

5. A profile showing the placement of storm sewers with the location of all the following items:
    a. pipe size changes
    b. grade changes
    c. pipe intersections
    d. the design event hydraulic grade line
    e. the extreme event hydraulic grade line

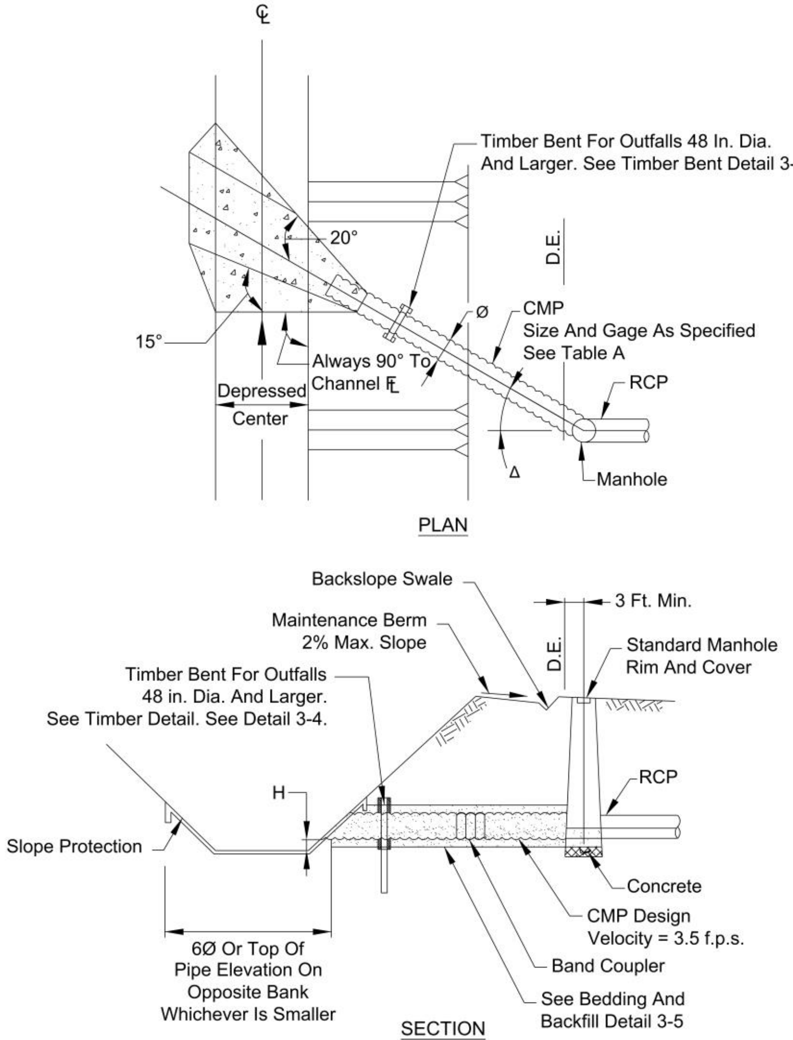


TABLE A

Size 2 ⅔" - ½" Corrugation	Pipe Gauge	Band Coupler Gage
24"	16	16
30"	16	16
36"	16	16
42"	14	16
48"	14	16
54"	12	14
60"	12	14
66"	10	12
78"	10	12
84"	8	10
96"	8	10

Δ : Prop. 24" To 42" Δ15°
Prop. 48" And Larger Δ = 30°

H : For Pipe Sizes 24" to 42"
H=3' Max. And 1' Min.
For Pipe Sizes 48" And Larger
H=1' Max. And Min.

TABLE B

Pipe Diameter ø	Slope	Velocity
24"	0.6%	3.25 f.p.s
36"	0.3%	3.00 f.p.s
42"	0.2%	2.75 f.p.s
48"	0.2%	3.00 f.p.s
54"	0.2%	3.25 f.p.s

POLICY, CRITERIA, & PROCEDURE MANUAL

TYPICAL STORM SEWER OUTFALL STRUCTURE LAYOUT

APRIL 2021

DETAIL 5-1

6 Detention

6.1 General

Volumetric storage of stormwater runoff is a common mitigation technique used to offset the impact of increased peak discharge due to development. This section of the manual presents information on storage techniques including guidance for the design of appropriate storm runoff storage facilities. Drainage design for rural subdivisions is presented in Chapter 10.

Regional detention is encouraged and preferable to scattered on-site detention facilities which are often inadequately maintained. Regional detention is appropriate when the following conditions exist:

- Land is available at appropriate locations, considering the hydraulics and hydrology of the drainage area.
- There exists an entity willing and able to absorb the up-front costs of acquiring the land and constructing the detention facilities and collect fees over time from those benefiting from the facilities.
- Adequate conveyance is provided from surrounding areas to the detention facility.
- Earthen slopes will be revegetated immediately after construction to reduce erosion.

6.2 Detention Storage Types

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

6.2.1 Retention Storage

In a retention storage facility, runoff is captured during a storm and released only after the event is over and the downstream water surface has subsided. A retention storage system is seldom used and when it is, special outlet devices or pumps are usually required. The use of retention storage requires the approval of the Drainage Regulating Entity. Automatic release gates with manual override ability are required at outfalls. The submitted design shall include recirculation pumps to maintain appropriate water quality in the retained portion of the facility. Operating instructions, including a requirement to operate pumps daily, shall be included in the construction documents and on the maintenance agreement. An auxiliary water source is required to maintain the normal pool in the retention storage portion of the facility. A maintenance agreement outlining routine maintenance by the property owner shall be required for any retention storage facility.

Retention storage shall be wholly maintained within a drainage easement. Private maintenance shall be required. Access by the County shall always be allowed. If the private property owner does not properly maintain the facility to its design capacity and function, the County shall have the right, but not the responsibility, to enter the property and maintain the facility. The County reserves the right to lien the property for the cost of the maintenance upon failure of the property owner to perform prompt, adequate maintenance.

6.2.2 Detention Storage

The vast majority of flood control storage in Brazoria County is achieved by detention facilities. The purpose of detention storage is to hold storm runoff and release it continuously at a rate that mitigates a development’s increase in peak discharge. Using a flow-limiting outlet structure, downstream peak flows can be controlled. Design of these facilities is discussed in subsequent sections.

6.2.3 On-line Storage

A detention basin where the entire hydrograph passes through the basin. This type of basin is best at controlling the rising limb of the hydrograph and delaying the time-to-peak discharge. On-line detention basins can be on-site detention basins, or if they are open to a channel they are sometimes referred to as “flow-through” detention basins.

6.2.4 In-line Storage

Detention storage is provided within a channel right-of-way and only near the headwaters of a drainage area with only the immediate landowner(s) draining to it. The channel is either oversized and/or changed to elevate the water surface inside it by a control structure or increasing roughness in order to slow the storm water and prevent downstream flooding.

6.2.5 Off-channel Storage

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

6.3 Design Procedures

The following design procedures are intended to ensure that new development, with detention, will not cause any adverse impacts on existing flooding conditions downstream. (Note: the design engineers shall contact the Drainage Regulating

Entity for any specific requirements for the watershed in which the proposed facility is to be located.)

6.3.1 For Drainage Areas < 50 Acres

The maximum allowable release rate from the detention facility during the 100-year storm event is the rate of runoff from the drainage area prior to development. The acre-feet of flood control storage, S, to be provided by the facility for the 100-year storm event is:

$$\boldsymbol{S = I^{1/2} * A} \qquad (6\text{-}1)$$

Where,

I = Average percent imperviousness of the area draining into the facility divided by 100 (see also Table 2-1);

A = Drainage area of the facility (ac)

The size of the outlet structure required to pass the maximum allowable release rate for the 2-year, 25-year, and 100-year storm events shall be computed assuming outlet control. Establish a maximum ponding level in the detention facility for the 100-year storm event and then estimate the 25-year and 2-year water surface elevations based on the volume assumptions listed below and utilize the receiving stream tailwater at the downstream end of the outlet pipe or at a depth in the outlet channel associated with the maximum release flowrate, whichever is higher.

- For the 25-year storm event, use 75% of the 100-year design volume in the detention facility to estimate the 25-year design volume and water surface elevation.
- For the 2-year storm event, use 30% of the 100-year design volume in the detention facility to estimate the 2-year design volume and water surface elevation.

Detention ponds designed to operate in series, or in conjunction with other peak discharge timing considerations, shall be sized as described in Section 6.3.3.

6.3.2 For Drainage Areas > 50 Acres and < 640 Acres

To design detention facilities serving watersheds with drainage areas between 50 and 640 acres, the Malcom Small Watershed Method presented in Section 2.3.1 shall be used to develop the inflow hydrographs and required storage volume for the facility. The procedure for sizing the pond outlet pipe for each design storms is described in Section 6.3.1.

Detention ponds designed to operate in series, or in conjunction with other peak discharge timing considerations, shall be sized as described in Section 6.3.3.

6.3.3 For Drainage Areas > 640 Acres

A detailed hydrologic analysis utilizing HEC-HMS or another method that has been approved by the Drainage Regulating Entity, shall be required to develop inflow and outflow hydrographs at the detention location. The HEC-HMS model shall then be used to verify the size of the facility and the outlet structure and to mitigate the project's impact on downstream flooding conditions. Once existing conditions are established, the impact of the proposed development including the detention facility shall be analyzed for the 2-year, 25-year, and 100-year frequency storm events, utilizing NOAA Atlas 14 precipitation data. The 2-year and 25-year event storms shall be used for determining the sizing of the outflow structure to prevent impacts to downstream properties. The 100-year will be used for sizing the required detention volume and configuring an emergency spillway. The detention facility will be sized to allow an appropriate release rate that will not cause any increase in flood levels downstream.

6.3.4 Design Tailwater Depth

In order to route the inflow hydrograph through the detention facility in the HEC-HMS model, a relationship shall be established between the volume of storage in the pond and the corresponding amount of discharge through the outflow structure. In most cases in Brazoria County, this relationship is directly dependent on the elevation of the tailwater at the outlet of the outflow structure. For the purpose of establishing an outflow rating curve, detention facilities that are evaluated using computer models shall use a variable tailwater condition based on the frequency storm being analyzed. The Design Engineer shall submit calculations to the Drainage Regulatory Entity to substantiate the design. The variable tailwater stage hydrograph can be developed using one of the following methods:

- Use an available (and current) HEC-RAS unsteady flow model, approved by the Drainage Regulatory Entity, of the receiving channel to define a stage hydrograph for a variable tailwater condition. See Section 3.4.3. This is the preferred method, but the lack of availability of a HEC-RAS unsteady flow model may require one of the following two methods.
- If no HEC-RAS unsteady flow model is available, apply a 100-year flow hydrograph developed in HEC-HMS to a rating curve from a HEC-RAS steady flow model. To develop the rating curve, run HEC-RAS profiles for 10%, 20%, 30%, …, 120% of the 100-year flow, and develop a discharge vs. elevation relationship at the location of the outfall. From this relationship, convert the HEC-HMS flow hydrograph to a stage hydrograph which can be used as a variable tailwater condition. Re-run the HEC-HMS

model for the 2-year and 25-year and repeat the effort to define variable tailwater.

- If no HEC-RAS steady flow model is available, it is possible that a HEC-HMS model is also not available. Develop a 100-year flow hydrograph for the watershed of the receiving stream at the point of outfall from the detention basin. This can be done with a single subbasin model to represent the watershed. To develop a rating curve, use Manning's Equation to determine elevations for 10%, 20%, 30%, …, 120% of the 100-year flow, and develop a discharge vs. elevation relationship at the location of the outfall. From this relationship, convert the HEC-HMS flow hydrograph to a stage hydrograph which can be used as a variable tailwater condition. Re-run the HEC-HMS model for the 2-year and 25-year and repeat the effort to define variable tailwater.

In certain situations where the assumption for the variable tailwater may prove to be unreasonable, coincidental peak with the design storm event can be considered for the determination of tailwater elevation using Table 6-1. A coincidental peak analysis is useful to reduce the starting HGL based on the ratio of the storm system's drainage area to the watershed area of the receiving stream. For the determination of hydraulic gradient and the sizing of storm drain conduits, a tailwater elevation, which can be reasonably expected to occur coincident with the design storm event, shall be used. Provide calculations for each coincidental occurrence and the more restrictive results based on the area ratio shall be used.

Table 6-1 Frequencies for Coincidental Occurrence

Area Ratio	2-Year design		5-Year design		25-Year design		100-Year design	
	Mainstream	Tributary	Main Stream	Tributary	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	2	1	5	2	25	2	100
	2	1	5	1	25	2	100	2
1000:1	1	2	2	5	5	25	10	100
	2	1	5	2	25	5	100	10
100:1	2	2	2	5	10	25	25	100
	2	2	5	5	25	10	100	25
10:1	2	2	5	5	10	25	50	100
	2	2	5	5	25	10	100	50
1:1	2	2	5	5	25	25	100	100
	2	2	5	5	25	25	100	100

Flap gates shall stay within the private easement of the basin (not on private property). "Bladder" pneumatic gates are also allowed with the consent of the Drainage Regulating Entity. Routine maintenance of the flap gate shall be included in a facilities maintenance agreement prior to acceptance of construction.

6.3.5 Downstream Impact Analysis Requirements

A downstream impact analysis is required for proposed residential developments equal to or greater than 5000 square feet of disturbed land and all other types of land use. The downstream analysis should be conducted using HEC-HMS and HEC-RAS at least one mile downstream of the proposed development. Table 6-2 summarizes the requirements of the downstream analysis. Proposed developments shall not create a net loss of storage volume below the 100-year floodplain. Offsite discharges shall be contained within a drainage easement upon leaving the proposed project site.

Table 6-2 Downstream Analysis Requirements

Category	Value
Return Events Evaluated	2-Year, 25-Year, and 100-Year Storms
Discharge	Increase less than 0.40 cfs
Water Surface Elevation	Less than 0.10 ft in all conditions Less than 0.00 ft at existing habitable structures
Velocity	No more than 5% increase in velocity up to maximum permissible velocity as defined in Section 3.3.1.1

6.3.6 Sizing of Pond Storage and Outflow Structure

To size detention or retention facilities, a minimum of 6 inches of freeboard is required if the average storage depth is 3 feet or less. For ponds with an average storage depth greater than 3 feet, a minimum of 12 inches of freeboard shall be maintained during the 100-year storm event. The average depth is from the top down to the normal pool level for wet pond and from top to the bottom of pond for dry pond. Freeboard shall be measured from the minimum elevation of the top of the detention or retention facility berm to the 100-year storm water surface elevation.

The maximum side slope allowed is 4 (horizontal):1(vertical) for long term stability and maintenance.

Minimum berm widths around a detention basin are presented in Table 6-3. Add an additional ten feet if benches, trees, equipment, or other obstructive features are located on the berm.

Table 6-3 Minimum Berm Widths

Detention Basins	Min. Berm Width
Grass-lined	20 feet
Grass-lined where side slopes are 8 (horizontal):1(vertical) or flatter	15 feet
Lined with riprap or partially concrete-lined	Same as grass-lined channel
Fully concrete-lined	20 feet

The volume of water held in public infrastructure (storm drainpipes or roadside ditches) shall not be considered as available detention volume.

The minimum allowable diameter for an outflow pipe of a detention facility is 24 inches. For outfall to a roadside ditch, an 18-inch diameter pipe may be utilized upon approval by the Drainage Regulating Entity. The end treatment of an outfall pipe shall be cut to match the roadside ditch side slope. The outfall pipe embedment shall include a minimum of 1 foot of stabilized sand around the pipe. Where outfall configuration includes an orifice, minimum opening size shall be a 6-inch diameter. Routine maintenance of the outfall structure shall be included in a facilities maintenance agreement prior to acceptance of construction.

The entity that regulates the proposed outfall shall be consulted before designing a drainage system. Roadside ditches on county roads may be used as drainage outfalls only when all the following conditions are met:

1. The roadside ditches are the only existing means of drainage.
2. Property being developed drains naturally to the roadside ditches.
3. Runoff after development is limited to the runoff that occurred prior to development.

6.3.7 Allowances for Extreme Storm Events

An emergency spillway, overflow structure, or swale (collectively referred to as the "emergency spillway") shall be sized with the following criteria:

- Set the emergency spillway crest elevation at or above the 100-year pond elevation.
- Size the emergency spillway to carry the 100-year flow so that the 100-year detention storage does not exceed the top-of-bank of the facility.
- Assume that the principal outlet structure is completely blocked.

- Size the emergency spillway assuming a normal pool level (if a wet pond) or a dry condition (if a dry pond) at the beginning of the storm.
- A minimum of 12 inches of freeboard above the spillway crest is required for an emergency overflow/extreme event condition.
- If the spillway is not immediately adjacent to a receiving stream, obtain a flowage easement to provide a clear path for conveyance without affecting adjacent property owners.

In places where a dam has been utilized to provide detention, due consideration shall be given to the consequences of a failure. If a significant hazard exists, the dam shall be adequately designed to prevent such hazards.

In addition, detention facilities with an outfall berm greater than 6 feet in height are subject to Title 30 Texas Administrative Code (TAC) Chapter 299 (Sub chapters A through E, latest edition) and all subsequent changes. The height of a detention facility or dam is defined as the distance from the lowest point on the crest of the dam (or embankment), excluding spillways, to the lowest elevation on the centerline or downstream toe of the dam (or embankment) including the natural stream channel. Subchapters A through E of Chapter 299 classify dam sizes and hazard potential and specify required failure analyses and spillway design flood criteria.

6.3.8 Erosion Controls

The same types of erosion protection required in earthen channels shall be incorporated in detention design including the use of backslope swales and drainage systems, proper revegetation, and pond surface lining where necessary, as outlined in Section 3.3. Extra care shall be taken to provide proper protection at pipe outfalls into the facility, pond outlet structures, and overflow spillways where excessive turbulence and velocities will encourage erosion.

6.4 Multipurpose Land Use

When a dual use facility is proposed, a joint use agreement is required between the Public Drainage Regulatory Entity and the entity sponsoring the secondary use. In all cases, maintenance shall be the responsibility of the private property owner and not the Drainage Regulatory Entity.

6.4.1 Approval of Facilities

Each stormwater detention facility will be reviewed and approved only if:

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

2. Provisions are made for the facility to be adequately maintained and a maintenance agreement is completed between the Drainage Regulatory Entity and the private property owner; and
3. If walking paths or other amenities are anticipated, sufficient details of each improvement shall be provided to Drainage Regulatory Entity for review and comment. The trail or path geometry and location may be required to utilize special specifications or provide access for maintenance vehicles to cross the facility.

6.4.2 Maintenance

Each development that provides detention shall make provisions to ensure future maintenance of the detention facility. Typically, a property owners association, LID, WCID, or MUD will be established and given the responsibility to maintain the drainage facility. The entity responsible for the maintenance of the facility shall be noted on the plat or plans. In all cases, maintenance shall be the responsibility of the private property owner and not the Drainage Regulatory Entity.

The Drainage Regulatory Entity has the right to inspect the facility and determine if the facility is maintained to designed conditions. Facilities and attendant maintenance berms shall be wholly contained within a drainage easement. Private maintenance shall be required. Access by the public Drainage Regulatory Entity shall be allowed at all times. If the private property owner does not properly maintain the facility to its design capacity and function, the Drainage Regulatory Entity shall have the right, but not the responsibility, to enter the property and maintain the facility. The Drainage Regulatory Entity reserves the right to lien the property for the cost of the maintenance upon failure of the property owner to perform prompt, adequate maintenance.

A 20-foot-wide unobstructed access and maintenance easement shall be provided from street, road, or adequate access-way to any drainage ditch, channel, or detention pond. This is in addition to the dedication required for the pond itself. The drainage district will not be responsible for any damage to curbs leading from streets to access ways. Table 6-3 Minimum Berm Widths shows the minimum criteria for maintenance berms in different development scenarios.

If guard rails or other impediments will block access to drainage ditches or detention facilities, adequate provisions shall be provided to allow reasonable access to the channel or drainage facility by Drainage Regulatory Entities personnel and equipment.

The drainage district will not be responsible for any damage to curbs leading from streets to access ways.

6.5 Pump Detention

Pumped detention systems will not be maintained by the Drainage Regulatory Entity under any circumstances and will be approved for use only under the following conditions:

1. A gravity system is not feasible based on provided engineering and economic calculations.
2. A maximum of 50% of the required storage volume is pumped detention.
3. At least two pumps are provided, each of which is sized to pump the design flow rate. If a triplex system is used, any two of the three pumps shall be capable of pumping the design flow rate.
4. Gravity and pumped flow shall not exceed maximum allowable outflow rate.
5. Auto shut off pumps may be considered. Auto shut off pumps shall be approved by Drainage Regulatory Entity.
6. The selected design outflow rate shall not aggravate downstream flooding. (Example: A pump system designed to discharge at the existing 100-year flow rate each time the system comes on-line could aggravate flooding for more frequent storm events.)
7. Fencing of the control panel is provided to prevent unauthorized operation and vandalism.
8. Adequate assurance is provided that the system will be operated and maintained on a continuous basis. A maintenance agreement shall be executed between the facility owner and the Drainage Regulatory Entity.
9. Emergency auxiliary power supply is provided and maintained.
10. A maintenance plan for operation of pumped system is required.

6.5.1 Allowable Drain Time Requirements

Minimum detention volume is 0.65 acre-feet per acre of new development. Allowable drain time is defined as the maximum allowable time to drain 80% of the detention basin volume. This is required to preserve detention storage for successive storm events which could affect the drainage system. Drain time is evaluated without a tailwater condition (free outfall), starting at the maximum water surface elevation in the detention basin from a 100-year storm event.

Maximum drain time is 48 hours (2 days) to drain 80% of the volume. If the maximum outflow rate results in a longer drain time, an increase in detention volume will also be required:

1. Increase by 5% if drain time is 3 days.

2. Increase by 10% if drain time is 4 days.
3. Increase by 15% if drain time is 5 days.
4. Increase by 20% if drain time is 6 days.
5. Increase by 25% if drain time is 7 days.

In no case shall a drain time longer than 7 days (168 hours) be allowed in meeting the minimum design requirements.

If a pumped detention system is desired, review of the preliminary conceptual design by the Drainage Regulatory Entity shall be obtained before any detailed engineering is performed.

6.6 Geotechnical Investigation

Before initiating final design of a detention pond, a detailed soils investigation by a professional geotechnical engineer, licensed in the State of Texas, shall be undertaken. The following minimum requirements shall be addressed:

1. Stability of the basin side slopes for short term and long-term conditions. (If basin depth ≤ 5 feet, a slope stability analysis is not required, however, a geotechnical report is still required to address the other issues.)
2. Stability of the permanent pool side slopes.
3. Evaluation of bottom instability due to excess hydrostatic pressure.
4. Control of groundwater.
5. Identification of dispersive soils.
6. Potential erosion problems.
7. Constructability issues.
8. Evaluation of inflow and outflow structures.
9. If a dam is to be constructed, the following shall be required:
    a. adequate investigation of potential seepage problems through the dam
    b. attendant control requirements
    c. availability of suitable embankment material
    d. stability requirement for the dam itself
10. Investigation into the potential for structural movement on areas adjacent to the pond may be required. This is mainly due to the induced loads from exiting or proposed structures and methods of controlling it.

6.7 General Requirements for Detention Pond Construction

The structural design of detention facilities is very similar to the design of open channels. For this reason, all requirements from Chapter 3 pertaining to the design of lined or unlined channels shall also apply to lined or unlined detention facilities. In addition, the following design requirements taken from the Criteria Manual for Design of Flood Control and Drainage Facilities in Harris County, Texas (the latest edition) are applicable:

1. Dry Pond Bottom Design – A pilot channel shall be provided in detention facilities to ensure that proper and complete drainage of the storage facility will occur. Concrete pilot channels shall have a minimum depth of one foot and a minimum flowline slope of 0.001 ft/ft. Unlined pilot channels shall have a minimum depth of two feet, a minimum flowline slope of 0.002 ft/ft, and maximum side slopes of 3 to 1.
2. The bottom slopes of the detention basin shall be graded toward the pilot channel at a minimum slope of 0.01 ft/ft. Detention basins which make use of a channel section for detention storage may not be required to have a pilot channel but shall be built in accordance with the requirements for open channels as outlined in Chapter 3.
3. Wet Pond Bottom Design – Ponds with a permanent pool shall include a bottom shelf to reduce the risk of falling into the water by running or rolling down the side slope. The bottom shelf shall be located one foot above the static water surface (normal pool level), have a minimum width of 10 feet, and a cross slope of 0.02 ft/ft. Wet bottom ponds shall have a minimum water depth of 6 feet to prevent the growth of vegetation.
4. Shallow pools may be used around the edges of deeper pools to support aquatic plants and habitat, and to improve water quality. However, shallow pools alone cannot be used in a pond bottom due to maintenance issues.
5. Outlet Structure - The outlet structure for a detention pond is subject to higher head water conditions and erosive velocities for prolonged periods of time. For this reason, the erosion protective measures are very important.
6. Reinforced concrete pipe used in the outlet structure shall conform to ASTM C-76 Class III with compression type rubber gasket joints conforming to ASTM C-443. Pipes, culverts and conduits used in the outlet structures shall be carefully constructed with sufficient compaction of the backfill material around the pipe structure as recommended in the geotechnical analysis. Generally, compaction density shall be the same as the rest of the structure. The use of pressure grouting around the outlet conduit shall be considered where soil types or conditions may prevent satisfactory backfill compaction. Pressure grouting shall also be used where headwater depths could cause backfill to wash out around the pipe.

7. Vertical Walls – Where vertical walls are included in the design of a detention basin, the applicable requirements in Section 3.2.2.5 of this manual shall be included. In addition, all vertical wall designs shall require a geotechnical analysis of surrounding soils and a structural foundation design for the wall.

7 Leveed Areas

The components of the levee system shall include an internal drainage system, a levee, a pump station with adequate storage capacity, and a gravity outlet with an outfall channel to a river or the Gulf. The Drainage Regulatory Entity's design criteria for each component is defined in the following sections.

The Drainage Regulatory Entity's minimum design standards shall be governed by the rules and regulations as established by the Federal Emergency Management Agency (FEMA) and the United States Army Corps of Engineers (USACE) including any updates as they occur, and any additional requirements contained in this document. The engineer is advised to check the current FEMA and USACE rules and regulations. Maintenance of these facilities will not be the responsibility of the Brazoria County Drainage District, unless specifically noted otherwise on the construction documents and in a fully executed maintenance plan.

Levees within Brazoria County may be identified as either federal or non-federal levees on USACE National Levee Database website. Non-federal levees can be part of the USACE Levee Safety Program and/or part of the USACE Rehabilitation and Inspection Program (RIP). These programs were implemented to reduce flood risk to the public by developing national policies and standards for the design, operation, and maintenance of levee systems. Under Public Law 84-99 (PL 84-99), USACE has authority to supplement local efforts in the repair of flood control projects (e.g., levees) which are damaged by a flood. To be eligible for rehabilitation assistance under PL 84-99, projects constructed by non-federal interests shall meet certain criteria and standards set forth by USACE. The RIP requires non-federal sponsors to have an initial eligibility inspection as well as periodic inspections performed of their flood damage reduction system. As part of USACE's Civil Works Program, Section 408 provides that USACE may grant permission for another party to alter a Civil Works project upon a determination that the alteration proposed will not be injurious to the public interest and will not impair the function of the Civil Works project. This is applicable for situations where levees within the Levee Safety Program and PL 84-99 may require modernization in order to meet updated regulatory or technical standards.

7.1 Internal Drainage System

The internal drainage system for the leveed area shall include the network of channels, lakes, and storm sewers which drain the leveed area to the outfall facility. Refer to Chapter 3, Chapter 5, and Chapter 6 for construction requirements and design criteria.

7.2 Levee System

7.2.1 Frequency Criteria

The levee system shall include a levee embankment that will protect the development from the 100-year frequency flood event on the adjacent watercourse. Protection from the 100-year (1% Annual Exceedance Probability) frequency event shall account for water surface elevations and any associated wind and wave action, if applicable. For those areas located along the coast where protection by a levee system has been the chosen alternative, the design of the system shall provide protection from the hurricane storm surge. See Chapter 8 for more specific requirements.

7.2.2 Design Criteria

The following specific criteria and requirements shall apply to the design and construction of a levee:

1. A geotechnical investigation shall be required on the levee foundation (the existing natural ground). Soil borings shall be required with a maximum spacing of 1,000 feet and a minimum depth equal to twice the height of the levee embankment.
2. The foundation area shall be stripped for the full width of the levee. Stripping shall include removal of all grass, trees, and sub-surface root systems.
3. Embankment material shall be CH or CL as classified under the Unified Soil Classification System and shall have the following properties:
    a. Liquid Limit greater than or equal to 30.
    b. Plasticity Index greater than or equal to 15.
    c. Percent Passing No. 200 Sieve greater than or equal to 50.
4. A geotechnical investigation shall be required on the embankment material to determine the levee side slopes with respect to slope stability and methods employed to control subsurface seepage.
5. In addition to specific requirements identified in the geotechnical analyses, the embankment material shall be compacted to a minimum density of 95% using the standard proctor compaction test within 3% of the optimum moisture content. The embankment material shall be placed in lifts no more than 12 inches thick.
6. The levee top and side slopes shall be adequately protected by grass cover or other suitable material to protect against erosion. A minimum of 80% vegetative coverage shall be required at the time of construction inspection.

7. The minimum levee top width shall be 12 feet.
8. The levee side slope shall be one vertical to a minimum of three horizontal.
9. The minimum top of levee elevation shall be the 100-year water surface elevation on the adjacent watercourse plus the following freeboard requirements:
    a. A minimum freeboard of 3.0 feet above the water-surface level of the base flood shall be provided for riverine levees. The base flood elevation (BFE) is the 100-year or 1% annual exceedance probability (AEP) in the river at the location of the gravity outfall.
    b. An additional 1.0 foot of freeboard is required within 100 feet on either side of structures (such as bridges crossing over the levee) wherever the flow is constricted.
    c. An additional 1.0 foot of freeboard shall also be required at each end of the levee where it ties into high ground (see Item #10 below).
    d. Freeboard of less than 3.0 feet above the base flood elevation (BFE) will not be accepted under any circumstances.
10. The levee shall be continuous and shall either completely encompass the development or tie into natural ground located outside of the limits of the adjacent watercourse's 100-year flood plain.
11. All pipes and conduits passing through the levee shall have seepage control devices, flap gates, and slope protection. The design of these structures shall be compliant with FEMA's Technical Manual for Conduits through Embankments (FEMA 484).
12. The minimum right-of-way for the levee shall be from toe to toe. In addition, the establishment of an easement for maintenance and access, which may be located within the right-of-way, shall be required. Access shall be provided with either a minimum 15-foot easement adjacent to the levee, a minimum 15-foot levee top width or a minimum 15-foot horizontal berm on either side of the levee. A minimum 20-foot-wide easement shall be established in at least two locations to provide access to the levee right-of-way from an adjacent public road.
13. The levee shall be regularly inspected for any visual deficiencies. During flood events, continuous patrols of the levee shall be performed with special awareness for unusual wetness on the landward slope.
14. The levee system and appurtenant structures shall be maintained in such a way to ensure serviceability of the structures during normal and flood conditions.

A detailed discussion covering all aspects of the design and construction of levees can be found in the U.S. Army Corps of Engineers (USACE) Engineer Manual EM 1110-2-1913 (30 April 2000, or most current edition).

7.3 Pump Station

7.3.1 Frequency Criteria

To prevent flooding within leveed areas, pumps are required to remove interior drainage from volumetric storage locations when the exterior water level reaches a stage that prevents gravity outflow. This criterion presumes that the storm event causing high flood stages outside of the leveed area is the same storm event occurring over the leveed area. This applies to all areas of the county. The exterior stage shall be the 100-year water level. The design rainfall amounts to be used for sizing the required pump capacity will be those associated with the 100-year rainfall event. (See Table 2-5 and Table 2-6 for Atlas 14 rainfall amounts for Brazoria County).

7.3.2 Design Criteria

All leveed areas that are equipped with a pump station shall be capable of maintaining the design pumping capacity with its largest single pump inoperative. The pump station shall have a combination of storage volume pumping capacity adequate to maintain the runoff resulting from the 100-year frequency event below the maximum ponding level. The minimum capacity shall be determined such that within a 24-hour period, the pump station will be able to pump the total volume resulting from 1-1/2 inches of runoff from fully developed areas and 1 inch of runoff from undeveloped areas over the contributing watershed. All pump stations shall be equipped with auxiliary power for emergency usage. If a pump station is not provided, adequate storage volume below the maximum ponding level shall be provided to contain the entire design storm.

7.4 Gravity Outlet and Outfall Channel

An outlet shall be required to release the gravity flow from the leveed area through the outfall channel to the adjacent watercourse during low flow conditions on the receiving channel. The outlet shall be equipped with an automatically functioning gate to prevent any external flow from entering the leveed area.

The outlet and outfall channel shall be designed in accordance with Chapter 3. The velocities within the outfall channel at the adjacent watercourse shall not exceed 5 feet per second. For velocities larger than 5 feet per second, erosion protection is required as discussed in Chapter 3.

7.5 Review Process

When a levee system is required for development, the following information shall be submitted to the appropriate Drainage Regulatory Entity for review:

1. Preliminary Submittal
    a. A vicinity map showing the proposed levee location in relation to the 100-year flood plain and floodway of the adjacent watercourse
    b. The preliminary design of the levee cross-section based upon the geotechnical investigation
    c. The preliminary design of the pump station capacity
    d. Impact analysis
2. Final Submittal
    a. The final design of the levee cross-section and location
    b. The final design of the pump station capacity
    c. The hydraulic calculations showing that the maximum ponding elevation is not exceeded within the leveed area more than once in 100 years on average
    d. Digital models consistent with and supporting the submitted construction drawings
    e. The construction drawings and technical specifications for the levee and pump station along with final design computations for the levee, pump station and channels

8 Coastal Drainage Criteria

For the purposes of the Brazoria County drainage design manual, coastal areas are defined as regions impacted by the coastal 100-year storm surge as delineated by FEMA or an equivalent equal. Caution shall be used when evaluating specific situations where both tidal and riverine flooding may impact drainage.

If the Drainage Regulatory Entity determines further analysis or more stringent criteria is necessary when evaluating specific situations of coastal and riverine interaction, appropriate measures shall be taken to quantify and/or reduce impacts to drainage systems and properties.

In general, the design criteria presented in Chapters 1 – 7 applies to coastal areas with some exceptions. The guidance provided in Chapter 8 is unique to the coastal environment and details necessary changes to specific drainage design criteria described in Chapters 1 – 7.

Drainage design shall account for projected changes in Mean Sea Level (MSL). Sources such as NOAA, the United States Army Corps of Engineers (USACE), or the Intergovernmental Panel on Climate Change (IPCC) indicate an acceleration to the sea level rise rate over the next several decades.

The Design Engineer shall add the sea level rise projected to occur over the life of the project to the Mean Sea Level, Mean High-High Water (MHHW), or 2.33-year water surface elevation for analytical purposes. (Assume that the rate of increase for all tidal datums is equivalent to the rise for the Mean Sea Level.)

8.1 General

8.1.1 Outfalls

Because outfalls in coastal areas are subject to the effects of sea level rise and tides in general, the design shall assess the use of check valves which close during periods of high tide. Such assessment shall include the analysis of an interior drainage system that works with closed outfalls.

8.2 Hydrology

8.2.1 Design Storm Losses

The losses found in Section 2.2 and Section 2.3 can be used for coastal regions. However, this may be too conservative where soil types may be different than those inland due to increased sand content. Further investigation of appropriate loss parameters for the soil type in the project area may be required. The chosen loss parameters shall be documented by the Design Engineer and approved by the Drainage Regulatory Entity.

8.2.2 Flood Routing

Flood routing shall account for runoff storage reduction caused by high tides.

At a minimum, the drainage design shall apply the MHHW elevation as a hydraulic boundary condition. The applicant may use the National Oceanic and Atmospheric Administration (NOAA) National Water Level Observation Network to estimate a MHHW elevation.

For areas with critical infrastructure, the 2.33-year water surface elevation will be analyzed as the tailwater condition.

8.2.3 Rational Method

Runoff coefficient values lower than those shown in Table 2-1 may be needed where sandy soils typical of Brazoria County's coastal areas exist. Further investigation of appropriate runoff coefficients for the area may be required.

8.2.4 Hydrograph Technique for Small Watersheds

An increase in hydrologic losses in Table 2-7 may be warranted in areas where sandy soil conditions exist. Further investigation of appropriate losses for the soil type in the area may be required. Any adjustments to the hydrologic losses for the change in soil type shall be documented by the Design Engineer and approved by the Drainage Regulatory Entity before design.

The user may also adjust the design "time to peak" parameter in Equation 2-14 to account for the mild slopes that characterize coastal Brazoria County. The final time to peak parameter adjustments shall be documented by the Design Engineer and approved by the Drainage Regulatory Entity.

8.3 Open Channel Flow

8.3.1 Design Frequency

In areas where slab elevations are based on the FEMA 100-year storm surge water surface elevations, the Design Engineer is permitted to design and construct large channels that carry a more frequent design flow than the 100-year flow. This manual recommends a 25-year design storm as the alternative design flow rate.

8.3.2 Optimal Design Flow Conditions – Freeboard

In areas where slab elevations are based on the FEMA 100-year storm surge water surface elevations, a minimum freeboard for channels is not required.

8.4 Culverts and Bridges

8.4.1 Culvert Design Frequency

Culvert design frequency shall be consistent with the channel design frequency. See coastal criteria change for Section 3.2 Channel Design.

8.4.2 Bridge Design Considerations

In areas where slab elevations are based on the FEMA 100-year storm surge water surface elevations, the design engineer may utilize a more frequent design storm for the bridge analysis and design. The bridge design frequency shall be consistent with the channel design frequency. This manual recommends a 25-year design flow as the alternative design flow rate. The new design storm frequency shall be approved by the Drainage Regulatory Entity prior to the start of design.

8.4.3 Hydraulic Modeling

Any hydraulic modeling utilized in the design and analysis for bridges within the coastal region shall reflect the highly dynamic flow conditions in this region and include all two-dimensional flow patterns, wave action, and scour potential.

8.5 Storm Sewers and Overland Flow

8.5.1 Specific Design Flow Frequency

In areas where normal tidal conditions may create storm sewer hydraulic gradients which exceed the criteria outlined in Section 5.2.2, less stringent criteria may be used including a more frequent design storm event. If less stringent criteria are used, it shall be shown that vehicular access will not be significantly impaired during the 10-year storm event for both rainfall runoff and storm surge analysis. The new design storm frequency shall be approved by the Drainage Regulatory Entity prior to the start of design.

8.5.2 Extreme Event Analysis

In areas where slab elevations are based on the FEMA 100-year storm surge water surface elevations, criteria for storm sewer overflows in streets may be reduced. This is dependent upon the analysis showing that property damage will not result from the reduction in design. Slab elevations shall be designed consistently with all slab elevation criteria concerning internal 100-year drainage system design. Coordination with the Drainage Regulatory Entity prior to design is highly recommended.

9 Floodplain Development and Watershed Analysis

9.1 Development in the 100-Year Floodplain

The following paragraphs present information concerning the regulations for development in the 100-year floodplain, and the procedures and methods of evaluating the feasibility of flood plain development and obtaining approval for such a development. This chapter is intended to be complementary with the Drainage Regulatory Entity's Flood Damage Prevention Ordinance. In cases of discrepancy, the ordinance shall govern.

9.1.1 Floodplain Regulations

Development within the 100-year flood plain is regulated by the Floodplain Administrator having jurisdiction in the area. All areas within an incorporated municipality are subject to the regulation of that municipality. The Drainage Regulatory Entity's regulations place controls on the type and location of new construction within a designated 100-year floodplain and floodway. Following are the requirements for floodplain developments:

1. The lowest floor of any new construction in flood plain areas shall be 24 inches above the 100-year flood elevation, or 24 inches above the original natural ground, whichever is higher. "Original natural ground" is defined as the ground elevation before any fill was placed.
2. No fill or encroachment is permitted within the 100-year floodplain which will impair its ability to discharge the 100-year peak flow rate except where the effect on flood heights have been fully offset by stream improvements. All fill in the floodplain requires a 1-to-1 volumetric, excavated offset to compensate for any volume compromised in the floodplain.
3. Placement of fill material within the floodplain requires a permit from the Floodplain Administrator. Appropriate fill compaction data and hydrologic and hydraulic analysis data are required before a permit will be issued. Fill compaction documentation shall meet FEMA compaction requirements for fill in a floodplain.

9.1.2 Floodplain Development Requirements and Procedures

The following design requirements shall be followed when planning a development within the 100-year floodplain.

1. If developments impact the Brazos River floodplain, appropriate hydrologic and hydraulic models shall be obtained from Brazos River Authority (BRA) to model proposed conditions.
2. Land fill within the flood plain shall be minimized and the importation of fill material from outside of the flood plain is discouraged. When fill is used, conveyance improvements shall be provided to offset any lost volume below the 100-year floodplain water surface elevation.
3. Construction within the floodway is limited to structures which will not obstruct the 100-year flood flow unless fully-offsetting conveyance capacity is provided. Documentation of compliance through hydraulic modeling shall be required.
4. Any proposed development within the designated floodway is prohibited unless hydraulic models demonstrate that the proposed improvements will not increase water surface elevation.
5. A downstream impact analysis is required on all proposed development. The requirements of such an assessment are contained in Section 6.3.5 of this manual.

Specific procedures to be followed for analysis of development proposed within the floodplain are outlined below:

1. The FEMA effective 100-year floodplain and floodway shall be plotted on a map of the proposed development. If the development is in a FEMA Zone-A floodplain, the developer is required to provide hydrologic and hydraulic models to demonstrate no negative impacts from the proposed project.
2. Hydraulic profiles shall be developed for the proposed storm drainage system.
3. The effect of the proposed development and the encroachment into the floodplain area shall be incorporated into the hydraulic model. The hydraulic impact and the resulting floodplain must be determined and documented. All developments (including residential developments larger than 5000 square feet) within FEMA 100-yr Floodplain require to do hydraulic analysis.
4. The required channel improvements or other means of offsetting increases in floodplain elevations shall be incorporated into the hydraulic model. The resulting flood levels shall be determined to verify that the improvements offset the encroachment.
5. When an existing floodway is encroached, and once it has been determined that the proposed improvements adequately offset the encroachment, a revised floodway for the stream shall be computed and delineated.

6. All hydraulic model data shall be submitted with appropriate supporting information and computations to the Floodplain Administrator for review.
7. If required by the Floodplain Administrator, submittal to FEMA shall be required before construction permits will be issued.

9.1.3 Acceptable Alternative Courses of Action

To satisfy the Floodplain Administrator that no adverse impact will result, potential courses of action may be followed in the Downstream Impact Analysis:

1. Provide dedicated channel improvements or a storm drainage system through the area identified in the Downstream Impact Analysis. The improvements shall fully offset the increased flow rates caused by the proposed development, or;
2. A detention basin or other acceptable detention system may be designed to eliminate any increase in peak flow rates to the receiving stream.

9.1.4 Flood Routing Studies

Regarding routing studies to evaluate the impact on the downstream critical reaches, the following general requirements shall be followed:

1. Rainfall distribution over the watershed shall be in accordance with this manual. However, the Floodplain Administrator may require additional analyses under different rainfall assumptions if the Floodplain Administrator feels such analyses are warranted.
2. Channel improvements planned to be completed within a two-year period may be considered in the routing procedures.

10 Drainage Design Criteria for Rural Subdivisions

10.1 Purpose

The purpose of this design criteria is to offer an alternative drainage procedure that can be used when designing detention facilities for rural-type subdivisions.

Typically, rural developments consist of large acre lots with minimal drainage improvements. Little change to the natural storm runoff occurs as a result of rural subdivisions being developed. In recognition of this, this criterion has been developed to reduce the amount of on-site detention otherwise required by the criteria manual.

This is the minimal criterion for acceptance by the Drainage Regulatory Entity. Individual circumstances may warrant an enhanced drainage and/or detention system.

10.2 Qualifications

The following qualifications are established and shall be met in order to be considered a rural subdivision and utilize this alternative design criteria:

A. The subdivision shall have a minimum lot size of 1/2 acre;

B. The percent impervious shall not exceed the maximum allowable percent impervious cover based upon lot size (see Figure 10-1 Detention Storage Requirements for Rural Subdivision and Table 2-1);

C. A roadside ditch drainage system shall be utilized as opposed to curb and gutter drainage; and

D. Development shall not provide any major drainage improvements that would significantly alter the natural drainage pattern in the area for large flood events.

10.3 Design Criteria

The following design criteria shall be utilized for rural subdivisions:

A. Minimum slab elevations shall be the maximum of

   a. two (2) feet above finished grade; or

   b. two (2) feet above the 100-year floodplains; or

   c. one (1) foot above the crown of any downgradient roadway

B. Roadways

1. R.O.W. shall be a minimum of 60 feet wide.
2. Crown shall be a maximum of one (1) foot above natural ground.
3. Roadside drainage system - open ditch with maximum 4:1 side slope; equalizer pipes under roadway at least every 1000 feet (minimum of 24-inch diameter reinforced concrete pipe) if roadway blocks natural drainage path.

C. Lot drainage

a. Swales may be constructed along lot lines to provide for minimal drainage of lots. Other than lot line swales and building pads, lots shall not be significantly graded.

b. A minimum 7-foot drainage easement is required on all side and rear lines. Drainage Easements can be coincident with other side and rear-lot easements. Each lot shall drain to the street in front of that lot.

D. Detention Requirements

a. See Section 6.3.6 for amount of on-site detention required.

b. Discharge pipe shall be a maximum 24-inch diameter RCP, or equivalent and shall be sized for 2-year, 25-year, 100-year storm events.

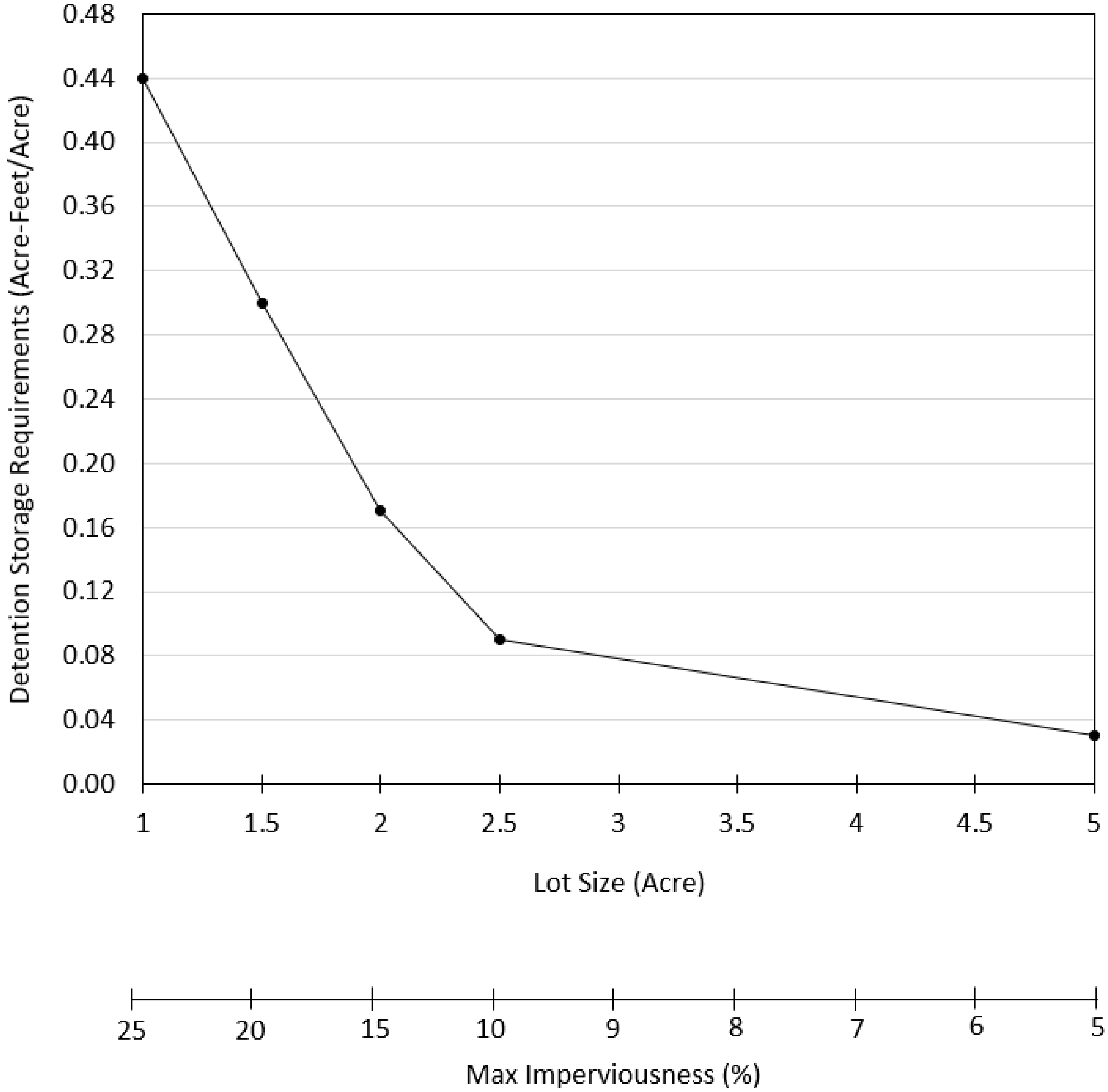


Figure 10-1 Detention Storage Requirements for Rural Subdivision

10.4 Submittals

A. Drainage area map showing existing drainage ways on or adjacent to property.

B. Map(s)/drawing(s) showing existing drainage patterns before development and proposed drainage patterns after development for both small storm events and large storm events.

C. Preliminary (and eventually final) plat with the following plat notes:

1. The maximum allowable impervious cover varies with lot size. Obtain the maximum percent imperviousness from Equation (2-1) for the corresponding average lot size shown on the plat.
2. The drainage and/or detention system has been designed with the assumption that this maximum percent imperviousness will not be exceeded. If this percentage is to be exceeded, a replat and/or redesign of the system may be necessary.
3. The minimum slab elevation shall be 2 feet above the 100-year flood plain elevation, or 2 feet above natural ground, whichever is higher.
4. This rural subdivision employs a natural drainage system, which is intended to provide drainage for the subdivisions that mirrors pre-development conditions.

11 Definitions

Abutment
A structure that supports the lateral load of an arch or span.

Annual Exceedance Probability (AEP)
The probability of exceedance in a given year.

Attenuation
The reduction of the peak of a hydrograph, causing the shape to become flat and wide.

Backwater
Water that is backed up or slowed compared to the average, natural flow. This phenomenon can be caused by temporary obstructions or an opposing current.

Backslope Drainage
A small swale system running parallel to a grass-lined channel that is used to receive and convey overland sheet flow to prevent channel bank erosion.

Bank
The defined sides of a channel, designated left and right banks looking downstream in the direction of flow.

Bankfull
The elevation at which the water level stage just begins to overfill the confines of the hydraulic structure (channel, detention basin, roadside ditch, etc.), and flows into the floodplain.

Basin Development Factor (BDF)
A hydrologic methodology measuring the extent of development of the drainage system in a watershed. A value of BDF = 0 represents a natural drainage condition and a value of BDF = 12 represents the maximum level of drainage capacity defined in terms of channel improvements, channel linings, storm sewers, and curb-and-gutter streets.

Berm
A flat, raised land surface bordering a body of water. Also termed embankment.

Boundary Condition
The defined parameters for a problem that govern what the solution can be.

Calibration
The determination and subsequent alterations that account for the differences between true values and values being supplied by models or other instrumentation.

Channel
A natural or manmade open system that conveys water, either during storm events or at all times. Rivers, streams, creeks, and tributaries are examples of natural channels while canals, ditches, and floodways are examples of manmade channels.

Coastal Areas
Regions impacted by the coastal 100-year storm surge as delineated by FEMA or an equivalent equal.

Conduit
An enclosed pipe or box, usually concrete, used to convey stormwater underground from one location to another.

Confluence
The intersection and convergence of two channels.

Contraction Scour
Scour that occurs when water accelerates due to the constricting of the flow area, where the downstream flow area is narrower than the upstream flow area. The increase in velocity can cause more erosion of the sediment lining the channel than when a channel is not constricted.

Dedication
A portion of a property set aside by the property owner to be used by the public. Dedications create public drainage easements that can be used to maintain channels and detention facilities.

Depression Storage
Water contained in natural low points in the land surface.

Design Storms
A defined hyetograph and total precipitation that represent the estimated runoff for a given hypothetical storm specified by the Drainage Regulatory Entity.

Detention Basin
A man-made storage basin that drains by gravity or pump during a runoff event. It temporarily detains the runoff to reduce peak downstream discharge.

Development
The improvement or subdivision of a tract of land exclusive of land being used for agricultural purposes. Improvement of land includes grading, paving, building of structures, or otherwise changing the runoff characteristics of the land.

Dike
Walls or embankments preventing overtopping.

Discharge
The volume of water that passes through a cross-sectional area in a specified unit of time, usually measured in cubic feet per second or gallons per minute. Also termed as flow.

Diversion
The interception and redirection of the flow of water from one channel to a channel in a different watershed.

Drainage Divide
The elevated surface that divides neighboring watersheds.

Drainage Regulatory Entity
The entity that has adopted and oversees enforcing these regulations described in this document (e.g., Brazoria County, drainage district, City, etc.).

Easement
A legally designated area of private property reserved for specific public use.

Eddy
The circular current of water caused by a structural obstruction in the flow path.

Erosion
A change in geometric configuration caused by the loss of existing soil.

Evapotranspiration
The loss of water due to evaporation from soil and water surfaces and the transpiration from plants.

Fetch
The distance of open water that can generate waves due to wind forces on the surface of the water.

Flood Fringe
The area outside of the floodway that is still within the floodplain.

Flood Wave
The distinct increase in water surface elevation followed by a drop.

Floodplain
The area outside the banks of a channel where floodwaters flow when they exceed the banks or capacity of the channel. Normally the floodplain is immediately adjacent to the channel but may extend laterally for a significant distance.

Floodway
The channel of a natural or manmade conveyance pathway and the adjacent land areas that are used to discharge flow from the main conveyance pathway in the event of a flood.

Freeboard
The vertical distance between the top edge of a hydraulic structure and the water surface it is containing.

Gabion
A wire cage binding together rocks, concrete, cement, soil, or other material used to stabilize water conveyance channels and shorelines.

Headwater
The water surface elevation immediately upstream of a hydraulic structure.

Hydraulic
Relating to the physical behavior or properties of runoff from a given rain event.

Hydraulic Grade Line
A line representative of the flow energy, it is a graphical representation of the water surface elevation at any point of an open channel.

Hydrologic
Relating to the quantity of runoff produced from a given rainfall event.

Hydrograph
Graphical representation of rate of flow over a period of time.

Hydrology
The study of the interaction of water with the topography

Hyetograph
Graphical representation of rainfall intensity over a period of time.

Impervious Cover
Surfaces that do not absorb rainfall. Also termed impervious area or impervious surface.

Infiltration
The permeation of water into the soil under the ground's surface. The rate of infiltration decreases as the soil becomes more saturated.

Intensity
The amount of rainfall experienced over a given time period. Usually expressed in inches per hour.

Interception
Precipitation captured by buildings, leaves, etc., before it reaches the land surface.

Inundation
The condition of being flooded.

Loss Rate
The rate at which a portion of rainfall is “lost” due to interception, depression storage, infiltration, and evaporation.

Meander
The gentle curving and winding of a channel.

Outfall
Downstream end of a pipe discharging into a channel or roadside ditch from a storm sewer system or a detention or retention basin.

Outlet Structure
Structure usually composed of pipes, weirs, spillways, and/or pumps designed to drain a detention or retention basin.

Overbank
A geological deposit of silt or other sediment on a floodplain caused by the overtopping of water.

Overland Flow
The flow of runoff over the land surface, not through a channel or other designated conveyance system.

Parameters
A numerical representation of characteristics of modeled events and locations.

Peak Flow
Rate of flow at the highest point of a hydrograph. Also termed as peak discharge.

Peak Runoff
The maximum runoff capacity for design of a hydraulic structure meant to carry or detain the runoff.

Ponding
The volume of rainfall runoff that is unable to move downstream by gravity.

Rainfall Intensity
The rainfall total divided by a given time interval, usually measured in inches per hour.

Rainfall Loss Rate
The portion of the total amount of rainfall that is included in a hydrologic runoff calculation over a given period of time.

Recurrence Interval
The reciprocal of AEP. A return period that marks the statistical average interval of time between the given flood occurrences, often denoted in a percentage.

Reservoir
A natural or manmade area for the storage and regulation of water. Ponds, lakes, and basins are types of reservoir.

Retention Basin
Similar to a detention basin except the runoff is held for an indeterminate amount of time, likely until after the rainfall event has receded.

Riprap
Rock, loose stone, or other material used to prevent erosion of shorelines, stream beds, and other channels.

Roughness Coefficient
A dimensionless value used in hydraulic calculation to approximate the impact of different types of physical characteristics within a channel or floodplain.

Routing
The alteration of the shape and timing of a runoff hydrograph as it moves downstream through a drainage system.

Runoff
Excess rainfall which runs off the land and which is defined as the rainfall minus the losses. This is the portion of a rainfall event which hydraulic structures are designed to contain.

Runoff Coefficient
A constant used to describe the expected amount of runoff produced from a given rainfall event.

Scour
Erosion near the base of structures caused by the fast movement of water.

Side Slopes
The angle of the side of a channel. Expressed in the change in horizontal dimension over the change in vertical dimension.

Slope Paving
Smooth concrete placed inside a drainage channel to prevent erosion.

Stage
The height of water surface elevation relative to a declared datum.

Storm Surge
An increase in water surface elevation due to changes in atmospheric pressure and wind caused by a tropical storm.

Swale
Natural or manmade shallow channel with gradual side slopes.

Tailwater
The water surface elevation immediately downstream from a hydraulic structure.

Time of Concentration
The travel time of a single particle of water from the farthest point of the watershed to the point of interest.

Unit Hydrograph
The base level for defining a hydrograph from a given watershed. It is a graphical representation of the surface runoff due to one inch of rainfall excess applied uniformly over the watershed in a specified time interval.

Unsteady Flow
Change in a flow hydrograph over time through a creek or channel.

Watershed
A defined area where all overland flow runoff is conveyed to the same outlet. Similar terms include basin, drainage basin, or drainage area.

Wave Height
The vertical distance between the trough and crest of the wave.

Weep Holes
Small openings in the structural siding to allow for water to drain from within the structure.