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2.0 HYDROLOGY

2.1 GENERAL

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

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

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

Because of its versatility and accuracy, the widely used computer program HEC-HMS version 4.3 (or newer) is recommended as the primary tool for modeling storm runoff hydrographs in Fort Bend County for newly developed models. Versions of HEC-HMS must be consistent throughout each project. Accordingly, the hydrologic design techniques described in this manual incorporate many of the routines contained in HEC-HMS. The recommended hydrologic modeling for the county as presented in this section is based on the Clark Unit Hydrograph technique with adequate design storms and rainfall loss rates. To derive the parameters used to compute the Clark Unit Hydrograph, Fort Bend 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 U.S. Geological Survey Water-Supply Paper 2207 in 1983. Sections 2.4.3 and 2.4.4 offer instructions and an example of the application of BDF to the development of Clark's TC and R. The HEC-HMS models obtained from the Fort Bend County Drainage District (in the watersheds where they are available) must be used in updating the individual watershed drainage studies.

For areas less than 200 acres, the widely used Rational Method provides a useful means of determining peak discharges. In situations requiring determination of a complete flood hydrograph, and not just a peak discharge, the Malcom Small Watershed Method can be utilized. However, the timing of the Malcom hydrograph is not related to any temporal distribution of a frequency storm. As an alternative when consistency with the design frequency storms of receiving channels is desired, the engineer may determine the time of concentration (TC) and the peak discharge (Qp) with the Rational Method, and then use HEC-HMS with the frequency storm of interest to analyze the catchment by using TC and iterating Clark's Storage coefficient (R) to produce Qp. This will produce a flow hydrograph based on the data determined with the Rational Method.

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

HEC-HMS modeling with the BDF parameters for the watershed is required for new development or drainage areas greater than 100 acres to ensure the development causes no adverse impact to a watershed for events including the 100-year, 25-year and 10-year rainfall events. Additional analysis for the 500-year rainfall event may be necessary in certain special situations (such as defined overflows, watershed rerouting, inline detention, or other special situations) and must be included in the analysis if requested by the Fort Bend County Drainage District Chief Engineer. Section 6 and Section 8 of this Manual define certain conditions under which a development may provide detention without a HEC-HMS analysis.

If the engineer desires to use another method, it is recommended that the Fort Bend County Drainage District Engineer be consulted prior to design.

2.2 RATIONAL METHOD

The Rational Method is acceptable for determining peak storm runoff rates for small watersheds that have a drainage system unaffected by complex hydrologic situations such as ponding areas, storage basins and watershed transfers (overflows) of storm runoff. This widely used method provides satisfactory results if understood and applied correctly. It is generally recommended that in Fort Bend County the Rational Method be used only for areas smaller than 200 acres. The method is to be used to calculate peak runoff rates and/or storm sewer design flow in developments less than 100 acres, while its application in sites between 100 and 200 acres is limited to calculating peak discharges for the design of storm sewers.

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

$$Q = C\ i\ A \qquad (2\text{-}1)$$

Where

Q — is the peak rate of runoff in cubic feet per second. From the formula, Q results in units of acre-inch per hour. However, the conversion factor of ac-in/hr to cfs is 1.0 and Q is therefore typically identified in units of cfs;

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

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

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

Basic assumptions associated with the Rational Method are:

1. The computed peak rate of runoff at the design point is a function of the average rainfall intensity 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 time of concentration (duration).

3. The time of concentration is the critical storm duration and is discussed under paragraph 2.2.2 of this manual.

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

5. Rainfall intensity is uniform during the storm duration.

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

2.2.1 <u>Runoff Coefficient (C)</u>

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

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

Coefficients for specific surface types can be used to develop a composite runoff coefficient based in part on the percentage of different types of surfaces in the drainage area. This procedure is often applied to typical “sample” blocks as a guide to selection of reasonable values of the coefficient for an entire area. Table 2-1 presents recommended values for the runoff coefficient “C” for various residential districts and specific surface types. These values were derived from numerous sources (see References 9, 21, 33, and 34) and have been correlated with the updated impervious cover values.

2.2.2 Rainfall Intensity (i)

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

The time of concentration used in the rational equation is the critical storm duration for the point of interest. The critical duration is the time associated with the peak runoff from all or part of the upstream drainage area to the point of interest. The assumption for small catchments represented by the Rational Formula is that runoff from a watershed typically reaches a peak at about the time when the entire drainage area is contributing; in which case, the time of concentration is the time for water to flow from the most remote point in the watershed to the point of interest. However, the runoff rate may reach a peak prior to the time the entire upstream drainage area is contributing. In this instance, only the portions of the drainage area able to contribute flow to the point of interest during the critical time of concentration should be used in determining the peak discharge. A trial and error procedure can be used to determine the critical time of concentration. This is done by varying the time of concentration to maximize the peak discharge associated with the given area and a given design frequency.

The time of concentration depends on the slope of the flow paths as runoff moves through the watershed, the roughness of the surface, retardation 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.

$$T_c = T_{of} + T_{ch} \tag{2-2}$$

Where

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

The time of concentration for the overland flow portion, T_{of}, can be approximated by the Kerby equation:

$$T_{of} = \frac{K\,(L \times N)^{0.467}}{S^{0.235}} \tag{2-3}$$

Where

- K is a unit conversion factor equal to 0.828 for customary U.S. units;
- L is the overland flow length in feet;
- S is the slope of the travel path in feet/feet;
- N is the retardation coefficient depending on the type of surface and obtained from Table 2-1.

The time of concentration for the channel or storm sewer portion is to be calculated based on the velocity obtained with Manning's equation. A template to be used in the time of concentration calculations is included in Table 2-2, which must be included in plans and drainage reports submitted to Fort Bend County for review.

$$V = \frac{1.49}{n} R^{0.667} S^{0.5} \tag{2-4}$$

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

Where

V is the velocity in the channel in feet per second;

n is the Manning's roughness coefficient of the channel and obtained from Table 3-1

R is the hydraulic radius in feet calculated as the area of the cross section of the channel divided by the wetted perimeter, both assuming full bank or full conduit conditions;

S is the slope of the channel in feet/feet;

Lch is the length of the channel in feet;

Tch is the travel time in the channel or storm sewer in minutes.

For the purpose of calculating peak flow in storm sewers in urbanized watersheds only, the time of concentration at the point where the contributing area increases can be approximated by the following equation:

$$Tc = 10\ A^{0.1761} + 15 \tag{2-6}$$

where

A is the drainage area in acres at the point the flow is being calculated.

Table 2-3 shows rainfall depth for various frequencies and durations and is useful for determining intensities when calculating runoff using the Rational Method. However, a direct method for determining intensity data is through an equation that is based on the time of concentration. The following "e-b-d" equation is provided for this method.

$$i = \frac{b}{(Tc+d)^e} \tag{2-7}$$

Where

- i is the rainfall intensity in inches per hour for 2-year, 3-year, 5-year, 10-year, 25-year, 50-year, 100-year, or 500-year event in inches per hour;
- Tc is the time of concentration in minutes;
- e, b, d are coefficients based on return interval and obtained from Table 2-4. These coefficients are valid when the time of concentration is shorter than 3.0 hours.

The relationship between the rainfall intensity and storm duration for the 2-year up to the 500-year frequency storms is shown in Figure 2-2. Table 2-3 presents the rainfall amounts for a variety of durations and frequencies with the corresponding e-b-d values summarized in Table 2-4. Rainfall intensities for various frequencies and durations can be determined based on Equation 2-7. For example, if a 25-year, 15-minute intensity is required, utilize Tc = 15 minutes along with the e-b-d values for the 25-year recurrence interval from Table 2-4 (e = 0.6222, b = 51.65, d = 5.09) to yield the intensity of 7.99 inches per hour.

2.2.3 Drainage Area (A)

The size and shape of the drainage area must be determined. The area may be determined through the use of topographic maps (2014 or newer LiDAR), supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. A drainage area map shall be provided for each project. The drainage area contributing to the system being designed and drainage subarea contributing to each inlet point shall be identified. The outlines of the drainage divides must follow actual topography rather than the artificial land divisions as used in the design of sanitary sewers. The drainage divide lines are determined by the pavement slopes, locations of downspouts, paved and unpaved yards, grading of lawns and many other features that are introduced by the urbanization process.

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

2.3 HYDROGRAPH DEVELOPMENT FOR SMALL WATERSHEDS

Runoff hydrographs based on the peak flow obtained with the rational method may need to be developed for detention design in sites between 50 and 100 acres. A technique for hydrograph development which is useful in small watersheds has been presented by H.R. Malcom. This method may be used in the general evaluation of storm water detention facility function, however all storm water detention facilities must be designed to meet the requirements presented in Fort Bend County Drainage Criteria Manual, Chapter 6.

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

Malcom's Method consists of the following equations:

$$T_p = \frac{V}{1.39\,Q_p} \qquad (2\text{-}8)$$

$$q_i = \frac{Q_p}{2}\left[1 - cos\left(\frac{\pi t_i}{T_p}\right)\right] \quad \text{for } t_i \le 1.25\,T_p \qquad (2\text{-}9)$$

$$q_i = 4.34 Q_p e^{\left(-1.30\frac{t_i}{T_p}\right)} \quad \text{for } t_i > 1.25\,T_p \qquad (2\text{-}10)$$

*Calculator must be in radian mode.

Where

Q_p is the peak design flow rate in cubic feet per second;

T_p is the time to Q_p in seconds;

V is the total volume of runoff for the design storm in cubic feet, which is calculated as the depth of excess rainfall from Table 2-5 multiplied by the drainage area with proper conversion units;

t_i and q_i are the respective time and flow rates which determine the shape of the hydrograph.

A plot of a hydrograph illustrating these parameters is included as Figure 2-3.

The peak design flow rate can be calculated directly with the Rational Method if the area is smaller than 100 acres. The total volume of runoff is dependent on the level of development of the area (i.e. percent of impervious cover). Typical loss rate totals for the 10-, 25- and 100-year, 24-hour rainfall events are included in Table 2-5.

2.4 RAINFALL-RUNOFF COMPUTATIONS USING HEC-HMS

A stream network model which simulates the runoff response of a river basin to rainfall over that basin can be developed utilizing the HEC-HMS computer program (version 4.3 or later) by the appropriate combination of hydrograph and routing computations. The following sections describe the elements required to develop a HEC-HMS computer model.

2.4.1 Design Storm Rainfall

Design storm rainfall can be described in terms of frequency, duration, areal extent and distribution of intensity with time. A design storm's rainfall distribution in time should be handled in the HEC-HMS by offsetting the intensity position of the hyetograph by 67% in HEC-HMS if either the watershed is shared with Harris County or there is no existing model for the watershed. If there is an existing model for the watershed with the intensity position at 50%, then for consistency, the updated model for the watershed should have the intensity position at 50%. However, the engineer is encouraged to update models to incorporate intensity positions to 67% where practicable. The engineer's choice for duration is dependent upon the physical characteristics, location and study objectives. In most cases, design will be based on a 24-hour duration storm event. Design storms used in HEC-HMS will be point rainfall amounts and not subject to depth-area-reduction. It is often necessary to increment design rainfall hyetographs in five-minute intervals to meet the design needs of small drainage areas having short times of concentration.

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" with updated, detailed, 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 TP-40 and Technical Memorandum NWS Hydro-35. Data from TP-40 and NWS Hydro-35 is superseded by Atlas 14 and no longer should be used for new development.

Using NOAA Atlas 14 data, Table 2-3 was developed. This table gives depth vs. duration data for a variety of storm frequencies, from the 2-year to the 500-year storm. The data in Table 2-3 is Partial Duration Series data (as opposed to Annual Maximum Series data). Partial Duration Series is preferred for use in design of smaller drainage systems (such as storm sewers and roadside ditches), while Annual Maximum Series is preferable for larger drainage systems (such as channels and regional detention facilities). However, since the difference between Partial Duration and Annual Maximum rainfall values is negligible for storms greater than a 10-year event, Partial Duration Series can be used for all storms analyzed in Fort Bend County. To use Partial Duration Series in HEC-HMS, the "Annual-Partial Conversion" should be set to "--None--" in HEC-HMS Version 4.3 (in earlier versions, "Input Type" and "Output Type" should both be set to "Partial Duration").

2.4.2 Design Storm Losses

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

The Green-Ampt loss method is one of several loss methods included in HEC-HMS. The Green-Ampt loss method, using the Simple Canopy method, is recommended for calculation of abstractions in Fort Bend County. This method assumes that the soil is initially at a uniform moisture content and infiltration occurs with piston displacement.

Following are the description of the parameters of the Green-Ampt loss method:

1. **Initial Canopy Storage:** the percentage of the canopy that is full of water at the beginning of the simulation.
2. **Max Canopy Storage:** the maximum amount of water that can be stored in the canopy, held on leaves before through-fall to the surface begins. It is expressed as an effective depth of water in inches.
3. **Crop Coefficient:** a ratio applied to the potential evapotranspiration when computing the amount of water to extract from the soil.

4. **Initial Content:** the initial saturation of the soil at the beginning of the simulation, specified in terms of a volume ratio. This parameter is heavily dependent on the antecedent moisture in the soil.
5. **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.
6. **Suction or Wetting Front Suction:** This parameter is a function of the soil texture and is expressed in inches.
7. **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 group.
8. **Impervious %:** the percentage of the sub-basin which is directly connected impervious area can be specified. No loss calculations are carried out on the impervious area; all precipitation on that portion of the sub-basin becomes excess precipitation and subject to direct runoff.

Green-Ampt parameter values within Fort Bend County are in the process of being defined and will be provided in a subsequent version of this document. All other sections of this document remain unaffected. Table 2-6 is a placeholder for future values.

For the portions of the Willow Fork, Barker Reservoir, Long Point Creek, Brays Bayou, Keegans Bayou, Sims Bayou, and Clear Creek watersheds that are in Fort Bend County and flow into Harris County, Harris County loss rates should be used, and the Engineer should refer to the most recent version of the HCFCD H&H Guidelines Manual for current information.

Typical values for the percentage of impervious cover corresponding to various types of development in Fort Bend County are given in Table 2-1. These values should be used only when the general type of planned development is known; once the actual level of development has been determined for a specific area, a refined value should be used to reflect the actual percent of impervious cover.

2.4.3 Design Storm Runoff

2.4.3.1 Determining Base TC and R Values

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

To determine appropriate TC and R values for watersheds in Fort Bend County, the Basin Development Factor (BDF) method should be used. The BDF is a measure of the extent of development of the drainage system within a basin, and ranges from a value of 0 for a completely undeveloped basin to 12 for a basin with fully improved drainage. BDF reflects improvements in the drainage system itself and does not account directly for impervious cover. The following factors are considered when determining BDF:

- Natural Channel (N): main drainage channel and principal tributaries that remain in a natural state. This may also apply to channels which were once modified, but have not been maintained and are taking on characteristics of a natural channel.
- Channel Improvements (I): straightening, enlarging, deepening, and clearing of the main drainage channel and principal tributaries.
- Channel Linings (C): lining of the main drainage channel and principal tributaries with an impervious material such as concrete.
- Undeveloped (U): areas that are primarily undeveloped. This would include wooded areas, rangeland, and agricultural areas that are undeveloped and typically not graded to drain efficiently.
- Open Space (OS): space that is graded to drain efficiently, but not otherwise improved. Golf courses, city parks, and regional detention basins would be considered open space. Detention basins within residential or commercial development should be considered part of that development's drainage characterization (impervious cover is calculated separately).
- Roadside Ditches (R): areas that are served by roadside ditches. These tend to be industrial or large lot residential.

- Storm Sewers pre-1987 ($SS_{pre\text{-}1987}$): enclosing drainage ditches and swales in storm sewers to improve efficiency. Pre-1987 developments feature smaller and more widely-spaced inlets.
- Storm Sewers post-1987 ($SS_{post\text{-}1987}$): enclosing drainage ditches and swales in storm sewers to greatly improve efficiency. Post-1987 developments feature large inlets that are much more closely spaced than pre-1987 developments.

To simplify the quantification of these factors, the Fort Bend 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 total channel length, and the ratio of improved (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.

$$BDF = \frac{(I\times3)+(C\times6)}{N+I+C} + \frac{(OS\times1)+(R\times1.5)+\left(SS_{pre-1987}\times3\right)+\left(SS_{post-1987}\times6\right)}{U+OS+R+SS_{pre-1987}+SS_{post-1987}} \tag{2-11}$$

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 %)

$SS_{pre\text{-}1987}$ = pre-1987 developed area served by storm sewer (ac or %)

$SS_{post\text{-}1987}$ = post-1987 developed area served by storm sewer (ac or %)

The result of Equation 2-11 is then used in the following equations to determine base TC and R values:

$$T_r = 10^{[(-0.05228\times BDF)+0.4028\log_{10}A+0.3926]} \tag{2-12}$$

$$TC_{BDF} = T_r + \frac{\sqrt{A}}{2} \tag{2-13}$$

$$R_{BDF} = 8.271e^{-0.1167\times BDF} \times A^{0.3856} \tag{2-14}$$

Where

BDF is the previously calculated Basin Development Factor (0 to 12)

A is the drainage area to point of interest in square miles;

T_r is the lag time in hours;

TC_{BDF} is the time of concentration based on BDF in hours;

R_{BDF} is the Clark storage coefficient or residence time based on BDF in hours.

2.4.3.2 Slope Adjustments

The values of the time of concentration and storage coefficient can vary significantly depending on watershed slopes. To account for these differences, 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 should average the slopes of several representative perpendicular slopes or can divide the watershed up into large grid cells (typical grid cells of 10 acres or 660 feet by 660 feet seem to work well) 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.

$$K_S = -0.162 \ln(S \times S_o) + 1.5232 \qquad (2\text{-}15)$$

Where

S is the channel slope in ft/mile measured along the entire watercourse, excluding drops in flowline such as control structures;

S_o is the overland slope in ft/mile, average of multiple representative “perpendicular” slopes;

$S \times S_o$ is the maximum of 26 and the product of both numbers (i.e., if $S \times S_o \leq 26$, use 26);

K_S is slope factor to be applied to time of concentration and Clark storage coefficient, which should be less than 1.0.

2.4.3.3 Detention Adjustments

The base TC and R values must 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-builts or 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 must 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. Also, large storage areas such as sand or gravel pits should 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.

$$C_f = 3 \times 10^{-5} \times DR^2 - 0.00095 \times DR + 1 \tag{2-16}$$

Where

DR is the detention rate for watershed or subwatershed in acre-feet per square mile;

C_f is the correction factor for detention (for DR $\geq$ 10) to be applied to both TC and R

2.4.3.4 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 must 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 much 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.

$$RM_2 = 1.33 \times DPP^{0.242} \quad (2\text{-}17)$$

$$RM_5 = 1.31 \times DPP^{0.214} \quad (2\text{-}18)$$

$$RM_{10} = 1.28 \times DPP^{0.199} \quad (2\text{-}19)$$

$$RM_{25} = 1.25 \times DPP^{0.171} \quad (2\text{-}20)$$

$$RM_{50} = 1.23 \times DPP^{0.153} \quad (2\text{-}21)$$

$$RM_{100} = 1.21 \times DPP^{0.132} \quad (2\text{-}22)$$

$$RM_{200} = 1.19 \times DPP^{0.113} \quad (2\text{-}23)$$

$$RM_{500} = 1.17 \times DPP^{0.086} \quad (2\text{-}24)$$

Where

DPP is the percentage of the watershed affected by ponding (between 0 to 100);

RMx is the adjustment factor for the x-year frequency storm.

2.4.3.5 Final TC and R Values

To determine the final TC and R values to be used in HMS, the following two equations may be applied, using the T_r value from Equation 2-12 above:

$$TC = K_S \times C_f \times \left(T_r + \frac{\sqrt{A}}{2}\right) \quad (2\text{-}25)$$

$$R = K_S \times C_f \times RM_n \times (8.271e^{-0.1167 \times BDF} \times A^{0.3856}) \quad (2\text{-}26)$$

Where

K_S is the slope factor to be applied to both time of concentration and Clark storage coefficient;

C_f is the correction factor for detention to be applied to both TC and R for DR $\geq$ 10;

T_r is the lag time in hours;

A is the watershed area to point of interest in square miles;

RM_n is the ponding factor applied to Clark storage coefficient (n=return period) for DPP $\geq$ 20%;

TC is the adjusted time of concentration in hours;

R is the adjusted Clark storage coefficient or residence time in hours.

These parameters may then be input into the HEC-HMS program to model the runoff process. Input of the time-area curve is handled internally by HEC-HMS unless the engineer specifies a particular time-area relationship. An example of the step-by-step procedure for the development of a design runoff hydrograph is presented in Section 2.4.4. A spreadsheet for calculating TC and R by the above method is available from the Fort Bend County Drainage District.

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

2.4.4 Procedure for Developing a Design Runoff Hydrograph

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

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

   Example: For this example, a discharge hydrograph for the 100-year, 24-hour storm event is needed to determine the design flow of a major channel in Fort Bend County *within the Clear Creek watershed (drains into Harris County). The drainage area is 691 acres or 1.08 square miles.*

2. Develop the design storm hyetograph. This process can be carried out internally by HEC-HMS as discussed in Section 2.4.1. It is required that depth-duration data as presented in Table 2-3 be input into HEC-HMS.

   <u>Example</u>: *Table 2-3 was used to assign the appropriate depth of rainfall for each of the various durations as follows, which are then input into the HEC-HMS frequency storm table.*

100-Year Frequency Design Storm	
Duration	**Depth (in)**
5 min	1.26
15 min	2.50
60 min	4.80
2 hours	6.91
3 hours	8.47
6 hours	11.20
12 hours	13.80
24 hours	16.50

3. Determine losses. This procedure is carried out internally by HEC-HMS, but it is required that the values of the variables presented in Section 2.4.2 of this manual be input in HEC-HMS.

   <u>Example</u>: *For the Clear Creek watershed, the following values should be input:*

Variable	**Value**
Initial Canopy Storage	0%
Max Canopy Storage (in)	0.1
Crop Coefficient	1.0

Initial Content	0.075
Saturated Content	0.46
Suction (in)	12.45
Conductivity (in/hr)	0.024
Percent Impervious	15%

4. Determine the Basin Development Factor for the watershed, as described in Section 2.4.3:

   <u>Example</u>: The BDF parameters of the watershed, which is also illustrated in Figure 2-5, 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
   $SS_{pre\text{-}1987}$ (pre-1987 storm sewer development) = 33 ac
   $SS_{post\text{-}1987}$ (post-1987 storm sewer development) = 160 ac

$$\mathrm{BDF} = \frac{(7988 \times 3) + (0 \times 6)}{1553 + 7988 + 0} + \frac{(377 \times 1) + (43 \times 1.5) + (33 \times 3) + (160 \times 6)}{78 + 377 + 43 + 33 + 160} = 4.68$$

5. Determine base TC and R values using BDF and input in HEC-HMS. The Clark parameters are determined by solution of Equations 2-12, 2-13, and 2-14.

   <u>Example</u>: The TC and R calculations for this example watershed are:

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

$$\mathrm{TC_{BDF}} = 1.45 + \frac{\sqrt{1.08}}{2} = 1.97 \text{ hours}$$

$$\mathrm{R_{BDF}} = 8.271\mathrm{e}^{-0.1167\times4.68} \times 1.08^{0.3856} = 4.93 \text{ hours}$$

6. Determine slope adjustment factor using Equation 2-15:

   Example: In this example watershed, the channel slope (S) is 3 ft / mile and the overland slope (So) estimated as the average of the slopes of several overland flow paths perpendicular to the channel is 7 ft/mile. The product SxSo is 3x7= 21, which is less than 26. Then, a value of 26 is entered as SxSo in equation 2-15.

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

7. Determine detention adjustment factor using Equation 2-16:

   Example: The subwatershed is determined to have a detention volume of 103 ac-ft that is not modeled directly. The total area of the subwatershed is 1.08 square miles, yielding a DR value of 95.4 ac-ft/square mile.

   $$C_f = 3\times10^{-5}\times95.4^2 - 0.00095\times95.4 + 1 = 1.18$$

8. Determine ponding adjustments using Equation 2-22:

   Example: Based on aerial photography and LiDAR data, it was determined that 31% of the watershed is affected by ponding.

   $$\mathrm{RM}_{100} = 1.21\times31^{0.132} = 1.904$$

9. Determine Final TC and R values using equations 2-25 and 2-26:

   $$TC = 1.00\times1.18\times\left(1.45 + \frac{\sqrt{1.08}}{2}\right) = 2.33 \text{ hours}$$

   $$R = 1.00\times1.18\times1.904\times(8.271e^{-0.1167\times4.68}\times1.08^{0.3856}) = 11.10 \text{ hours}$$

These values are then input into the Clark Unit Hydrograph transform table in HEC-HMS for the relevant watershed.

2.4.5 <u>Flood Routing</u>

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

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

The method to be used depends on the information available to obtain channel and floodplain storage. If a HEC-RAS steady flow model is available for the stream being studied, the recommended technique for both channel and reservoir routing is the Modified Puls method. If an unsteady-flow HEC-RAS model is available, it is recommended to complete the routing using the Muskingum method in HEC-HMS with parameters obtained from the unsteady-flow HEC-RAS model.

Alternatively, the engineer may elect to use unsteady-flow HEC-RAS modeling to account for all channel storage without HEC-HMS routing. Where no HEC-RAS models exist to develop channel routing, a different method may be used, but should be justified by the engineer with information about methods and assumptions. A discussion of the Modified Puls and Muskingum methods is presented in the next two subsections.

a) Modified Puls

The Modified Puls method assumes a non-variable discharge-storage relationship and a constant level pool in the storage reach or reservoir of interest. The HEC-HMS program provides a routine for this flood routing technique. The required storage-discharge relationship for channel routing can be obtained by using the HEC-RAS steady-flow backwater program for a variety of flow conditions. Care must be taken in developing these storage-discharge relationships with HEC-RAS. Cross-sections need to be provided that adequately define <u>all</u> of the flood plain storage available at various water levels. However, only the <u>effective</u> flow area of the cross-section should be used to establish the proper discharge-water level relationship. In using this technique, it is recommended that the engineer use percentages of the 100-year flow to develop the rating data (e.g., 10%, 20%, 30%,…120%). Storage can be sensitive to change in flow, especially in the low range of flows. Accordingly, enough points should be added to capture the variation of this curve in the range of low flow.

The required storage-elevation-discharge relationships for this routing technique can be obtained by use of elevation and storage data estimated or measured from topographic data, and by use of elevation and discharge data developed through the use of structure/spillway options in HEC-HMS or a rating curve developed outside of HEC-HMS that adequately represents the hydraulic properties of the structure/spillway. For a discussion of the Modified Puls routing technique and other methodologies, the engineer is referred to the *<u>Handbook of Applied Hydrology</u>*, by Ven Te Chow, 1964.

b) Muskingum

The Muskingum method assumes both wedge storage (related to the sloping water surface) and prism storage (related to the flow of relatively uniform depth below the wedge) to address the variable discharge-storage relationship along a river reach as the water rises and falls with the passing of the hydrograph through the reach. 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. The value of K is typically defined as the travel time (in hours) of the flood wave through the channel 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). K and x are assumed to be constant throughout the full range of flows. The required K and x values for this routing technique can be obtained by use of a spreadsheet (available from the Fort Bend County Drainage District) based on a least-squares regression analysis. The HEC-RAS unsteady flow model can be used to develop hydrographs to calibrate K and x values in this spreadsheet. For a discussion of the Muskingum routing technique and other methodologies, the engineer is referred to *Applied Hydrology*, by V.T. Chow, D.R. Maidment, and L.W. Mays, 1988.

TABLE 2-1

TYPICAL AVERAGE VALUES

FOR IMPERVIOUS COVER AND RATIONAL METHOD PARAMETERS

Percent Impervious Cover, Runoff Coefficient and Retardation Coefficient*			
Land Use	**Percent Impervious**	**Runoff Coefficient C**	**Kerby Retardation Coefficient N**
Commercial/Multifamily	85	0.85	0.02
Industrial	72	0.75	0.02
Detention (Wet or Dry)	85	0.85	NA
Major Thoroughfares	90	0.90	0.02
Open Space – Row crops	0	0.20	0.20
Open Space – Pasture Land	0	0.15	0.40
Open Space – Wooded/forested	0	0.15	0.80
Open Space – Parks/green space	5	0.30	0.40
Cemeteries	0	0.15	0.40
Churches (up to 5 acre parcel)	60	0.50	0.10
Residential - 1/9 Acre			
Residential - 1/8 Acre			
Residential - 1/7 Acre	60	0.60	0.02
Residential - 1/6 Acre			
Residential - 1/5 Acre			
Residential - 1/4 Acre	50	0.55	0.02
Residential - 1/3 Acre	45	0.50	0.02
Residential - 1/2 Acre	38	0.45	0.02
Residential - 1 Acre	22	0.35	0.02
Residential ≥ 5 Acres	5	0.20	0.02
Schools	40	0.45	0.02

* NOTE: Local streets are included in impervious cover for residential areas.

TABLE 2-2

TEMPLATE FOR RATIONAL METHOD CALCULATIONS

Rational Method Peak Flow Calculation			
Overland flow Pasture Land	K	0.828	
	L	360	feet
	S	0.001	ft/ft
	N	0.4	
	Tof	43	min
Channel flow	n	0.10	
	R	2	feet
	s	0.005	ft/ft
	V	1.7	ft/s
	L	1000	ft
	Tch	10	min
Flow Calculations 100-yr storm Pasture land	Tc	53	min
	i	5.2	in/h
	C	0.4	
	A	10	Acres
	Qp	21	cfs

TABLE 2-3

POINT RAINFALL AMOUNTS (INCHES) FOR

VARYING DURATIONS AND FREQUENCIES

IN FORT BEND COUNTY, TEXAS

	Rainfall Frequency						
Duration	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
5 minute	0.59	0.73	0.84	1.00	1.13	1.26	1.57
10 minute	0.94	1.16	1.34	1.60	1.80	2.01	2.47
15 minute	1.19	1.46	1.69	2.00	2.25	2.50	3.11
30 minute	1.70	2.08	2.39	2.83	3.16	3.50	4.40
1 hour	2.26	2.78	3.22	3.83	4.30	4.80	6.20
2 hour	2.83	3.53	4.19	5.16	5.99	6.91	9.45
3 hour	3.17	4.00	4.82	6.08	7.19	8.47	12.00
6 hour	3.77	4.86	5.97	7.72	9.33	11.20	16.30
12 hour	4.40	5.79	7.20	9.41	11.40	13.80	20.50
1 day	5.09	6.82	8.55	11.20	13.70	16.50	24.50
2 day	5.86	7.98	10.10	13.40	16.30	19.70	28.20
3 day	6.38	8.72	11.00	14.50	17.70	21.30	30.00
4 day	6.77	9.23	11.60	15.30	18.50	22.10	30.90
7 day	7.64	10.30	12.70	16.50	19.90	23.50	32.20

Source: NOAA Atlas 14 Volume 11 Version 2, September 2018, Partial Duration Series Rainfall Depths (Lat: 29.5427º, Long: -95.5013º)

TABLE 2-4

EBD COEFFICIENTS FOR VARIOUS FREQUENCY STORMS

TO CALCULATE RAINFALL INTENSITY

Return Period	e	b	d
2-Year	0.7122	45.19	8.51
3-Year*	0.7063	48.73	8.37
5-Year	0.7033	53.74	8.30
10-Year	0.6771	55.66	7.43
25-Year	0.6222	51.65	5.09
50-Year	0.5782	47.69	3.04
100-Year	0.5274	42.99	1.08
500-Year	0.4782	45.22	0.20

Maximum Time of Concentration of 3 hours

**The 3-year values are provided for existing storm sewer systems for comparison purposes only.*

TABLE 2-5

EXCESS RAINFALL FOR COMPUTING RUNOFF VOLUMES

Percent Impervious	5-YR, 24-HR Rainfall	Losses	Excess Rainfall	10-YR, 24-HR Rainfall	Losses	Excess Rainfall	25-YR, 24-HR Rainfall	Losses	Excess Rainfall	100-YR, 24-HR Rainfall	Losses	Excess Rainfall
	in	in	in	in	in	in	in	in	in	in	in	in
0	6.82	**TBD**	**TBD**	8.55	**TBD**	**TBD**	11.20	**TBD**	**TBD**	16.50	**TBD**	**TBD**
20	6.82	**TBD**	**TBD**	8.55	**TBD**	**TBD**	11.20	**TBD**	**TBD**	16.50	**TBD**	**TBD**
40	6.82	**TBD**	**TBD**	8.55	**TBD**	**TBD**	11.20	**TBD**	**TBD**	16.50	**TBD**	**TBD**
60	6.82	**TBD**	**TBD**	8.55	**TBD**	**TBD**	11.20	**TBD**	**TBD**	16.50	**TBD**	**TBD**
80	6.82	**TBD**	**TBD**	8.55	**TBD**	**TBD**	11.20	**TBD**	**TBD**	16.50	**TBD**	**TBD**

Green-Ampt parameter values within Fort Bend County are in the process of being defined, and will be provided in a subsequent version of this document.

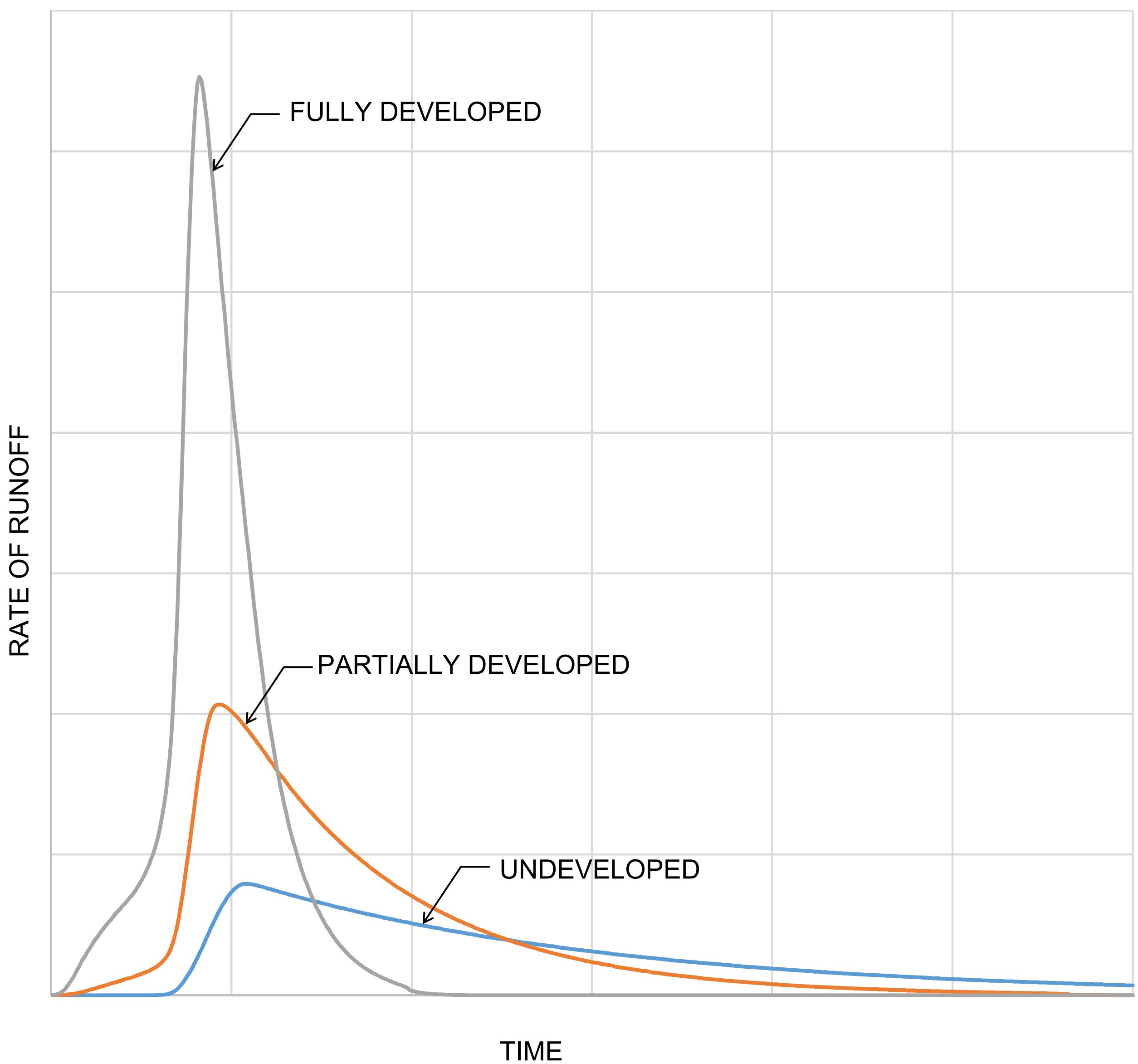
TABLE 2-6

GREEN-AMPT LOSS PARAMETERS FOR SOIL ZONES

IN FORT BEND COUNTY, TEXAS

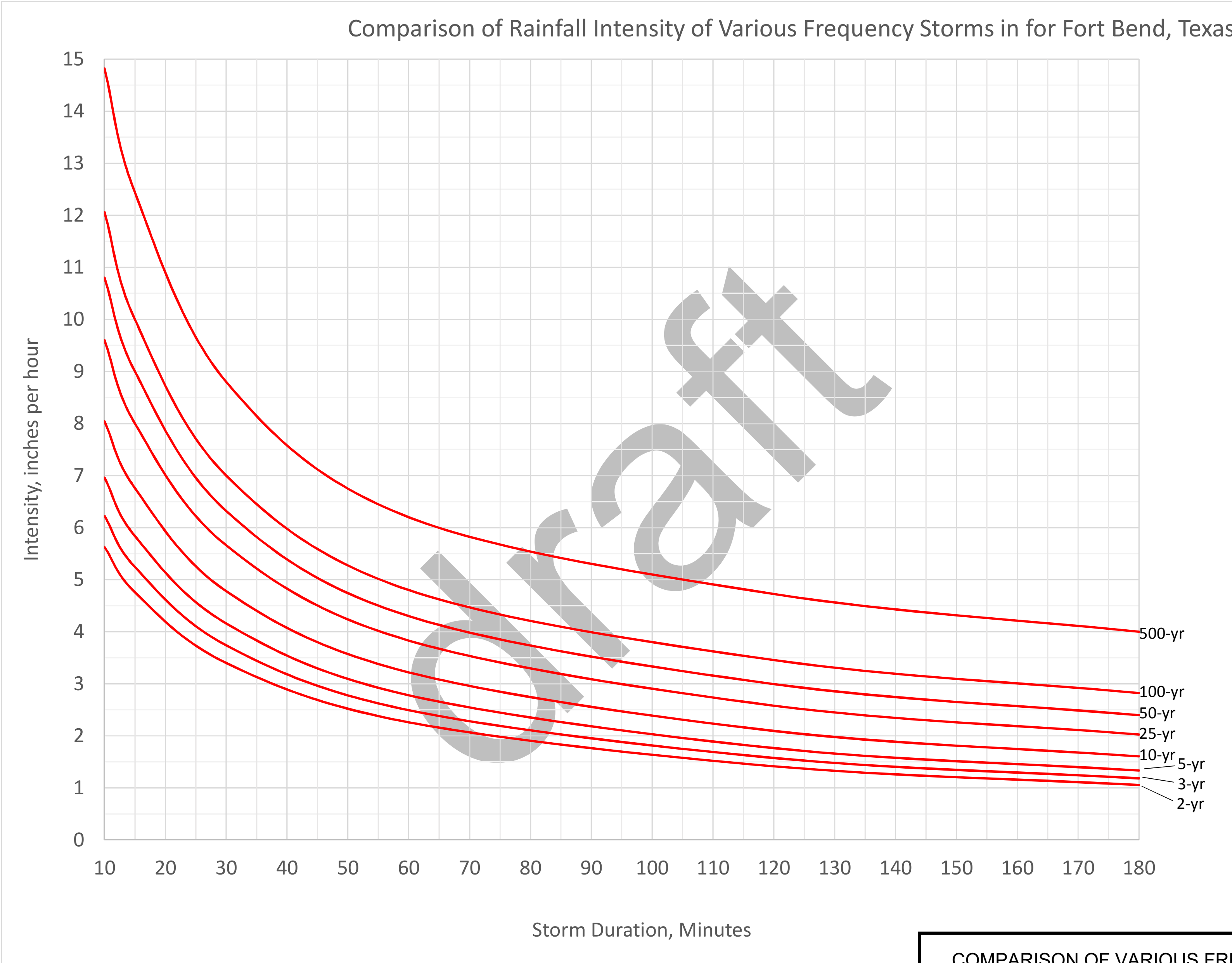
Zone*	**Initial Canopy Storage**	**Max Canopy Storage**	**Crop Coefficient**	**Initial Content**	**Saturated Content**	**Suction**	**Conductivity**
	(%)	**(in)**		**(in)**		**(in)**	**(in/hr)**
Zone 1	**TBD**	**TBD**	**TBD**	**TBD**	**TBD**	**TBD**	**TBD**

Green-Ampt parameter values within Fort Bend County are in the process of being defined, and will be provided in a subsequent version of this document.



EXAMPLE BASED ON A 15.0 SQ-MI WATERSHED

EFFECTS OF URBANIZATION ON STORM HYDROGRAPH	
SEPTEMBER 2021	FIGURE 2-1



COMPARISON OF VARIOUS FREQUENCY STORMS IN FORT BEND COUNTY

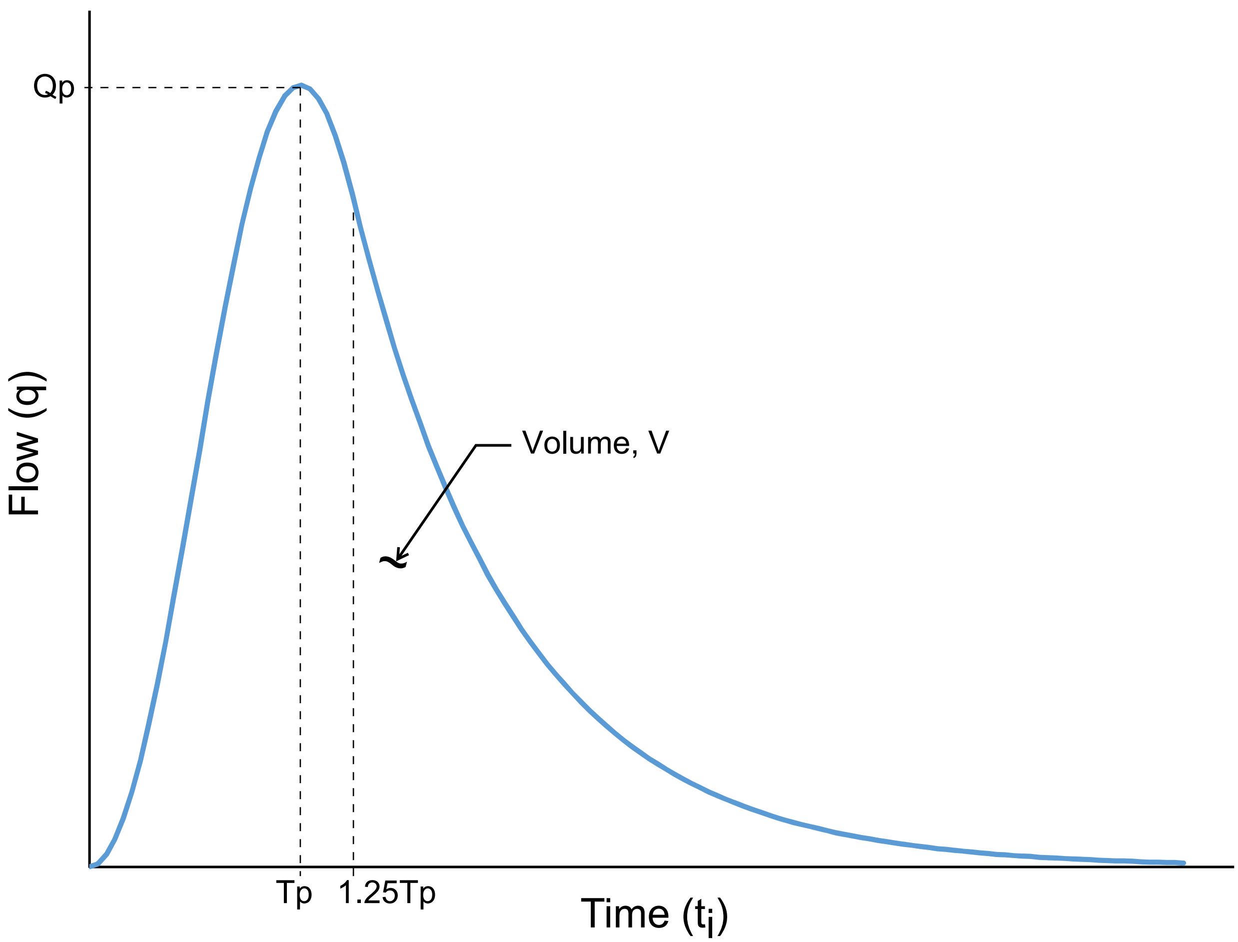
SEPTEMBER 2021	FIGURE 2-2

$$T_p = \frac{V}{1.39\,Q_p}$$

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

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

*Calculator must be in radian mode.

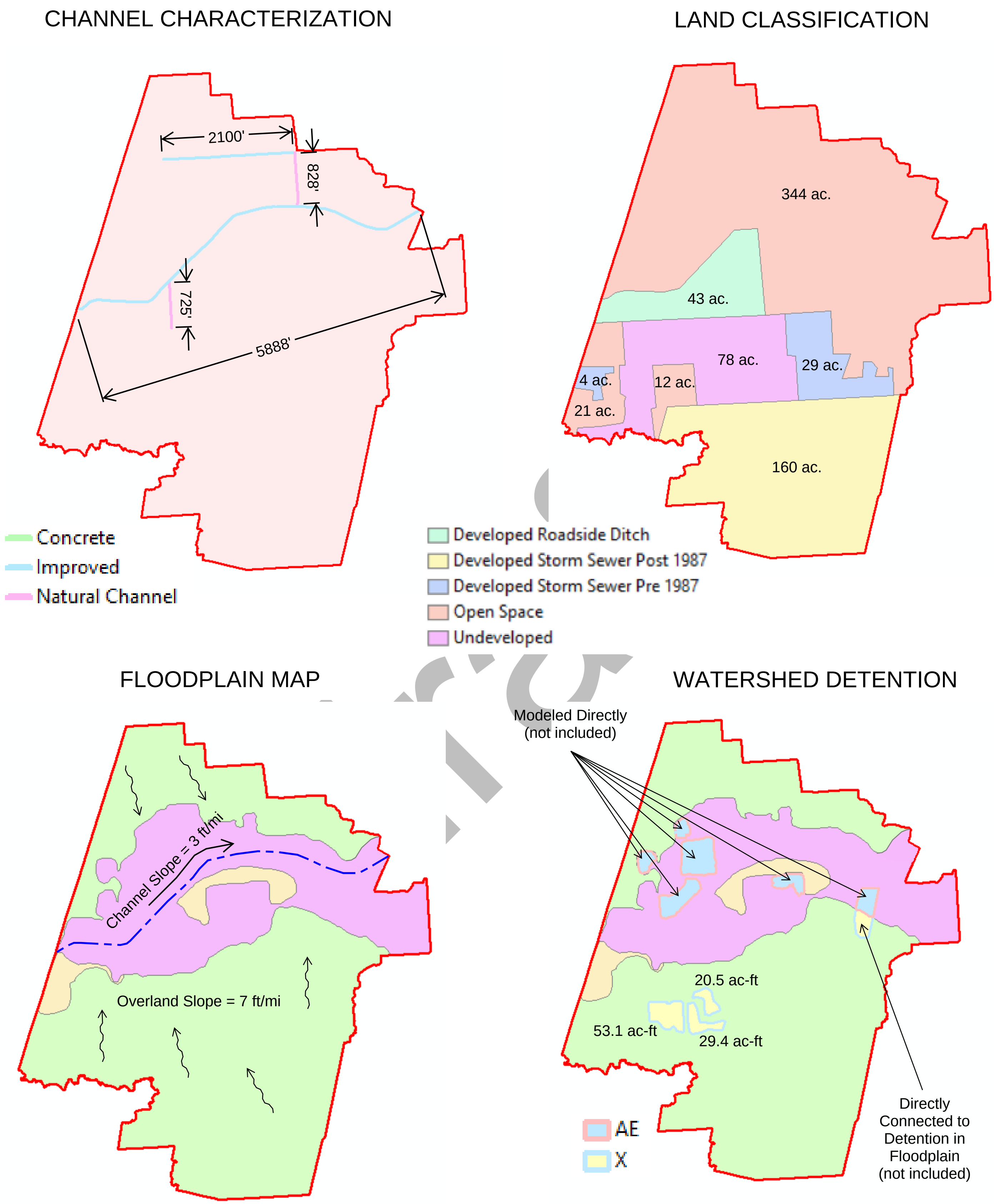


Source: Policy, Criteria, and Procedure Manual for the Approval and Acceptance of Infrastructure. Harris County Flood Control District, October 2018

MALCOM'S METHOD FOR

HYDROGRAPH DEVELOPMENT

SEPTEMBER 2021	FIGURE 2-3



EXAMPLE WATERSHED FOR APPLICATION OF BASIN DEVELOPMENT FACTOR

SEPTEMBER 2021

FIGURE 2-4

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Section **Page**

3.0 OPEN CHANNEL FLOW

3.1 GENERAL

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

In a major drainage system, open channels offer significant advantages over closed conduits considering cost, flow capacity, flood storage, recreation, and aesthetics. However, open channels require considerable right-of-way and maintenance. Careful consideration must be given in the design process to ensure that benefits are maximized, and disadvantages minimized. For design approaches not included in this manual, the Engineer should review and discuss with the Fort Bend County Drainage District Engineer prior to the design effort.

Channel analysis 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 Fort Bend County Drainage District.

All open channel hydraulic computations shall be in HEC-RAS version 5.0.7 (or newer) or XP-SWMM for newly developed models. Versions of HEC-RAS shall be consistent throughout each project. Software programs other than HEC-RAS or XP-SWMM, such as Infoworks, ICPR, FLO-2D, MIKE 11/MIKE FLOOD, Quick 2.1.0, WSPRO, etc., will be considered on a case by case basis, upon written agreement with the Fort Bend County Drainage District Engineer. At the discretion of the Fort Bend County Drainage District Engineer, conversion of alternative software models into HEC-RAS format may be required.

The Fort Bend County Drainage District will make available the most recent update of the models developed as part of the Fort Bend County Master Plan and a digital version of a County-Wide ponding map for use by designers and engineers to comply with the requirements of this Section. Where Fort Bend County Master Plan models exist, they must be used for submittals to the Fort Bend County Drainage District regardless of the availability of an effective FEMA model. FEMA submittals must be based on the FEMA effective models in addition to the modeling required with the Fort Bend County Master Plan models. Changes to the FEMA effective model should reflect the project as approved by the Fort Bend County Drainage District with the Master Plan models.

3.2 OPEN CHANNEL HYDRAULICS OVERVIEW

Flow conditions in an open channel may be characterized as steady or unsteady, uniform or varied, subcritical or supercritical. The characterization of the flow will determine the type of analysis required for submittal review by Fort Bend County Drainage District.

3.2.1 Steady Flow Techniques

Steady flow occurs when the velocity of successive fluid particles at any location is the same for successive periods of time. Therefore, the velocity is constant with respect to time ($dv/dt = 0$), although it may vary at different locations in the channel. This statement implies that the flow rate, Q, must also be constant with respect to time. Steady-state flow use is limited to situations where water surface elevations for a specified discharge rate, without consideration of the timing, is the main purpose of the analysis.

3.2.2 Uniform Flow

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

A true state of uniform flow is difficult to obtain under most conditions. Nevertheless, when a channel is sufficiently long and uniform such that the flow depth is not changing (i.e. the channel resistance and gravity forces balance each other), the flow may be assumed to be uniform for design purposes. In this instance, a steady state modeling approach is an acceptable methodology. A steady-state analysis is acceptable if the design storm is contained within the channel banks, attenuation of peak discharges is not intended in the channel, and design discharges are unchanged within the limits of the proposed drainage improvement.

3.2.3 Varied Flow

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

3.2.4 Unsteady Flow

For unsteady flow, the flow at any point is not constant with respect to time. The unsteady option in HEC-RAS is recommended when one or more of the following situations are present:

1. Discharges and/or elevations have rapid changes
2. Channels have slopes greater than 5 feet per mile
3. Flow attenuation is expected due to flat topography
4. Varying tailwater or backwater effects dominate
5. Flood forecasting for major rivers is to be considered
6. River systems are large and complex
7. Channel overbanks provide significant storage that may produce attenuation of peak discharge
8. Storage basins, particularly basins in series, or off-line basins are being analyzed
9. Timing and attenuation of peak discharges are of interest

Unsteady flow modeling shall be utilized to demonstrate no adverse impact to flood risk to meet the requirements of this Manual. However, detention storage shall be provided for developments as outlined in Section 6.0, even if a detailed unsteady flow analysis shows no increase in peak surface water elevations.

3.2.5 Two-Dimensional Flow

Two-dimensional flow is a form of unsteady-state flow. In areas where design discharges are not contained within the limits of the channel banks, flow in the overbanks may provide storage, resulting in flow attenuation and changes in the characteristics of the hydrographs. This condition is common in Fort Bend County where existing channels do not completely contain the design storms. These overbank flow effects can often be captured in one-dimensional unsteady state models. However, one-dimensional modeling begins to falter when flow travels traverse from the channel or there are complex flow patterns in the overbanks. When it is desired to understand the location, depth or timing of discharges in overbank flow, a two-dimensional flow analysis shall be considered. Special conditions which may require two-dimensional flow analysis include:

1. Overflows between watersheds
2. Widespread flow with complex patterns such as islands within the floodplain
3. Prevalent ineffective flow areas in the overbanks
4. Redirection of flow by proposed fill
5. Split flow conditions where portion of the flow leaves the channel and continues downstream in the overbanks

At discretion of the Fort Bend County Drainage District Engineer, two-dimensional modeling may be required where special conditions exist. The Engineer shall coordinate with Fort Bend County Drainage Engineer on any two-dimensional flow modeling required for the project. The Engineer is encouraged to follow adopted two-dimensional guidelines by other agencies such as Harris County Flood Control District or the U.S. Army Corps of Engineers' Hydrologic Engineering Center.

3.2.6 Subcritical or Supercritical Flow

The celerity of small gravity waves in a shallow channel is defined by the term $(gy)^{1/2}$ where g is the acceleration due to gravity and y is the depth. When the velocity of flow in a channel exceeds this value, the flow is supercritical. When it is less than this value, the flow is subcritical. Hence, the ratio of velocity of flow to celerity $(v/(gy)^{1/2})$, known as Froude Number, is less than 1.0 for subcritical flow and greater than 1.0 for supercritical flow. Supercritical flow is generally characterized by high velocities, shallow depths and an unstable flow regime, while subcritical flow is characterized by slower velocities and greater depths. The most important distinction between these two states of flow is that the effect of a disturbance in the channel, such as a bridge constriction, cannot be propagated upstream in supercritical flow as it can in subcritical flow. Therefore, subcritical flow is controlled by downstream channel conditions while supercritical flow is controlled by upstream channel conditions.

3.2.7 Critical Depth

When the velocity of flow in a channel is equal to the velocity of a gravity wave $(gy)^{1/2}$, critical flow at critical depth exists. Hence, for critical flow, the value of the Froude Number is 1.0. Flow at or around critical is characterized by instability and should be avoided in channel design except at specific flow transition points such as weirs and sluice gates. Near critical flow, small changes in hydraulic conditions such as bridges or changes in cross-section area will cause exaggerated changes in depth and velocity, creating potentially erosive conditions in earthen channels. Critical depth conditions may occur at significant changes in channel bottom slope or cross-sectional geometry. A transition from subcritical flow to supercritical always pass at some point through critical depth, while a transition from supercritical to subcritical flow occurs in the form of a hydraulic jump.

The output of the hydraulic models provide locations where critical flow occurs. The Engineer should identify the locations of critical flow and determine if this condition is expected in the design or if revisions to the model or the design are required.

3.2.8 Manning's Equation

Manning's equation is an empirical equation which relates friction slope, flow depth, channel roughness, and channel cross-sectional shape to flow rate. The friction slope is a measure of the rate at which energy is being lost due to resistance at the interface between flow and the channel surface. When the channel slope and the friction slope are equal, the flow is uniform and Manning's equation may be used to determine the depth for this condition (normal depth). Manning's equation is as follows:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad (3\text{-}1)$$

where

> V = velocity of flow (ft/sec)
> n = Manning's coefficient of roughness
> A = cross-sectional area of the flow (ft^2)
> R = hydraulic radius of the channel (ft) (flow area/wetted perimeter)
> S_f = friction slope, the rate at which energy is lost due to channel resistance

Alternatively,

$$Q = \frac{1.49}{n} A\, R^{2/3}\, S^{1/2} \qquad (3\text{-}2)$$

where Q = total discharge (cfs)

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

3.2.9 Manning's "n" Value

Manning's "n" value is an experimentally derived constant which represents the effect of channel roughness in the Manning's equation. Considerable care must be given to the selection of an appropriate "n" value for a given channel due to its significant effect on the character of the flow and resulting values for velocity and depth of flow. Tables 3-1 and 3-2 provides a listing of "n" values for one-dimensional and two-dimensional modeling respectively for various channel conditions. Deviation from these values requires explanation of justification and written acknowledgement from the Fort Bend County Drainage District Engineer.

3.3 CHANNEL DESIGN

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

3.3.1 Design Flow and Freeboard Requirements

Channels designed to convey a flow greater than or equal to 30 cfs or those with a total depth greater than or equal to 2.5 feet are subject to the design guidelines of the Section 3.3 of this Chapter. Open channels shall be designed to contain the runoff from the 100-year frequency 24-hour duration storm within the channel banks while also providing at least one foot of freeboard when passing this flow. Freeboard must be checked at the points of discharges from lateral pipes accounting for the added flow. The necessity for additional freeboard shall be considered on the outside channel edge along curves or as directed by Fort Bend County Drainage District Engineer

3.3.2 Required Analyses

In order to ensure that the design is adequate for various flow conditions, profiles of water surface elevations for new channels or channel modifications shall be provided for the 2-, 5-, 10-, 25-, 50-, and 100-year storm events. Other events may be required by the Fort Bend County Drainage District Engineer, depending on site-specific local conditions.

In cases where channel modifications are necessary as part of a proposed development, hydraulic modeling shall demonstrate the project results in no increase in water surface elevations upstream or downstream of the proposed project as required in Section 3.4. For new development or when the new channel discharges into an existing channel with limited capacity, storm water detention shall be provided so that there is no adverse impact to flood risk in the receiving channel. Please refer to Section 6.0 for the requirements for storm water detention.

3.3.3 Submittals

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

1. A vicinity map of the site and subject reach. The subject reach is defined as the stretch of channel necessary for any altered flow profile to match the upstream and downstream existing profiles. Pertinent roadway name callouts shall be provided in the vicinity map.

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

3. A watershed map showing the full extents of the existing and proposed drainage area boundary for the subject reach, along with all subarea delineations and all areas of existing or proposed development.

4. Discharge calculations for storms denoted in this section, specifying methodology and key assumptions used, including time of concentration, impervious cover assumptions, rainfall source information and routing methodology as needed for discharges at key locations within the subject reach. Section 2 of this Manual shall be utilized for all discharge calculation information. Rational Method calculations shall only be used for drainage areas less than 100 acres. At a minimum, discharge calculations are required for locations upstream and downstream of the subject channel reach.

5. Hydraulic calculations specifying methodology used. All assumptions and values of the design parameters must be clearly stated, including source of geometric information, topography, elevation datum, Manning's n-values, and bridge/culvert modeling approach.

6. A tabulation of water surface elevations and average velocities for each return event, summarizing pre- and post-project conditions. Maximum allowable velocities by channel material are shown in Table 3-3.

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

   a. All pertinent water surface profiles as noted in Section 3.3.1 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.)

   A sample of the water surface elevation profiles is included as Figure 3-1.

8. A map delineating existing and proposed rights-of-way, adjacent property lines and ownership information.

9. Benchmark, elevation, datum and year of adjustment. For consistency with the Fort Bend County Master Plan models, the datum of all hydraulic models submitted for review shall be the 1988 North American Vertical Datum, 2001 adjustment. Use of another datum requires approval from the Fort Bend County Drainage District Engineer.

10. Typical existing and proposed cross-sections.

11. A list of environmental permits required from the U.S. Army Corps on Engineers with a statement that the project owner or project engineer is responsible for acquiring these permits. Approvals from the Fort Bend County Drainage Engineer are limited to compliance with the requirements of this Manual and does not include approvals for other permits required by other agencies.

12. A geotechnical report addressing groundwater conditions, erosion and slope stability. The geotechnical report shall be part of the construction documents.

3.3.4 <u>Design Considerations</u>

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

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

In areas where land values are extremely high, or right-of-way is limited, lined channels may be the design of choice. However, lined channels can be significantly more expensive to construct. In addition, they tend to produce higher velocities and provide less storage, possibly resulting in higher peak discharges or erosive conditions.

3.3.4.1 Optimal Design Flow Characteristics

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

1. Velocity – Excessive velocities can cause erosion and may pose a threat to safety or adjacent structures. Low velocities may allow sediment deposition and subsequent channel clogging and loss of available channel capacity. Table 3-3 provides maximum allowable velocities. Minimum velocities are those produced by a channel slope of 0.05 percent.

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

3. Freeboard – Selection of a safe amount of freeboard should be based on confidence in the design discharge estimates, stability of the flow profile and the expected damage from water overflowing channel banks. Free board requirements are listed in Section 3.3.1.

3.3.4.2 Optimal Channel Configuration Characteristics

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

1. Invert Slope – Slope of the channel invert is generally governed by topography and the energy head required for flow. Since invert slope directly affects channel velocities, channels should have enough grade to prevent significant sediment deposition. However, grades should not be steep enough to create erosion concerns. In Fort Bend County, the minimum recommended channel invert slope shall be 0.05 percent. The Fort Bend County Drainage District Engineer shall provide written acknowledgement for channel designs flatter than 0.05 percent.

The maximum channel invert slope shall be limited by maximum flow velocities as shown in Table 3-3. Appropriate channel drop structures may be used to limit channel invert slope in steep areas.

2. Side Slope – Slope stability analyses shall be performed for short term, rapid drawdown, and long-term conditions. The criteria for safety factors against slope failures are 1.3 for the short term (end of construction) condition, 1.25 for the rapid drawdown condition, and 1.5 for the long-term condition. Regardless of the results of this analysis, the slope shall not be steeper than 4 (horizontal): 1 (vertical), which is the practical limit for mowing equipment.

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

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

5. Manning's "n" Value – Tables 3-1 and 3-2 of this Section provide guidance regarding the Manning's roughness coefficient used in man-made channels. Alternative values to those noted in this section shall be acknowledged in writing by the FBCDD Engineer.

6. Confluences – The angle of intersection between the tributary and main channels shall be between 15º and 45º. Angles in excess of 45º are discouraged and require approval by the Fort Bend County Drainage District Engineer. Angles in excess of 90º are not permitted.

7. Maintenance Access – Access to the entire length of the channel for maintenance shall be provided for new channels. This access path shall be as noted in Table 3-4 and be located within the right-of-way of the channel or a dedicated drainage easement. These access requirements may include crossing across the channel, stabilized access roads, or concrete lining. The Fort Bend County Drainage

District Engineer may provide guidance for access points and location.

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

9. <u>Location</u> – Channels should be located at enough distance away from existing and proposed roads, buildings, and other infrastructure to protect the stability of those items. Where channels cross roadways, adequate slope stabilization and erosion control measures shall be provided. Culvert design shall conform to the requirements of Chapter 4.

3.3.5 Minimum Requirements for Specific Types of Channels

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

3.3.5.1 Grass-Lined Channels

The following are minimum requirements for vegetated earthen channels:

1. Maximum side slopes shall be as determined by a slope stability study but no steeper than 4:1. A signed and sealed geotechnical engineering report shall be provided with the design plans.

2. Minimum bottom width shall be six (6) feet.

3. Backslope interceptor structures shall be provided at a maximum of 800-foot intervals to prevent sheet flow over the channel side slopes. Refer to Figure 3-2 for a detail.

4. Channel slopes must be vegetated with a perennial grass cover (typically Common

Bermuda) immediately after construction to minimize erosion. Approximately 70 percent coverage of vegetation is required within three weeks of first placement and 100 percent within six weeks. The contractor shall be responsible for the maintenance of the vegetation coverage for a period of 12 months after construction is complete and accepted by the Project Engineer or the Fort Bend County Drainage District Engineer.

3.3.5.2 Concrete-Lined Trapezoidal Channels

The following are minimum requirements for concrete-lined channels:

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

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

3. The slope concrete design shall conform to the recommendations of the geotechnical study and meet the following minimum requirements:

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

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

    c. The maximum side slopes for any concrete lined areas shall be 2:1 and shall ensure that the escape stairways are included per Section 3.3.5.3 (6).

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

    e. Backslope drainage structures shall be provided in partially lined channels.

For fully lined channels, backslope drainage is to be considered but is not required.

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

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

h. Control joints shall be provided at approximately twenty-five feet on center. Sealing agents shall be utilized to prevent moisture infiltration.

3.3.5.3 Rectangular Concrete Pilot Channels

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

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

2. The structural steel design shall conform to ASTM A 615, Grade 60 steel.

3. Minimum bottom width shall be eight feet.

4. Bottom widths twelve feet or greater shall be graded 1% toward the channel center line.

5. The maximum height of the vertical walls shall be four feet. Heights above four feet shall be installed in no more than two-foot increments with two-foot (minimum) flat shelf between increments. Exceptions will be considered on a case-by-case basis.

6. Escape stairways shall be located at the upstream side of all street crossings and at

intervals not exceeding 1,400 feet.

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

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

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

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

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

3.4 EROSION

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

1. At all points of discharge from closed system outfalls.

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

requirements in Section 3.3.4.2.

3. Around bridges and culverts where channel transitions create increased flow velocities.

4. When the channel slope is steep enough to cause critical depth and/or excessive flow velocities.

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

6. In areas with channel side slopes steeper than 4:1

7. At locations of change in cross-sectional geometry

8. At the point of change in bed or bank material type (i.e. gabion to earthen, rock to vegetated, etc.)

9. At channel confluences.

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

Sound engineering judgment and experience should be used in locating areas requiring erosion protection. It is often prudent to analyze potential erosion sites following a significant flow event to pinpoint areas of concern. It is the discretion of the Fort Bend County Drainage District Engineer to require erosion protection in specific locations.

3.4.1 Minimum Erosion Protection Requirements

Minimum requirements for Fort Bend County are as follows:

3.4.1.1 Confluences

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

3.4.1.2 Bends

When required, erosion protection must extend along the outside bank of the bend and at least 20 feet downstream of the point of tangency. Additional protection on the channel bottom and inside bank, or beyond 20 feet downstream, shall be required if maximum allowable velocities are exceeded. Refer to Table 3-3 for the maximum velocities.

3.4.1.3 Culverts

In areas where outlet velocities exceed 5 feet per second on to a vegetated earthen channel, channel lining or an energy dissipation structure shall be required.

3.4.1.4 Outfalls

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

3.4.2 <u>Structural Erosion Controls</u>

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

3.4.2.1 Riprap

The use of riprap is an allowable erosion control measure only in those locations where concrete slope paving is not feasible. Placement of riprap is not allowed within FBCDD's ROW without the consent of the Fort Bend County Drainage District Engineer. Riprap is defined as broken concrete rubble or large stone. A discussion of riprap design can be found in Hydraulic Design of Flood Control Channels, EM 1110-2-1601, U.S. Department of the Army, Corps of Engineers, 1994 (or latest version). Erosion protection construction plans shall include methodology and calculations for selection, sizing and placement of rip rap.

3.4.2.2 Concrete Slope Paving

Minimum requirements for partially or fully concrete-lined channels are presented in Section 3.3.5.2

3.4.2.3 Backslope Drainage Systems

The use of backslope drains and swales with engineered channels is required in Fort Bend County. These systems collect overland flow from channel overbanks and other areas not draining to a storm sewer collection system. Their purpose is to prevent excessive overland flow from eroding vegetated channel side slopes as it enters the channel. Subject to Fort Bend County Drainage District's Engineer written acceptance, back-slope drains may not be required in undeveloped or sparsely developed areas.

The design engineer should carefully consider the drainage area to be intercepted by such systems, particularly when the channel passes through large areas of undeveloped acreage where large quantities of naturally occurring sheet flow could overload the backslope swale and drainage system. In these areas, drain spacing and backslope drainage pipe requirements may have to be modified to account for the conditions. In these cases, discussion with the Fort Bend County Drainage District Engineer is strongly recommended. Refer to Figure 3-2 for backslope drainage design requirements.

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

General requirements for backslope drains and swales are as follows:

1. Minimum backslope drain pipe shall be 24” in diameter.
2. Maximum spacing is 800 feet (or 400 feet to the swale high point).
3. The drain structure and swale centerline shall be six feet inside the channel right-of-way line.
4. Minimum design depth in swale is 0.5 feet.
5. Maximum design depth in swale is 2.0 feet.
6. Minimum gradient for swale invert is 0.2%.
7. Swale should have side slopes no steeper than 3:1.

A typical detail of backslope interceptor is presented in Figure 3-2.

3.4.2.4 Sloped Drops

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

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

When subcritical flow approaches a drop, depth decreases and velocity increases as the flow nears critical depth. After the drop, the flow becomes supercritical and a hydraulic jump occurs prior to the transition into the receiving channel. Accordingly, appropriate erosion protection must be provided such that excessive flow velocities are not unprotected in any reach of channel. Erosion protection shall extend a minimum recommended distance of 20 feet upstream of a drop.

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

3.4.2.5 Baffled Chutes

Baffled chutes are used in drainageways when a relatively large change in elevation is necessary. The baffle blocks prevent undue acceleration of the flow as it passes down the chute by interrupting flow velocity and forcing the flow to pass though critical depth and a hydraulic jump. Baffled chutes are generally laid out with a maximum steepness of 2:1 slope and can be designed to discharge up to 60 cfs per foot of channel width. The lower end of the chute is constructed to below streambed level and backfilled as necessary thereby minimizing degradation or scour of the streambed. No tailwater or stilling basin is required as velocities will remain moderate.

A simplified step-by-step procedure for the design of baffled chutes can be found in Section 9.8 of the U.S. Department of the Interior Bureau of Reclamation publication Design of Small Dams. For a more detailed discussion, the engineer is referred to U.S. Department of the Interior Bureau of Reclamation publication, Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25. Baffled chute basins shall be designed according to the guidelines of these documents. As of the date of publication of this Manual, these documents can be found at the following links:

Design of Small Dams:
https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallDams.pdf

Hydraulic Design of Stilling Basins:
https://www.usbr.gov/tsc/techreferences/hydraulics_lab/pubs/EM/EM25.pdf

3.5 MODELING GUIGELINES

Modern computer software for hydraulic modeling of open channels has many options available to represent various conditions and include a library of theoretical equations to assist the engineers in building a robust model. At the same time, there are decisions that are made at discretion of the engineer that may have a tangible impact on modeling results. From cross sections alignment, to losses coefficients, to modeling approaches for culverts and bridges, the engineer faces multiple options that require good judgements and sound application of engineering concepts to ensure the model is effective in representing the specific hydraulic conditions of the site.

Several documents have been produced as guidance for hydraulic modeling, which are intended to assist engineers in the model configuration and selecting proper options. Some level of consistency is also achieved if the same guidelines are followed by the various engineer responsible for several models in a region. The following documents are applicable for modeling in Fort Bend County:

1. HEC-RAS Unsteady Modeling Guidelines by the Harris County Flood Control District, July 2018.

2. Two-Dimensional Modeling Guidelines by the Harris County Flood Control District, July 2018.

3. HEC-RAS Hydraulic Reference Manual by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers.

4. HEC-RAS 2D Modeling User's Manual by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers.

The Engineer is encouraged to follow the above guidelines for the development of hydraulic modeling submitted for review by the Fort Bend County Drainage District Engineer. At minimum, the modeling guidelines for small models and steady state hydraulic modeling included in the next section must be considered.

3.6 WATER SURFACE PROFILES

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

3.6.1 Uniform Flow

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

3.6.2 Gradually Varied Flow

In most channel flow situations, the state of flow is gradually varied. In other words, the depth is gradually changing with longitudinal distance along the channel due to an imbalance between the forces of gravity and channel resistance. The recommended means for determining flow profiles under these conditions is with the use of a hydraulic modeling computer software. The primary accepted computer model for calculating gradually varied flow profiles is HEC-RAS.

The HEC-RAS model can readily accommodate modifications in channel design and losses at bridges, culverts, drop structures, and transitions. The program begins computation at a cross-section of known or estimated water surface elevation and proceeds upstream for subcritical flow, and downstream for supercritical flow. Most channels in Fort Bend County operate in sub-critical flow regime due to the topographic conditions in the county.

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

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

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

3. Errors can occur with the improper use of energy losses values; thus loss coefficients should be chosen carefully. The engineer should carefully select a specific bridge analysis methodology and understand its operation and limitations. Proper care should be taken to

ascertain that losses coefficient are reasonable.

3.6.3 Rapidly Varied Flow

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

3.6.4 Energy Losses

Analysis of flow profiles in open channels must include proper consideration of energy losses due to local disturbances such as bridges, drop structures, transitions and confluences. In many cases, such head losses are adequately handled with empirical coefficients. The following guidelines should be followed for typical sources of non-frictional energy loss.

3.6.5 Expansions and Contractions

Losses at transitions should be considered according to the channel geometry. In steady-state flow analyses, a guidance for loss coefficients is provided in the HEC-RAS User Manual. In unsteady-state flow analyses, transitions losses are accounted for in the momentum equation and calculation of loss coefficient for transitions may not be required. The Engineer should verify that all losses are properly accounted for depending on the modeling approach.

3.6.5.1 Bends

Channel bends results in added energy loss. Due to the typical low velocities of the channels in Fort Bend County, these losses may be considered negligible in most cases. Bend losses shall be accounted for if the radius is less than 100 feet and the channel turns more than 45 degrees.

The HEC-RAS program does not make allowances for energy losses at bends and modeling of these losses needs to be manually added. Guidance from the U.S Army Corps of Engineers

(Hydraulic Design of Flood Control Channels, EM 1110-2-1601) indicates that losses can be accounted by increasing the Manning's roughness coefficient by 0.001 for every 20 degrees of curvature per 100 feet of channel Alternatively, the geometry editor of HEC-RAS has the option to add user-defined loss coefficient for each cross section. Any bend loss analysis should be clearly documented in the submitted analysis.

3.6.5.2 Bridges

There are various methods available to compute losses associated with flow through a bridge. Sources of energy loss in bridges include flow resistance, channel transitions, and direct obstructions to the flow such as piers. Each bridge should be examined individually to determine the best approach. The bridge routines found in HEC-RAS are recommended for their versatility and flexibility. Additional information on HEC-RAS analysis of bridges and culverts can be found in the HEC-RAS manuals which are available on-line. Additional requirement for bridge analysis are part of Section 4.0 of this Manual.

3.6.6 Supercritical Flow Transitions

The design engineer should be aware that if flow through a transition is supercritical, standing waves, or a hydraulic jump, will be generated and additional freeboard will be necessary to safely contain the flow. For a discussion of the analysis of supercritical flow in transitions, the engineer is referred to the U.S. Army Corps of Engineer's publication Hydraulic Design of Flood Control Channels, EM 1110-2-1601, 1994 (or latest version).

3.7 FLOOD MITIGATION REQUIREMENTS

3.7.1 Hydraulic Impact Studies for Channels

Impacts to flood risks must be evaluated in all projects that alter the flow or modify channels or natural streams (including their floodplains) in Fort Bend County. Proper hydrologic and hydraulic analysis shall be completed to verify the proposed changes will not result in adverse impacts upstream or downstream of the project. If the analyses indicate there is the potential for increased flood risk, proper mitigation shall be included to prevent adverse impact and the effectiveness of the mitigation shall be demonstrated with a hydraulic analysis.

For some channels, the flood risk is identified in FEMA's Flood Insurance Rate Maps or is documented in other studies, while in many other channels, the flood risk has not been determined or officially published. In these latter streams, the flood risk is present, but is unknown. Regardless of the designation of a stream in the Flood Insurance Rate Maps or the status of previous studies, proper analysis of flood risk and mitigation should be conducted for all projects that have potential to change the water surface elevation or increase the flow in Fort Bend County drainage channels. Accordingly, detailed hydraulic impact analyses that demonstrate no adverse impact to flood risk for the 5, 10, and 100-year storm events are required for the approval of projects located in the following areas:

1. Streams, or tributaries to streams included in the Fort Bend County Drainage Master Plan Study.
2. FEMA Special Flood Hazard Areas Zones A, AE, AO, and AH.
3. Areas with 100-year ponding greater than 2.0 feet in the Fort Bend County-Wide Ponding Map.
4. Fort Bend County Drainage District's Right-of-Way.

For Zones A, the results obtained in the Hydraulic Impact Study may impose additional requirements for the development according to 44 CFR § 60.3 (b), paragraphs (3), (4) and (5) and the County's Floodplain Management Regulations. For developments in Zones A (including manufactured homes parks and subdivisions) greater than 50 lots or five acres, whichever is the lesser, the results of the hydraulic impact studies may be used to document the base flood elevations for the site provided no other studies are available (Refer to 44 CFR § 60.3 (b) (3)). The Engineer is advised to consult with the County's Floodplain Administrator about any additional requirement for the development given the additional information provided by the Hydraulic Impact Studies required in this Section.

3.7.2 **Requirements for Unstudied Streams**

If a portion of the project site falls within an area with ponding greater than 2.0 feet, per the County-Wide Ponding Map, detailed hydrologic and hydraulic analyses shall be required to determine the floodplain for the 100-year 24-hour event. The same shall be required if the drainage area for the receiving stream at the most downstream point of floodplain encroachment is one square mile or greater, regardless of the flood risk designation in the Flood Insurance Rate Maps. Backwater effects from nearby streams with known 100-year water elevation shall be considered in the analyses.

The modeling approach and software used for these required studies should be selected based on site-specific conditions. The study shall use the methodologies and criteria outlined in this Manual. Unless otherwise approved by the Fort Bend County Drainage District Engineer, analysis shall be unsteady-state flow or two-dimensional modeling. The drainage report shall include a justification for the modeling approach selected. The limits of the delineated floodplain in these studies become subject to the mitigation requirements described in Section 3.7.4.

3.7.3 Floodplain Mitigation for Fill

Floodplain mitigation shall be provided for any fill placed in the following areas that reduces the storage capacity of the flooding source:

1. 100-year floodplains as estimated in the models of the Fort Bend County Master Plan
2. Areas with 100-year ponding greater than 2.0 feet in the Fort Bend County-Wide Ponding Map
3. FEMA Special Flood Hazard Areas A, AE, AO, and AH.
4. New delineated floodplains per the requirements of Section 3.7.2

A different set of requirements to mitigate fill in the conveyance zone of the Brazos River is included in Section 3.7.4.

The proposed mitigation plan shall offset all impacts due to fill placed in the 100-year floodplain, therefore resulting in no adverse impacts to the water surface elevations or increases in velocities in the receiving stream that exceed the permissible maximum per Table 3-3. In addition to the requirements of no adverse impact of this Manual, the project shall comply with all regulations set forth by the County's Floodplain Administrator for floodplain management.

Development of sites 50 acres or less, that follow the guidelines from Section 6.4.1 and are located outside a conveyance zone, may provide compensatory storage for the fill at 1:1 ratio for the fill placed below the 100-year floodplain elevation in lieu of the hydraulic analysis. Volume can be quantified from the Fort Bend County Master Plan models, County-Wide Ponding Map, or newly delineated floodplains.

The preferred mitigation approach for fill placed in the floodplain is through an off-line detention basin directly connected to the affected stream; However, on-site mitigation is also accepted. Mitigation within the channel is only allowed with prior approval from the Fort Bend County Drainage District Engineer and must follow the requirements defined under Section 6.4.12.3.

The mitigation volume shall be below natural ground and shall be hydraulically connected to the flooding source. Any mitigation added to the project to meet the requirements of this Section is separate from the detention established in Section 6.0, meaning floodplain mitigation volume cannot be counted as detention volume. Any unused volume of the detention pond during the 100-year event that is above the channel's 100-year base flood elevation, including the freeboard, cannot be counted as floodplain mitigation for fill. Mitigation measures must be incorporated into a detailed hydraulic unsteady flow model as described further below.

A Conveyance Zone is defined as an area that actively conveys runoff, where the product of the depth times the velocity of flow is greater than or equal to 2.5. No fill is allowed within a conveyance zone without a detailed analysis as specified below.

Floodplain fill mitigation may be accomplished by implementing engineered mitigation measures evaluated with detailed hydraulic unsteady-flow modeling (in HEC-RAS, XP-SWMM, Infoworks ICM, or ICPR 4) that demonstrate the proposed project and mitigation measures result in no increase in water elevation and in acceptable velocities, per Table 3-3, for the 10-year and 100-year event upstream and downstream of the project. This analysis must include the following:

a. Existing conditions: represent conditions and floodplain storage available before the project.
b. Proposed conditions:
    i. For detention ponds (Section 6) located outside of an identified 100-year floodplain, and not influenced by tailwater conditions, the analysis focuses on evaluating impacts to conveyance along the receiving stream and must represent conditions with the project in place, including proposed fill, modifications to receiving channel, proposed mitigation infrastructure on-site

or off-site, and infrastructure that provides connectivity between the receiving stream and the mitigation infrastructure. Flows generated by the proposed project can be omitted, as there is no interaction between the detention pond and mitigation infrastructure for fill placed in the floodplain.

ii. For detention ponds (Section 6) located within an identified 100-year floodplain, or influenced by tailwater conditions, a comprehensive evaluation of the system is required. In addition to the elements required for 3.7.3.b.i, flows generated by the proposed project and detention pond must be incorporated.

3.7.4 <u>Floodplain Mitigation for Fill in the Floodplains of the Brazos River</u>

The Fort Bend County Drainage District has updated the hydrologic and hydraulic (H&H) models for the Brazos River based on recent storm events and Atlas 14 rainfall. This information is available at the FBCDD website or can be requested from the District. The previous drainage criteria manual included a conveyance zone within the Brazos River and did not require mitigation of fill placed in the floodplain outside the conveyance zone.

Based on the results of the updated H&H models for the Brazos River, a conveyance zone is no longer applicable, and a no adverse impact policy applies within the Brazos River floodplain. Future development must mitigate fill placed within the Brazos River floodplain and conduct an evaluation of impacts to conveyance. The level of detail for this evaluation varies depending on the type of development within the Brazos River floodplain, as presented below.

1. Single residential lot development (1 structure):
   a. On slab or with fill
      i. Fill placed above natural ground should be compensated with excavation occurring in the vicinity of the fill, resulting in no net fill when measured at the Base Flood Elevation (BFE) of the Brazos River.
      ii. A simplified Manning's equation calculation may be used to demonstrate no impact to Brazos River conveyance. Manning's n-values shall be consistent with values listed in Table 3-2.
      iii. Mitigation of conveyance impacts must follow flow patterns of the Brazos River floodplain.

iv. Accessory structures as defined by flood damage prevention regulations should be coordinated with Fort Bend County Engineering.

b. Pier and Beam

i. In order to take credit for conveyance mitigation, open foundations are required, and the distance between piers/columns should result in, at minimum, a 75 percent opening around the perimeter of the building footprint below the Base Flood Elevation (BFE). Areas below elevated buildings must be free of obstructions.

ii. Closed foundations, such as solid masonry or concrete walls must follow the same requirements for 3.7.4.1.a.

2. Rural Subdivisions:

a. Net fill should be zero, measured at the BFE of the Brazos River.

b. Evaluation of impacts using a two-dimensional HEC-RAS model is ideal. In any case, mitigation of conveyance impacts must follow flow patterns of the Brazos River floodplain, and account for all fill placed, including roads.

c. Mitigation for conveyance must be provided within an established easement that is maintained in perpetuity by others. Mitigation within individual lots is not acceptable.

d. Placement of culverts across proposed roads should be coordinated with Fort Bend County Engineering

e. Mitigation must be designed by a Licensed Engineer.

3. Large Developments: Levee improvement district or master planned communities.

a. A complete No Adverse Impact analysis must be completed to evaluate impacts to the Brazos River and any local streams in the vicinity of the project.

b. The recently completed hydraulic model (1-D/2-D HEC-RAS) for the Brazos River, along with the best available Hydrologic and Hydraulic models for the local streams in the vicinity of the project should be used for this evaluation.

c. The analysis must demonstrate no adverse impact to the Brazos River and local streams in the vicinity of the project.

3.8 RIGHT-OF-WAY

All new drainage facilities shall consider the existing drainage upstream. In addition, new development must provide the ultimate planned right-of-way width based on anticipated upstream development watershed conditions. Anticipated upstream developed conditions means the greater of:

(1) the existing conditions flow as estimated by a master plan model;
(2) Developed mitigated flows from the channel's contributing area; this assumes future development within the contributing area provides on-site detention to mitigate increases in flow down to pre-development conditions;
(3) future undetained developed flows if being conveyed to a downstream detention pond considering an adopted Future Land Use Plan and regional detention plans, if available.

See Chapter 2 for guidelines for fully developed discharge calculations.

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

3.9 UTILITY CROSSINGS

Prior to design, the FBCDD Engineer should be contacted for information pertaining to the ultimate channel cross-section and right-of-way requirements. In addition, written County approval must be obtained for all future utility lines crossing Fort Bend County flood control facilities. All utility crossing shall have a minimum of five feet cover below the channel flow line. All manholes required for the utility conduit shall be located outside of the ultimate Fort Bend County right-of-way.

3.10 LOW WATER CROSSINGS

Low water crossings over intermittent streams in rural areas shall maintain the capacity of the pre-existing channel below the top of bank and shall be designed with the minimum requirements noted in Figure 3-7. Calculations of the capacities before and after the installation of the low water crossing shall be provided with the design.

TABLE 3-1

VALUES OF THE MANNING ROUGHNESS COEFFICIENT – "n" FOR ONE-DIMENSIONAL MODELING

Description	Recommended Manning's n-Value for 1D Channel Modeling
Channel	
Maintained Grass Lined Channel	0.04
Unmaintained or Natural Channel	0.05 - 0.10
Concrete Lined Channel	0.015
Open Water	0.02
Overbanks	
Developed High Intensity (Parking lots, or impervious areas)*	0.03
Developed Med Intensity (Developed <0.5 ac lots)	0.12
Developed Low Intensity (Developed >0.5 ac lots)	0.10
Developed Open Space (Parks, Golf Course, Cemeteries)	0.05
Barren Lands	0.03
Forest/Shrubs (Dense, max depth <6')	0.15
Forest/Shrubs (Dense, max depth >6')	0.25
Pasture/Grasslands	0.07
Cultivated Crops	0.06
Wetlands	0.15
Building**	10
Pavement	0.02

**Additional detail of adding large building footprints may be required*
***Optional to Use Ineffective Flow Area in Lieu of High N-value*

TABLE 3-2

VALUES OF THE MANNING ROUGHNESS COEFFICIENT – “n” FOR OVERBANK AREAS IN TWO-DIMENSIONAL MODELING

Land Classification	**HGAC Code**	**Recommended 2D Manning's n-Value for Rain On Grid (Precipitation Assigned)**	**Recommended 2D Manning's n-Value for 1D/2D Simulations (Flow Hydrographs Assigned)**
Open Water	1	0.02	0.02
Developed High Intensity	2	0.03	0.03
Developed Med Intensity	3	0.18	0.12
Developed Low Intensity	4	0.16	0.10
Developed Open Space	5	0.06	0.05
Barren Lands	6	0.04	0.03
Forest/Shrubs	7	0.25	0.15
Pasture/Grasslands	8	0.22	0.08
Cultivated Crops	9	0.17	0.08
Wetlands	10	0.25	0.15
Building	N/A	10	10
Pavement	N/A	0.02	0.02
Grass Lined Channel	N/A	0.04	0.04
Concrete Lined Channel	N/A	0.015	0.015
Generic Landuse 1*	N/A	0.12	0.06
Generic Landuse 2*	N/A	0.14	0.07
Generic Landuse 3*	N/A	0.20	0.09

**Generic land use is to be used in special cases where n-values do not fit within other land classifications*

TABLE 3-3

MAXIMUM VELOCITIES FOR CHANNEL DESIGN

Channel Description	Maximum Velocity (fps)
Channel	
Grass-Lined: Some Sand and/or Dispersive Clay	3.0
Grass-Lined: Mostly Clay	5.0
Riprap-Lined – Gradation 1	7.0
Riprap-Lined – Gradation 2	9.0
Articulated Concrete Block Lined	9.0
Concrete-Lined	11.0
Overbanks and Existing Natural or Overgrown Channels	Site Specific
Conduit	
Concrete Pipe or Box	8.0
Corrugated Metal Pipe	6.0

Source: Harris County Flood Control District. Policy and Criteria Manual, July 2019

TABLE 3-4

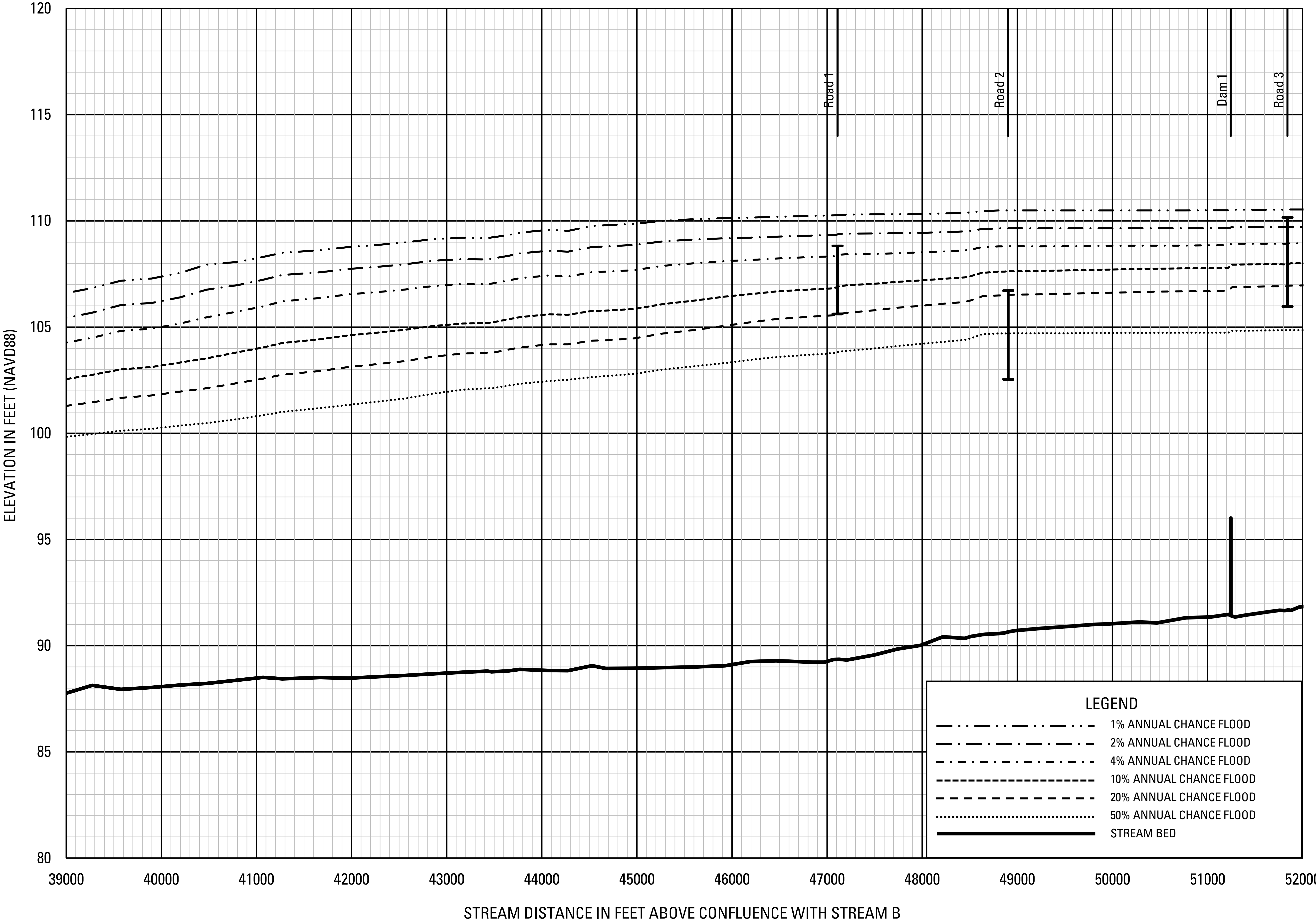
RIGHT-OF-WAY REQUIREMENTS FOR FORT BEND COUNTY, TEXAS [1]

Concrete lined, Grass or Earthen Channels		
Top Width of Channel TW	**Maintenance Berm on Each Side** [2]	**ROW**
TW ≤ 30 feet	15 feet	30 + TW
30 < TW ≤ 60 feet	20 feet	40 + TW
TW > 60 feet	30 feet	60 + TW

Notes:

(1) Right of Way listed in this table assume berm will be used for maintenance vehicles only. Additions of trails or tree buffers will require wider ROW and approval from FBCDD.

(2) Backslope swale not required for concrete lined channels.



FLOOD PROFILES

EXAMPLE CREEK

FORT BEND COUNTY MASTER DRAINAGE PLAN

FORT BEND COUNTY, TX

AND INCORPORATED AREAS

FIG. 3-1

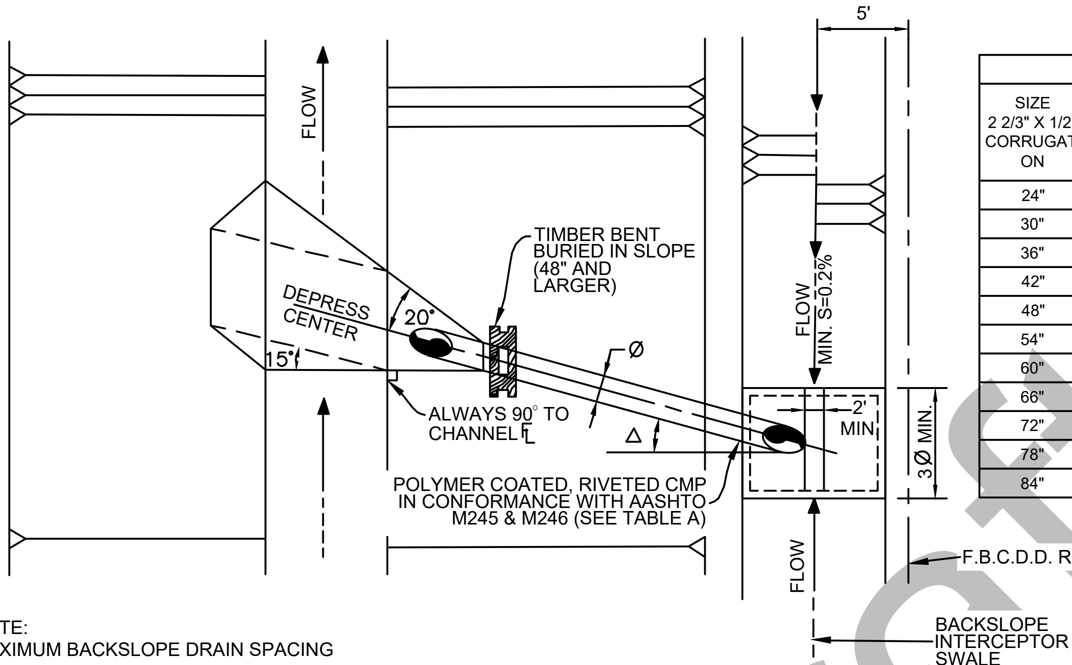


TABLE A					
SIZE 2 2/3" X 1/2" CORRUGATION	PIPE GAUGE	BAND COUPLER GAUGE	SIZE 3"X1" & 5"X1" CORRUGATION	PIPE GAUGE	BAND COUPLER GAUGE
24"	16	16			
30"	16	16			
36"	16	16			
42"	14	16			
48"	14	16	48"	16	18
54"	12	14	54"	16	18
60"	12	14	60"	16	18
66"	10	12	66"	16	18
72"	10	12	72"	16	18
78"	8	10	78"	14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

TABLE B		
PIPE DIA.	SLOPE	VELOCITY
24"	0.6%	3.25 F.P.S.
36"	0.3%	3.00 F.P.S.
42"	0.2%	2.75 F.P.S.
48"	0.2%	3.00 F.P.S.
54"	0.2%	3.25 F.P.S.

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

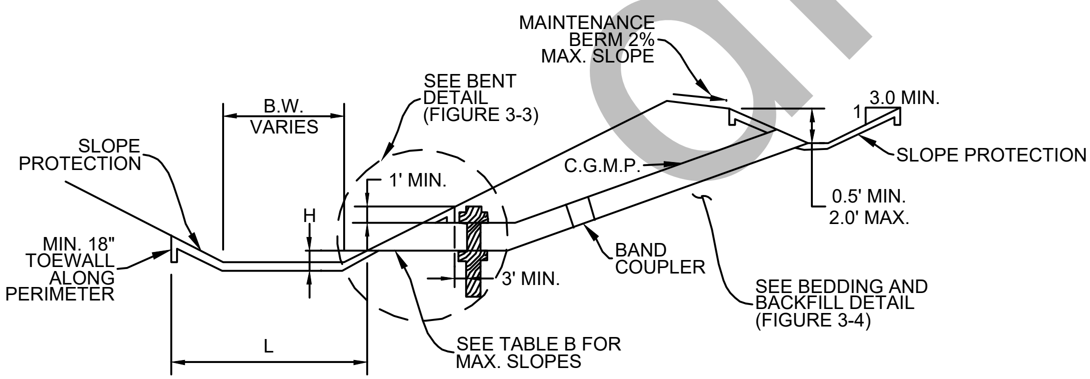
Δ: PROP. 24" TO 42" Δ= 15°
PROP. 48" AND LARGER Δ= 30°

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

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

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

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

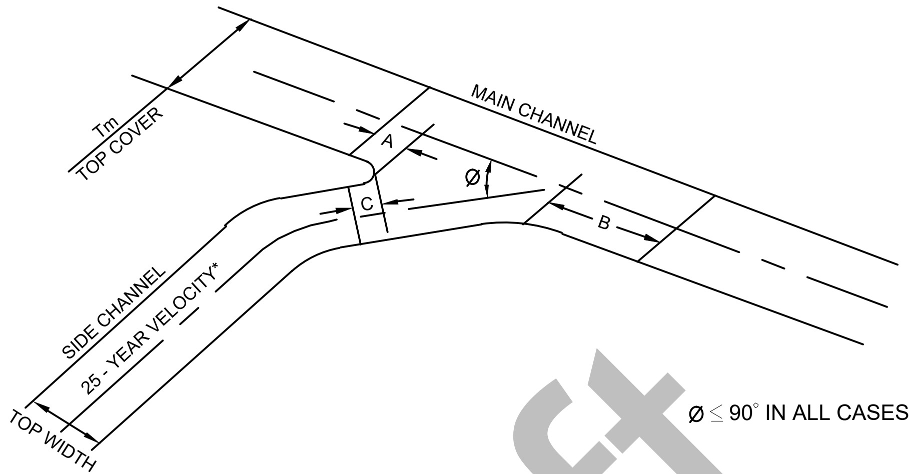


NOTE:
CONCRETE SLOPE PAVING SHALL HAVE A MINIMUM THICKNESS OF 4". MINIMUM REINFORCING STEEL SHALL BE #3 REBAR AT 18" 0.C. OR 6X6XW4.0XW4.0 WELDED WIRE FABRIC

REVISIONS 1. 7-13-1988
2. 5-20-1997
3. 12-01-2010

TYPICAL BACKSLOPE DRAIN DETAIL
FOR
FORT BEND COUNTY, TEXAS

APRIL 2022 | FIGURE 3-2



Ø ≤ 90° IN ALL CASES

MINIMUM EXTENT OF EROSION PROTECTION

LOCATION	DISTANCE (FT)
A	20
B	LONGER OF 50' OR 0.75 X Tm / Tan(Ø)
C	20

25-YEAR VELOCITY* IN SIDE CHANNEL (FEET PER SECOND)	ANGLE OF INTERSECTION Ø 15° - 45°	45° - 90°
4 OR MORE	PROTECTION	PROTECTION
2 - 4	NO PROTECTION	PROTECTION
2 OR LESS	NO PROTECTION	NO PROTECTION

* NOTE: 25 - YEAR VELOCITY IN SIDE CHANNEL ASSUMING NO BACKWATER FROM MAIN CHANNEL.

NOTE: EROSION PROTECTION MUST BE PROVIDED TO THE LEVEL OF THE 25-YEAR WATER SURFACE ELEVATION.

SOURCE: CRITERIA MANUAL FOR DESIGN OF FLOOD CONTROL AND DRAINAGE FACILITIES IN HARRIS COUNTY, TX, FEB., 1984.

REQUIRED EROSION PROTECTION AT CHANNEL CONFLUENCE	
APRIL 2022	FIGURE 3-3

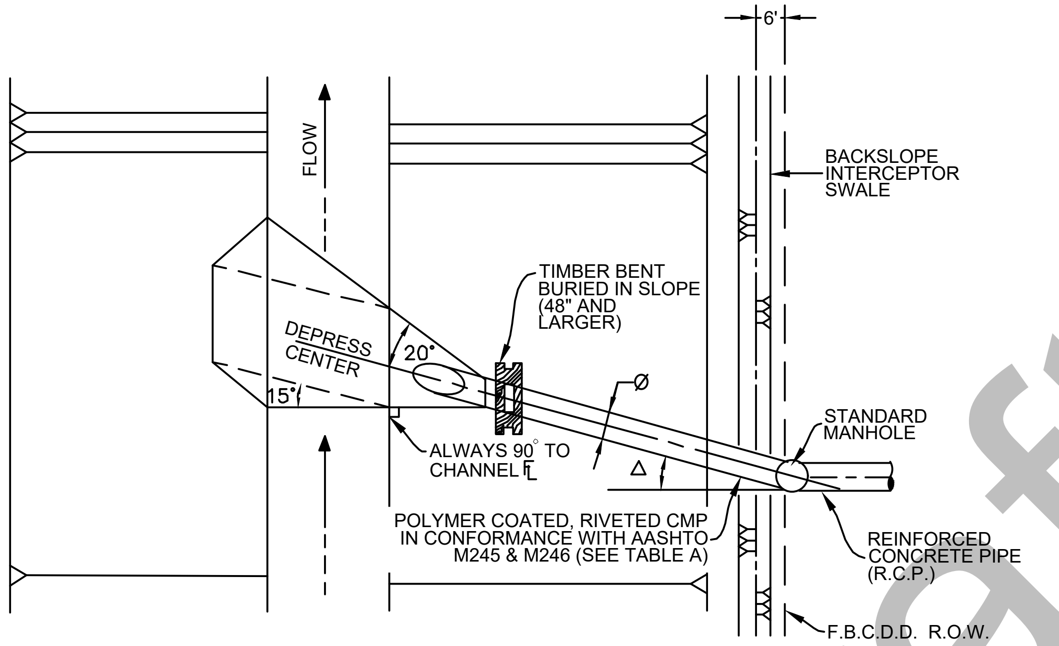


TABLE A					
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30"	16	16			
36"	16	16			
42"	14	16			
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54"	12	14	54"	16	18
60"	12	14	60"	16	18
66"	10	12	66"	16	18
72"	10	12	72"	16	18
78"	8	10	78"	14	16
84"	8	10	84"	14	16

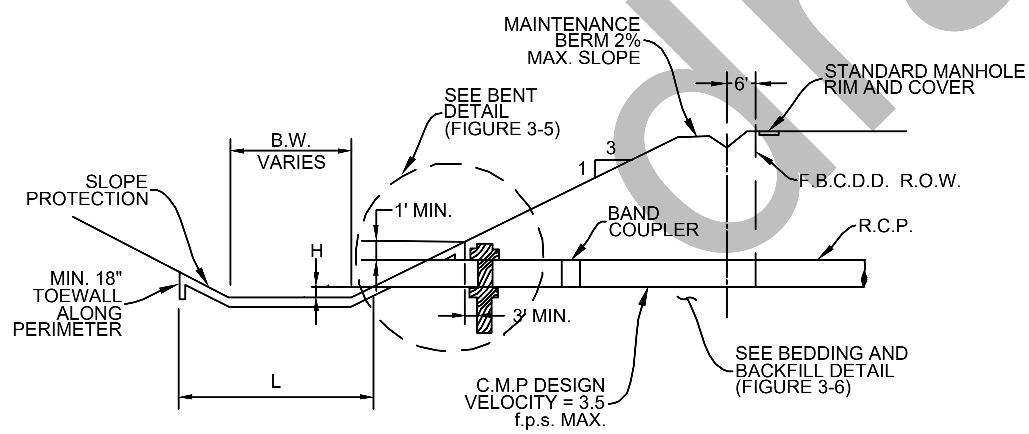
H: FOR PIPE SIZES 24" TO 42"
H=3' MAX. AND 1' MIN.

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

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

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

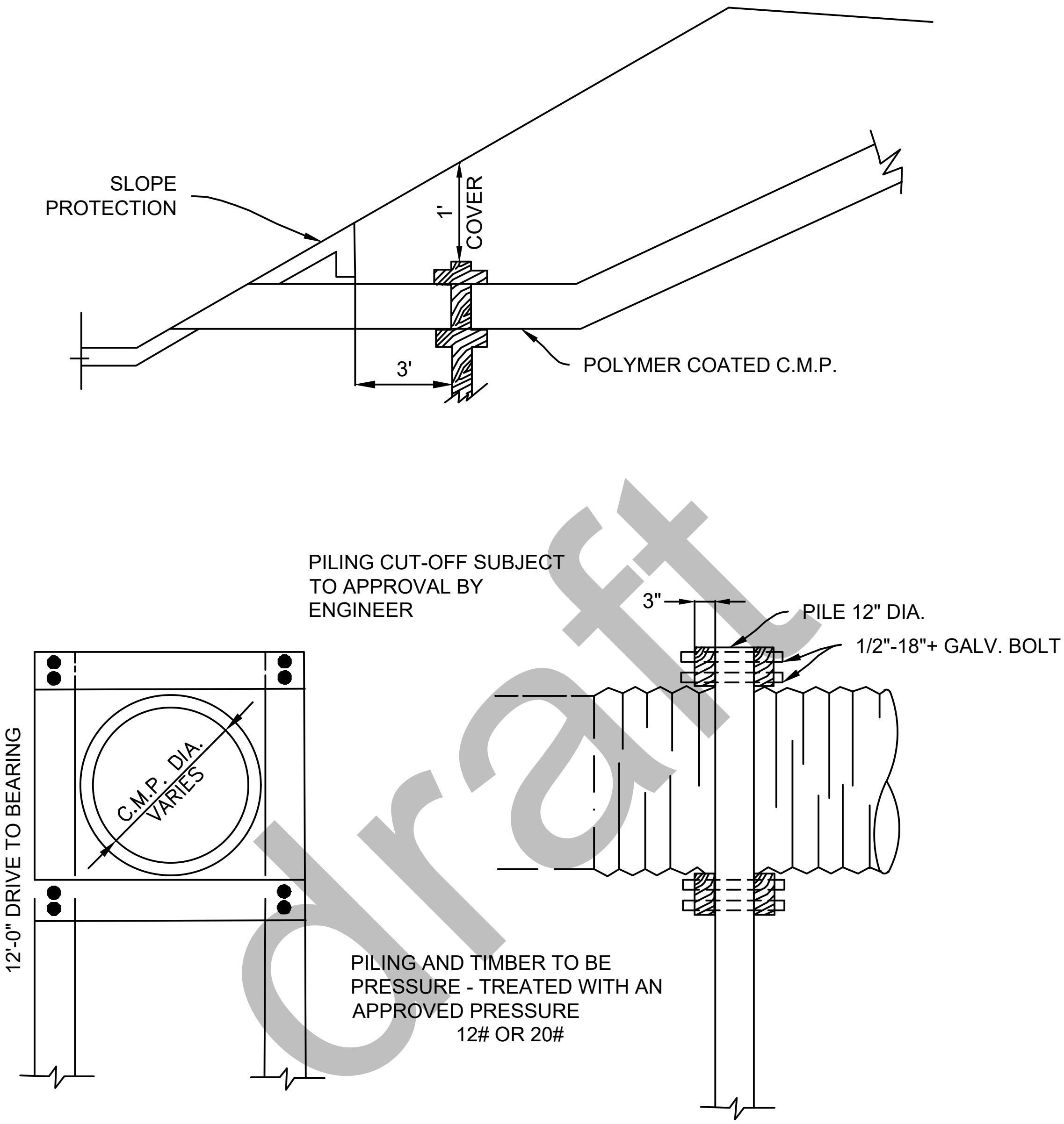
Δ: PROP. 24" TO 42"Δ= 15°
PROP. 48" AND LARGERΔ= 30°



NOTE:
CONCRETE SLOPE PAVING SHALL HAVE A MINIMUM THICKNESS OF 4". MINIMUM REINFORCING STEEL SHALL BE #3 REBAR AT 18" 0.C. OR 6X6XW4.0XW4.0 WELDED WIRE FABRIC

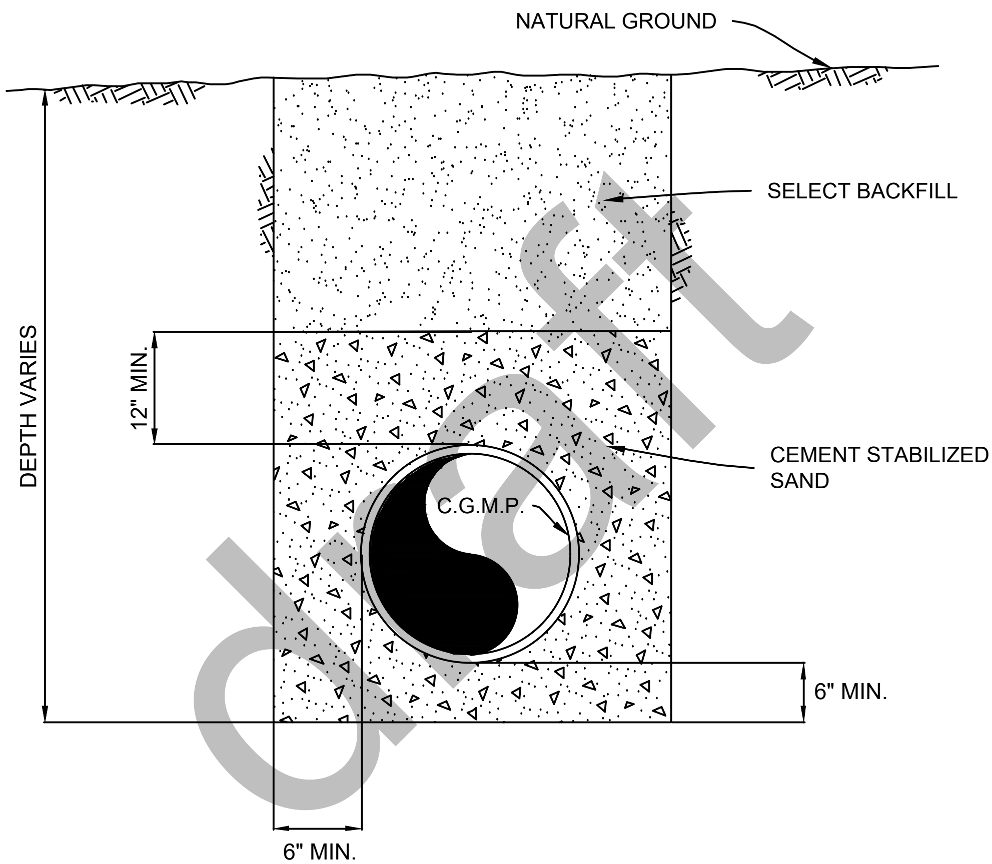
REVISIONS: 1. 7-13-1988
2. 5-20-1997
3. 12-01-2010

TYPICAL STORM SEWER OUTFALL DETAIL FOR FORT BEND COUNTY, TEXAS	
APRIL 2022	FIGURE 3-4



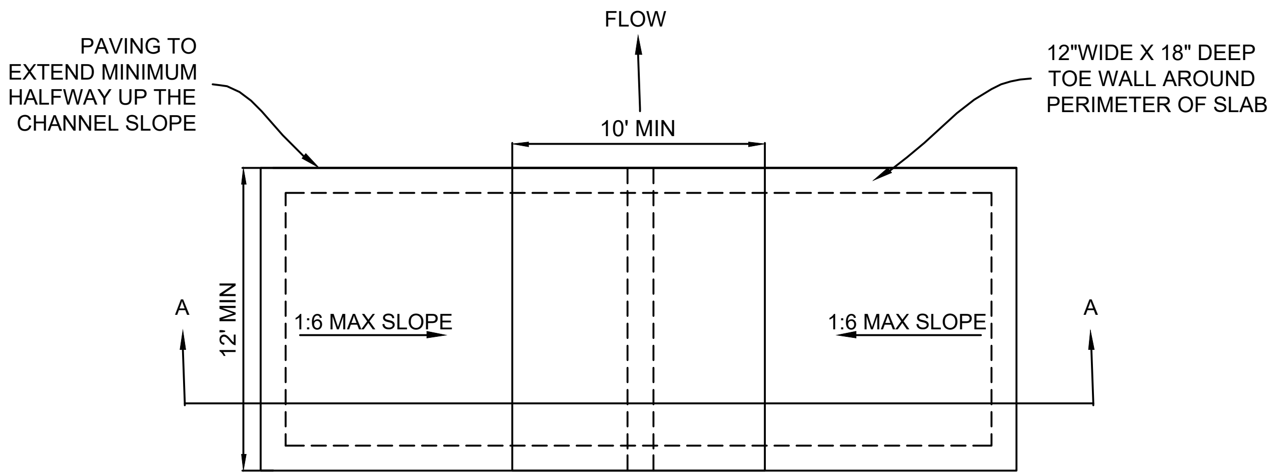
BENT FOR CORRUGATED METAL PIPE OUTFALL
48-INCH AND LARGER

TYPICAL BENT DETAIL FOR C.G.M.P. OUTFALL FOR FORT BEND COUNTY, TEXAS	
APRIL 2022	FIGURE 3-5

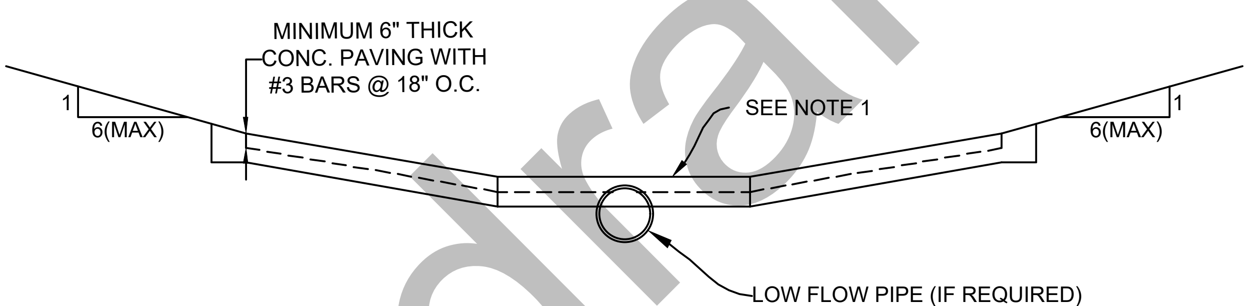


BEDDING AND BACKFILL DETAIL

TYPICAL BEDDING AND BACKFILL DETAIL FOR C.G.M.P. OUTFALL FOR FORT BEND COUNTY, TEXAS	
APRIL 2022	FIGURE 3-6



PLAN VIEW



SECTION A-A

MINIMUM LOW WATER CROSSING

NOTES:

1. TOP OF SLAB ELEVATION SHALL BE LOWER THAN THE PRE-EXISITNG CHANNEL FLOWLINE. IF THIS REQUIREMENT IS MET, LOW FLOW PIPE IS NOT REQUIRED.

2. LOW FLOW PIPE SIZE TO BE COORDINATED AND APPROVED BY FBCDD. MUST CONFIRM NO DECREASE IN CROSS SECTIONAL AREA OF CHANNEL.

3. LOW FLOW PIPE SHALL BE WRAPPED AND BEDDED IN MINIMUM 6" THICK STABILIZED SAND.

LOW WATER CROSSING STANDARD DETAIL	
APRIL 2022	FIGURE 3-7

TABLE OF CONTENTS

4.0 CULVERTS AND BRIDGES

4.1 GENERAL

The most economical mean of moving open channel flow beneath a road or railroad is generally with culverts. Discussion in this section will address procedures for determining an adequate culvert size and shape given a design discharge and allowable headwater elevation. The design procedures for the culverts referenced in this section pertain to those in the main channels as well as those in roadside ditches. In addition, this section includes a brief discussion of the hydraulic and hydrologic considerations pertinent to bridge design.

This Section applies to new development and to new crossings over channels designed per the requirements of this Manual. Existing culverts and bridges built prior to the adoption of this Manual may not meet the requirements of this Section, but this condition shall not be construed as a deficiency or deemed non-compliant. Compliance with design guidelines is based on the design criteria in effect at the time of the approvals for the construction of the bridge or culvert.

Upsizing existing culverts or bridgesthat reduce water surface elevations upstream of the crossing may result in increased conveyance and peaks flow downstream. All modifications to existing culverts or bridges shall result in no adverse impact both upstream and downstream. However, there could be exceptional conditions where the increased downstream flow can be accommodated in the channel or detention ponds, and the upsizing may be beneficial for an overall reduction in flood risk. Changes that would result in an increase in water elevations shall be coordinated with the Fort Bend County Drainage District Engineer early in the design process.

The Fort Bend County Drainage District Engineer requires all designs to accommodate ultimate development watershed conditions either for undeveloped flows to be detained downstream or after proper detention is provided. 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 documented in the drainage report and shown in the construction plans. The ultimate right-of-way is required even for an interim phase of construction.

New culverts and bridges or proposed changes to existing structures in identified areas of flood risk per Section 3.7 of this Manual are subject to hydraulic impact analysis to demonstrate the crossing will result in no increase in water surface elevations. The evaluation of the proposed crossing shall also account for the proposed roadway embankment. In these areas, the hydraulic design and analysis shall extend to the entire width of the floodplain and include elements to avoid obstructions to the flow in the floodplain such as relief culverts at appropriate intervals, mitigation for fill, channel improvements, wider or longer bridges, or other mitigation measures.

4.2 CULVERT DESIGN CRITERIA

4.2.1 Design Frequency and Freeboard

All new culverts in Fort Bend County (that are not culvert driveways in roadside ditches) shall be designed to pass the 100-year flow for fully developed conditions (either for undeveloped flows to be detained downstream or after proper detention is provided) without overtopping the road.

Driveways culverts shall be designed to pass the 5-year event or the bankfull capacity, whichever is less, without overtopping the driveway and do not have a minimum required freeboard. Refer to Section 5.5 for capacity requirements for new roadside ditches.

4.2.2 Culvert Alignment

It is recommended that alignment of roadways completed in the planning of the development consider the hydraulics of each crossing. Skewed crossings are discouraged because larger ineffective areas, added contraction and expansion losses, and erosion potential. If a skewed culvert cannot be avoided, the hydraulic analysis should properly account the effective flow area in the contraction and expansion reaches and reflect the actual hydraulic length of the culvert. Section 4.5 includes guidance for skewed crossings.

Culverts shall be aligned parallel to the longitudinal axis of the channel to ensure maximum hydraulic efficiency and minimum erosion. In areas where a change in alignment is necessary, the turn shall be made upstream in the natural channel and appropriate erosion protection shall be provided as described in Section 3.4.

4.2.3 Culvert Length

Culvert lengths shall accommodate future widenings at the discretion and request of the Fort Bend County Drainage District or the Fort Bend County Engineer. Accordingly, culverts length may need to account for proposed embankments, and culvert size may need to be determined based on the ultimate length of the culvert. If the roadway is proposed to be widened in phases, culvert calculations for each known phase of construction shall be presented in the drainage report and the design drawings.

Narrowing of the pavement width or roadway section at culvert crossings is not allowed. Culverts may not extend beyond the limits of the roadway right-of-way.

4.2.4 Headwalls

Headwalls and endwalls shall be utilized to control erosion and scour, to anchor the culvert against lateral pressures, to minimize hydraulic losses due to abrupt cross-sectional area changes, and to ensure bank stability. All headwalls shall be constructed of reinforced concrete and may be straight and parallel to the channel, flared or warped, with or without aprons, as required by site and hydraulic conditions. Protective guardrails shall be included along culvert headwalls. Table 4-1 provides general guidelines for choosing headwall geometric configuration.

4.2.5 Minimum Culvert Sizes

For driveway culverts at roadside ditches, the minimum diameter is 18 inches. For all other channels, 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 intended to minimize incidental flow obstruction.

4.2.6 Manning's "n" Values

The Manning's "n" value to be used in the hydraulic calculations shall be the same used in storm sewer calculations listed in Table 5-1. The open channel portion of the model shall use the Manning's "n" values from Tables 3-1 and 3-2.

4.2.7 Erosion

Flow velocity is generally higher in a culvert than in the adjacent natural channel due to the contraction effect. For this reason, the tendency for erosion must be addressed at both entrances of the culvert. Proper erosion protection for both entrances must be provided following the requirements of Section 3.4. Velocities greater than 9 fps require concrete lining. Entrance or exit velocities in excess of 11 fps are not permitted without energy dissipator structures.

4.2.8 Structural Requirements

The structural design of the culvert shall meet the following requirements:

1. All precast reinforced concrete pipe shall conform to ASTM C76.

2. All precast reinforced concrete box culverts shall conform to ASTM C1433.

3. All corrugated metal pipes shall conform to ASTM A760.

4. Culverts should be designed to resist the anticipated load for the crossing with adequate factors of safety. The minimum design load shall be ASSHTO HS20-44.

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

6. Joint sealing material for precast concrete culverts shall comply with ASTM C990, Standard Specification for Joints for Concrete Pipe, Manholes, and Precast Box Sections Using Preformed Flexible Joint Sealants.

7. 6-inch thick bedding layer of stabilized sand is required for all precast concrete box culverts. Bedding material shall conform to TxDOT Technical Specifications (Item 400, Section 3.3.4) or City of Houston's Technical Specification 02321.

4.3 CULVERT HYDRAULIC DESIGN

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

4.3.1 Culvert Design Methodology

The hydraulics of the culvert shall be analyzed with either HEC-RAS or the Federal Highway Administration's Culvert Hydraulic Analysis Program HY-8. Use of other programs shall be approved by the Fort Bend Drainage District Engineer. The preferred modeling software is HEC-RAS. HY-8 is a culvert analysis software, not a backwater analysis model, and its use is acceptable only for simple crossings where all the following conditions are met:

1. The drainage area is 100 acres or less, and there are no significant increases in drainage area (outfalls, stream confluences) 800 feet downstream of the culvert;
2. All barrels in the crossing are of the same size, shape, and material;
3. All barrels in the crossing have the same flow line upstream and downstream (or the difference in flow lines between barrels is negligible so they can be assumed to be equal);
4. Backwater effects from another downstream crossing, confluences, or hydraulic structure are not present;
5. The downstream boundary condition can be represented with a regular channel section, a simplified irregular section, or a rating curve; and
6. The culvert is not subject to Flood Mitigation Requirements per Section 3.7 and therefore, impacts to the upstream water surface elevation as a result of the culvert do not need to be calculated.
7. The design of a new crossing should be consistent with the sizes of nearby existing crossing both upstream and downstream. For example, a new crossing should not be smaller than an existing upstream crossing unless downsizing is properly justified.

If the only objective of the model is to verify the hydraulic capacity of the culvert, analysis shall be completed with steady-state flow using the peak discharges during the design storm. Peak discharges shall be calculated with the appropriate methodology for the size of the watershed following the guidance of Section 2.0. The open channel portion of the analysis shall follow the guidance of Section 3.0 of this Manual.

If the culvert location is subject to the Flood Mitigation Requirements per Section 3.7, a hydraulic impact analysis is to be completed following the requirements of Section 3.7.

If the culvert is part of a hydraulic model developed to analyze a channel or floodplain to meet the requirements of other sections of this Manual, then the results of that hydraulic model (unsteady-state flow or 2D modeling) related to the culvert capacity (design frequencies, headwater, flow, and outlet velocity) shall be documented in the drainage report and the design sheets of the culvert.

4.3.2 Culvert Flow Types

The hydraulic capacity of a culvert is defined as inlet-controlled or outlet-controlled. Inlet control means that the discharge through the culvert is limited by the hydraulic and physical characteristics of the inlet alone. These characteristics include barrel shape, barrel cross-sectional area, and the type of inlet edge. When the culvert is inlet-controlled, the barrel has supercritical flow (either in its entire length or in the upstream end with a hydraulic jump) and changes in the conditions downstream of the culvert do not impact its capacity (as long as the flow in the barrel remains supercritical). In outlet control conditions, the discharge capacity of the culvert is dependent on all the hydraulic variables of the structure. These include headwater depth, tailwater depth as well as barrel shape, cross-sectional area, barrel roughness, slope, and length.

Due to the flat terrain in Fort Bend County, most of the culverts are outlet-controlled with the downstream tailwater condition being a relevant parameter in determining the capacity of culverts. Even culverts calculated to be inlet-controlled can become outlet-controlled with a drastic increase in the downstream tailwater depth. For these reasons, special attention is required to obtain good estimates of the downstream boundary conditions. See Section 4.3.3 includes considerations to determine tailwater conditions.

4.3.2.1 Inlet-Controlled Flow

Under inlet control, the culvert entrance may or may not be submerged. However, in all cases, inlet-controlled flow through the culvert barrel is free surface flow. When the culvert inlet is submerged and the culvert remain inlet-controlled, the capacity of culvert is calculated with equations similar those used for sluice gates.

4.3.2.2 Outlet-Controlled Flow

Outlet-controlled culverts will operate with the barrel full or partially full for part or all the barrel length. The tailwater may or may not submerge the culvert. Figure 4-1 represents the various hydraulic elements of flow through a culvert.

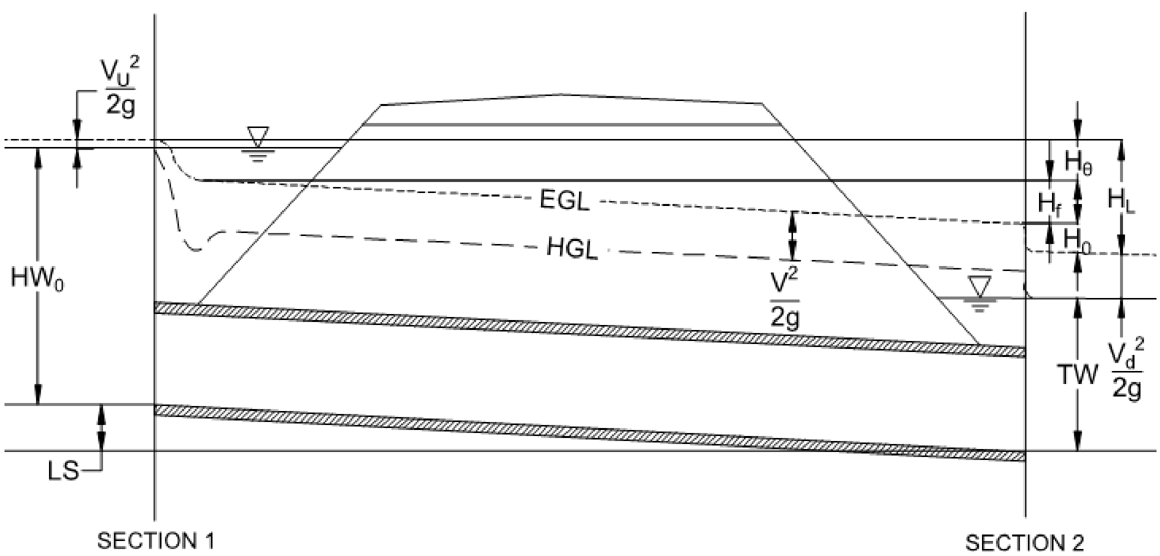


Figure 4-1 Hydraulic Elements of Flow through Culvert
(Adapted from Hydraulic Design of Highway Culverts, Federal Highway Administration, 3rd Edition, April 2012)

If the culvert is flowing full, the energy required to pass a given quantity of water is stored in the head (H). From energy considerations it can be shown that H is the difference between the energy line at the outlet and the energy grade line at the inlet. Due to low velocities in most entrance pools and the difficulty in determining velocity head in any flow, the upstream water surface elevation (HW_0) can be assumed to be coincident with energy line. Figure 4-1 reveals the following relationship for full flow conditions:

$$H = HW_0 + \frac{V^2}{2g} + LS_0 - TW \qquad (4\text{-}1)$$

Where HW_0 = Upstream headwater

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

L = culvert barrel length (feet)

S_0 = slope of the barrel in ft/ft

TW = tailwater depth

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

$$H = H_e + H_f + H_v \qquad (4\text{-}2)$$

The entrance loss (H_e) is expressed in terms of the velocity head multiplied by an entrance loss coefficient k_e. The velocity head (H_v) is equal to $V^2/2g$ where V is the mean velocity of flow in the barrel. An expression for the friction loss (H_f) is derived from Manning's equation:

$$H_f = \frac{29\,L\,n^2V^2}{2g\,R^{1.33}} \qquad (4\text{-}3)$$

Where n = Manning's roughness coefficient

L = culvert barrel length (feet)

R = the hydraulic radius (feet)

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

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

Rearranging Equation 4-1 and replacing Hv and He in terms of the velocity head, it is seen that for full flow conditions:

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

Rearranging Equation 4-1, the following expression for HW_0 is derived:

$$HW_0 = H - \frac{V^2}{2g} - LS_0 + TW \qquad (4\text{-}5)$$

When the culvert outlet is submerged by the tailwater, the above equation can be solved directly to determine HW_0. However, when the tailwater is below the crown of the culvert, the barrel behaves like an open channel subject to a backwater effect. In this case, the headwater HW_0 is found with the energy loss across the barrel in the same way a backwater analysis is performed. As water elevation in the barrel increases traveling upstream, the culvert may or may not become pressurized, and if it does, calculations of pressurized flow are applicable for the pressurized portion of the barrel. Computer programs like HEC-RAS or HY-8 calculate the headwater HW_0 for given the hydraulic characteristics of the culvert and tailwater conditions.

4.3.3 <u>Tailwater Depth</u>

As noted in Equation (4-5), an increase in tailwater when the culvert is outlet-controlled results in an increase in the upstream headwater. Because headwater is dependent on downstream conditions, the engineer shall obtain and consider in the design the hydraulic conditions downstream of the crossing that may influence the tailwater depth downstream of the culvert. Such conditions may include other culverts, bridges, water level fluctuations in ponds or reservoirs, hydraulic structures (weirs, drop structures, etc.), and channel expansion and contractions among others.

Analysis shall extend past 800 feet downstream of the culvert to estimate backwater impacts from any structure within that distance. If there are no downstream structures within 800 feet, the tailwater depth depends on the channel characteristics downstream of the culvert. In this case, the tailwater depth may be assumed to be the normal depth in the downstream channel. Then, the channel hydraulic parameters such as slope and Manning's "n" used in normal depth calculations should be properly defined and documented. It is recommended that engineer completes a sensitivity analysis of the hydraulic characteristics of the channel as part of the tailwater evaluation and the culvert design, and documents the findings in the drainage report and the design sheets of the culvert. Parameters to be included in the sensitivity analysis include Manning's roughness coefficient and channel slope.

4.3.4 Headwater Depth

In all culvert design, headwater, or depth of ponding at the entrance to the culvert, is an important factor in culvert capacity. The headwater depth (HW) is the vertical distance from the culvert entrance invert to the energy line of the approaching flow. As noted in Section 4.3.2.2, the water surface elevation shall be assumed to be coincident with energy line. Freeboard calculations shall be based on the energy line. Resulting headwater depth and associated headwater elevation should be clearly listed in the drainage report and the design sheets of the culvert.

4.3.5 Conditions at Entrance

Culvert performance is significantly affected by inlet efficiency, especially for conditions of inlet-controlled flow. Changes in the culvert edge geometry can significantly change discharge capacity. Selection of an inlet type is contingent on the relative weightings the engineer assigns to the effect on peak flows, cost, and topography. In other words, the ideal inlet geometry is not necessarily the most efficient. Both HEC-RAS and HY-8 have a library on inlet types that can provide information in selecting loss coefficients for the culvert calculations. The library and coefficients are based on the Manual for Hydraulic Design of Highway Culverts by the Federal Highway Administration. The engineer shall refer to this Manual and the User's Manual of HEC-RAS and HY-8 for the proper selection of the inlet condition in the model.

4.4 BRIDGES

4.4.1 Bridge Design Considerations

Bridges shall be designed to pass the 100-year design flow without causing adverse impacts to the floodplain or erosive conditions in the channel or detention basin. For new bridges, the low chord (at the center of the bridge) must be at least 1.5 feet above the 100-year water surface elevation. At no point shall the low chord of the new bridge be less than one foot above the 100-year water surface elevation.

Bridge lengths must be sized to accommodate future channel widenings at the discretion and request of the FBCDD or the Fort Bend County Engineer. Energy losses due to flow transitions shall be minimized. In addition, provision shall be made for anticipated future channel enlargements. The design engineer shall coordinate with the Fort Bend County Drainage District Engineer for determination of the full ROW width to be utilized. Right-of-Way requirements are shown in Table 3-3 of this Manual.

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

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

4.4.2 Bents and 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, should be eliminated entirely from the channel bottom.

4.4.3 Interim Channels

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

4.4.4 Erosion Protection

Increased turbulence and velocities associated with flow in the vicinity of bridges requires the use of erosion protection in affected areas. Requirement for erosion protection location and design are found in Section 3.5.

Scour protection measures are required in all areas where scour is anticipated. The potential scour depth can be calculated utilizing either HEC-RAS scour module or Hydraulic Engineering Circular No. 18 (HEC-18) entitled "Evaluating Scour at Bridges" by the Federal Highway Administration. The hydraulic engineer shall coordinate with the geotechnical engineer the soil information and parameters required to complete the scour analysis such D50, soil classification, and others.

4.5 HEC-RAS CULVERT AND BRIDGE MODELING

All bridge analyses are to be computed in HEC-RAS version 5.0.7 (or newer), This modeling software is preferred because it has options to model both pressure flow and open channel flow for bridges and culverts, provides options for several methods (energy and momentum) and is well documented. Versions of HEC-RAS shall be consistent throughout each project.

Modeling of bridges and culverts in HEC-RAS shall conform to the following guidelines, which are based on the HEC-RAS Hydraulic Reference Manual:

1. The bridge deck/roadway should be defined at the centerline of the road. The roadway centerline shall extend the entire width of the roadway cross-section, including sags along the profile.

2. Bounding cross-sections upstream and downstream are to be aligned along the toe of the roadway embankment, the location where flow encroachments limit the hydraulic capacity cross-sectional area and should not include the roadway embankment.

3. Two additional bounding cross-sections should be placed upstream and downstream of initial bonding cross section assuming proper channel expansion and contraction ratios (width of channel to length of channel). Recommendations for contraction and expansion ratios based on site-specific hydraulic parameters are listed in the HEC-RAS Hydraulic Reference Manual.

4. Abutment information should be input using the Sloping Abutment Data Editor, instead of the bounding cross sections or the Deck/Roadway Editor.

5. Ineffective flow areas should be located to model the contraction of the flow as it approaches and leave the bridge. The depth and width of the ineffective flow areas varies based on engineering judgement of the design engineer. Horizontally, the flow contracts from the channel width to the edges of the bridge opening and the ineffective area is considered to be permanent upstream of the bridge. Vertically, the effective area is below the low chord. However, once the road overtops, flow over the roadway is considered effective at the lowest elevation along the bridge deck and the entire channel downstream becomes effective. For this reason, ineffective flow areas downstream are considered non-permanent. The elevation at which the downstream flow becomes effective may not be centered over the channel section if a roadway sag elevation in the overbanks is lower than the bridge deck elevation.

6. For 1D steady-state models, expansion and contraction coefficients shall be 0.3 and 0.5, respectively, for all cross-section from the section downstream of the point of flow encroachment and continue to the cross-section where flow has transitioned back to the full-flow cross-section.

7. For bridges skewed to the normal direction of flow, all geometric input shall be adjusted to represent the values calculated with the impact of the skew, including culvert or bridge opening widths and pier locations.

8. When it is anticipated that a bridge will be overtopped by discharges contained within the model, High Flow Methods should be selected and proper parameters entered in the Bridge Modeling Approach Editor.

9. Skew bridge crossings are generally handled by adjusting the bridge dimensions to reflect an equivalent bridge section perpendicular to the flow. As noted in Figure 4-2, the hydraulic opening perpendicular to the flow (W_B) is narrower than the bridge span (b). The length of the bridge deck and the width of the piers should also be adjusted to model the equivalent distances that are perpendicular to the flow. The skewed-bridge option in HEC-RAS can be used, but the Engineer is encouraged to refer to the

documentation available in the Hydraulic Reference and Users Manual to understand the changes made to geometry and the limitations.

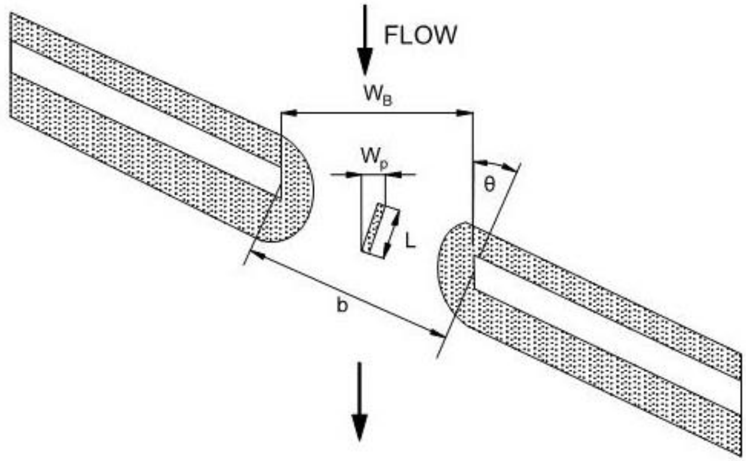


Figure 4-2. Geometry of a Skewed Bridge.
(Adapted from HEC-RAS Hydraulic Reference Manual, January 2010)

10. Tailwater conditions (Section 4.3.3) shall be considered in the modeling of bridges.

The Fort Bend County Drainage District will make available the most recent update of the models developed as part of the Fort Bend County Master Plan and a digital version of a County-Wide ponding map for use by designers and engineers to comply with the requirements of this Section. Where Fort Bend County Master Plan models exist, they must be used for submittals to the Fort Bend County Drainage District regardless of the availability of an effective FEMA model. FEMA submittals must be based on the FEMA effective models in addition to the modeling required with the Fort Bend County Master Plan models. Changes to the FEMA effective model should reflect the project as approved by the Fort Bend County Drainage District with the Master Plan models.

The Fort Bend County Drainage District Engineer may require conversion from any other model into HEC-RAS version 5.0.7 or later format. Models other than HEC-RAS for bridge analysis require approval from the Fort Bend County Drainage District Engineer.

4.6 SPECIAL CROSSINGS STRUCTURES

4.6.1 Inverted Siphons

Inverted siphons are underground closed conduits used to convey water under existing utilities, railroads or other obstacles where placement of a culvert following the slope of the channel is not feasible. All other alternatives should be considered, and siphons are relegated as the last resort when all other alternatives are less favorable. This type of crossing is only allowed with approval from the Fort Bend County Drainage District Engineer and shall be maintained by a perpetual agency with drainage and maintenance responsibilities other than FBCDD.

Siphons convey water by gravity running full and under pressure. Then, the hydraulics and design consideration follow the principles of pressurized conduits. The following are guidelines for the design of siphons:

1. All siphons under roads and utilities shall have a minimum of five feet of cover, excluding the pavement. Siphons crossing concrete lined canals shall have a minimum of one-foot cover between the canal lining and the top of pipe. For crossings under railroads, the Engineer shall consult with the railroad owner to ascertain the minimum cover requirements.

2. The minimum size of the siphon shall be 30 inches and the minimum pipe slope in the downstream direction shall be 0.5%.

3. Given the low-lying profile of siphon piping, it is important to carefully consider what provisions are necessary to prevent trash, debris, and sediment from clogging the system. Opportunities for mitigating this concern include, but are not necessarily limited to: a trash rack system affixed to reinforced concrete end treatment of the siphon piping, constructing a reinforced concrete headwall structure and vault to situate the piping parallel to the direction of flow, and additional pipes to provide redundancy in the case that one pipe becomes non-functional due to maintenance or blockage.

4. Conduit velocities during the design event shall be between 3.0 and 10 feet per second. This reduces stagnation and deposition of sediments and limits erosion potential at the

outfall.

5. Careful consideration should be given to what form of transition grading and pipe end treatment is appropriate for the siphon inlet and outlet.

6. The siphon shall contain elements to allow for operations and maintenance activities (e.g., dewatering, cleaning, inspections, etc). These elements shall be able to stop the flow from both upstream and downstream ends with slide gates and/or stop logs so that dewatering can take place. The design shall allow to deploy all flow blockage elements (slide gates and stop logs) at the same time.

7. Access ways shall be located within the channel right-of-way (as opposed to the utility right-of-way) and be wide enough to allow for personnel entry.

8. The Engineer shall explore inlet and outlet configurations which incorporate reinforced concrete structures. Implementing reinforced concrete structures introduce the following opportunities:

    i. Training walls can be erected, inclusive of stop log slots.

    ii. Structures could be configured to include a lower box (vault) commensurate with the profile of the siphon piping. The vertical wall of the vault could provide a means to mount a slide gate (and frame) operable from the top of bank.

    iii. Vertical walls of the access vault could function as end treatment for the pipes, which eliminates the need for separate inlet and outlet structures.

9. If an access structure is not feasible, both inlet and outlet shall have configurations that, at a minimum, reduce head losses and prevent erosion. This is can be done by gradually contracting the channel at the upstream end and expanding the siphon outlet at the downstream end. The transition shall result in a gradual change from the channel slope to the pipe slope. Concrete slope paving of the transitions is required to reduce erosion. Rip-rap protection is limited only where concrete paving is not feasible.

6. The length of the siphon may need to be extended beyond the conflict being crossed and beyond the ultimate ROW held by others (for example, to account for future roadway expansions, future utility work, etc.). The Engineer shall coordinate with the appropriate stakeholders the need for lengthening the siphon system to avoid future modifications as a result of future projects by others.

7. Head loss calculations across the siphon shall consider entrance and outlet losses, energy losses at pipe runners, friction losses, and energy losses across vertical and horizontal bends.

8. Inlet and outlet ends shall be provided with horizontal safety pipe runners with a clear spacing between pipe no greater than 30 inches.

9. Pipe shall be structurally designed with material and bedding adequate for the anticipated loads.

10. Given the depth requirements associated with inverted siphons (i.e., cover depth, promotion of positive drainage, etc.), the Engineer shall be responsible for understanding the native soils and associated groundwater table. The design of the siphon and associated concrete structures shall be able to withstand uplift buoyancy pressures.

11. Provisions for deterring public entry, as well as to safeguard maintenance personnel, are warranted at all stormwater siphon facilities. This could necessitate features such as: access gates, perimeter fencing, signage, and safety railing.

12. The Engineer shall consider perched water conditions that may be encountered by the Contractor during construction. A thorough understanding of potential water encounters shall be based upon experience with similar projects, topography, field surveys, and geotechnical field exploration and analysis. Mitigation options for perched water conditions include: the continual conveyance of water in the channel, the control of surface water, control of groundwater, and control of any existing or proposed bypass piping or channels.

13. The Engineer shall coordinate with all applicable agencies the permissible pipe installation method. Typical pipe installation methods include but are not limited to open-cutting (direct bury) and direct jacking.

A typical design of an inverted siphon is illustrated in Figure 4-1.

TABLE 4-1

HEADWALL GUIDELINES

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

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

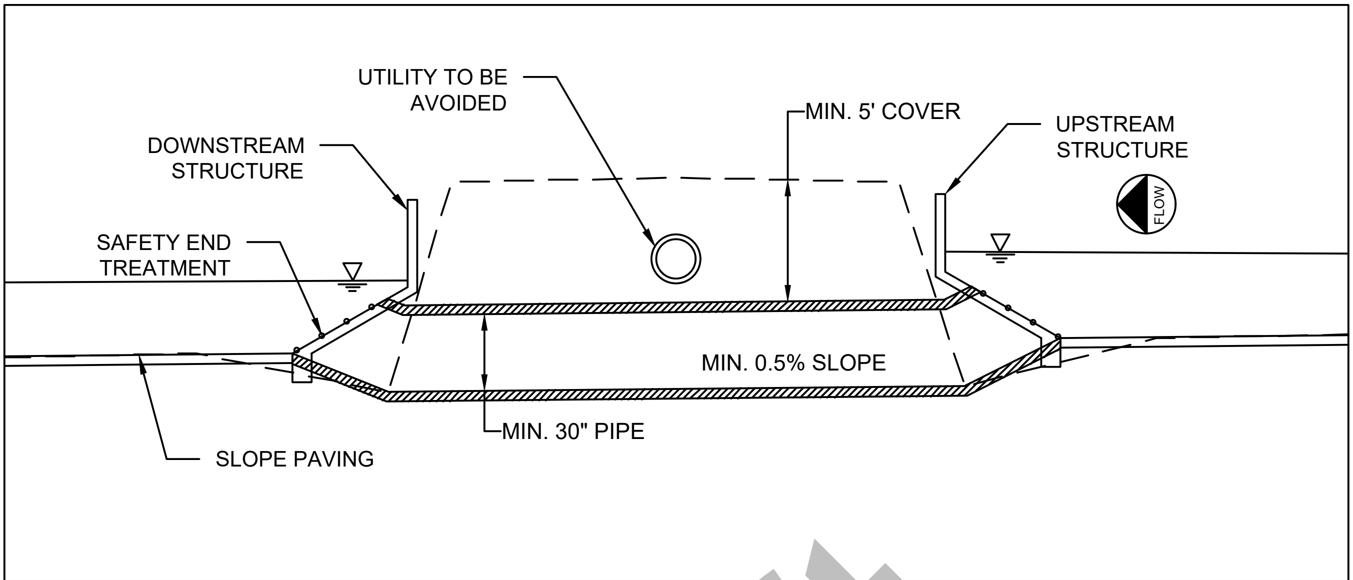
Flared Headwall and Endwall
1. Channel is well defined.
2. Approach velocities are greater than 6 fps.
3. Medium amounts of debris exist.

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

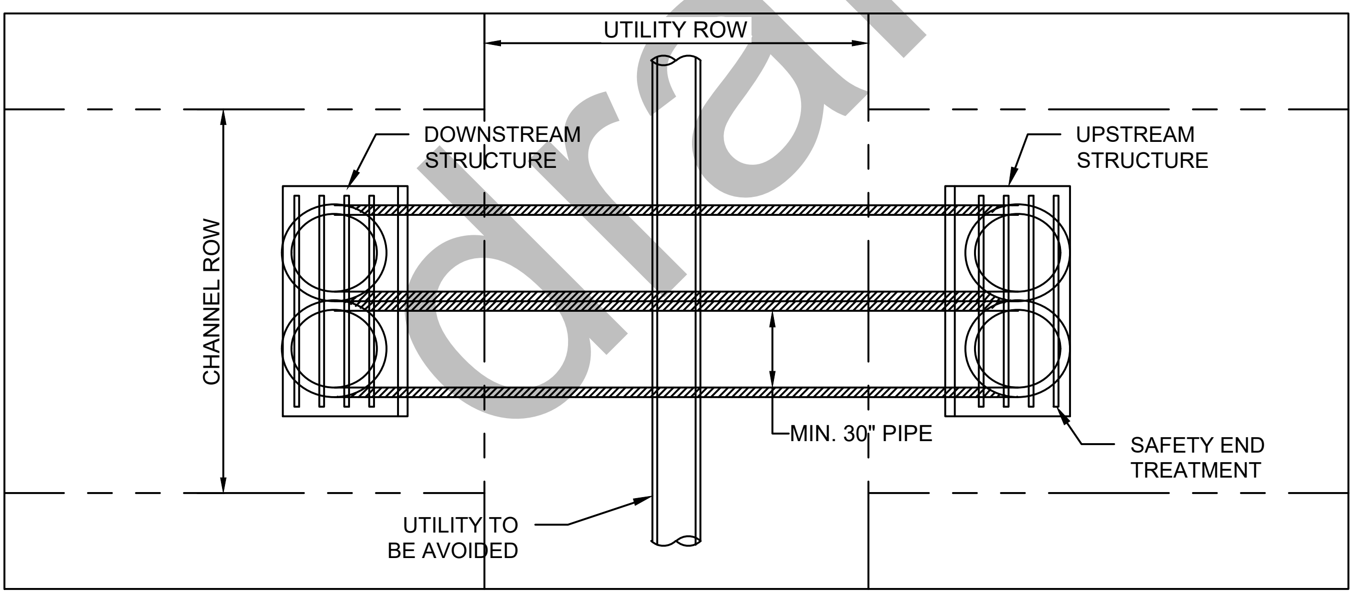
Warped Headwall and Endwall
1. Channel is well defined and concrete lined.
2. Approach velocities are greater than 8 fps.
3. Medium amounts of debris exist.

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

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



PROFILE VIEW



PLAN VIEW

NOTES:

(1) NOT DEPICTED IN THIS FIGURE, SIPHON IS TO INCLUDE THE FOLLOWING:

A) STRUCTURES FOR ACCESS, INSPECTION AND MAINTENANCE ON BOTH UPSTREAM AND DOWNSTREAM ENDS, WIDE ENOUGH TO ALLOW FOR PERSONNEL ENTRY.

B) ABILITY TO DEWATER THE SIPHON BY BLOCKING THE FLOW ON BOTH ENDS WITH SLIDE GATES AND STOP LOGS.

(1) ACCESS (NOT SHOWN IN FIGURE) COULD BE PROVIDED WITH A LOWER VAULT COMMENSURATE WITH THE PROFILE OF THE SIPHON. A VERTICAL WALL CAN BE USED TO MOUNT A SLIDE GATE AND FRAME OPERABLE FROM THE TOP OF BANK.

(2) ACCESS (NOT SHOWN IN FIGURE) SHALL BE CONSTRUCTED WITHIN THE CHANNEL R.O.W. AND COULD ADDITIONALLY FUNCTION AS END TREATMENT OF PIPES, ELIMINATING THE NEED OF THE STRUCTURES SHOWN.

INVERTED SIPHON DETAIL FOR FORT BEND COUNTY, TEXAS	
SEPTEMBER 2021	FIGURE 4-1

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5.0 STORM SEWERS AND OVERLAND FLOW

5.1 GENERAL

Due to the flat terrain in Fort Bend County, it is infeasible for certain areas to convey the runoff from extreme rainfall events entirely via an underground storm sewer system. Local flooding will occur in areas away from the primary drainage channels because it is uneconomical to provide a storm sewer pipe large enough to carry extreme storm events. For this reason, a sheet flow analysis is required so that street design and alignment assure that excess runoff from extreme storm events will be conveyed to a detention pond or primary channels safely. The development or project is not allowed to increase the water surface elevation or flows into a receiving channel, ditch or drainage system and accordingly, adequate mitigation shall be required along with all supporting calculations. Sheet flow corridors shall be designated, extreme event swales shall be provided, and all required right-of-way shall be established. Special consideration must also be given for off-site sheet flows and their impacts on planned developments and surrounding properties.

The discussion presented in this Section is centered primarily on curb-and-gutter streets with underground storm sewers. Roadside ditch systems for new development are acceptable in certain instances but are not preferred. Roadside ditches calculations are based on the guidelines of Section 3.0, Open Channels and shall be designed to meet the criteria of Section 5.5.

5.2 RUNOFF ANALYSES

5.2.1 Frequency Considerations

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

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

Figure 5-1 illustrates the hydraulic grade line of a storm sewer for two conditions. Assuming the receiving system is at the design tailwater condition, Part A of Figure 5-1 shows that the hydraulic grade line for the standard design condition remains at or below the gutter level at all inlets. For this condition, there is no street ponding and the storm sewers are functioning at or below their maximum capacity. Part B of Figure 5-1 shows the case where the tailwater condition is higher than the design level of Part A. Street ponding begins to occur throughout the storm sewer drainage system as the storm sewers are unable to operate at their design capacity. This local flooding situation could also occur when the tailwater is below design conditions if local rainfall is in excess of that used in the design of the storm sewer system. As this widespread street ponding starts to occur, provisions must be made to route the excess runoff to a detention pond or channel and minimize the risk of structural flooding.

5.2.2 <u>General Design Guidelines</u>

1. Storm sewers shall be designed to carry the design storm peak flow (See Section 5.2.3 for design frequency). To obtain the design storm peak flow, the Rational Method can be used for drainage areas less than 200 acres. To obtain peak flows for larger drainage areas, hydrologic modeling with the Basin Development Factor in HEC-HMS shall be used. A detailed description of these techniques is provided in Section 2.0.

2. Ponding elevations for storms up to the 100-year event shall not exceed 1.0 foot above the lowest top of curb in front of the lot. Top of slab elevations shall be at least 2.0 feet above the lowest top of curb in front of the lot. (If these two requirements are met, the slab elevation is at least 1.0 foot above the maximum ponding.) Figure 5-2 illustrates the maximum ponding elevation in relation to the top of curb and top of slab elevation. Refer to Section 5.5.2 for the same criteria for roadside ditches.

3. Fill and grading shall be limited to the parcel of the project. Agreements with adjoining property owner(s) shall be required if these activities cross the property line.

4. The grading of the development or lots shall conform to the project plans and drainage area boundaries established in the approved master drainage report for the development and direct the runoff to a designated drainage system and storm water detention facilities that has been designed to convey the project flows.

5. The grading of the site or lots shall be from the back of the lots or development to the front of the lots adjacent to a public right-of-way as shown in Figure 5-3. The portion of the parcel that can drain to the back of lot (the side opposite to the public right-of-way) shall be no greater than a 14-foot wide buffer. For large sites where this requirement is not practical, the Engineer shall consult with the Fort Bend County Drainage District Engineer for an exception.

6. When filling lots adjacent to a channel or detention pond, a transition from the back of the lot to natural ground at the channel or detention pond right-of-way must occur and shall not hinder maintenance operations within the right-of-way. No fill shall be placed in the right-of-way. The design should also minimize the amount of the back of lot draining directly to the right-of-way. Approval by the Fort Bend County Drainage District Engineer of the preliminary design of projects near major channels should be obtained before any detailed engineering is performed.

7. Slopes steeper that 5:1 shall be entirely located in reserve dedicated to a perpetual agency (other than FBCDD) with maintenance responsibilities. The minimum width of such reserve shall be 15 feet and the maximum slope shall be 3:1.

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

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

5.2.3 Specific Design Flow Frequency Criteria

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

1. The design flows shall be the 5-year event based on the applicable hydrologic method according to the size of the watershed. The conduit shall be designed in accordance with the methodology outlined in Section 5.3.

2. The design shall be checked for the 100-year to ensure the ponding elevation is below the limits set in Section 5.2.2.

3. The tailwater conditions for storm sewers analysis shall be as specified in Section 5.3.2.

4. Adequate inlet capacity per Section 5.3.7 requirements shall be provided to allow for entry into the closed-conduit system of runoff required to meet the hydraulic grade line and maximum ponding requirements.

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

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

7. An extreme event overflow pathway shall be designed and provided, as needed, to convey flow beyond the capacity of the storm sewer to the designated detention pond.

5.2.4 Submittals

Before a storm sewer design is reviewed by the Fort Bend County Drainage District Engineer, the following items shall be presented in the design plans and be supported by the drainage report:

a. A contour and drainage area map showing all pertinent subareas, including all contributing off-site areas. A note should be included related to the frequency storms where offsite drainage areas are accounted for in the flow that enters the system. Typically, offsite drainage areas are considered in the calculations of extreme events like the 100-year storm but in some cases, the 5-year storm may result in runoff from offsite areas depending on the capacity of the offsite drainage systems. Justification shall be provided when such offsite areas are not accounted for in the calculations.

b. A listing of all relevant hydrologic design flow calculations, including all contributing offsite flows.

c. Calculations for determining the hydraulic gradients with their profiles along the pipes for the 5-year design event and the 100-year check event. This should include documentation of the tailwater conditions used in the calculations.

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

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

5.3 STORM SEWERS

5.3.1 Design Criteria

The following specific criteria and requirements shall apply to the design and construction of storm sewers in Fort Bend County.

1. The minimum diameter of a pipe in a sewer line in public right of ways shall be 24 inches.

2. The Manning's "n" value to be used in the design are listed in Table 5-1. If a different material is used, calculations shall include the source of the coefficient used.

3. The minimum velocity in any section of the storm sewer flowing full shall be 3 fps. The maximum recommended velocity inside a conduit is 8 fps. Velocities above 8 fps are permissible only inside concrete conduits provided the hydraulic grade line is at least two feet below the gutter line and the velocity is gradually reduced to obtain acceptable velocities at the outfall.

4. Proper erosion protection for the outfall must be provided following the requirements of Section 3.4. Outfall velocities greater than 9 fps require concrete lining of the outfall and outfall velocities in excess of 11 fps require special approvals from the Fort Bend County Drainage District Engineer.

5. Provisions shall be made for all adjacent undeveloped areas with natural drainage patterns directing overland flow into and across planned development. Drainage in the adjacent properties up to the 100-year event should not be adversely impacted.

6. All storm sewers and appurtenant construction shall conform to the City of Houston Department of Public Works publication Specifications for Sewer Construction, Storm Sewer Details, and all subsequent revisions. All outfalls into ditches, channels, streams or detention ponds shall include the use of erosion protection in accordance with Figures 3-4 and 3-5.

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

8. All storm sewers shall follow the alignment of the right-of-way or easement without hindering access for maintenance.

9. Where easements are dedicated only to storm sewers, the pipe should be centered within the limits of the easement.

10. All pipes shall be designed in a straight alignment between manholes. The maximum separation between manholes shall be 500 feet.

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

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

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

5.3.2 Tailwater Conditions

Assumed water elevation at the downstream end of the pipe is an important variable in the design of storm sewers as it determines the capacity of the system. A low tailwater condition results in more capacity than a high tailwater condition as the system has more energy available. It should be considered that the receiving channels fill up with discharges from storm sewers and therefore, storm sewers typically reach the peak flow before the receiving systems reach the maximum water elevation during the storm. The tailwater elevation depends on the size of the development and the drainage area for ultimate development conditions as follows:

1. Developments of 50 acres or less with a detention pond designed per Section 6.4.1 and not part of a Master Planned development:

The tailwater condition is based on the percentage of the 100-year detention depth as noted in Table 5-2. The 5-year tailwater condition applies for the calculations of the design event even if the storm drain has a capacity greater than the 5-year storm.

The detention depth is measured from the bottom of the pond to the maximum water surface elevation for the 5-year or 100-year events. For purposes of this rule, the 100-year detention depth is calculated based on the applicable requirements for detention specified in Section 6.0.

For submerged outfalls, the detention depth is measured from the permanent water surface elevation to the maximum water surface elevation for the storm being analyzed. The tailwater elevation cannot be lower than the top of pipe or the permanent water surface elevation of a submerged pipe.

Example: A 20-acre development with 60% impervious cover is being designed. The proposed storm sewer outfall into the pond is a 36-inch pipe with invert at elevation 90 feet msl. (Thus, the top of pipe is at elevation 93 feet msl.)

The development is smaller than 50 acres and follows Section 6.4.1. An elevation-volume relationship for the pond is as follows:

WSEL (feet-msl)	Depth (feet)	Volume (acre-feet)
90.00	0.00	0.0
91.00	1.00	2.8
92.00	2.00	5.8
93.00	3.00	8.9
94.00	4.00	12.1
95.00	5.00	15.4
96.00	6.00	19.0
96.78	6.78	21.8*
97.00	7.00	22.6
98.00	8.00	26.4
99.00	9.00	30.4

*100-Year Detention Volume

Per Section 6.4.1, a detention rate of 1.09 acre-feet per acre applies for this project, this storage is based on the 100-year event. The detention pond should provide a minimum volume of 21.8 acre-feet excluding the required freeboard. Using the same elevation-volume relationship, the water surface elevation for the required storage is 96.78 feet, which corresponds to a depth of 6.78 feet.

<u>5-Year Tailwater Calculation</u>: 25% of the 100-yr depth corresponds to 1.7 feet at a water surface elevation of 91.70 feet msl. This elevation is lower than the top of pipe and therefore, the top of pipe elevation of 93.0 should be used as the tailwater for the 5-year event.

<u>100-Year Tailwater Calculation</u>: The tailwater to be used for the calculation of the 100-year storm is 60% of 6.78 feet, or 4.07 feet, which corresponds to a tailwater elevation of 94.07 feet msl.

The elevation-volume table included in the plans and in the drainage report shall include sufficient points to illustrate this relationship. In this example, shall include values at elevation 96.78 (21.8 acre-feet) in addition to other elevations used in other storms analyzed per Section 6.0.

2. Developments greater than 50 acres discharging to a detention pond designed per Sections 6.4.2 and 6.4.3, or Master Planned Developments outfalling to a detention pond regardless of the size:

   The tailwater condition is a function of the volume of the storm being analyzed. The tailwater shall be the water surface elevation at 50% of the peak volume stored in the pond for the frequency storm being evaluated or the top of pipe, whichever is higher. The elevation-volume relationship of the detention pond shall be included in the design plans and in the drainage report. All elevations used to calculate the tailwater conditions and the corresponding storage should be part of the table.

For submerged outfalls, the detention volume refers to the volume that fills up above the permanent water surface elevation, and the tailwater cannot be lower than the permanent water surface elevation.

The 5-year tailwater condition applies for the calculations of the design event even if the storm pipes have a capacity greater than the 5-year storm.

Example: a 36-inch storm sewer discharges to a dry detention pond in which the peak volume stored during the 5-year design storm is 30 acre-feet at elevation 48 feet. The flow line of the storm sewer is at elevation 40 feet, then, the top of pipe is at elevation 43 feet. The tailwater elevation for the calculation of the hydraulic grade line for the 5-year event in the storm sewer shall be the water surface elevation at 15 acre-feet (50% of 30 acre-feet), which is determined to be at elevation 45.0 using the storage-elevation relationship of the pond. This is higher than the top of pipe and therefore, should be adopted as the applicable tailwater conditions of the 5-year storm sewer calculations. The elevation-volume table included in the plans and in the drainage report shall include sufficient points to illustrate this relationship and in this example, shall include values at elevations 45.0 (15 acre-feet) and 48.0 (30 acre-feet) in addition to other elevations used in other storms analyzed per Section 6.0.

3. Storm sewers discharging to a channel:

The tailwater is based on the ratio of the drainage area of the storm sewer to the drainage area of the channel at the point of discharge according to Equation 5-1 below:

$$\text{Ytw} = \text{Yf}\left(\frac{\text{As}}{\text{Ach}}\right)^{1/3} \qquad \text{Eq. 5-1}$$

where:

Ytw is the resulting tailwater depth in feet;

Yf is the depth in the receiving channel for the storm frequency being analyzed in feet;

Ach is the drainage of the channel at the point of discharge; and

As is the contributing drainage area of the storm system.

Equation 5-1 uses the ratio of drainage areas (Ach to As) so these variables must be in the same units. The tailwater used in the analysis cannot be lower than the top of pipe. Direct discharges to a channel without proper detention for mitigation will not be allowed.

Example: a storm sewer with a drainage area of 250 acres discharges into a channel (As = 250 acres). That channel has drainage area of 1,000 acres at the point of the discharge from the storm sewer (Ach=1,000 acres) and the maximum depth in the channel during the 100-year storm is 12 feet (Yf=12 feet). The 100-year tailwater depth is calculated using Equation 5-1.

$$\mathrm{Ytw} = \mathrm{Yf}\left(\frac{\mathrm{As}}{\mathrm{Ach}}\right)^{1/3}$$

$$\mathrm{Ytw} = 12\left(\frac{250}{1000}\right)^{1/3}$$

$$\mathrm{Ytw} = 7.6 \text{ feet}$$

The elevation corresponding to a depth of 7.6 feet in the receiving channel should be used as the tailwater condition for the 100-year storm calculations for the storm sewer that discharges at that location. If the tailwater depth is below the top of pipe, the top of pipe should be adopted.

4. Proposed storm sewers connecting to an existing storm sewer:

In calculating the new system, the tailwater for the 5-year design event shall be the lowest gutter elevation of the two existing inlets near the proposed connection. The tailwater for the 100-year overflow calculations shall be the top of curb, or in absence of a curb, the natural ground. The Engineer has the option to use the tailwater elevation obtained with hydraulic grade line calculations of the existing system for the 5-year and 100-year events, accounting for the entire drainage area including the proposed connection. Hydraulic grade calculations of the new system will be carried out starting at the connection point with the assumed tailwater and design flows no greater than the maximum allowable. The maximum allowable flow shall be as determined in Section 5.3.3.

5.3.3 Connections to Existing Storm Sewers

When the new storm system connects to an existing system, the Engineer shall obtain the following from the owner agency of the existing system (County, Municipal Utility District, or other) or from approved design drawings:

a) Original design flow of the existing storm sewer just downstream of the point of connection;
b) Original design flow of the existing storm sewer just upstream of the point of connection;
c) Original drainage area maps used in the design of the existing system;
d) Flow allocation to the new connection based on prorated contributing drainage area or other methods;

Note that the original design flow can be for a storm frequency other than the 5-year storm required in Section 5.2.3. For example, if the existing system was approved with the prior Fort Bend County Drainage Criteria Manual, the original design storm is most likely the 3-year event. The Engineer shall confirm from the available drawings that the hydraulic grade for the original design flow does not exceed the gutter line along any point of the existing system downstream of the connection point.

If drawings and/or information about the original design flow of the existing system at the point of connection are not available, the Engineer shall determine the capacity of the new system at the connection point as the maximum flow that be carried by the existing system for a distance of 1,000 feet without having the hydraulic grade above the gutter line at any point.

The maximum allowable inflow from the new system into the existing system during the 5-year storm shall be the minimum of the following:

a) The 5-year event runoff calculated for the new system;
b) The maximum additional flow that can be received by the existing system without exceeding the capacity just downstream of the connection point;

c) The prorated or allocated flow in the original design to the new system's drainage area, if available.

Storm water detention shall be provided in the new system to reduce the discharges from the 5-year runoff to the maximum allowable inflow into the existing system. Additional detention for the 100-year event may be required following the guidelines of Section 6.0.

The Fort Bend County Drainage District will consider allowing previously approved storm systems to be completed with original design criteria on a case-by-case basis and at the discretion of the Fort Bend County Drainage District Engineer.

Example: A new proposed development in a two-acre site is connecting to an existing 36-inch storm sewer built in 1995. The following is known about the existing system, according to the design plans:

a. The original design flows (1995) are 30 cfs at the manhole immediately upstream from the proposed connection and 36 cfs at the proposed connection.

b. The assumed contributing drainage areas for these two manholes are 25 acres at the upstream manhole and 30 acres at the downstream manhole.

c. The two acres of the site are accounted for in the drainage area of the existing storm sewer system.

From parts a and b, it is concluded that the flow between the two manholes increases by 6 cfs assuming the drainage area increases by 5 acres. This equates to a flow rate of 1.2 cfs/acre in the area between the two manholes.

The contributing runoff from the new site for the 5-year event is calculated using the Rational Method. Under the current guidelines of this Manual, the runoff coefficient is C=0.5. The Rational Method calculations are:

	Acres	Tc (min)	I (in/hr)	C	Q	cfs/acre
Upstream MH	25	32.6	3.95	0.5	49	2.0
Downstream MH	30	33.2	3.91	0.5	59	2.0

Then, the 5-year event as defined with the current rainfall data for Fort Bend County produces 2.0 cfs/acre. This exceeds the allocated during the original design. Therefore, detention is to be provided at the site to reduce the runoff from 5-year event from 2.0 cfs/acre to 1.2 cfs/acre based on the drainage area included in the original calculations. In this two-acre site, detention shall reduce the 5-year outflow to a maximum of 2.4 cfs. Hydraulic grade lines calculations for the new storm sewer can start with a tailwater shown in the original design plans.

If new calculations of the existing system and proposed development with the 5-year and 100-year events as prescribed in this Manual demonstrate that all applicable criteria are met, detention for the 5-year may not be required. The site is still subject to the detention requirements outlined in Section 6.0.

5.3.4 General Design Methodology

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

1. Determine the applicable tailwater conditions for the design flow per Section 5.3.2 and 5.3.3.

2. Determine the design flow rates for all sections of storm sewer.

3. Determine the minimum size that carries the design flow for all sections of storm sewer using Manning's equation and assuming uniform flow conditions.

4. Begin calculation assuming the applicable tailwater conditions at the outfall and plot the hydraulic gradient for the design storm. Include all relevant energy losses. The hydraulic gradient for the design storm must be below the roadway gutter flowline elevation. Consider discontinuities in the hydraulic gradient due to drops of the storm sewer at manholes. If the hydraulic gradient is not continuous, use the top of pipe as the tailwater condition for the segment upstream of the drop.

5. Upsize conduits as needed to meet the requirements of the maximum ponding or sheet flow depth for the 100-year check event.

5.3.5 <u>Head Losses</u>

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

5.3.5.1 Head Losses at Structures

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

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

where

V_1 = velocity in the upstream pipe (fps).

V_2 = velocity in the downstream pipe (fps).

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

5.3.5.2 Entrance Losses

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

$$\text{head loss} = K\frac{V^2}{2g} \qquad (5\text{-}3)$$

where

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

V = flow velocity in pipe (fps).

5.3.6 Manholes and Junction Boxes

Manholes or junction boxes shall be placed at all pipe size or cross section changes, pipe sewer intersections, street intersections, and pipe sewer grade changes.

5.3.7 Inlets

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

5.3.7.1 Inlet Capacity

The capacity of inlets shall be enough to capture the runoff from the design storm without exceeding the maximum spread width (Section 5.3.7.3) and to keep ponding during the 100-year below the maximum level. The capacity of an inlet depends if the water enters the storm sewer as weir flow or orifice flow. Weir flow occurs when the depth of water does not exceed the opening of the inlet while a fully submerged inlet behaves as an orifice. When the depth of the opening is reached, flow can be assumed to start transitioning or be completely considered orifice flow. Inlet capacity calculations shall consider the type of regime under the storm being analyzed. Engineer shall refer to the Hydraulic Design Manual from the Texas Department of Transportation for equations related to capacity of inlets.

5.3.7.2 Inlet Spacing

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

5.3.7.3 Maximum Spread

Spread of flow around the gutter will be calculated with the applicable equation listed in the most recent release of the Texas Department of Transportation Hydraulic Design Manual. The maximum spread for the design event shall be as follows:

1. On a residential street, it shall be distance from the curb to the centerline or the crown of the roadway, whichever is less.
2. On a roadway with two or more lanes in each direction, it shall be the full width of the outside lane, leaving one or more lanes passable.

5.4 STREET DRAINAGE OF STORM SEWER OVERFLOWS

When the capacity of the underground system is exceeded and street ponding begins to occur, careful planning can reduce or eliminate the flood hazard for adjacent properties. Street layout and pavement grades along with extreme event swales are the key components in developing a successful system which can handle the 100-year storm without excessive ponding and is able to convey any runoff in excess of the 100-year event to the designated detention pond or outfall channel. The following design methodology and example is derived from typical development conditions found in Fort Bend County.

5.4.1 Land Plan and Street Layout

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

Some examples of undesirable sheet flow patterns are depicted in Figure 5-4 and include:

a) Cul-de-sac streets sloping downhill causing a problem where sheet flow can only escape through building lots.

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

c) Many streets "T-ing" into one street which is lower than the intercepting streets causing a problem where sheet flow down the streets can escape only through existing building lots.

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

5.4.2 100-Year Event Analysis Methodology

The design engineer must check the storm drainage system can convey flows from a 100-year storm event without ponding water in the street at levels that exceed the maximum allowable level. The 100-year discharge can be obtained by following the procedures outlined in Section 2.4. The design engineer shall calculate the 100-year hydraulic grade line using the applicable tailwater conditions per Section 5.3.2. If the maximum ponding level is exceeded, the engineer must consider upsizing the conduits and/or increasing the capacity of the overland portion to lower the maximum ponding to the acceptable limits.

The engineer can account for the portion of flows that would be carried by the sewer system in addition to the street system, assuming the applicable tailwater condition for the 100-year event. In order to reduce the amount of overland flow, the engineer may upsize the storm sewer system, in which case, hydraulic grade calculations must be rechecked to verify the ponding depth is below the acceptable ponding limit. The final design should convey the 100-year in a combination of storm sewers and overland flow without exceeding the maximum ponding depths.

The engineer must also verify that there are no increases in flows and water elevation into the receiving channel. Storm water detention shall be provided according to the guidelines of Section 6.0 to avoid adverse impacts to the receiving channels. Outfalling directly into a receiving channel is not allowed without appropriate mitigation and approval by the Fort Bend County Drainage District.

5.4.3 Conveyance of Extreme Event Runoff to Detention Ponds

Overland flow must be routed through a detention pond before outfalling into an existing channel. This may be done by using additional pipe capacity, additional inlets, and/or by using a surface overflow swale. The swale shall have capacity to pass the required flow during the 100-year with at least one foot of freeboard between the water elevation and adjacent top of slabs and shall be located with an easement dedicated to a permanent agency with maintenance responsibilities (MUD, LID etc). The easement shall be wide enough to allow access for maintaining the system.

Since an overflow surface swale system would be utilized only under extreme storm events and would not be utilized under most circumstances, all precautions must be taken to ensure that this relief system will function when needed. Overflow swales within the right-of-way of detention ponds and outfall channels shall have a minimum 10-foot bottom width and side slopes no steeper than 6(H):1(V). These swales shall be designed to use interlocking concrete blocks or concrete slope paving to protect from erosion.

In special circumstances and at discretion of the Fort Bend County Drainage District Engineer, the conveyance to the detention pond and the overflow from extreme events will need to be analyzed for the 500-year event. In such special cases, the 500-year event shall result in no structural flooding.

5.4.4 Design Procedure for Pipe Outlet

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

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

1. In this example, a drainage area of five acres at the end of the street joins a larger storm sewer. The total drainage area after the confluence of the small system with

the larger system is 40 acres. After the confluence, a 60-inch storm sewer pipe discharges the runoff into the detention pond.

2. Determine the 100-year peak flow at the point of interest from all existing and future contributing drainage areas for 100% of the proposed development conditions. In this example, the 100-year flow from the smaller street drainage area is 22 cfs. The total flow entering the outfall pipe is 158 cfs.

3. Determine the applicable tailwater elevation in the receiving detention pond. The 100-year elevation in the detention pond is at 98.0 with a depth of 8.0 feet. This means the bottom is at elevation 90.0. Using Table 5-1, the tailwater to be used for the analysis of the extreme event storm sewer design is 60% of the 100-year detention depth or 4.8 feet, which is elevation 94.8. Because this is lower than the top of pipe of a 60-inch pipe, then the top of pipe resulting in a depth of 5.0 feet or elevation 95.0 is used.

4. Determine the maximum energy head, H, available between the outfall point and the ponding area at the end of the street by subtracting the maximum allowable ponding elevation in the ponding area from the tailwater elevation.

   If the top of curb is at elevation 100.0, the lowest slab elevation is 102.0 (two feet above the top of curb) and the maximum 100-year ponding elevation is 101.0 (one foot over the top of curb). Then, there is 6.0 feet of head available (H) between the tailwater and the maximum ponding elevation.

5. Compute the head loss using the following equation:

$$h_p = \frac{4.66 Q^2 n^2 L}{D^{16/3}} \tag{5-4}$$

where

h_p = Head loss in feet

Q = 100-year discharge in cubic feet per second

n = Manning's "n" value

D = Diameter of pipe in feet

L = Length of pipe in feet

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

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

$$h_{LOSS1} = \frac{4.66\,(158)^2}{5.0^{16/3}}\,((0.024)^2\,(65) + (0.013)^2\,(120))$$

$$= 1.25 \text{ feet}$$

The head losses in the 80-ft, 30-inch pipe that connects to the contributing street are:

$$h_{LOSS2} = \frac{4.66\,(22)^2}{2.5^{16/3}}\,((0.013)^2\,(80)) = 0.22 \text{ feet}$$

6. Compute the head loss through the leads, h_{LOSS3}, using Equation 5-4. Experience has shown that the 24-inch diameter leads generally cause excessive head loss. The 30-inch diameter leads are satisfactory in most cases, while the 36-inch leads are too large for the most common street inlets type "B-B" and "C-1". The 24-inch diameter was selected for initial calculations.

Estimate the portion of 100-year runoff from street flowing through each lead. Assume the 22 cfs to be divided between two equal leads as follows:

Lead 1 20-foot lead with a flow of 11 cfs.
Lead 2 20-foot lead with a flow of 11 cfs.

The head loss at each lead is:

$$h_{L_1} = h_{L_2} = \frac{4.66\, n^2\, Q^2\, L}{D^{16/3}} = \frac{4.66\,(0.013)^2\,(11)^2\,(20)}{2.0^{16/3}}$$

$= 0.19$ foot

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

$$h_i = H = h_p - \Sigma\, h_{LOSS} \tag{5-5}$$

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

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

The energy head available at each inlet is:

$$h_i = 6.0 - (1.25+0.22+0.19) = 4.34 \text{ feet}$$

The number is positive, so the pipe will not need to be increased. The hydraulic grade at each inlet is 95.0 + 1.25 + 0.22 + 0.19 = 96.66. The gutter line elevation is 99.6, which is above the hydraulic grade line.

In this case, the maximum head available for each inlet is based on the maximum ponding elevation of 101.0, resulting in a maximum head at each inlet of 1.4 feet.

8. Determine the type of inlets required to handle the portion of the 100-year flow reaching the ponding area. The flow through the inlet(s) must be equal to or greater than the flows estimated in Step 6 for each lead. Since the inlet opening is submerged with a depth of 1.4 feet above the gutter line, the orifice equation can be used to compute the flow into each inlet.

$$Q = CA\,(2gh_i)^{1/2} \tag{5-6}$$

where

Q = discharge in cubic feet per second

C = orifice coefficient (0.67 for submerged inlets per TxDOT Hydraulic Manual)

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

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

h_i = head from the maximum ponding to the centroid of the orifice opening. The centroid is about three inches above the gutter line.

Type “B-B” are selected for the two inlets. The flow per inlet is:

$$Q_{B\text{-}B} = 0.67\ (2.14) \times (64.4\ (1.4\text{-}0.25))^{1/2} = 12 \text{ cfs}$$

The $Q_{B\text{-}B}$ inlet capacity is greater than the required for the extreme event runoff. Thus, it is shown that a Type “B-B” inlet at both Inlets 1 and 2 will convey the 100-year sheet flow to underground system and the storm sewer will convey the water to the channel with the energy head available. The design can be optimized to reduce cost, but recalculations with smaller pipes are required. Otherwise, the design is complete.

9. If one of the criteria is not met, repeat Steps 3 through 8 until the combination of storm sewer pipe, leads, and inlets adequately conveys the 100-year sheet flow to the channel with the energy head available, and is the most economical.

Calculations can be completed with a hydraulic modeling software capable of modeling both pipe flow and surface flow.

5.4.5 Design Procedure for Overflow Swales

Calculations shall be provided that substantiate the design of an overflow swale to meet the 100-year requirements. The design procedure recommended for sizing of the surface swale is similar to the procedure for the pipe outfall as described in Section 5.4.4. First, the appropriate values from steps one and two are computed, then the required extreme event swale cross-section is determined by normal depth calculations, sizing the swale such that an acceptable water surface (considering the required free board) is achieved.

5.5 ROADSIDE DITCHES

Under certain conditions, roadside ditch drainage is acceptable as an alternative to curb-and-gutter systems. However, a similar potential for flooding exists when flow in roadside ditches exceeds capacity. Provisions must be made to assure that the amount of water ponded behind an elevated roadway does not reach damaging levels. (See Figure 5-6 for typical roadside ditch drain detail.)

Projects or developments that drain to existing roadside ditches are only allocated a pro rata share of the existing ditch capacity, for which an example is presented in Section 5.5.3. Detention shall be provided to limit the discharge from the new development into the existing roadside ditch to the prorated capacity of the ditch's existing bank full condition. In addition, developments less than 50 acres in size must be designed per FBCDCM, Sec. 6.4.1 which limits discharge to no greater than 0.125 cfs per acre of contributing drainage area of the detention pond. Additional detention is to be provided to avoid increases to the existing condition runoff in the roadway during the 100-year event.

5.5.1 Preliminary Approval

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

5.5.2 Design Criteria

The following requirements must also be met in the design of new roadside ditch systems:

1. Roadside ditches shall be designed so that the 5-year flow does not exceed the top of bank and 100-year water elevation or sheet flow depth is less than 1.0 foot above the crown of the road. (Refer to Figure 5-2)

2. The minimum top of slab elevation shall be one foot above maximum ponding/sheet flow elevation or one foot above the crown of the road, whichever is higher. (Refer to Figure 5-2)

3. The design 5-year and 100-year flows shall be determined based on the projected land use and the runoff rates calculated in Section 2.4.

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

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

6. The "n" coefficient for the ditch calculations shall be a minimum of 0.04. All values shall be justified in the drainage report.

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

8. Hydraulic design computations must be submitted for each drainage ditch system. Computations shall include the effect of current, and future driveway culverts, which shall be sized considering design flow and ditch depth.

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

10. Erosion control methods shall be utilized in the ditch designs where velocities of flow are calculated to be greater than 5 fps or where soil conditions dictate their need.

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

5.5.3 Prorated Ditch Capacity Calculation Example

An existing drainage roadside ditch has a drainage area of 20 acres at the downstream end of a new proposed development, or the point at which the proposed new development entirely contributes to the ditch. The capacity of that ditch at the downstream end of the proposed development, accounting for all driveway culverts, is 40 cfs, meaning that a flow greater than 40 cfs would cause overflow above the top of bank at that location. The capacity of 40 cfs governs the design for events up to and including the 100-year event.

Design calculation shows that the 3 acres of the proposed development is included in the 20 acres of drainage area. Then, the share of the new development of the capacity of the ditch is 3/20 or 15% of 40 cfs. Then, the runoff from the new development is limited to 6 cfs. Detention shall be provided to limit the discharge from the new development into the existing roadside ditch to not exceed the prorated capacity of the ditch's existing bank full condition. In addition, because the development is less than 50 acres in size, it must be designed per FBCDCM, Sec. 6.4.1 which limits discharge to no greater than 0.125 cfs per acre of contributing drainage area of the detention pond. Added detention is required to avoid increases to the runoff into the roadway right-of-way during the 100-year event.

TABLE 5-1

ROUGHNESS COEFFICIENT (MANNING'S N VALUES)

Material	Manning's n
Concrete pipes	0.013
Concrete boxes	0.015
CMP	0.024
Corrugated profile polyethylene pipe (Smooth Interior)	0.012

TABLE 5-2

TAILWATER CONDITIONS FOR DEVELOPMENTS THAT FOLLOW SECTION 6.4.1 WITH OUTFALL INTO A DETENTION POND

Storm	Tailwater
5-year	25% of the design 100-year detention depth or the top of pipe, whichever is higher
100-year	60% of the design 100-year detention depth or the top of pipe, whichever is higher

TABLE 5-3

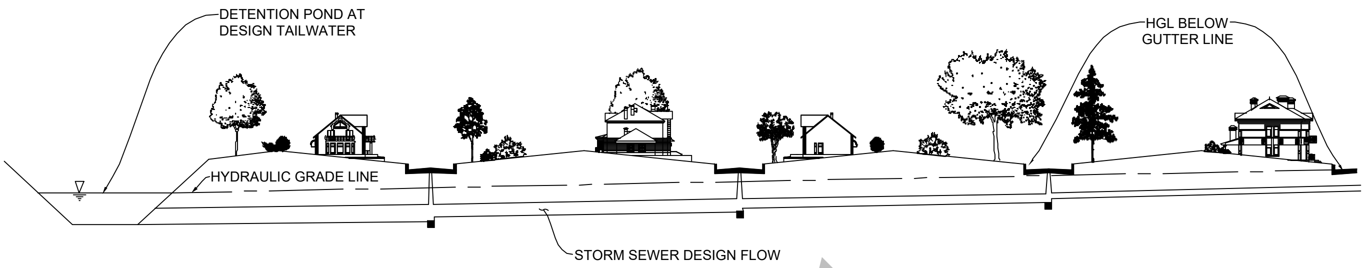
COEFFICIENTS AT STRUCTURES

Type of Structure	Coefficient K
Inlet on main line	0.50
Inlet on main line with branch lateral	0.25
Manhole on main line with 22-1/2° lateral	0.75
Manhole on main line with 45° lateral	0,50
Manhole on main line with 60° lateral	0.35
Manhole on main line with 90° lateral	0.25

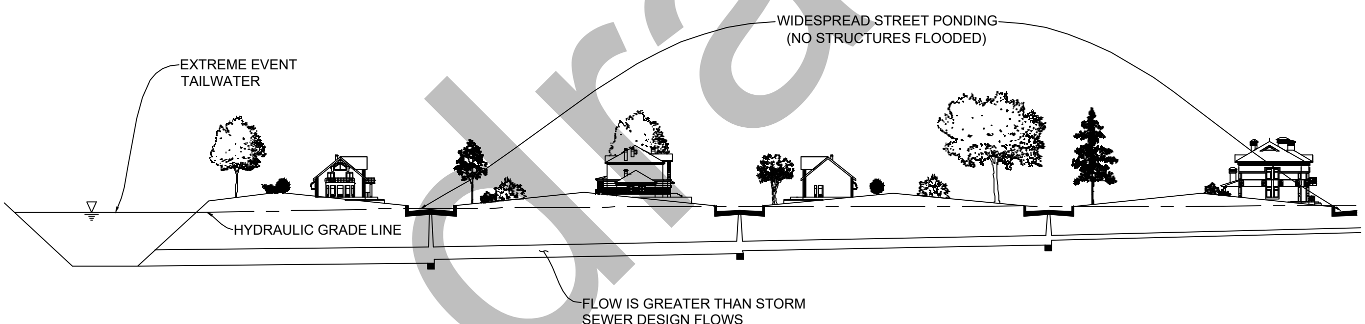
TABLE 5-4

COEFFICIENTS FOR ENTRANCE LOSSES

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



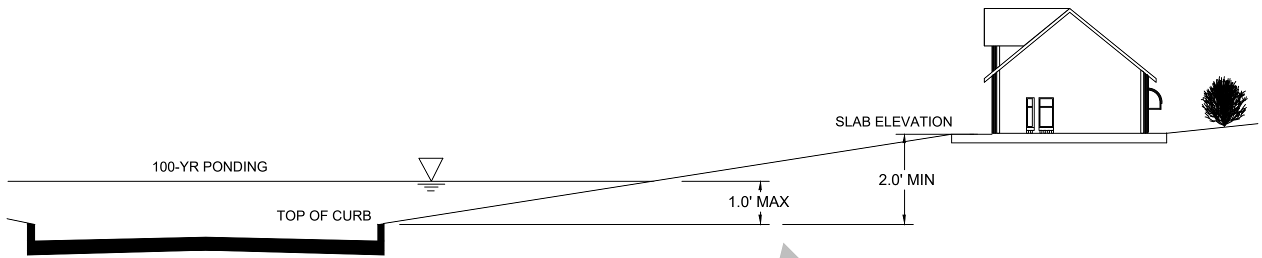
A) STORM SEWER AT STANDARD DESIGN FLOW CONDITIONS



B) SYSTEM OPERATING DURING THE 100-YEAR OR EXTREME EVENT

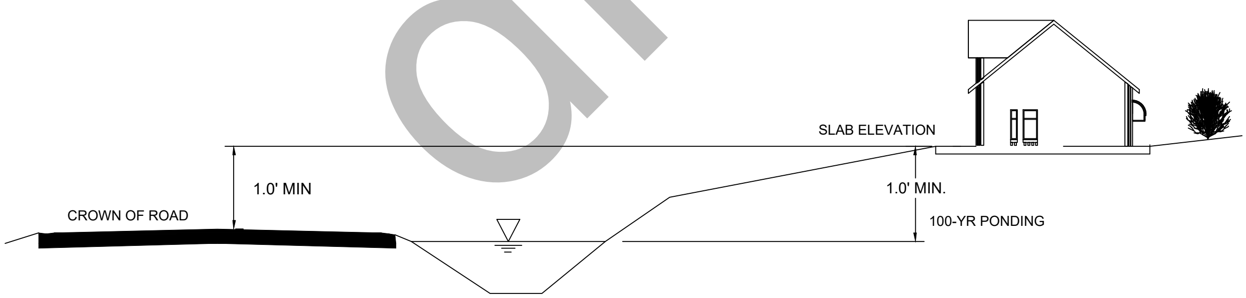
STORM SEWER-TAILWATER INTERACTION	
SEPTEMBER 2021	FIGURE 5-1

WHEN THESE SEPARATIONS ARE MET, THE TOP OF SLAB IS AT LEAST 1.0 FOOT ABOVE
THE 100-YEAR PONDING ELEVATION.



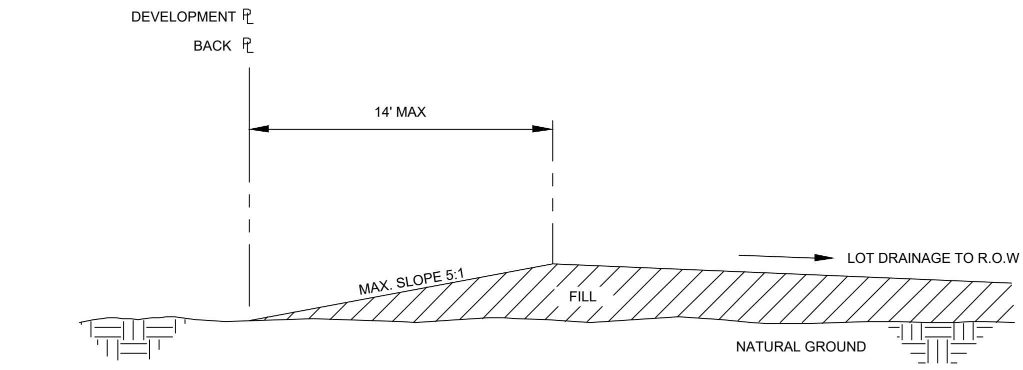
A) CURB AND GUTTER SYSTEM

TOP OF SLAB SHALL BE 1.0 FOOT ABOVE THE CROWN OF THE ROAD OR 1.0' ABOVE THE 100-YEAR WATER
SURFACE ELEVATION., WHICHEVER IS HIGHER

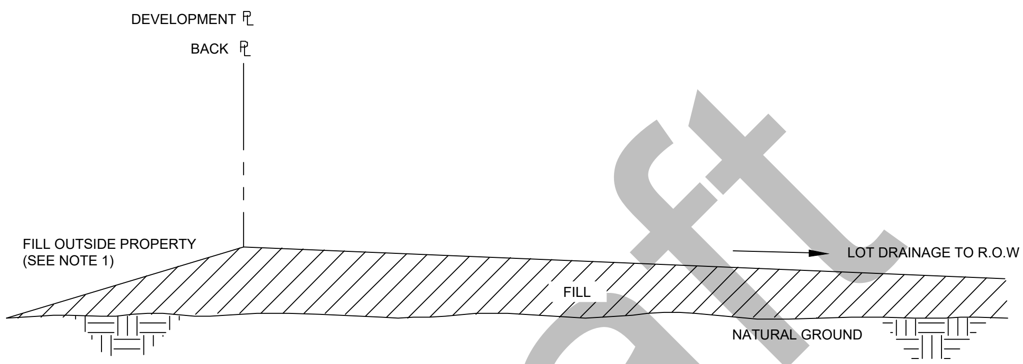


B) ROADSIDE DITCH

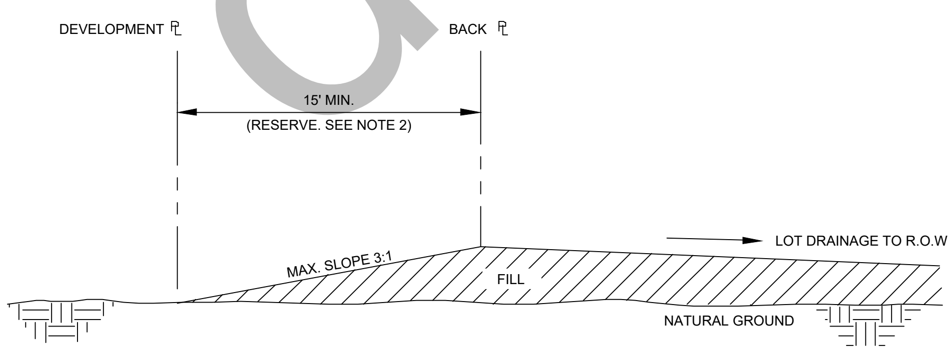
MAXIMUM 100-YR PONDING AND MINIMUM SLAB ELEVATION	
SEPTEMBER 2021	FIGURE 5-2



A) SMALL LOT RESIDENTIAL



B) SMALL LOT RESIDENTIAL
(ALTERNATIVE)



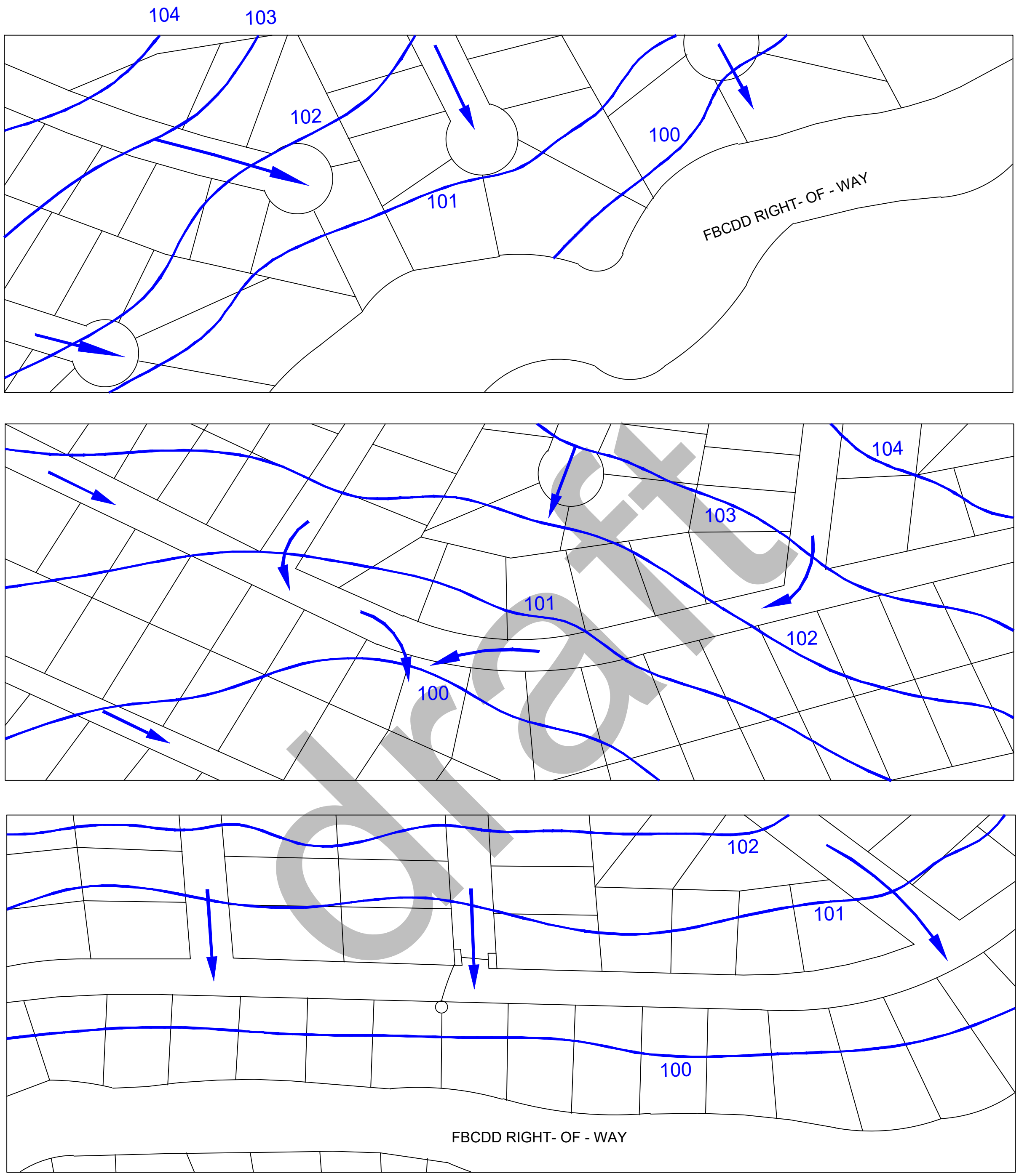
C) SMALL LOT RESIDENTIAL WITH EASEMENT RESERVE

NOTES:

(1) WITH AGREEMENT FROM ADJACENT LAND OWNER, FILL MAY BE PLACED ON THE ADJACENT PROPERTY CREATING A DRAINAGE DIVIDE AT THE BACK OF LOT.

(2) RESERVE SHALL BE DEDICATED TO AN ENTITY WITH PERPETUAL MAINTENANCE RESPONSIBILITIES.

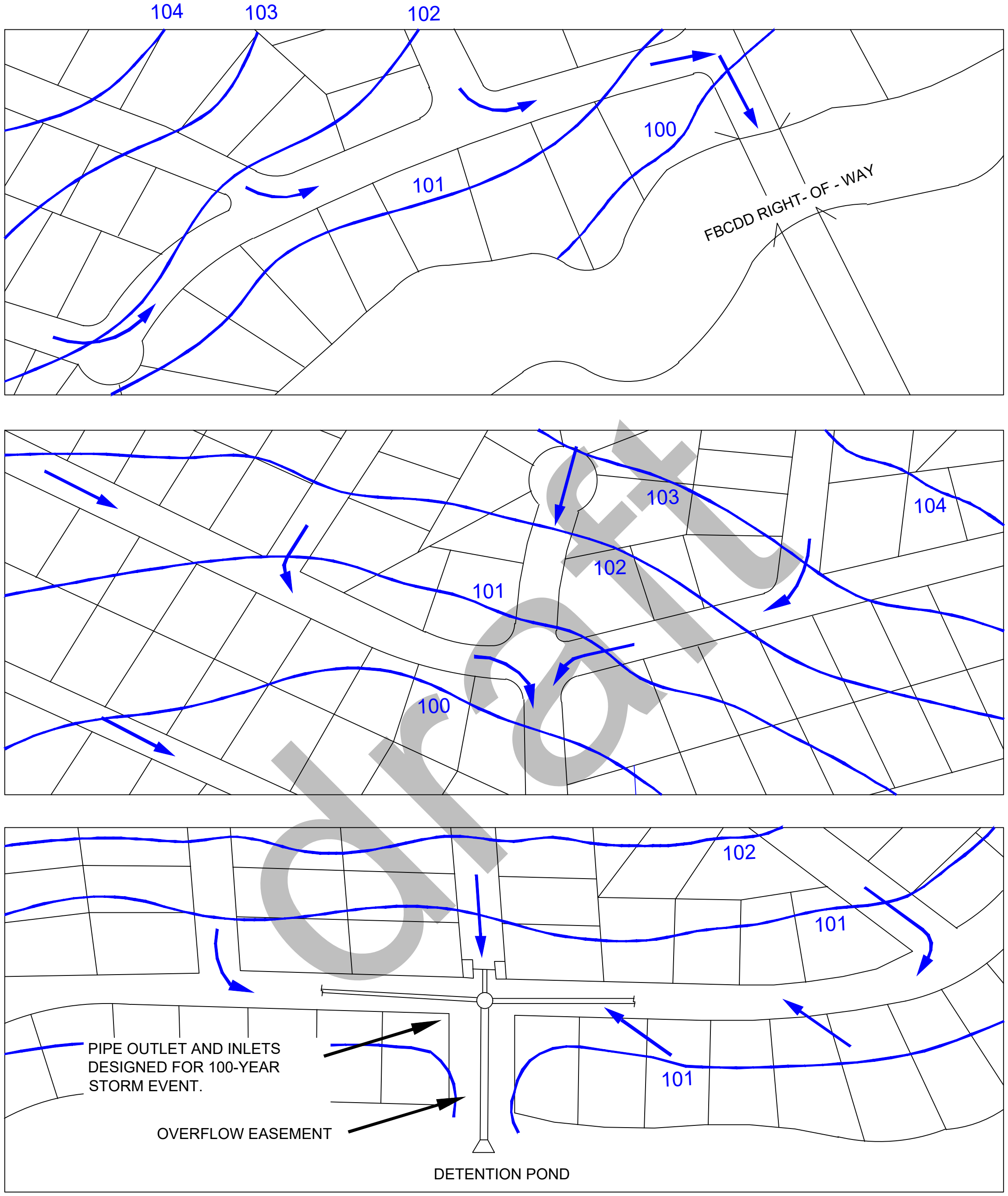
BACK OF LOT GRADING RECOMMENDATIONS	
SEPTEMBER 2021	FIGURE 5-3



UNDESIRABLE SHEET FLOW PATTERNS FOR FORT BEND COUNTY, TEXAS

SEPTEMBER 2021

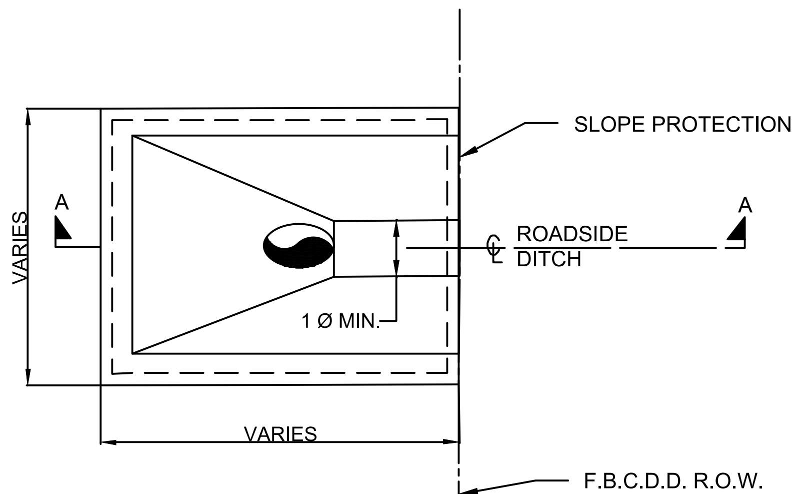
FIGURE 5-4



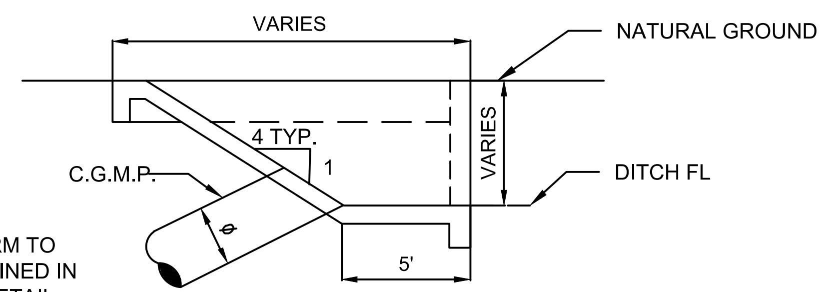
ACCEPTABLE SHEET FLOW PATTERNS FOR FORT BEND COUNTY, TEXAS

SEPTEMBER 2021	FIGURE 5-5

NOTE:
WHERE BACKSLOPE INTERCEPTOR
SWALES FLOW TOWARDS ROADSIDE
DITCH, MODIFY TO ACCOMMODATE
FLOW.



PLAN
N.T.S



OUTFALL TO CONFORM TO
SPECIFICATIONS DEFINED IN
BACKSLOPE DRAIN DETAIL
(FIGURE 3-2)

SECTION A-A
N.T.S.

TYPICAL ROADSIDE DITCH DRAIN DETAIL FOR FORT BEND COUNTY, TEXAS	
SEPTEMBER 2021	FIGURE 5-6

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6.0 STORM RUNOFF STORAGE

6.1 GENERAL

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

6.2 MASTER PLANS

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

If a master plan is not available for the watershed in which the engineer is trying to develop or design a project, the engineer may be able to develop localized master planning information related to the site location within the watershed. That planning should include close coordination with the FBCDD Engineer. The master plan shall provide for anticipated future upstream development (as defined in Section 3.8) including full conveyance flow, to facilitate potential future channel improvements along the channel. The plan shall provide for adequate channel sizing and maintenance access (or “maintenance berms”) under anticipated future development assuming storm sewer outfall depth requirements. This planning is needed to ensure that adequate right of way, channel sizing and channel depth is provided for each project.

6.3 STORAGE CLASSIFICATION

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

6.3.1 Retention Storage

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

6.3.2 Detention Storage

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

6.3.3 In-line Storage

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

6.3.4 Off-line Storage

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

6.4 DESIGN PROCEDURES

The following design procedures are intended to ensure that new development with detention will result in no adverse impact to flood risk. (Note: The design engineers should contact the FBCDD Engineer for any specific requirements for the watershed in which the proposed facility is to be located.)

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

6.4.1 For Drainage Areas <50 Acres

The maximum allowable release rate from the detention facility during the 100-year storm event is 0.165 cfs/acre. When outfalling into a roadside ditch, the release rate shall be the lesser of the 0.165 cfs/acre or the proposed development's pro rata share of the bank full capacity (or the most restrictive flow rate at any downstream point) of the receiving ditch, considering the ditch at bank-full for the design tailwater condition. Supporting documentation should be submitted that demonstrates the calculations used to determine this share.

The percentage of impervious area used for the storage calculation shall include all areas that are paved or where gravel or crushed stone is used, all rooftops and other covered areas, and all other impervious surfaces, including the portion of the detention pond below the 100-year design water surface elevation.

The detention rate in acre-feet per acre for the 100-year storm event is shown in Figure 6-1 and Table 6-1. The acre-feet of flood control storage, S, to be provided by the facility for the 100-year storm event is calculated by multiplying the detention rate by the entire area served by the detention pond. Receiving ditches with reduced capacity will result in a lower release rate, which results in an increase in the detention rate, as shown in Figure 6-1 and Table 6-1. If used pumped detention, all requirements listed under Section 6.6 must be met.

The size of the outlet pipe that is required to pass the maximum allowable release rate during the 100-year storm is to be computed assuming outlet control (See Section 4.3.5), by establishing a maximum ponding level in the detention facility during the 100-year storm and assuming a tailwater at the top of the downstream end of the outlet pipe or at a depth in the outlet channel associated with the maximum release flow rate, whichever is higher. Restrictors smaller than four inches in diameter are not allowed. If the resulting release through a 4-inch restrictor exceeds the allowable release rate, the calculated detention volume shall be based on the allowable release rate through a hypothetically smaller restrictor.

6.4.2 For Drainage Areas ≥50 Acres and <640 Acres

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

6.4.3 For Drainage Areas ≥640 Acres

Detention ponds shall be analyzed for the 5-, 10-, and 100-year storm events and the design engineer shall determine no adverse impact by ensuring no increases in flow rates and/or water surface elevations. The 5-year event will be used for determining the sizing of the outflow structure to prevent downstream impacts and to define tailwater conditions for storm sewers entering the detention basin (see Section 5.3.2). The 10-year will also be used for sizing the outflow structure to prevent downstream impacts. The 100-year will be used for sizing both the required detention volume and the outflow structure to prevent downstream impacts, as well as determining tailwater conditions for storm sewers entering the detention basin (see Section 5.3.2) and configuring an emergency spillway (see Section 6.4.10).

The proposed detention system shall also be evaluated for a 500-yr flood to ensure no structural flooding for all structures served by the detention pond.

Once existing conditions are established, the new development with the detention facility shall be designed such that there is no increase in 10-, and 100-year flow rate and water surface elevation at any point along the receiving channel. If the receiving channel has less than a 10-year capacity, then the design of the detention facility shall also consider the 5-year event. Inflow and outflow hydrographs for detention analysis shall be determined following guidance from Chapter 2.

For detention ponds located outside of an identified 100-yr floodplain from either the Master Drainage Plan or the Flood Insurance Rate Map, whichever is wider, and not influenced by tailwater conditions, the HEC-HMS computer model can be used to size the facility and the outlet structure to ensure no adverse impacts. For detention ponds located within an identified 100-year floodplain, or influenced by tailwater conditions, unsteady-flow HEC-RAS, XP-SWMM, EPA-SWMM, Infoworks ICM, or ICPR 4.0 shall be used to size the facility and outlet structure to ensure no adverse impacts. Two-Dimensional (2D) components may be needed to adequately capture existing and/or proposed conditions. Refer to Section 3.7.3 for additional requirements if placing fill in the 100-year floodplain.

Additional models may be considered, however they should be presented to the FBCDD for approval prior to starting the analysis.

6.4.4 Minimum Detention Rate

The proposed detention system shall provide a detention rate no less than 0.65 ac-ft/acre of proposed development, regardless of the location within the County. This rate is exclusive of additional storage needed for floodplain fill mitigation (Section 3.7).

6.4.5 Maximum Release Rate

The maximum allowable release rate shall not exceed the release rate under existing conditions, nor the maximum release rate as determined by the FBCDD. Exceptions will be considered on a case-by-case basis, and require specific approval from FBCDD.

6.4.6 Design Tailwater Depth

In order for the adequate evaluation and sizing of detention facilities to be completed, tailwater conditions from the receiving stream or system shall be incorporated into the analysis. As described in 6.4.3, a detention pond not influenced by tailwater conditions can be evaluated using HEC-HMS; otherwise, unsteady flow HEC-RAS shall be used.

Incorporating the proposed development and detention facility into an unsteady-flow HEC-RAS model, should adequately capture tailwater conditions; this is the preferred method. If no unsteady-flow HEC-RAS model for the receiving stream is available, one shall be developed to adequately establish existing conditions.

If using HEC-HMS, variable tailwater conditions shall be established when evaluating each storm event. The variable tailwater stage hydrograph can be developed with the following method:

- Apply a 100-year flow hydrograph developed in HEC-HMS to a rating curve developed from a steady-flow HEC-RAS model that represents the receiving channel. This can be done with a single subbasin model to represent the watershed upstream of the outfall into the receiving stream.
- To develop the rating curve, run HEC-RAS profiles for 10% to 120% of the 100-year flow in 10% increments, 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 the variable tailwater condition when evaluating the pond.
- Re-run the HEC-HMS model for the 5-year and 10-year, and repeat this effort to define the 5-year and 10-year variable tailwaters.

In certain situations where this assumption for the variable tailwater may be found to not be reasonable, an alternative tailwater condition can be presented for approval to the FBCDD.

<u>**6.4.7 Allowable Drain Time Requirements**</u>

Allowable drain time is defined as the maximum allowable time to drain 80% of the peak 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 tailwater condition (free outfall), starting at the maximum water surface elevation in the detention basin from a 100-year storm event. The maximum drain time should be 48 hours to drain 80% of the volume. If a longer drain times are required, an increase in detention volume will be required:

- Increase volume by 5% if drain time exceeds 3 days.
- Increase volume by 10% if drain time exceeds 4 days.
- Increase volume by 15% if drain time exceeds 5 days.
- Increase volume by 20% if drain time exceeds 6 days.

- Increase volume by 25% if drain time exceeds 7 days.

The increased detention volume should not be included in the drain time evaluation, but must be documented in the drainage report and reflected in the design drawings. The maximum allowable drain time is 7 days. Detention ponds sized following the requirements of Section 6.4.1 are exempt from the allowable drain time requirement. Requirements for areas leveed areas are covered in Chapter 7 of this manual.

6.4.8 Final Sizing of Pond Storage and Outflow Structure

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

The lowest gutter elevations for streets surrounding or adjacent to the detention facility shall be designed to have minimum elevations above the 100-year pond elevation in the detention basin. No floodwater volume in streets or parking areas during a 100-year event can be counted as part of the required detention volume of the facility, with the exception of parking areas for developed sites less than 50 acres.

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

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

6.4.8.1 Accommodating Offsite Flows

Offsite flows draining to a detention facility are often overlooked, but can have a substantial effect on the total storage requirement for the detention basin. Although detention is not required for any offsite flow entering the site, the additional flow and/or volume needs to be accounted for as the offsite runoff passes through the proposed development.

To identify the size of a contributing offsite runoff to the detention facility, hydrologic methods described in Chapter 2 of this manual may be employed for the area draining from offsite. If difficulties arise in determining the offsite area, a 2D rain-on-mesh analysis can be used, with 100-year rainfall to define flow patterns and drainage boundaries. The use of the 100-year rainfall will allow the engineer to determine whether high points in topography may be overtopped during significant events leading to a larger (or smaller) drainage area for the offsite flow.

While not always possible, it is preferred that offsite drainage be diverted or rerouted so that it does not pass through the detention basin. This will allow the design engineer to size the detention basin volume for the onsite development only. For offsite flow that is rerouted around the detention pond, all proposed development shall evaluate pre-development drainage patterns of adjacent off-site, upstream property and provide accommodations to ensure no adverse impact post-development. This evaluation should specifically include, but is not limited to, an analysis of pre-development 100-year sheet flow conditions across the proposed development and the design of necessary drainage infrastructure to sufficiently intercept and convey these flows around/through the new development.

When not possible, flows should be accounted for in the hydrologic and hydraulic modeling of the detention basin, and included in the calculations of pre-development as well as post-development conditions flow, as this will set the threshold for allowable outflow from the detention basin.

The analysis of offsite flow onto a proposed development site should include calculations to demonstrate that any infrastructure used to intercept/convey offsite flows is sized adequately to not back water onto or otherwise cause impacts to the offsite areas.

6.4.8.2 Development Within an Overflow Zone

All proposed developments located within an area identified as part of an overflow zone shall be evaluated using 2D modeling. The hydraulic analysis must be completed in a way that demonstrates no adverse impact on offsite properties for flood risk up to and including the 100-yr storm event, and the design engineer shall strive to maintain existing flow patterns as closely as possible. No concentrating of flows leaving the property will be permitted without the notification/legal agreement of the neighboring (receiving) property owners.

Development shall also be checked for changes to flood risk during the 500-yr storm. At the discretion of the FBCDD Engineer, a no adverse impact for the 500-yr storm may be required.

6.4.8.3 Detention Associated with Rerouted Drainage Areas

Occasionally, it may be necessary to reroute drainage from one area to another. When doing so, this may require additional storage in the system to mitigate runoff down to a level that the receiving system is able to accommodate. Examples of this may be when drainage areas are diverted into an existing engineered drainage system, an adjacent stream, or into an area protected by a levee.

If a drainage area is diverted into an existing engineered drainage system (existing development), the following should be considered:

- Do not exceed the release rate of existing area drainage into the engineered system. For example, if 10 acres currently drain into an existing storm sewer location, but due to a diversion of runoff, a total of 50 acres is being proposed to drain to that same storm sewer location, the peak flow into the storm sewer must not exceed release of the original 10 acres.

- Provide additional storage in a detention pond equivalent to 100% of the 100-year runoff being diverted/rerouted.
- Meet the drain time requirement specified in Section 6.4.7.
- Extend analysis past receiving development. FBCDD should be consulted on how far to extend the analysis.

If additional drainage area is diverted into an area that then outfalls into an adjacent stream, consider the following criteria:

- For areas less than 50 acres, provide detention so that the release rate does not exceed the runoff rate from the existing service area and provide additional storage equal to 100% of the 100-year runoff from the added area.
- For areas greater than 50 acres, complete a detailed no adverse impact study with unsteady-flow HEC-RAS.

Lastly, if rerouting a drainage area into an existing area protected by levees, the following criteria applies:

- If the additional drainage area was accounted for in the original drainage design as undeveloped, do not exceed the original design release rate and provide adequate storage to <u>retain</u> (no release) 100% of the increased runoff volume under proposed conditions during the 100-year storm.
- If the additional area was not accounted for in the original drainage design of levee, the engineer has two options:
    1) Retain 100% of volume from the additional area on-site and release only when the downstream system is empty. No study is required; however, the engineer will need to obtain approval from the Levee Improvement District (LID).
    2) Update the hydrologic and hydraulic analysis of the LID, in accordance with Chapter 7 of this manual, evaluate gravity flow and coincident events, and demonstrate no adverse impact. Again, the engineer will need to obtain approval from the LID.

6.4.9 Allowances for Extreme Storm Events

Design considerations must be given to storm events in excess of the 100-year flood. An emergency spillway, overflow structure, or swale must be provided as necessary to effectively handle the extreme storm event. The emergency spillway, overflow structure, or swale (collectively referred to as the “emergency spillway”) should be designed with the following criteria:

- Set the emergency spillway crest elevation at or above the 100-year pond elevation.
- Size the emergency spillway with a maximum of one foot of depth to carry the 100-year flow so that the 100-year detention storage does not exceed the top-of-bank of the facility. If emergency spillway is flowing into a street or swale, a discharge coefficient of 0.5 should be used for these calculations. If emergency spillway is over an embankment or discharging directly into a stream, use a discharge coefficient of 2.6.
- Assume that the principal outlet structure is clogged.
- 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.
- All slabs will be required to be a minimum of two feet above the 100-year water surface elevation under normal conditions. Under emergency overflow/extreme event conditions, one foot of freeboard shall be maintained.
- 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.

See Chapter 5 of this manual for additional criteria for extreme event swale design and sizing.

6.4.10 Erosion Controls

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

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

6.4.11 Special Provisions

Several unique situations exist in the design of detention within Fort Bend County which require special attention. Each condition listed below has provisions that are in addition to the previously stated detention criteria.

6.4.11.1 Interconnected Ponds

Detention ponds which are interconnected (i.e., which drain to other ponds) shall be analyzed in detail using one of the following programs: HEC-RAS, XP-SWMM, EPA-SWMM, Infoworks ICM, or ICPR (Version 4 or newer). The use of any other computer programs for analysis of interconnected ponds will require pre-approval from the FBCDD. In no case will the use of HEC-HMS be permitted due to the lack of backflow capabilities within HEC-HMS.

Each pond shall be analyzed as a separate pond connected by a culvert/structure to another pond. The assumption that adjacent or connected ponds will function in unison as a single volume will not be permitted due to the likelihood that the interconnected volumes will vary with time.

To meet the minimum required detention volume, volume shall be calculated based on max pool elevation in each pond from the analysis.

A system of extreme event flow paths for interconnected ponds must ensure runoff from a 500-year event can be routed through the proposed system without resulting in structural flooding. Deviation from the general flow path from one pond to the next will require pre-approval from FBCDD.

In situations where the connection between ponds is intended to serve as a restrictor to increase the design water surface elevation on the upstream pond by more than 2-feet, the engineer shall avoid using the conduit under the proposed roadway as a restrictor. Instead, the restriction should be located upstream of the roadway crossing in a separate structure that also includes an overflow spillway for water surface elevations greater than those experienced during the 100-year event. The conduit under the roadway in between the two ponds should be sized to convey the unrestricted 100-yr flows with a maximum headloss of 6-inches. Figure 6-3 offers an alternative to accommodate this situation.

The outfall for the interconnected pond system shall be sized in accordance with Sections 6.4.8 and 6.4.9.

The design engineer shall provide a map of the defined extreme event flow paths as part of the submittal to FBCDD for review, as well as obtain a flowage easement to provide a clear path for conveyance without affecting adjacent property owners.

6.4.11.2 Offsite / Off-Line Detention

For detention basins located offsite and which are off-line facilities, no adverse impact must be demonstrated through the use of unsteady flow HEC-RAS modeling of the detention basin and receiving channel.

The unsteady-flow HEC-RAS model must adequately account for any fill in the floodplain, interaction between the receiving stream and the detention pond, change in drainage patterns, runoff generated from the site, and runoff generated by the drainage area contributing to the receiving channel.

6.4.11.3 Offsite / In-Line Detention Along a Channel

In-line detention will not be permitted without specific approval from the FBCDD, which will consider such requests on a case-by-case basis. If approved, an unsteady-flow HEC-RAS analysis will be required which demonstrates no adverse impacts to flood levels.

Additionally, such in-line basins are likely subject to environmental permits (Clean Water Act, 404 permits, jurisdictional waters, etc.). The design engineer is responsible for obtaining such permits. FBCDD does not regulate environmental permits. Approval of the detention design by FBCDD does not pertain to environmental regulation and the owner is responsible to obtain all environmental permits required for the project.

If the in-line detention is located within a designated floodway, it is the responsibility of the engineer to coordinate with the local floodplain administrator to determine whether a revision to the Flood Insurance Rate Map is warranted. FBCDD is not the local floodplain administrator.

Due consideration must be given to the consequences of a failure, and if a significant hazard exists, the control structure must be adequately designed to prevent such hazards. In addition, detention facilities which measure greater than six feet in height are subject to Title 30 Texas Administrative Code (TAC) Chapter 299 (Subchapters A through E), effective January 1, 2009, and all subsequent changes. The height of a control structure, detention facility or dam is defined as the distance from the lowest point on the crest of the dam (or embankment), excluding spillways, to the lowest elevation on the centerline or downstream toe of the dam (or embankment) including the natural stream channel. Subchapters A through E of Chapter 299 classifies dam sizes and hazard potential and specify required failure analyses and spillway design flood criteria. The following is a link to a copy of these sections of the TAC (https://texreg.sos.state.tx.us/public/readtac$ext.ViewTAC?tac_view=4&ti=30&pt=1&ch=299).

If the in-line structure meets the TCEQ definition of a "dam," the design engineer will need to coordinate with TCEQ and supply that agency with any required additional documentation/analyses to meet TCEQ regulations for dam safety. It is not necessary to provide FBCDD with this additional information.

6.5 MULTIPURPOSE LAND USE

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

If proposed development is less than 50-acres, a parking lot may be used as part of the detention system, provided that the maximum depth of water over an inlet does not exceed nine (9) inches and the maximum depth in the parking stall does not exceed six (6) inches.

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

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

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

1. Safe to the public

2. Easy to maintain (refer to pertinent Section 6.4 of the *Harris County Flood Control District Policy, Criteria, and Procedure Manual* relating to wet bottom detention facilities; please note that the use of 3:1 side slopes are subject to verification by a geotechnical engineer)

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

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

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

2. Provisions are made for the facility to be adequately maintained per Section 6.8.1.

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

6.6 PUMPED DETENTION

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

1. Pumped detention is permissible only when a gravity system is not feasible from an engineering and economic standpoint.

2. At least 50% of the total pond volume shall be drained by gravity. Gravity outflow shall be provided above the pumped storage.

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

4. The selected design outflow rate must not aggravate downstream flooding. As an 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. This may require multiple pumps that turn on and off as stages rise and fall, respectively, within the pond, resulting in no impact to 5-, 10-, or 100-year flood flows on the receiving system.

5. The drainage report must include a section describing how the pump station should be operated.

6. Pumped detention is subject to maximum drain time requirements (see Section 6.4.7). If this condition conflicts with #4 above, the engineer will need to contact the FBCDD for direction.

7. Pumped detention shall be routed to a junction box, stilling basin, or manhole to dissipate the energy from the pump outlet prior to gravity flow into a channel. Limited gravity flow should be achieved by an appropriately sized restrictor pipe draining the junction box, stilling basin, or manhole. The outflow velocity into the channel shall not exceed three feet per second (3 fps).

8. Detailed hydrologic and hydraulic analyses shall be performed to determine the detention volume needed and to size the pumps and gravity outflow structure. These calculations shall clearly show how the pump system and gravity outflows work to satisfy the outflow criteria.

9. Backflow prevention through the use of flapgates or tideflex valves should be considered to prevent high tailwater conditions downstream of the pump system from entering the pumped storage area.

10. Redundancy and backup power shall be provided for emergency conditions. Installation of a quick connect for a mobile generator shall also be provided.

11. Pumped detention systems can be operated and maintained by a Municipal Utility District or any other public agency with mandated maintenance responsibilities.

12. An operation and maintenance plan shall be prepared and provided to the owner. This plan shall include guidance on staged operation of multiple pumps to reduce the risk of surcharge to the electrical system, which can cause failure or reduction of efficiency in pump operation.

13. Adequate assurance shall be provided that the system will be operated and maintained on a continuous basis.

14. Fencing and locking of the control panel shall be provided to discourage unauthorized operation and vandalism.

15. Emergency contact information for the owners(s), engineer, and operator responsible for operations and maintenance shall be provided to the FBCDD. FBCDD will refer calls received during floods concerning pumped detention basins and their service area to these individuals.

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

6.7 GEOTECHNICAL INVESTIGATION

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

1. The ground water conditions at the proposed site;

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

3. If a dam is to be constructed, adequate investigation shall be undertaken with respect to potential seepage problems under and/or through the dam or around conduits penetrating the embankment, attendant seepage control requirements, the suitability of foundation materials, the availability of suitable embankment material, the lift thickness during construction, and the stability requirements for the dam itself;

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

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

6.8 GENERAL REQUIREMENTS FOR DETENTION FACILITIES

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

In addition, the following guidelines are applicable:

1. 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 3:1.

   The bottom slopes of the detention basin should be graded toward the pilot channel at a minimum slope of 0.01 ft/ft.

   In the event of approval by FBCDD for in-line detention, the detention facility may not be required to have a pilot channel, but should be built in accordance with the requirements for open channels as outlined in Chapter 3. The exception to Chapter 3 requirements is that side slopes in detention basins may have a maximum slope of 3:1.

   In the case of vertical-walled detention, the maximum height of the vertical walls shall be four feet. Heights above four feet shall be installed in no more than two-foot increments with a two-foot wide (minimum) flat shelf between increments. Maintenance access is required for maintenance equipment to conduct operations in the bottom of the detention facility. Exceptions will be considered on a case-by-case basis.

2. Wet Pond Bottom Design – Ponds with a permanent pool should 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.

   Walls at the water's edge (bulkheads) are permissible only if the owner of the detention basin agrees to maintain them. A bottom shelf shall be included with the water edge wall.

   Permanent pools shall be a minimum of 6 feet deep and a maximum of 8 feet deep depending on soils, geometry, and habitat goals.

   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 cannot be used alone in a pond bottom due to maintenance issues.

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

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

6.8.1 Maintenance

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

A 30-foot wide access and maintenance easement shall be provided from street, road or adequate access way to and around any drainage ditch, channel or the entire detention pond. This is in addition to the dedication required for the pond itself. Figure 6-3 shows the minimum criteria for maintenance berms in different development scenarios.

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

6.9 STORM WATER QUALITY BMPs AND PHASE II NPDES PERMIT

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

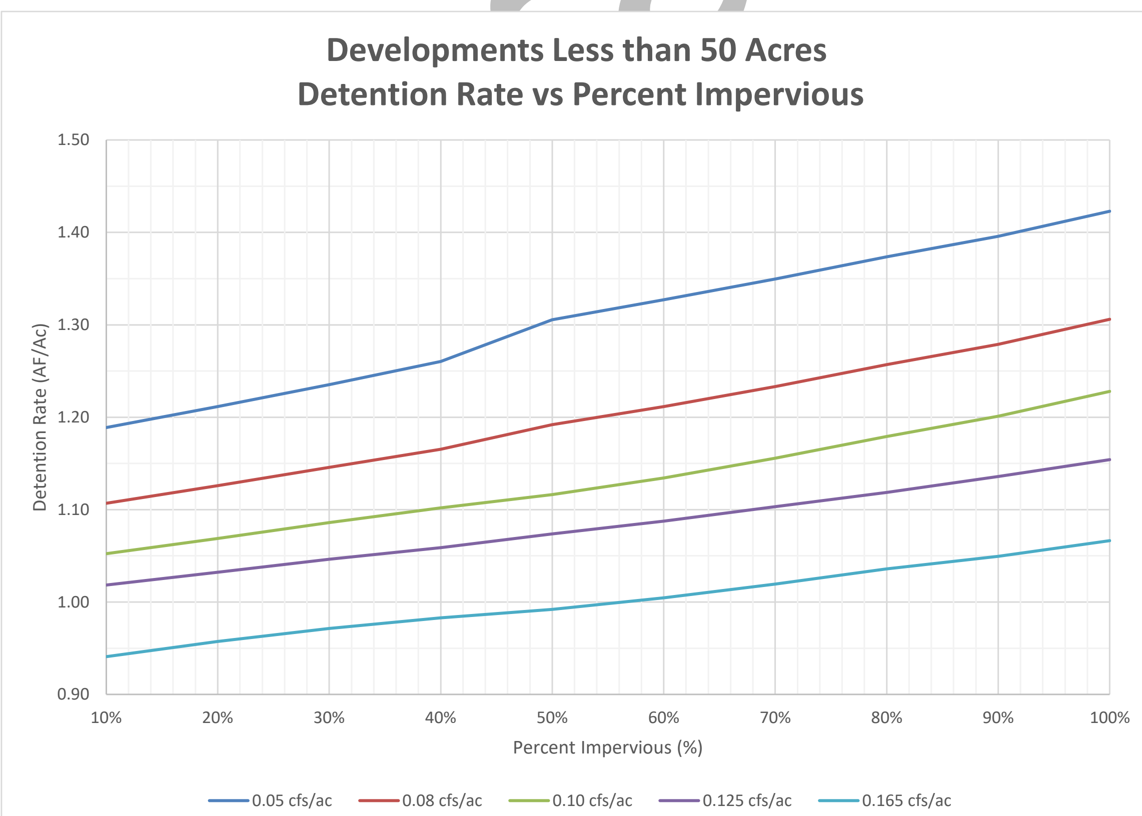
TABLE 6-1

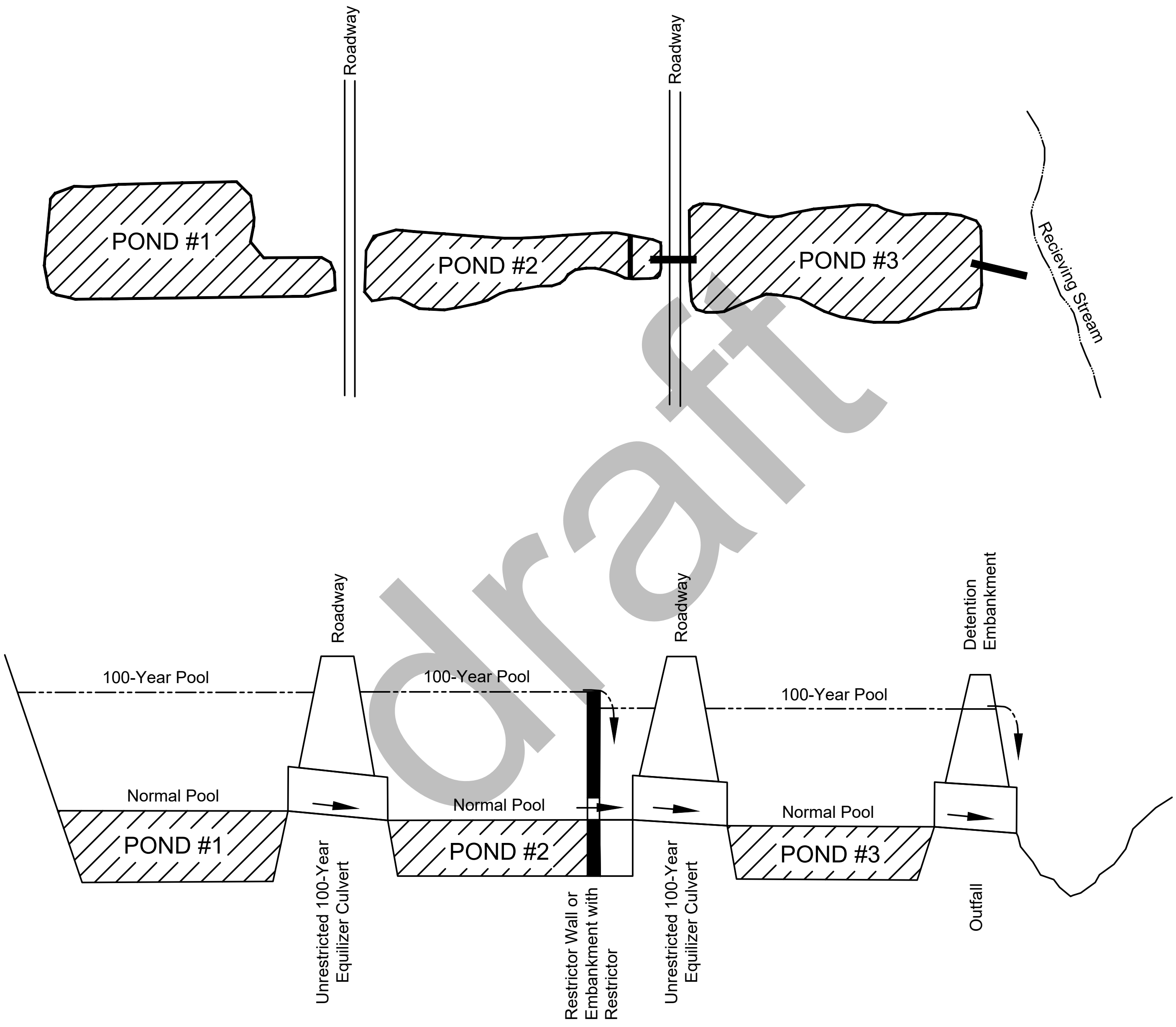
MINIMUM REQUIRED DETENTION VOLUME FOR DRAINAGE AREAS LESS THAN 50 ACRES

	Release rate (cfs/acre)				
% Impervious	**0.05 cfs/ac**	**0.08 cfs/ac**	**0.10 cfs/ac**	**0.125 cfs/ac**	**0.165 cfs/ac**
10%	1.189	1.107	1.052	1.018	0.941
20%	1.211	1.126	1.069	1.032	0.957
30%	1.235	1.146	1.086	1.046	0.971
40%	1.260	1.165	1.102	1.059	0.983
50%	1.305	1.192	1.116	1.074	0.992
60%	1.327	1.211	1.134	1.088	1.004
70%	1.350	1.233	1.156	1.103	1.020
80%	1.374	1.257	1.179	1.119	1.036
90%	1.396	1.279	1.201	1.136	1.049
100%	1.423	1.306	1.228	1.154	1.066

FIGURE 6-1

MINIMUM REQUIRED DETENTION VOLUME FOR DRAINAGE AREAS LESS THAN 50 ACRES





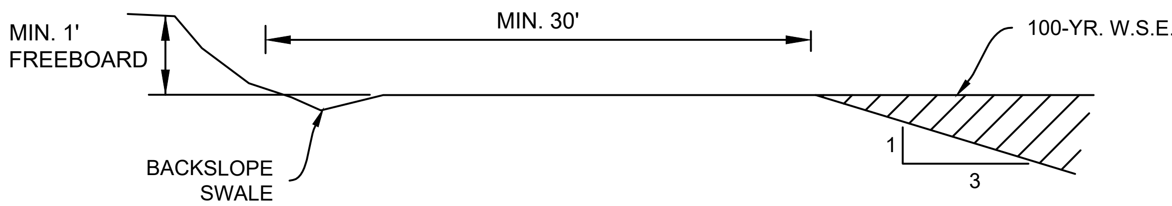
NOT TO SCALE

OPTION FOR LIMITING INUNDATION OF ROADWAYS DURING EMERGENCY OVERFLOWS

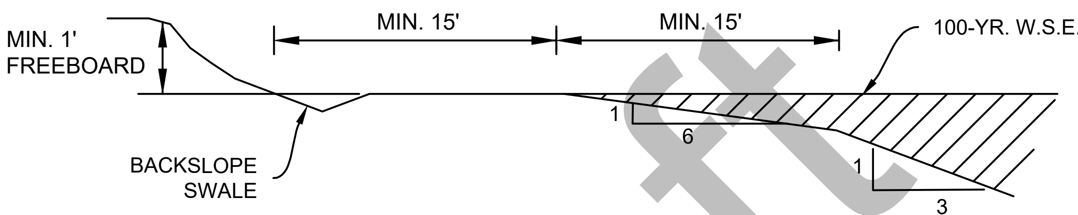
APRIL 2022

FIGURE 6-2

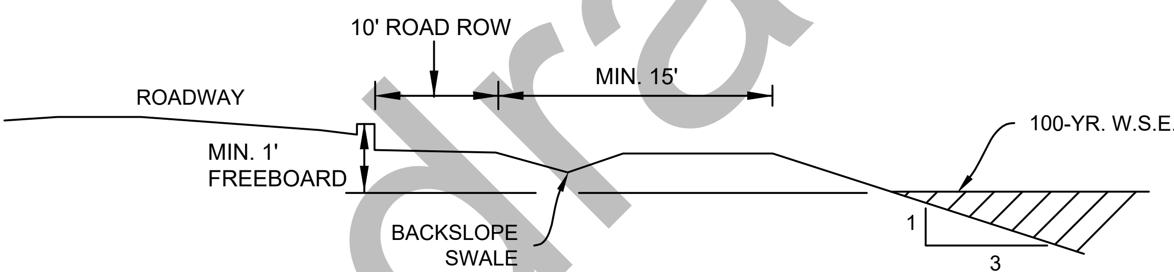
PER FBCDCM:



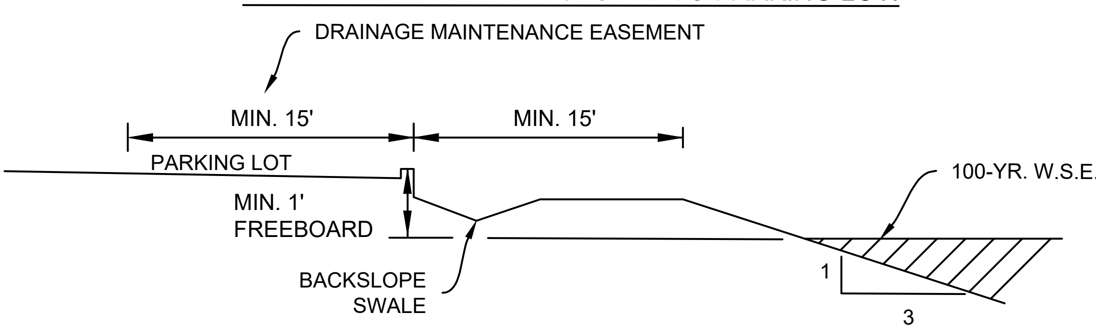
ALLOWED WITH MUD APPROVAL:



MAINTENANCE BERM ADJACENT TO ROAD ROW:



MAINTENANCE BERM ADJACENT TO PARKING LOT:



DETENTION POND MAINTENANCE BERM MINIMUM CRITERIA	
APRIL 2022	FIGURE 6-3

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7.0 LEVEED AREAS

7.1 GENERAL

Flood plains, which cover a significant area within Fort Bend County, may be developed within the floodplain if a levee system is constructed to protect the area from high water levels on the adjacent watercourse (usually the Brazos River), and no adverse impact is demonstrated. The components of the levee system shall include an internal drainage system, a levee, a pump station, adequate storage capacity and a gravity outlet with an outfall channel to the river. New leveed areas without a pump station will not be allowed. Mechanisms shall be provided at the gravity outlet pipe to prevent backflow from the river into the protected area. The Fort Bend County design criteria for each component are defined in the following sections.

This Section shall be applicable in its entirety for new levee-protected areas. New development and/or new drainage systems within established Levee Improvement Districts shall comply with this Section with freeboard requirements as noted in Section 7.4.2.

This section does not apply to fully built-out Levee Improvement Districts; however, the Engineer shall evaluate and document how the internal drainage system performs under the probabilistic approach described in Section 7.6 in submittals for drainage upgrades. Although not mandatory for built-out levee systems, the Fort Bend County Drainage District encourages Levee Improvement Districts to upgrade the existing facilities in accordance to the guidelines of this Section.

Design of new levees and pump stations, regardless of the date of plat approval, must follow the design standards outlined in Sections 7.5 and 7.7. At discretion of the Fort Bend County Drainage District, design of levees and related drainage elements in levee-protected areas, including construction plans and pump station design drawings, may be subject to a third-party review on the technical elements and compliance with the requirements of this Section.

7.2 APPLICABLE FEDERAL DESIGN GUIDELINES

The County'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. In general, FEMA is not responsible for building, maintaining, operating, or certifying levee systems. FEMA does, however, develop and enforce the regulatory and procedural requirements that are used to determine whether a completed levee system should be credited with providing 100-year (1% Annual Exceedance Probability) flood protection. These requirements are documented in Section 65.10 of the National Flood Insurance Program (NFIP) regulations. FEMA also assists with mapping and floodplain management for issues related to levees. USACE addresses a range of operation and maintenance, risk communication, risk management, and risk reduction issues as part of its responsibilities under its Levee Safety Program. Depending on the levee system, FEMA and USACE may be involved with the levee sponsor and community independently or, when a levee system overlaps both agency programs, jointly. The engineer is advised to check the current FEMA and USACE rules and regulations. Operation and maintenance of these facilities is not the responsibility of the Fort Bend County Drainage District.

Levees within Fort Bend County may be identified as either federal or non-federal levees on USACE Nation Levee Database website. Owners of non-federal levees, at their own discretion, may elect to 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 project (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 must meet certain criteria and standards set forth by USACE. The RIP requires non-federal sponsors to have an initial Eligibility Inspection and 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 usefulness 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.

7.3 DESIGN FREQUENCY EVENTS

7.3.1 Levees

The levee system shall include a levee embankment that will protect the development from the 100-year frequency flood event on the adjacent watercourse at a minimum. Protection shall consider the water surface elevation on the watercourse and any associated wind and wave action. A minimum free board above the 100-year water surface elevation in the adjacent watercourse shall be provided in accordance with this Section.

7.3.2 Internal Drainage System

The landside drainage system shall be capable to evacuate the runoff from the one-percent annual chance rainfall event in the landside area entirely by gravity when the system is not influenced by tailwater effect of the watercourse. When operating entirely by gravity, the system is subject to the requirements of Sections 3.0 and 5.0 of this Manual.

The internal drainage system, composed of a gravity drainage system, storage and pumps, shall have a combined capacity to evacuate the runoff produced from the landside of the levee so that the joint probability of flooding considering the risk from riverine and local runoff sources is less than one percent every year. Additional analysis required to meet these criteria are listed in Section 7.6. Considerations to obtain an optimal combination of storage and pump capacity are included in Section 7.6.2.

7.4 FREEBOARD REQUIREMENTS

7.4.1 Levees

All new levee districts in Fort Bend County shall have a minimum of four feet of freeboard at all locations including U-shaped levees and tie-ins to natural ground. (This requirement exceeds by one foot the minimum freeboard required by FEMA in Section 65.10 of the National Flood Insurance Program.) An additional 1.0 foot of freeboard is required within 100 feet on either side of structures that are situated riverward of the levee or wherever the flow is constricted.

Although not mandatory for built-out levee systems, the Fort Bend County Drainage District encourages existing Levee Improvement Districts to upgrade their existing facilities in accordance to the guidelines of this Section.

Occasionally, exceptions to the minimum riverine freeboard requirements noted above may be approved at the discretion of the Fort Bend County Drainage District after a review of the following information:

1. Engineering analyses that demonstrate adequate protection with a lesser freeboard are provided.

2. Evaluation of the uncertainty in the estimated base flood elevation profile including at a minimum: a) an assessment of statistical confidence limits of the 1% AEP discharge; b) changes in stage-discharge relationships; and c) sources, potential, and magnitude of debris and sediment.

3. Analyses that demonstrate the levee will remain structurally stable during the base flood when additional loading considerations are imposed.

Under no circumstances will freeboard of less than 2.0 feet above the BFE be accepted.

7.4.2 <u>Minimum Slab Elevation Requirements</u>

For new sections of development, in a newly formed Levee Improvement District, slab elevations shall be located at least 24 inches above the water elevation from events that have 1% annual chance of exceedance. Possible events with 1% annual chance include a local severe storm without backwater effects from the Brazos or a coincident event of high-water level in the Brazos and a heavy local storm, as described in Section 7.6.

When new sections of development are proposed inside an existing Levee Improvement District, improvements to the internal drainage system and/or pump station shall be required to ensure existing structures meet 1% criteria using Atlas 14 freeboard requirements as referenced below:

1. Pre-January 1, 2020: No structural flooding

2. After January 1, 2020: 24 inches, or as mandated in future guidance in Fort Bend County after this Manual becomes effective.

For fully built-out Levee Improvement Districts, the Engineer shall evaluate the water elevations and/or freeboard during the 1% annual chance coincident events, and the Engineer is encouraged to consider and propose upgrades to prevent structural flooding of existing structures during storm events that have 1% annual chance of exceedance.

7.5 DESIGN CRITERIA FOR LEVEES

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

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

2. The foundation area shall be cleared, grubbed, and stripped for the full width of the levee to remove all objects and/or obstructions including the removal of all grass, trees, and surface root systems.

3. Embankment material shall be CH or CL as classified under the Unified Soil Classification System and shall generally have the following properties:
    - Liquid Limit greater than or equal to 30.
    - Plasticity Index greater than or equal to 15.
    - 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. The embankment material shall be compacted to a minimum density of 95% using the standard proctor compaction test at approximately plus or minus three percent optimum moisture content. The embankment material shall be placed and compacted in loose lifts of not more than 12 inches thick.

6. The levee top and side slopes shall be adequately protected by grass cover or other suitable material to safeguard against erosion.

7. The minimum levee top width shall be twelve feet, or as required to facilitate operations and maintenance activities.

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

9. The levee shall be continuous and shall either completely encompass the development or tie into natural ground located outside of the limits of the adjacent watercourse's 100-year floodplain with consideration of freeboard requirements per Section 7.4.

10. All conduits passing through the levee shall have at least two means of closure for redundancy. As an example, a flap gate on the river side and a sluice gate on the land side (normally open and ready to be closed in case the flap gate does not prevent back-flow) is a configuration that tends to work well in most cases and allows for pump testing. However. the Engineer shall consider options for closure suitable for the project conditions.

11. All pipes and conduits passing through the levee shall have seepage control devices, backflow control provisions, and slope protection. The design of these structures shall be compliant with FEMA's technical manual for conduits through embankments (FEMA 484).

12. Where pipes or conduits are to be constructed through new or existing levees:

    a. Seepage rings or collars should not be provided for the purpose of increasing seepage resistance. Except as provided herein, such features should only be included as necessary for coupling of pipe sections or to accommodate differential movement on yielding foundations. When needed for these purposes, collars with a minimum projection from the pipe surface should be used.

b. A 0.45-m (18-in) annular thickness of drainage fill should be provided around the landside third of the pipe, regardless of the size and type of pipe to be used, where landside levee zoning does not provide for such drainage fill. For pipe installations within the levee foundation, the 0.45-m (18-in) annular thickness of drainage fill shall also be provided, to include a landside outlet through a blind drain to ground surface at the levee toe, connection with previous underseepage features, or through an annular drainage fill outlet to ground surface around a manhole structure. Figure 7-2 shows typical sections of drainage structures through levees and various control structure arrangements.

Note: Seepage control devices have been employed in the past to prevent piping or erosion along the outside wall of the pipe. The term "seepage control devices" usually referred to metal diaphragms (seepage fins) or concrete collars that extended from the pipe into the backfill material. The diaphragms and collars were often referred to as "seepage rings." However, many piping failures have occurred in the past where seepage rings were used. Assessment of these failures indicated that the presence of seepage rings often results in poorly compacted backfill at its contact with the structure.

13. The minimum right-of-way for the levee shall be extended 50 feet away from the river side toe and 30 feet from the land side toe to provide enough space to maintain vegetation as required by USACE PL84-99 and to provide space for raising the levee if needed in the future. If the levee exceeds the requirements of this Section, the right-of-way width may be reduced subject to a justification by the Engineer and approval by the Fort Bend County Drainage District, but under no circumstances, the right-of-way shall be less than 30 feet away from each toe. In addition, the establishment of an easement for maintenance and access, which may be located within the right-of-way, shall be required.

14. Additional easements should be established in at least two locations to provide controlled access to the levee from nearby public roads.

15. The levee should be regularly inspected for any visual deficiencies. During flood events, continuous patrols of the levee should be performed with special awareness for unusual wetness on the landward slope.

16. The levee system and appurtenant structures shall be maintained in such a way to ensure serviceability of the structures during normal and flood conditions.

7.6 DESIGN CAPACITY OF PUMPING SYSTEMS

In this Section, the term failure refers to a situation where the critical elevation in the landside area is reached. The critical elevation is defined as the lowest slab elevation in the protected area minus the adopted internal freeboard, defined in Section 5.0 for new developments. Figure 7-1 illustrates the relative elevation of the different variables involved in assessing the flood risk of a levee-protected area.

The design of the interior drainage system of a levee-protected area shall consider the influence of two sources on flood risk: the first source of interest is the watercourse itself, which may rise to the point where the capacity of the gravity flow drainage in the landside is impacted or eliminated. The second source is the runoff from a storm event occurring in the landside system which may be high enough to cause structural flooding if the combined capacity of pumping and gravity flow is exceeded.

Each of these two flooding sources has its own probability of occurrence, and the interior drainage system, composed of a gravity outfall and pumps, shall be designed to obtain a joint probability of failure of less than one percent in any given year. In the case of a levee system protecting from a flooding source other than the Brazos River, the pumping system shall be assessed assuming the two flooding sources are equal to a 1% annual exceedance probability (100-yr), 24-hour event. For the Brazos River, the probability of each source can be found as follows:

1. For the first source, the watercourse, the probability distribution is described by a Flood Frequency Analysis of water levels at the location of the gravity outfall. For the Brazos River, this probability distribution is described by the water surface elevation profiles along the Brazos River for various frequency events (2-yr to 500-

yr), included in Appendix 1 of this Manual. This information is applicable to the Brazos River only and may be different as the outfall is farther away from the river. It is recommended that the water surface elevations at the location of the gravity outfall be requested from the Fort Bend County Drainage District, especially for situations where the outfall is not adjacent to the river.

2. For the second source of flooding, the runoff from the landside of the levee, the probability is defined by the depth-duration-frequency curves for Fort Bend County included in Section 2.0 of this Manual.

For the purpose of assessing the flood risk in a levee-protected area along the Brazos River, the two sources of flooding are assumed to be independent from each other. This assumption is reasonable because of the difference in the order of magnitude between the Brazos River and the landside system watersheds. Because the Brazos River watershed is on the order of thousands of square miles, the peak stages in Fort Bend County occur several days or even weeks after a rainfall event in the upper portion of the watershed. On the other hand, drainage areas of the local systems in the protected areas are measured in acres or a few square miles and the runoff peaks within the same day of the storm. The contribution of local rainfall to the peak flow of the Brazos River is very small, if not negligible.

Using probability concepts, the probability that two independent events will occur at the same time is equal to the product of the probabilities of each event. Then, for each peak water surface elevation in the Brazos River, there is a corresponding storm event in the landside contributing area that results in a joint probability (i.e., both events happening at the same time) of one percent in any given year (100-year). For example, if the Brazos River peak elevation is greater than or equal to the 4% annual exceedance probability AEP), or 25-year flood elevation, the storm in the landside of the levee (local system) would need to exceed the 25% AEP (4-year) flood event to have a combined annual probability of one percent. Likewise, the probability of having the water elevation in the Brazos exceeding the 20% AEP (5-year) water elevation and a local rainfall event exceeding the 5% AEP (20-year) flood event at the same time is equal to one percent.

The flood risk analysis should consider that there are numerous combinations of peak water elevations and local rainfall that have a combined probability of one percent and that the consequences from each combination is different. In these analyses, the base flood elevation (1% annual exceedance) and other frequency elevations shall be measured at the location of the gravity outfall. As water surface elevation in the Brazos River increases, the capacity of the gravity flow is decreased, and at some point, pumping is required to avoid structural flooding. As water level in the Brazos River increases, the capacity lost in the gravity flow is not necessarily the same as the capacity gained by additional pumping. This trade-off depends on the configuration of the system and the pumps operating rules. Then, simulations of the entire internal drainage and pumping systems should be completed for a range of scenarios that have one percent annual chance of occurrence, assuming that the system starts empty. Peak water elevations in the landside system and available freeboard should be calculated for each scenario. The goal of these simulations is to verify there is not a system failure (i.e., the Critical Elevation is not exceeded) for any of the simulated scenarios that have 1% annual chance of occurrence.

It is required that the Engineer completes at a minimum seven simulations of the landside drainage system, which shall be inclusive of the pump station, the available storage, and the internal drainage system. The seven simulations cover various scenarios, some of them allowing for partial gravity flow out of the leveed area, which could occur for situations when the Brazos River water surface elevation is below the Critical Elevation; and other scenarios that would rely solely on the pump station. It is important to clarify that partial gravity flow may not be feasible in all levee systems as this depends on the configuration and operation of the gravity outfall, and associated gates, during high water levels on the Brazos River.

In order to assist in the development of the seven scenarios and to document the design process of the interior drainage system of a levee-protected area, a spreadsheet was developed and is available from the Fort Bend County Drainage District. The use of this spreadsheet is mandatory for all new leveed areas or modifications to drainage systems in existing levee-protected areas.

The spreadsheet requires as input the water surface elevation in the Brazos River for various annual frequencies (2-, 5-, 10-, 25-, 50-, 100-, and 500- year) at the existing or proposed outfall location, the lowest slab elevation, the target freeboard, and the elevations and size of the gravity outfall. The spreadsheet then summarizes the precipitation depths that shall be used to evaluate the internal drainage system; these precipitation values correspond to a storm event with a duration of 24-hours. The simulations of the internal drainage system should be completed following the requirements outlined in Section 5.0, and avoid the use of level pool routing software, such as HEC-HMS. The peak water surface elevations in the landside system, which may occur in locations away from the pump station, should be documented for all seven scenarios. The scenario with the minimum freeboard shall be adopted as the controlling scenario, and shall meet or exceed the requirements specified in this section.

The spreadsheet is formatted to produce an output in 11x17 format for easy inclusion in the drainage report that will be submitted for review and approval to the Fort Bend County Drainage District.

The design engineer should include the following information in the drainage report:

- Controlling Slab: Lowest slab elevation within leveed area
- Critical Elevation: Lowest Slab minus Freeboard; This is the maximum allowable water surface elevation when designing the pump station.
- Maximum tailwater elevation: The highest elevation between: 1) the WSEL on the Brazos River, or 2) the controlling high point along the discharge pipeline.
- Maximum internal WSEL for each one of the coincident events evaluated.
- For each pump station provide:
    - Number of pumps
    - Type of pumps
    - On/Off elevations for each pump
    - Pump curves with enough detail within the operating range
    - Total pump capacity (gpm and cfs) for a pumping head corresponding to the difference between the maximum tailwater elevation, and the critical elevation.
    - Firm pump capacity (gpm and cfs) assuming one of the largest pumps is out of commission.
    - Hydraulic profile showing key elevations, water surface elevations, flow rates, and head losses under various conditions. At minimum include:
        - Intake elevation
        - Outfall elevation
        - Minimum water surface elevation for pumping
        - Controlling high point along the discharge pipeline

7.6.1 Drain Time

The pump station shall have a minimum capacity to drain the storage volume in less than 24 hours. The drain time is calculated as the time it takes for the pumps to lower the storage from the Critical Elevation to 20% of the total storage without the use of the redundant pumps and assuming no runoff inflow. This storage is not considered storm water detention and accordingly, the provision of drain time in Section 6 are not applicable to levee-protected areas.

7.6.2 Optimal Design Considerations

The design process of a levee system and its internal drainage system shall investigate the benefits of added storage in reducing the required capacity of the pumps. The runoff that cannot leave the system by gravity can be temporality held in a storage unit and be evacuated by pumping at an acceptable rate. Systems with small amounts of storage, may require large pumping systems, which are costly and, depending on the size, may require more specialized mechanical and electrical design. Increasing the storage will reduce the capacity of the pumps but at the same will increase the land that should be reserved for this storm water storage. Several combinations of available storage capacity with the corresponding changes in pumping capacity should be explored to select the alternative that is economically optimal and technically feasible.

7.6.3 Example of Pump Station Design Capacity

A levee-protected area is proposed in 2,500 acres of land adjacent to the Brazos River in Fort Bend County. Information about the frequency of floods in the Brazos River (as obtained from the hydraulic models and entered in the cells shaded in blue in the FBCDD spreadsheet) at the location of the gravity outfall is summarized in Table 7-1.

Table 7-1

Return Frequency Elevation Table for the Brazos River used in Example

Return Event (Years)	**Annual Exceedence Probability**	**Water Surface Elevation (feet msl)**
2	50%	55.96
5	20%	66.24
10	10%	68.34
25	4%	69.68
50	2%	70.97
100	1%	72.25
500	0.2%	74.10

The proposed development allows for a good amount of internal storage. The lowest slab elevation in this proposed development is 67.5 feet with a minimum freeboard of 2.0 feet.

Solution:

If the minimum freeboard is 2.0 feet and the lowest slab elevation is 67.5, the critical elevation is 65.5. If the Brazos River reaches the critical elevation, the entire outflow from the development will entirely be pumped. The return period of this elevation can be estimated by interpolation of the numbers in Table 7-1. The return period of the critical elevation is approximately 4.5 years, as calculated in the spreadsheet.

The Engineer will need to analyze the performance of the overall drainage system (internal drainage, storage, and pump stations) for at least seven 1% annual chance scenarios of 24-hour rainfall and Brazos River water elevations. The scenarios to be analyzed shall cover a whole range of water elevations in the Brazos River from the 2-year to the 100-year return period, which for this example are listed in Table 7-2. One scenario (Scenario 3) shall be for the Brazos River reaching the critical elevation, which in this example corresponds to a flood frequency of 4.5-years (22.1% annual chance) coincident with a 22.1-year rainfall (4.5% annual chance). The joint probability of having these two events concurrently is 0.221 x 0.045 = 0.01 or 1% annual chance. Table 7-2 shows all other scenarios have 1% annual chance.

Table 7-2

Scenarios to be Analyzed for Pump Capacity Design

Scenario	Brazos River Water Elevation	Local 24-hr Rainfall	
	Return Period (Years)	Return Period (Years)	Joint Probability
1	2.0	50.0	1%
2	2.8	36.1	1%
3	4.5	22.1	1%
4	5.0	20.0	1%
5	10.0	10.0	1%
6	25.0	4.0	1%
7	50.0	2.0	1%

The analysis of the internal drainage shall be completed with a dynamic hydraulic modeling software and shall account for the tailwater impact from the Brazos River to the local drainage, pumping operating rules, and accumulation of storage. The proposed configuration of the pump station and internal drainage system must be evaluated for all scenarios, and iterations of the design should be evaluated until the critical elevation is not exceeded by any scenario.

Each of the seven simulations will have a resulting peak water surface elevation in the internal system. The Engineer shall enter the resulting peak water elevations in the FBCDD spreadsheet and determine the storm that results in the lowest freeboard in the landside. The results from the spreadsheet for this example are listed in Table 7-3.

Table 7-3

Results of Freeboard for Different 1% Annual Chance Scenarios

Scenario	Brazos River Flos	Local 24-hr Rainfall	Condition	Peak WSEL*	Freeboard
	Return Year	Year		(ft)	(in)
1	2.0	50.0	Partial Gravity	64.70	33.6
2	2.8	36.1	Partial Gravity	65.10	28.8
3	4.5	22.1	Pumps Only	65.32	26.2
4	5.0	20.0	Pumps Only	65.20	27.6
5	10.0	10.0	Pumps Only	64.33	38.0
6	25.0	4.0	Pumps Only	63.00	54.0
7	50.0	2.0	Pumps Only	62.00	66.0

It can be seen the minimum free board is obtained with the 22-year rainfall in the landside of the system and a 4.5-year flood elevation in the Brazos River (Scenario 3). The minimum freeboard is 26 inches and therefore, this is the critical scenario adopted for the design. The summary table from the FBCDD Spreadsheet is included as Figure 7-3.

The resulting system includes a pump station with a firm capacity of 250,000 gpm or 557 cfs. With the added redundant pumps, the total capacity is 300,000 gpm or 668 cfs. The storage of the internal drainage system has a capacity of 930 acre-feet at an elevation of 65.32 feet (Scenario 3).

The Engineer shall verify the pumps have capacity to empty the storage under the 24-hour time limit. The elevation-capacity relationship indicates the storage at the maximum internal water surface elevation is 930 acre-feet. 20% of that volume is 186 acre-feet. Therefore, the volume to be pumped under the time limit is 744 acre-feet. Drain time calculations are dependent on pump operations and therefore should be obtained directly from the hydraulic model results; for simplification in this example, it is assumed that the pumps are able to maintain the firm capacity of 557 cfs while the storage is being emptied. Under these assumptions, the pump station will empty 744 acre-feet in 16.2 hours calculated as follows:

744 acre-feet / 557 cfs x (43,560 cf / acre-feet) x (1 hour / 3,600 sec) = 16.2 hours

This last step verified the pumps are able to drain the storage within the time limit, making the system to handle the next severe storm on the next day.

7.7 PUMP STATION DESIGN CRITERIA

All leveed areas within Fort Bend County that are equipped with a pump station shall have a minimum of three pumps and shall be capable of maintaining the design pumping capacity with its largest single pump inoperative. The design pumping capacity of the pump station shall be determined following the requirements from Section 7.6.

7.7.1 **Pump Station Design**

The purpose of this section is to provide information and criteria pertinent to the design and selection of mechanical and electrical systems for pump stations. Designs should be as simple as practicable. Increased complexity tends to increase initial and future operations and maintenance costs. Designs and equipment and material selection that require less frequent maintenance are recommended. The goal of the design should be to ensure minimal maintenance. The goal should also be to ensure any required maintenance is simple, accessible, uses readily available parts and supplies, and reduces operator, mechanic, and electrician burden.

During construction, the designer should be involved in the review of shop drawings and as-built drawings, preparation of the pump station operation and maintenance manual, and field and shop inspections. The designer should also be consulted when a field change is recommended or required.

It is the Owner's responsibility to ensure that the planning, design, construction, and operation of pump stations should take full consideration of the project's environmental requirements. Many of these requirements are dictated by federal or state statutes, by local sponsor's desires or by the U.S. Army Corps of Engineers (USACE) policy.

7.7.1.1 Intake and Sump Design

Intakes and sumps for drainage pump stations shall meet the requirements of the latest versions of the Hydraulic Institute Standard 9.8 – *Rotodynamic Pumps for Pump Intake Design* and the US Army Corps of Engineers manual EM 1110-2-3105 – *Mechanical and Electrical Design of Pump Stations*. Physical model studies shall be performed when directed by HI Standard 9.8.

Rectangular intakes should include divider walls between the pumps and be capable of isolating individual pump bays for maintenance. Isolation of intake bays may be omitted if designer can demonstrate all sump and pump maintenance can be performed without dewatering the pump bay. If screens are provided for individual bays, include openings between the bays to allow cross flow the event a bar screen becomes blocked.

For small stations below 5,000 gallons per minute (gpm), precast or concrete circular manhole stations can be utilized.

Trash racks should be placed near the inlet of the station and away from the pumps. Pump stations that are unmanned or that may not have adequate access during flood events should consider the use of automatic trash rakes.

7.7.1.2 Pump Types

Pump types that may be used for drainage stations include:

- Submersible pumps;
- Vertical line-shaft pumps;
- Horizontal split case pumps;
- Horizontal or vertical end-suction pumps.

7.7.1.3 Pump Tests

All pumps for flood-control pumping stations should have their performance verified by the manufacturer for the required pumped flow at the head conditions specified. If identical pumps, only one of them needs to be performance tested. Tests on similar pumps used for another station will not be acceptable as equivalent tests. Pumps should be tested over a range of heads at least 2 feet greater than the highest total head or at shutoff and extending down to the lowest head permitted by the test setup. The lowest head tested should be at least equal to the total head that occurs 95% of the operation time during low head pumping conditions.

Once flood-control pumping stations are operational, the design should account for regularly scheduled pump tests that occur, at minimum, annually or at the discretion of the Owner if more frequently.

7.7.1.4 Pump Drivers

Pumps may be operated by electric motor, diesel engine, or natural gas engine. If electric motors are used as the primary pump driver, backup-power in the form of an engine drive or generator must be provided. If diesel engines are used as the primary pump driver, the pump station shall have fuel stored on site to power all pumps continuously for three days. If natural gas engines are used as the primary pump driver and historical data indicates that utility supplied gas flow is sometimes interrupted for more than an hour, two days of gas storage shall be provided on site.

All engines must be designed for manual starting. Automatic starting can be incorporated as applicable and necessary. In addition, engines must comply with EPA regulations published in Title 40, Part 63, Subpart ZZZZ (63.6580) of the Code of Federal Regulations (CFR).

All engines shall be rated on continuous duty operation and be rated 10% in excess of the maximum operating requirements that include the maximum pump horsepower and all loses through the drive system.

7.7.1.5 Backflow Prevention

Backflow Preventers: If the pump discharge is below the maximum discharge pool, two means to prevent backflow must be provided, one of which must be backflow gates. These backflow prevention methods do not substitute for the need to have reverse rotation protection. Of the two means of preventing backflow, one means should be for normal use and the other for emergency use in the event of failure of the normal method. Air suppression systems should not be utilized as the normal means of preventing backflow. Discharge lines routed over the level of protection shall be prevented from siphonic backflow by providing air vents and vacuum breaker at the crest of the discharge.

Siphon breaker valves: If utilizing a siphonic recovery discharge, the design must include a siphon breaker valve at the crest of the discharge. A common layout is to have the siphon break be a bifurcated conduit coming from the discharge. One segment with a motorized valve for the primary valve for the siphon breaker piping, and a manually valve normally open. The other segment of the bifurcated line being a manual valve normally closed. The backup manual valves allow for manual manipulation of the siphon breaker function.

7.7.1.6 Discharge Piping

Pumps may discharge individually or into a common header. If a common header is used, each pump discharge shall include a check valve. An isolation valve is recommended unless designer can demonstrate pump and check valve maintenance can be performed in its absence. Isolation valve may still be required for backflow prevention depending on the piping arrangement, see paragraph 7.7.1.5.

Pipes discharging over or through a levee may only account for self-priming siphonic head recovery if the pipes are less than 60” diameter and the flow velocity at the crest of the pipe exceeds 7 ft/s. Where pipes exceed that diameter or unusual discharge arrangement exists a physical model of the discharge piping system shall be performed to confirm the design.

Pipes discharging through a levee or headwall should be avoided whenever possible however it is recognized that conditions may exist that would dictate their use. For those situations the designer should account for prolonged settlement, seepage, backflow prevention and the appropriate support system for the discharge piping.

The designer should also provide corrosion protection in the form of specialized coatings, cathodic protection or material selection for the integrity of the discharge piping.

7.7.1.7 Backup Power

Backup power for electric motor-driven pumps may be provided by either an engine coupled to each pump or by a whole station generator(s). Generators may be fueled by diesel or natural gas. Generators shall be sized to run all pumps concurrently, including starting and running the stand-by or “firm” pump while all other pumps are running. Diesel generators shall have fuel stored on site to power all pumps and additional station loads continuously for three days if the primary power is a diesel generator, and two days if the primary power is grid power.

For flood risk management pump stations, the potential for a lack of gas flow is a much greater concern. If historical data indicates that utility-supplied gas flow is sometimes interrupted for more than an hour, consider not using natural gas.

7.7.1.8 Instrumentation and Controls

Pumps shall be capable of automatically starting and stopping based on suction side water levels. Instrumentation should be included, at a minimum, to indicate canal, suction water levels before and after the trash rack and discharge water levels. Analog indicators, such as staff gauges, shall be installed to indicate suction and discharge water levels and be calibrated to the datum used in the drainage analysis.

Pump stations shall, at a minimum, include telemetry to remotely alert operators to power failure, pump trip, and other upset conditions. Designers should consider the inclusion of instrumentation and controls to remotely monitor the operation of the pump station.

Auxiliary systems to include, but not limited to, generators, siphon breaking valves, exhaust fans, and bar screen cleaning systems may require automatic capabilities at the Owner's discretion.

7.7.1.9 Site and Access

The pump station site shall be above the 500-year flood elevation as defined with gravity flow conditions. Motors, engines, electrical gear, fuel systems, and any other critical support systems that can be damaged by flooding shall be at least 2 feet above the 500-year flood water surface elevation. Access to the pump station site from the nearest state highway shall be above the adjacent watercourse (Brazos River) 100-year water surface elevation. If providing roadway access above the 100-year water surface elevation is impractical the designer should consider providing alterative means for site access during emergencies such as a boat ramp or increase the amount of fuel stored on site.

The pump station site should allow for maintenance access to pull pumps, bar screens, trash rake equipment, and any associated appurtenances. Depending on the proximity to the local populace, the designer should account for sound attenuation, appropriate lighting, security and impact to the surrounding environment.

7.7.1.10 Maintenance

A maintenance plan shall be developed that meets or exceeds the manufacturer's recommendations for periodic inspections and maintenance of the pumps and appurtenances. The Owner or Engineer shall keep records of the plan and detailed descriptions of all maintenance activities completed. The maintenance plan and related records shall be provided to FBCDD upon request.

All pump stations will require vehicular and personnel access during major rain events. If access is difficult for vehicle access during major rain events, then an operator room may be required at the discretion of the Owner, designed to meet applicable building codes.

7.8 REFERENCES

The following is a non-exhaustive list of public literature with brief descriptions that offer guidance for the design, operation, maintenance, and regulation of levees within Fort Bend County.

7.8.1 Levee Owner's Manual for Non-Federal Flood Control Works

The intent of the document is to provide the public sponsor of a flood control system, with clear and comprehensive guidance on the operation and maintenance of levees, floodwalls, and other flood control structures.

This manual also clearly explains the minimum requirements that the Corps has established for participation in the Rehabilitation and Inspection Program (RIP).

7.8.2 Public Law (PL) 84-99

This is the federal law that gives the Corps the legal authority to supplement local efforts in the repair of flood control projects that are damaged by floods.

7.8.3 Title 33, Code of Regulations, Part 203 (33 CFR 203)

This federal regulation establishes the Rehabilitation and Inspection Program (RIP), describes the types of assistance the Corps can provide, and describes the general operation, maintenance, and disaster preparedness that is expected of non-federal flood control works.

7.8.4 Title 33, Code of Regulations, Part 208 (33 CFR 208)

This federal regulation provides more specific details on the operation and maintenance required of Federally constructed FCWs. This regulation provides a guideline for O&M but does not provide the level of detail regarding specific requirements that the Levee Owner's manual does.

7.8.5 Engineer Regulation (ER) 500-1-1

This Corps-wide regulation expands on the previously mentioned laws and regulations, provides basic information on the Corps' implementation of Public Law 84-99.

7.8.6 Engineer Pamphlet (EP) 500-1-1

This Corps-wide pamphlet provides further detail on how the ER will be applied practically. It establishes the Inspection Guide and other applicable forms that are listed in the appendices to this manual.

7.8.7 Engineer Manual (EM) 1110-2-1205

This manual provides information regarding environmental engineering for flood control channels.

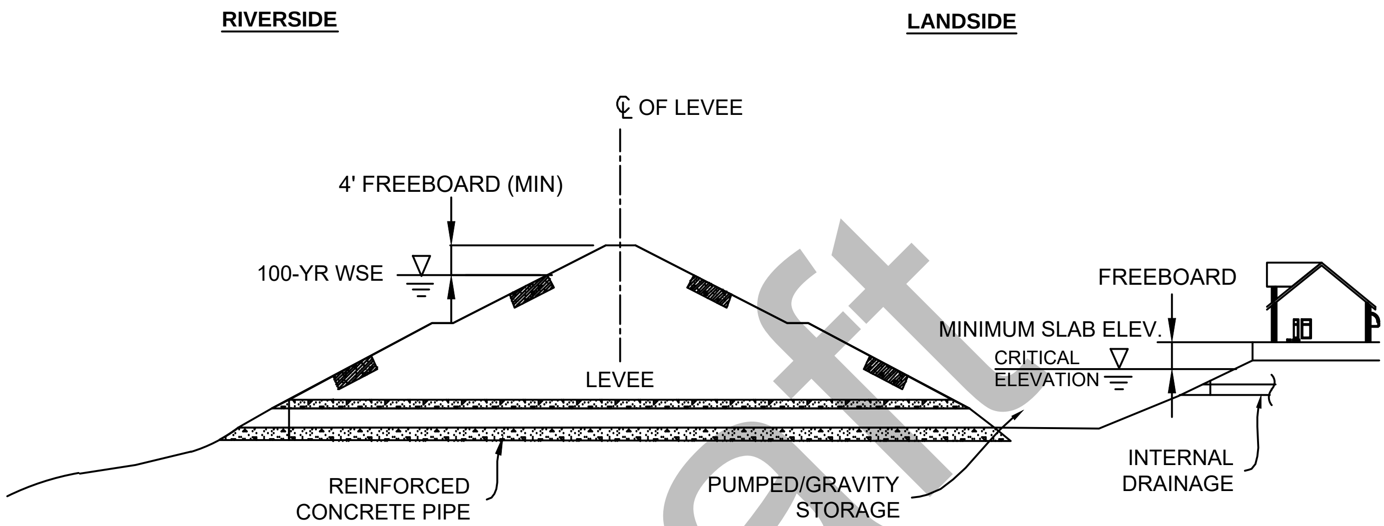
7.8.8 Engineer Circular (EC) 1165-2-218 (DRAFT form published 2/13/2020).

7.8.9 FEMA 484

This FEMA technical manual provides procedures and guidance for best practices associated with conduits through embankment dams.

7.8.10 FEMA P-1032

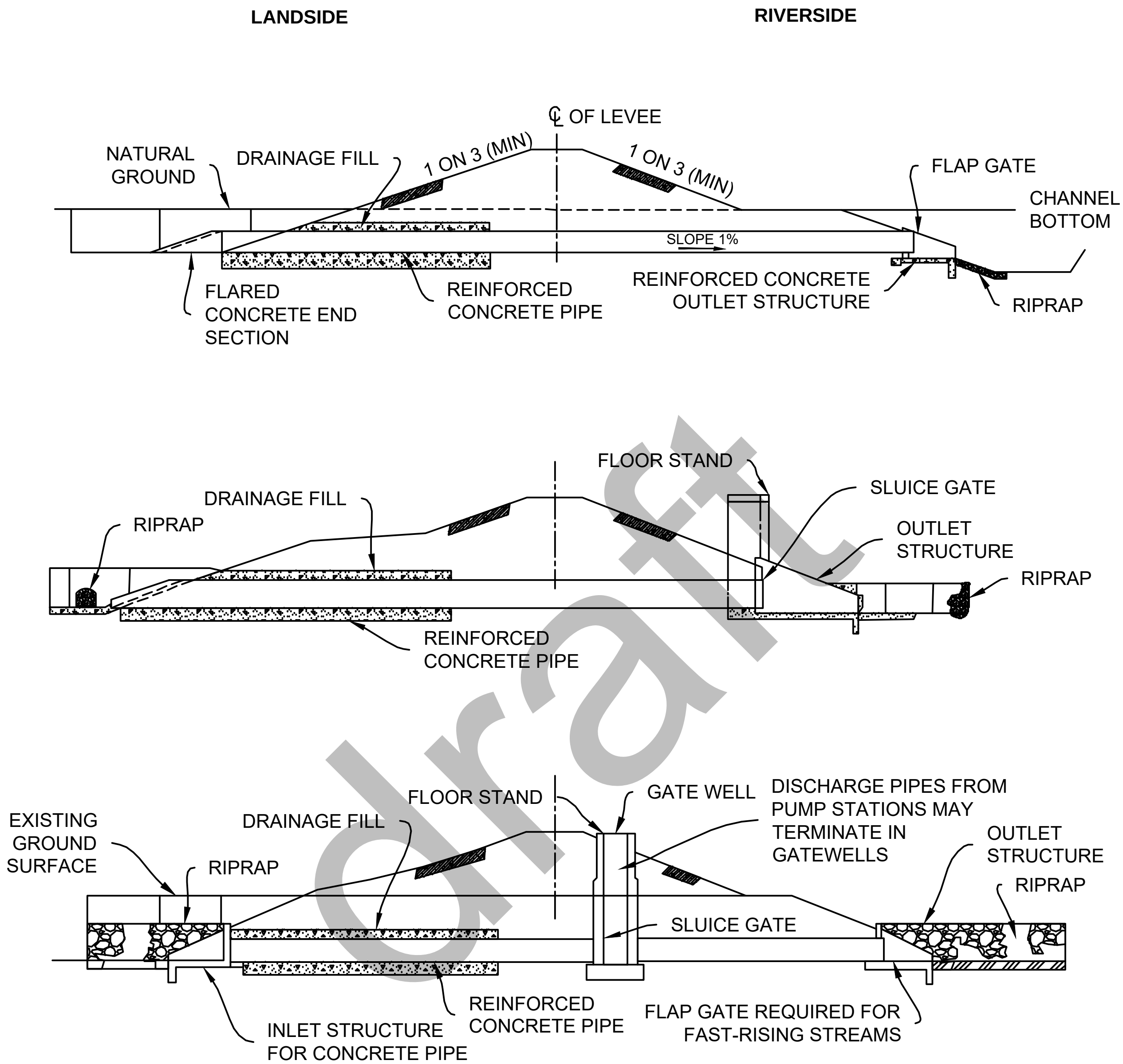
This document presents a summary of current Federal practices for monitoring and measuring seepage, identifying potential failure modes (PFMs) related to internal erosion, assessing risk related to internal erosion, and remediating internal erosion.



LEVEE SCHEMATIC CROSS SECTION

SEPTEMBER 2021

FIGURE 7-1



TYPICAL SECTION OF DRAINAGE STRUCTURES THROUGH LEVEES

SEPTEMBER 2021

FIGURE 7-2

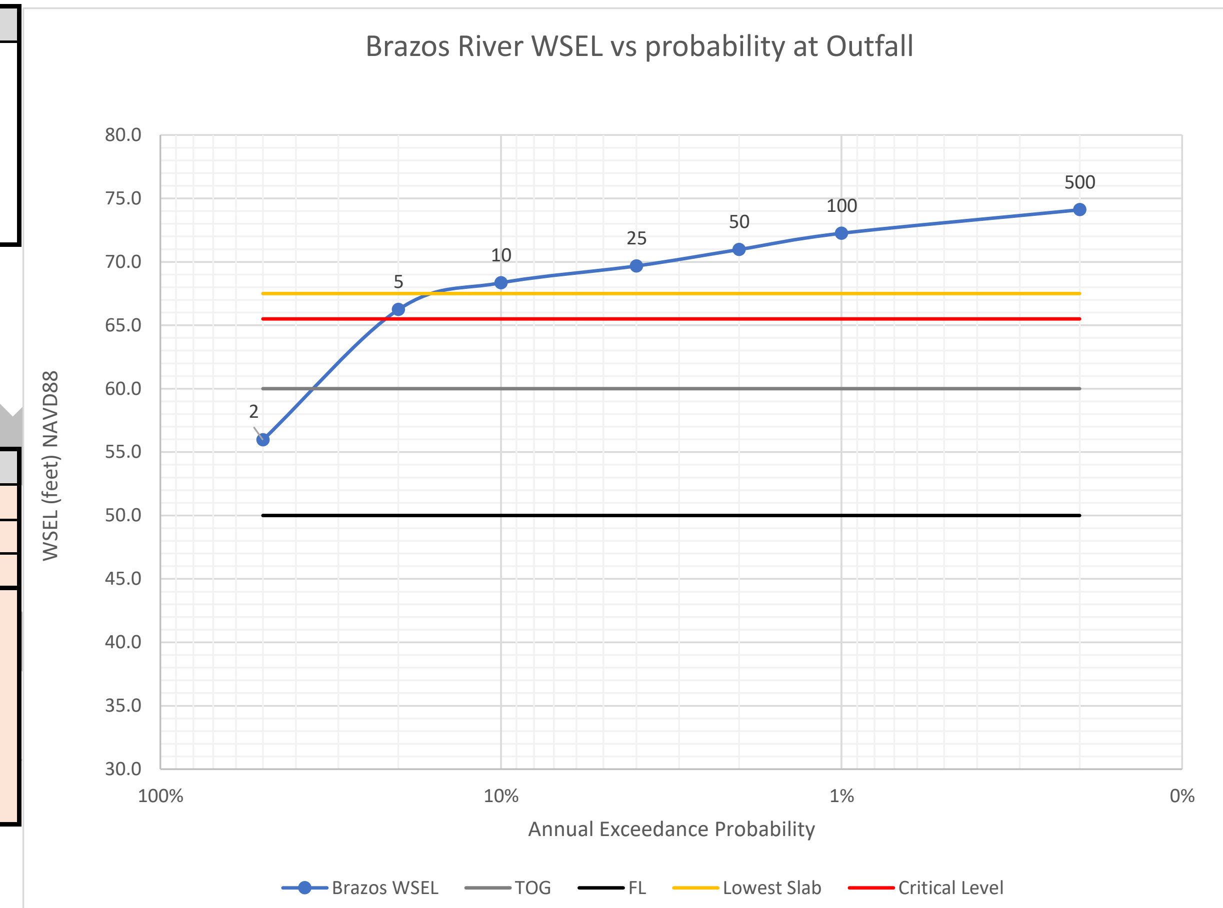
Project: Example for DCM
Description: Example sizing of a pump station serving a leveed area
Assumptions: Same process as outlined in DCM Chapter 7

Date: 9/30/2021
Firm: XYZ
Responsible PE: PE A
QC PE: PE B

From RAS models (NAVD88)		
Return event (yr)	AEP	WSEL (ft)
2	50%	55.96
5	20%	66.24
10	10%	68.34
25	4%	69.68
50	2%	70.97
100	1%	72.25
500	0.20%	74.10

Outlet Information (NAVD88)		
FL	50.00	Feet
H	10.0	Feet
TOG	60.00	Feet
Outlet fully submerged by		
Return event	2.62 yr	
Brazos AEP	38.2%	

Critical Level (NAVD88)			
		Brazos AEP	Brazos Return Event
Lowest Slab	67.5 feet →	14.0%	7.14 yr
Freeboard	24 inches		
Critical Level	65.50 feet	22.1%	4.51 yr



Coincident Events to be evaluated

Scenario	Brazos AEP	Brazos Return Event	Brazos WSEL	Local 24-hr Rainfall AEP	Return Event	5 min	15 min	1 hr	2 hr	3 hr	6 hr	12 hr	24 hr
1	50.0%	2.0	55.96	2.0%	50.0	1.13	2.25	4.30	5.99	7.19	9.33	11.40	13.70
2	36.1%	2.8	60.73	2.8%	36.1	1.08	2.15	4.12	5.67	6.76	8.71	10.63	12.73
3	22.1%	4.5	65.50	4.5%	22.1	0.99	1.97	3.78	5.08	5.97	7.57	9.22	10.97
4	20.0%	5.0	66.24	5.0%	20.0	0.97	1.95	3.73	5.00	5.87	7.43	9.04	10.76
5	10.0%	10.0	68.34	10.0%	10.0	0.84	1.69	3.22	4.19	4.82	5.97	7.20	8.55
6	4.0%	25.0	69.68	25.0%	4.0	0.70	1.42	2.69	3.41	3.86	4.68	5.56	6.53
7	2.0%	50.0	70.97	50.0%	2.0	0.59	1.19	2.26	2.83	3.17	3.77	4.40	5.09

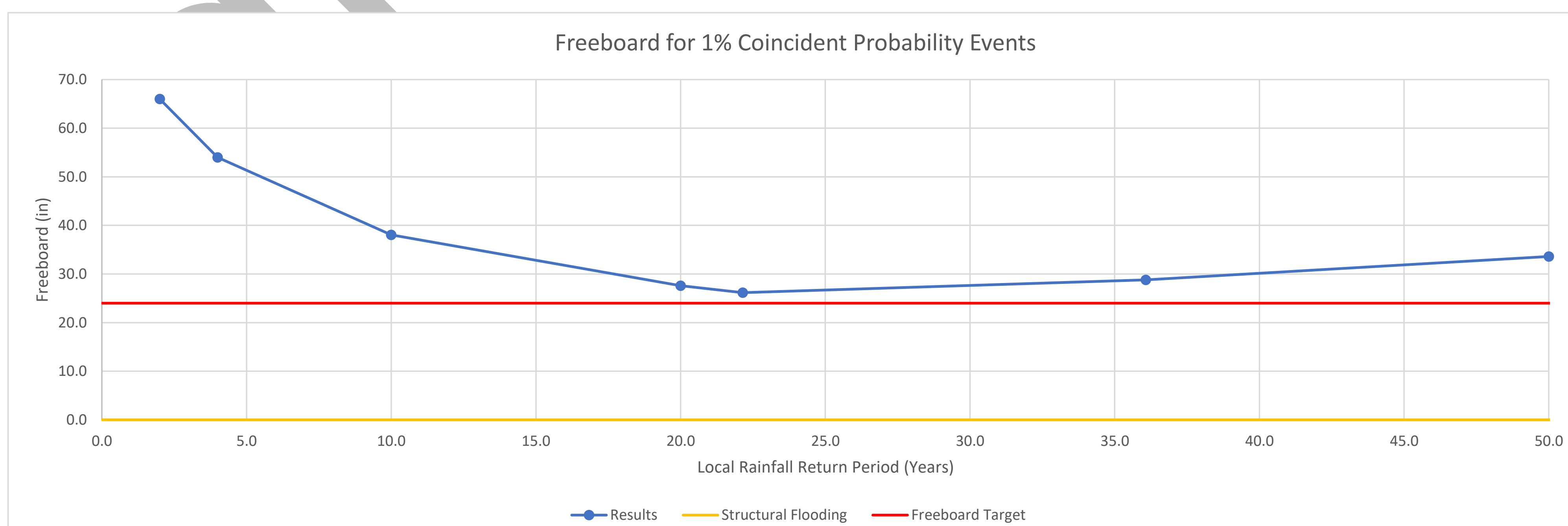
To account for partial gravity flow, include Brazos TW in the model, and appropriate assumptions regarding gates
Evaluate all coincident event scenarios. For any of these simulations, internal ponding level should not exceed critical level

Results of Coincident Probability Events (NAVD88)

Scenario	Brazos year	Local 24-hr Rainfall Year	Condition	Peak WSEL* (ft)	Freeboard (in)
1	2.0	50.0	Partial Gravity	64.70	33.6
2	2.8	36.1	Partial Gravity	65.10	28.8
3	4.5	22.1	Pumps Only	65.32	26.2
4	5.0	20.0	Pumps Only	65.20	27.6
5	10.0	10.0	Pumps Only	64.33	38.0
6	25.0	4.0	Pumps Only	63.00	54.0
7	50.0	2.0	Pumps Only	62.00	66.0

Minimum Freeboard 26.2 in

Firm Capacity		Total Capacity	
(GPM)	(CFS)	(GPM)	(CFS)
250,000	557	300,000	669



Input

All elevations should be in NAVD88 Datum
* Peak WSEL within system; not necessarily at P.S.

EXAMPLE OF PUMP CAPACITY CALCULATION FOR LEVEES

SEPTEMBER 2021 | FIGURE 7-3