APPENDIX A
CRITERIA & METHODOLOGY

1. BACKGROUND

1.1. INTRODUCTION

Over the years a number of methods have been used in Brazoria County and adjacent counties for discharge determination in the design and analysis of flood control facilities. The methods included various forms of the Rational Method, U.S. Soils and Conservation Society synthetic unit hydrograph analysis using existing stream gauging records and computer programs developed by the Corps of Engineers, and U.S. Geological Survey generalized regression equations developed for the area.

In the mid-1960's, the Harris County Flood Control District (HCFCD) and the City of Houston commissioned a detailed hydrologic study of Harris County which resulted in the development of discharge versus drainage area relationships and unit graph methodologies used for the design of flood control and drainage facilities.

In June of 2001, Tropical Storm Allison came ashore on the Upper Texas Coast and produced record rainfall amounts and pervasive flooding in Harris and surrounding counties, including the Clear Creek Watershed. In October of 2001, through a joint effort between FEMA and HCFCD, Harris County began the Tropical Storm Allison Recovery Project (TSARP).

Since that time, a variety of updated models and FEMA maps have been released. Some of these models and maps are still preliminary while others have been officially adopted. It is required that the effective (current) FEMA map shall be used for any project within the DISTRICT. Furthermore, any new or updated analysis within the DISTRICT shall consider the tail water effects based upon potentially higher water surface elevations in the models or effective maps within neighboring counties. If the project involved requires the approval of FEMA, then the requirements of FEMA shall supersede any DISTRICT requirements.

In the case that FEMA approval is not required for the project, design engineers should use the methodology presented in this Appendix to design drainage facilities in the DISTRICT.

2. HYDROLOGIC ANALYSIS OVERVIEW

The selection of an appropriate hydrologic methodology for all projects shall be carried out in accordance with Figure 2-1. The design engineer shall contact the appropriate reviewing agencies prior to preparing his analysis to obtain approval of the selected methodology. This shall include a meeting with the DISTRICT'S Engineer.

HEC-HMS was created at the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC). HEC-HMS has replaced HEC-1 as the standard rainfall-runoff model. For this reason, the design engineer shall get approval from the DISTRICT'S Engineer prior to using HEC-1 models. This applies to existing, revised, and proposed models. Please note that a rainfall runoff analysis using HEC-1 or HEC-HMS should only be used in cases where it is required for FEMA submittals or where a reviewing agency has determined that the design engineer must investigate the downstream impacts of the proposed project. In any case, for projects requiring FEMA approval, design engineers should use the most current effective model of the study stream.

2.1. PEAK DISCHARGE DETERMINATION

2.1.1. APPLICATION OF RUNOFF CALCULATION MODELS

2.1.1.1. ACCEPTABLE METHODOLOGY FOR AREAS LESS THAN TWO HUNDRED (200) ACRES

For areas up to two hundred (200) acres served by storm sewer or roadside ditch, peak discharges will be based on the Rational Method. If the modeling is associated with establishing a flood-prone area for purposes of a FEMA submittal, the models to be used must be acceptable to that agency.

2.1.1.2. ACCEPTABLE METHODOLOGY FOR AREAS GREATER THAN TWO HUNDRED (200) ACRES

Rainfall runoff modeling will be applied to areas greater than two hundred (200) acres in size. Again, if the modeling is associated with establishing a flood-prone area for purposes of a FEMA submittal, the models to be used must be acceptable to that agency.

2.1.2. RAINFALL DURATIONS FOR HYDROLOGIC MODELING

For design using HEC-HMS, the 24-hour design storm isohyet graph will be used for rainfall data for drainage areas larger than two hundred (200) acres.

2.1.3. APPLICATION OF THE RATIONAL METHOD

Use of the Rational Method for calculating the peak pre- development and post development runoff for a storm drainage system involves applying the following formula to runoff:

$$Q = CIA$$

Where Q	=	peak discharge (cfs)
C	=	runoff coefficient
A	=	area (acres)
I	=	rainfall intensity (inches per hour)

2.1.3.1. CALCULATION OF RUNOFF COEFFICIENT

The runoff coefficient "C" values in the Rational Method formula will vary based on the land use. Land use types and "C" values which can be used are as follows:

Land Use Type	Runoff Coefficient*
Paved Areas/Roofs	1.0
Residential Districts	
Lots more than ½ acre	0.40
Lots 1/4 - ½ acre	0.50
Lots 8,000 sf. – 1/4 acre	0.55
Lots 5,000 sf. – 8,000 sf.	0.60
Lots less than 5,000 sf.	0.70
Multi-Family areas	
Less than 20 DU/AC	0.75
20 DU/AC or Greater	0.85
Business Developments	0.95
Industrial Developments	0.95
Railroad Yard Areas	0.30
Parks / Open Areas	0.30

Rice Fields/Pastures	0.20
Lakes / Detention Facilities***	1.0

** Includes concrete, asphalt, gravel, limestone, crushed stone, and lime stabilized surfaces.

*** Includes wet and dry detention facilities. Area will be computed from the top of slope.

Composite "C" values may be allowed in mixed-use drainage areas. These values are obtained by calculating the weighted average of the contributing sub-areas as follows:

$$C = \frac{(C_1A_1 + C_2A_2 + C_3A_3 \ldots C_nA_n)}{(A_1 + A_2 + A_3 \ldots A_n)}$$

The calculations and an exhibit of surface types for use of composite "C" values shall be included with the drainage calculations and provided on the plans.

2.1.3.2. DETERMINATION OF TIME OF CONCENTRATION

The following method shall be used for determining the time of concentration

$$Tc = D / (60*v) + Ti$$

Where Tc = time of concentration (minutes)
Ti = initial time (minutes)
use 10 minutes for developed flows
use 15 minutes for undeveloped flows
D = travel distance on flow path (feet)
V = velocity (ft / sec)

Time of concentration shall be based upon the actual travel time from the most remote point in the drainage area to the point of runoff. The design engineer shall provide a sketch of the travel path with the calculations.

The following minimum and maximum velocities shall be used when calculating the time of concentration.

SURFACE TYPE	UNDEVELOPED FLOWS MIN V (fps)	DEVELOPED FLOWS MIN V (fps)
storm sewer	3.00	3.00
ditch / channel	2.00	2.50
paved area	1.50	1.50
bare ground	0.50	1.00
grass	0.35	0.50
thick vegetation	0.25	0.35

2.1.3.3. RAINFALL INTENSITY

The rainfall intensity shall be computed as follows:

$$I = b / (Tc + d) ^\wedge e$$

Where I = rainfall intensity (in / hr)
Tc = time of concentration (min)
b, d, e = coefficients per the table below

COEFFICIENT	100-YEAR	50-YEAR	25-YEAR	10-YEAR	5-YEAR	3-YEAR
Tc <= 60 min						
b	90.8	107.0	98.5	107.9	92.9	90.6
d	16.5	21.1	24.0	23.6	19.7	19.5
e	0.685	0.734	0.729	0.781	0.788	0.803
Tc > 60 min						
b	84.0	86.5	89.2	96.6	70.1	71.0
d	11.0	10.0	10.4	17.2	7.7	8.4
e	0.679	0.709	0.736	0.770	0.752	0.774

2.2. SMALL WATERSHED METHOD HYDROGRAPH METHODOLOGY

The small watershed method referred to in Figure 2-1 is the one developed by H.R. Malcolm and is described below.

2.2.1. INTRODUCTION

A technique for hydrograph development which is useful in the design of detention facilities serving relatively small watersheds has been presented by H.R. Malcolm. The methodology utilizes a pattern hydrograph which peaks at the design flow rate and which contains a runoff volume consistent with the design rainfall. The pattern hydrograph is a two-part function approximation to the dimensionless hydrograph proposed by the Bureau of Reclamation and the Soil Conservation Service.

This method shall be used for medium projects as defined in section 3.7.2. The minimum rate of detention shall be 0.65 ac-ft / ac when using this method.

2.2.2 EQUATIONS

The Small Watershed Hydrograph Method consists of the following equations:

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

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

$$q_i = 4.34Q_p e^{-1.30t_i / T_p} \qquad \text{for } t_i > 1.25T_p \ (3)$$

* Calculator must be in radian mode.

Where T_p is the time (in seconds) to Q_p, Q_p is the peak design flow rate (in cubic feet per second) for the subject drainage area, V is the total volume of runoff (in cubic feet) for the design storm, and t_i and q_i are the respective time (s) and flow rates (cfs) which determine the shape of the inflow hydrograph. All variables must be in consistent units.

2.2.3.1. APPLICATIONS

The peak flow rate, Qp, is obtained from the Rational Method Formula. For detention mitigation analyses the Rational Method should be applied in accordance with Section 2.1.3 of this Appendix, with the exception that all proposed developed runoff coefficients (C) given in that section should be inflated by 5%. The total volume of runoff (V) is the same as the rainfall excess. Table 2-1 below gives typical values for the rainfall excess based on percent impervious cover. The actual values may be interpolated from the table. See Table 2-3, Section 2.3.3, for determination of percent impervious cover.

Table 2-1. Typical Rainfall Excess Values To Be Used with Small Watershed Method

Impervious Cover	100-Year Rainfall Excess (in.)	10-Year Rainfall Excess (in)	3-Year Rainfall Excess (in)
100 %	13.5	8.3	6.1
90%	13.2	8.0	5.7
80%	13.0	7.8	5.4
70%	12.7	7.5	5.2
60%	12.4	7.3	5.0
50%	12.2	7.0	4.8
45%	12.0	6.9	4.7
40%	11.9	6.8	4.6
35%	11.8	6.7	4.5
30%	11.6	6.6	4.4

The Small Watershed Hydrograph Method should only be used where an impact analysis is not required for the total drainage system including the detention facility and outfall channel (as indicated in Figure 2-1). The Small Watershed Hydrograph Method cannot be used in conjunction with the HEC-HMS computer models of watersheds studied in the Flood Insurance Study. The time to peak of the Small Watershed Hydrograph Method is computed strictly to match volumes and has no relationship to the storm durations and rainfall distributions used in the Flood Insurance Study.

2.3. WATERSHED MODELING

In June of 2001, Tropical Storm Allison came ashore on the Upper Texas Coast and produced record rainfall amounts and pervasive flooding in Harris and surrounding counties, including the Clear Creek Watershed. In October of 2001, through a joint effort between FEMA and HCFCD, Harris County began the Tropical Storm Allison Recovery Project (TSARP). Since that time, a variety of updated models and FEMA maps have been released. Some of these models and maps are still preliminary while others have been officially adopted. It is required that the effective (current) FEMA map shall be used for any project within the DISTRICT. Furthermore, any new or updated analysis within the DISTRICT shall consider the tail water effects based upon potentially higher water surface elevations in the models or effective maps within neighboring counties. If the project involved requires the approval of FEMA, then the requirements of FEMA shall supersedes any DISTRICT requirements.

In the case that FEMA approval is not required for the project, design engineers should use the methodology presented in this Appendix to design drainage facilities in the DISTRICT.

2.3.1. RAINFALL FREQUENCY AND DURATION

The storm event used to establish regulatory flood plain and floodway limits in the Flood Insurance Study is the 100-year, 24-hour event. For planning purposes and establishing flood insurance rate zones the 10-, 50-, and 500-year events also require analysis. For projects requiring FEMA submittals, the rainfall depths in the most current effective model should be used. For all other projects requiring a rainfall runoff analysis, the depths should be based on Table 2-2, which includes the maximum values for each depth, duration and frequency from the TSARP, TP40 and Hydro 35 information.

Point rainfall amounts for various durations and frequencies for use in the DISTRICT are given in Table 2-2.

Table 2-2. Point Rainfall Depth (Inches) Duration-Frequency Values[1]

	Depth (in)				
	100-	25-	10-	5-	3-
Duration	**Year**	**Year**	**Year**	**Year**	**Year**
5 min.	1.20	1.00	0.90	0.80	0.70
30 min.	3.00	2.40	2.10	1.90	1.60
1 hr.	4.30	3.40	2.90	2.50	2.20
2 hr.	5.70	4.40	3.70	3.10	2.60
3 hr.	6.80	5.10	4.20	3.50	2.80
6 hr.	9.10	6.60	5.30	4.40	3.30
12 hr.	11.10	8.00	6.40	5.30	4.00
24 hr.	13.50	9.80	7.80	6.40	4.80

2.3.2. RAINFALL DEPTH-AREA RELATIONSHIP AND TEMPORAL DISTRIBUTION

In the initial stages of the TSARP it was necessary to address issues having to do with the use of the new USACE runoff model called HEC-HMS. HEC-HMS has replaced HEC-1 as the standard software for hydrologic analysis within the DISTRICT. Two important differences in between HEC-HMS and HEC-1 have to do with the use of depth-area indices to account for

[1] Source: TP-40, Hydro-35 and U.S.G.S.

point rainfall depths on large areas and the temporal distribution of rainfall (the rainfall hyetograph).

Furthermore, it is anticipated that HEC-1 will no longer be supported by the U.S. Army Corps of Engineers. For these reasons, the design engineer must get approval from the DISTRICT prior to using HEC-1 models. This applies to existing, revised, and proposed models. For projects requiring FEMA approval, the rainfall input of the most current effective model should be used. For projects not requiring FEMA submittals, the 67% duration peaking temporal rainfall distribution should be used.

2.3.3. LOSS RATES

Rainfall excess and runoff volume are dependent on factors such as rainfall volume, rainfall intensity, antecedent soil moisture, impervious cover, depression storage, interception, infiltration, and evaporation. The extent of impervious cover and depression storage is actually a measure of development and is discussed in the next section. The other factors are dependent on soil type, land use, vegetative cover, topography, time of year, temperature, etc.

For projects requiring FEMA approval, the loss input in the most current effective model should be used. For all other projects requiring a rainfall runoff analysis, the Green-Ampt loss function available in HEC-HMS shall be used. A detailed description of the Green-Ampt loss function can be found in USACE EM 1110-2-1417. The following parameters should be used to compute the Green-Ampt losses:

Initial Loss	=	0.1 inches
Volume Moisture Deficit	=	0.385
Wetting Front Suction	=	12.45 inches
Hydraulic Conductivity	=	0.024 in/hr

Additional development in the watershed is analyzed by increasing the value of the impervious cover parameter in the runoff model. Table 2-3 gives appropriate values of percent impervious based on land use types:

Table 2-3. Percent Impervious Cover For Land Use Types

Land Use	% Impervious
High Density	85%
Dry Detention Ponds	85%
Undeveloped	0%
Developed Green Areas	15%
Residential Small Lot (<1/4 acre or schools)	40%
Residential Large Lot (≥1/4 acre or older neighborhoods with limited roadside ditch capacity)	20%
Residential Rural Lot (≥5 acre ranch or farm)	5%
Isolated Transportation	90%
Water	100%
Light Industrial	60%
Airport	50%

2.4. UNIT HYDROGRAPH METHODOLOGY

The model that the Clear Creek Watershed flood insurance study is based on is the Clark unit hydrograph. In cases where FEMA submittals are required, the design engineer should use the Clark unit hydrograph method. In other cases, where a downstream impact analysis is required, consult the appropriate reviewing agencies on the applicability of the Clark unit hydrograph. In some cases, other unit hydrograph methods may be applicable.

The watershed parameters for the Clark unit hydrograph may be developed using the Harris County methodology. Design engineers should refer to the most current effective model and the most recent version of the HCFCD hydrology manual.

2.5. FLOOD HYDROGRAPH ROUTING

Flood routing is used to simulate the runoff hydrograph movement through a channel or reservoir system. Flood routing techniques vary greatly between hydrologic computer models and caution should be used in selecting a routing method, which adequately represents the channel storage conditions present in areas with extremely flat slopes, such as within the DISTRICT.

The HEC-1 and HEC-HMS programs employ several flood routing methods for characterizing the transfer of the runoff hydrograph through the drainage system of a watershed. The models developed for the Flood Insurance Study for the Clear Creek watershed use the Modified Puls Method of routing. This flood routing method is based on the continuity equation and a relationship between flow and storage or stage. The routing is modeled on an independent-reach basis from upstream to downstream. A detailed discussion of the Modified Puls Method can be found in the user's manual for either HEC-1 or HEC-HMS.

2.5.1. STORAGE –ROUTING COMPUTATIONS USING HEC-2 OR HEC-RAS

All of the Flood Insurance Study data submitted for the Clear Creek Watershed have utilized the HEC-2 or HEC-RAS computer program to generate the storage-discharge relationship required for HEC-1 or HEC-HMS to utilize the Modified Puls flood routing. Listed below is a suggested procedure by which the HEC-2 or HEC-RAS program can best be formatted to provide the most effective input and output data necessary for hydrologic studies.

a) Determine which routing reaches of the subject channel will need to be evaluated. Routing reaches that are defined in the Flood Insurance Study usually represent an area between outfalls of two significant drainage areas.

b) Review all the available data for the routing reaches of the subject stream.

c) Run HEC-1 or HEC-HMS for the 100-year storm event using preliminary channel routing data or alternate methods (i.e. Muskingum or Lag).

d) Multiply the preliminary 100-year peak discharges determined above by 0.20, 0.40, 0.60, 0.80, 1.00, and 1.20 to obtain a series of six discharges for each storage routing reach.

e) The discharges that have been developed are then input to the HEC-2 or HEC-RAS program. The discharges should be held constant throughout the subject routing reach. Outflows through a routing reach should not vary.

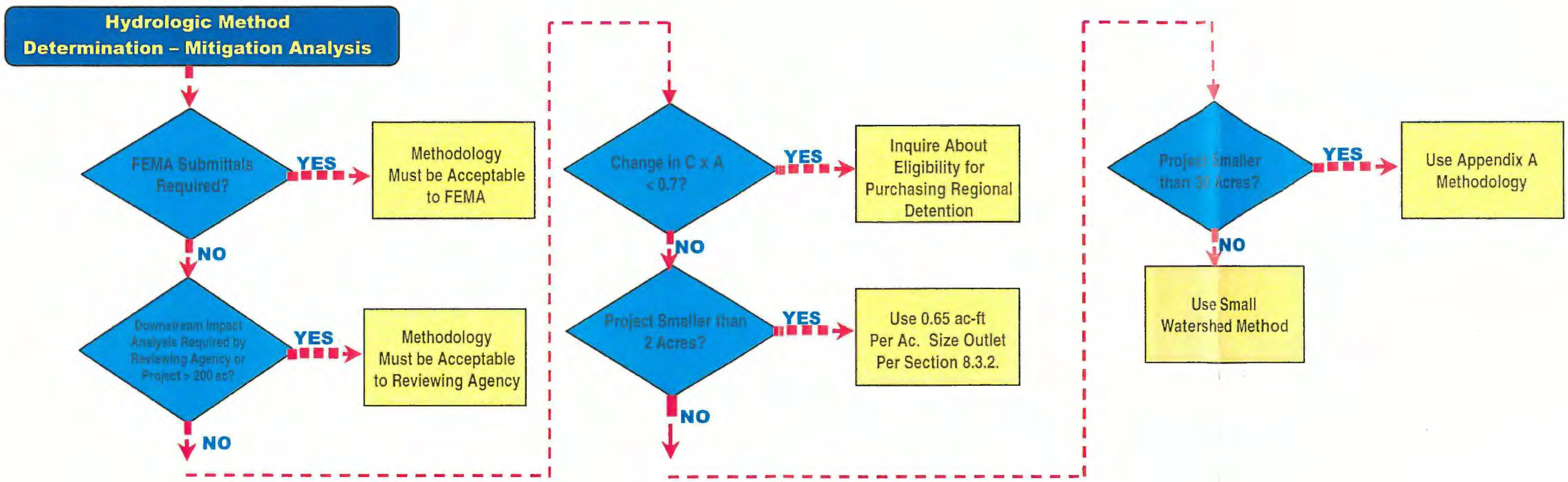
HEC-RAS has replaced HEC-2 as the standard model for this type of analysis. Furthermore, it is expected that HEC-2 will no longer be supported by the U.S. Army Corps of Engineers. For these reasons, the design engineer must get the DISTRICT'S Engineer's approval prior to using HEC-2 models. This applies to existing, revised, and proposed models.

The HEC-2 or HEC-RAS model used in the storage-outflow analysis should be reviewed to ensure that the analysis is correctly determining the total storage volume. Make sure that the ineffective flow areas are modeled appropriately. Also, if using HEC-2 make sure that any ET or X5 cards are removed from the input prior to running the storage-outflow multiple profile analysis.

FIGURE 2.1

Brazoria Drainage District No. 4

Figure 2-1. Hydrologic Method Determination – Mitigation Analysis



3. DETENTION SYSTEM DESIGN

3.1. INTRODUCTION

In situations where on-site storage of storm water runoff is the most effective way to allow development of properties without increasing the flood potential downstream, detention systems will be permitted. This section of the Appendix presents background information on storm water storage techniques and detailed guidelines and criteria for the design of storm water storage facilities.

3.2. GEOTECHNICAL DESIGN

Before initiating final design of detention ponds over six (6) feet deep and two (2) acres in size, a detailed soils investigation by a geotechnical engineer shall be undertaken. Geotechnical investigation, at a minimum, the study should address:

a) The ground water conditions at the proposed site.

b) The type of material to be excavated from the pond site and its suitability for fill material.

c) If a dam is to be constructed, adequate investigation of potential seepage problems through the dam and attendant control requirements, the availability of suitable embankment material and the stability requirements for the dam itself the DISTRICT will not accept or be responsible for any said dams.

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

e) Stability of the pond side slopes.

3.3. PUMPED DETENTION SYTEMS

All storm water detention facilities requiring mechanical pumping systems are generally prohibited, with the exception of pumping of dead storage (maintenance or amenity water stored at or below the discharge pipe control level). However, pumped detention may be allowed under the following conditions and with the following stipulations:

a) A combination pump and gravity systems shall be constructed.

b) The minimum detention rate shall be 0.70 ac-ft/ac.

c) The selected outfall rate shall not increase the elevation or the flow within the receiving system.

d) No more than 75% of the total pond capacity shall be pumped.

e) The discharge delivery system shall not have peak discharge and / or peak stages that exceed the pre-developed values at any point in time for the 3-year, 10-year, and 100-year design storm events.

f) Two pumps minimum shall be required, each capable of providing the design discharge rate. If three pumps are provided, any two pumps in combination must be capable of providing the design discharge rate.

g) Pumping from detention ponds into an existing storm sewer is prohibited unless the pre-developed land already drains into that system and that system has capacity for those undeveloped flows.

h) Pumped detention shall not be allowed for detention basins that collect public storm water runoff, except for detention basins owned, operated and maintained by the DISTRICT or any other governmental entity. Public storm water runoff shall be defined as runoff water that originates from the property of two or more property owners.

i) For pumped detention basins collecting non-public storm water runoff outside of the City of Pearland and within the DISTRICT'S jurisdiction, a cash amount equal to the fair market value of the pumps and installation costs shall be provided to the DISTRICT and placed into an escrow account prior to the approval of the final plat, or prior to the issuance of a building permit if platting is not required. This deposit shall remain in a permanent interest bearing escrow account for the DISTRICT'S use to maintain the pumped system in the event that the Owner fails to maintain the pumped system in accordance with the requirements of the DISTRICT.

j) Fencing of the control panel must be provided to prevent unauthorized operation and vandalism pursuant to the Texas Commission on Environmental Quality Standards. The DISTRICT must be provided access.

k) Adequate assurance must be provided that flooding would not occur for those cases that loss of power occurred during a 100-year flood event.

l) Sensors must be placed so that the pumps would remain off during a rain event.

m) Sensors must be placed so that pumping will not occur when the level of water in the receiving system is at or above ¼ of its full depth.

n) The Operator shall provide the DISTRICT with a quarterly operational report that shall indicate the operational times, total hours of operation, and the amount pumped. Said report shall be delivered to the DISTRICT office on the 15th day of the month after the end of each quarter.

o) The DISTRICT shall have the right to enter the property and inspect the operation of the system at any time for any reason.

p) Failure to maintain the pump station in working order is a violation of these "Rules, Regulations and Guidelines" and is subject to the Penalty provisions of Section 16- "Penalties" and the forfeiture of funds paid in escrow to the DISTRICT for pumped detention facilities.

The use of a pumped detention system must be approved by the DISTRICT prior to the Preliminary Drainage Plan being submitted.

The structural design of detention facilities is very similar in many ways to wide bottom channels. Therefore, the design requirements concerning side slopes and berms are as outlined in Section 4.2.3 for channels. Design considerations addressed specifically in this section deal with the facility bottom and outfall structure.

3.4. STRUCTURAL AND GEOMETRIC PARAMETERS

3.4.1. GENERAL

Two types of detention facilities are acceptable in the DISTRICT. The first is a naturalized facility in which standing shallow pools of water and muddy areas are allowed to exist along the bottom of the facility and support natural or wetland vegetation. This type of facility is only maintained around the sides and perimeter and involves special design considerations at the outfall structure. Designing this type of facility must be approved by the DISTRICT prior to submittal of the Preliminary Drainage Plan and must consider the aesthetics of the surrounding area.

The second type of detention facility is a manicured or well-maintained facility, which is mowed regularly and is designed to stay dry between rainfall events. This type of facility may be more aesthetically pleasing in heavily populated areas and is more amenable to multiple uses such as parks or ball fields. The design considerations for each facility are outlined below.

The following parameters shall apply to all detention facilities.

a) Side slopes shall be 4:1 or greater.

b) Minimum maintenance berms shall be as follows:

POND DEPTH	WIDTH OF BERM
0' - 2'	10'
>2' - 5'	15'
>5' - 10'	20'
> 10'	30'

c) When a detention facility is constructed adjacent to a street right-of-way or DISTRICT channel, a minimum 30 foot maintenance berm is required. This does not include the required channel maintenance berm of the DISTRICT channel or the distance from the street right-of-way line to the curb. The proposed maintenance berm shall not overlap into an adjacent DISTRICT easement, fee strip, or public right-of-way.

d) If the detention facility is to be dedicated to the DISTRICT, a minimum of 25 foot maintenance berm shall be required. Backslope swales and interceptor structures shall also be provided as per the DISTRICT details and requirements.

e) 95% turf germination shall be achieved on the maintenance berms, side slopes, and bottom to prevent erosion.

f) Maintenance berms shall not be encumbered by any other permanent improvements, easements, fee strips, or right-of-way.

3.4.1.2. WET DETENTION POND (STATIC WATER LEVEL)

Wet detention ponds must be approved by the DISTRICT prior to the design and preparation of construction plans. Any detention pond which is designed to hold water for any reason shall be considered to be a wet detention pond. It will be the responsibility of the Developer, MUD or Homeowners Association to own and maintain any wet detention ponds. The DISTRICT will not accept wet detention ponds for maintenance.

3.4.2. BOTTOM DESIGN FOR NATURALIZED DETENTION FACILITIES

The bottom of a detention facility, which is intentionally meant to support natural vegetation, should be designed as flat as practical to still maintain positive drainage to the outfall structure. Side slopes should be designed to allow for regular maintenance and be grass-lined with a 4 to 1 side slope. The bottom should be graded toward the outfall structure at a minimum transverse slope of 0.002 feet per foot. The remainder of the pond bottom shall be graded toward the flowline of the pond at a minimum slope of 0.01 feet per foot. Selected vegetation may be introduced to the bottom of the facility to encourage a particular habitat. Other design requirements for channels should be followed, including backslope drains, and erosion protection measures. A maintenance plan to remove trash debris and excessive siltation must be provided to and approved by the DISTRICT. Additional storage volume may be required by the DISTRICT to offset predicted siltation based on experiences with nearby storage facilities.

3.4.3. BOTTOM DESIGN FOR MANICURED DETENTION FACILITY

The design of the detention facility bottom to remain dry and aesthetically manicured is very important from the standpoint of long term maintenance. A pilot channel is required to facilitate complete drainage of the basin following a runoff event. A lined concrete channel should have a minimum depth of 4 inches and a minimum flowline slope of 0.002 feet per foot (Standard District Details).

Bottom slopes of the detention basins should be graded towards the low-flow pilot channel or outfall. The transverse slope of the bottom should be a minimum slope of 1%.

Detention basins which make use of a channel section for detention storage may not be required to have pilot channels, but should be built in accordance with the requirements for channels, including side slopes, maintenance berms, back slope drains and erosion protection measures previously discussed.

3.4.4. OUTLET STRUCTURE

The outlet structure for a detention facility is subject to higher than normal headwater conditions and erosive velocities for prolonged periods of time. For this reason the erosion protection 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. HDPE or aluminized steel pipes may also be used. 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. The use of seepage cutoff collars is not recommended since such collars are often inadequately installed and prevent satisfactory backfill compaction. A concrete control structure with a grate area equal to ten (10) times the outfall pipe area shall be constructed. Concrete or approved equal paving extending from the outfall area into the pond a distance of ten (10) feet shall be placed on the bottom of the facility for maintenance of the structure. Adequate steel grating around the outfall pipe intake must be designed to prevent clogging of the pipe from dead or displaced vegetation.

The concrete spillway for the 100-year discharge or greater flows shall extend down the bank to the bottom of the channel and up the far side as per the standard DISTRICT details.

3.4.5. EXTREME EVENT SPILLWAYS

The drainage system must be designed to adequately deal with an extreme rainfall event. The extreme event shall be defined as an event which includes or exceeds the 100-year flow. A sheet flow analysis shall be provided to show this extreme event will be conveyed to the detention pond and then to the receiving drainage system.

A concrete lined extreme event overflow swale shall be provided where this event enters and exits the detention pond. The swale shall be constructed as per the DISTRICT details.

Where providing this overflow swale between a detention pond and a DISTRICT maintained ditch or channel, the concrete lining shall extend into the bottom and up the far bank of that ditch or channel at a thirty (30) degree downstream angle in conformance with the standard DISTRICT details.

3.4.6. UNDERGROUND DETENTION SYSTEMS

With prior approval of the DISTRICT, underground detention systems may be utilized. All pipes must be at least forty-eight (48) inches in diameter.

For systems using buried arches or similar structures with rubble backfill, the void space in the rubble backfill shall not count as detention volume.

All underground detention systems must be accessible for inspection and cleaning. No such system may be used in conjunction with a pump system. Concrete overflow structures are required for all underground detention systems.

No underground detention system of any kind will be accepted by the DISTRICT for maintenance.

3.5. ADDITIONAL DESIGN CONSIDERATIONS

Ponds must have one (1) foot of freeboard. The following items describe additional design criteria associated with detention facilities.

3.5.1. EROSION CONTROL

Adequate erosion control and re-vegetation shall be accomplished during and following construction of the facility. The DISTRICT will not allow articulated block on filter fabric as an acceptable means of slope protection.

3.5.2. SAFETY, AESTHETIC CONSIDERATION, AND MULTI-PURPOSE USE

The use of a detention facility generally requires the commitment of a substantial land area. The DISTRICT recognizes that such a facility may be used for other purposes which are compatible with the primary intended purpose of providing flood control. Detention facilities may be utilized as parks and recreational facilities on a case-by-case basis as approved by the BOARD in advance. Also, a parking area may be used for a portion of the storage as long as the 100-year water depth is nine (9) inches or less at the inlets in areas where cars are parked. Street ponding is also allowed as long as the depth of water does not exceed nine (9) inches at the inlets as measured from the gutter. The proposed use and the facilities to be constructed within the facility area must be specifically approved by the DISTRICT. Any facilities constructed for a non-flood control use will not be maintained by the DISTRICT, nor will the DISTRICT be responsible for any damage to these facilities resulting from flooding. A note relieving the DISTRICT of any responsibility shall be provided on the construction plans.

3.5.3. STORM SEWER OUTFALLS

All storm sewer outfall structures should be constructed in accordance with the Standard District Details. Design criteria for outfall structures are as follows:

a) All storm sewer outfall pipes within the DISTRICT right-of-way must be reinforced concrete pipe, aluminized steel pipe, or HDPE with a minimum diameter of eighteen (18) inches.

b) All backslope drains shall be twenty-four (24) inch reinforced concrete pipe, aluminized steel pipe, or HDPE as shown in the Standard DISTRICT Details.

c) A standard manhole or junction box must be outside of the DISTRICT ultimate right-of-way. Where a road or railroad right-of-way is located adjacent to the channel, the manhole may be placed just inside the DISTRICT right-of-way.

d) The minimum radius of curvature for unlined and lined channel bends shall be as per the latest details. Bend losses and super elevation must be included in the hydraulic analysis of severe curves.

e) Erosion protection is required for all outfall pipes as per the Standard DISTRICT Details.

f) Wastewater Treatment plant outfalls shall have a paved invert and concrete slope paving in accordance with the Standard DISTRICT Details.

3.5.4. SPECIAL EROSION AND VELOCITY CONTROL STRUCTURES

3.5.4.1. GENERAL

Special erosion and velocity control structures will generally include stilling basins, baffled aprons, straight drop spillways, sloped drops and impact basins. Due to the hydraulic and earth loads encountered through these structures, the structural as well as the hydraulic design is very critical.

A geotechnical engineering investigation to determine the characteristics of the supporting soil is required for major hydraulic structures. For example, a two (2) foot sloped drop would not require a soils investigation, whereas a five (5) foot straight drop structure would.

3.5.4.2. STRAIGHT DROP SPILLWAY

Straight drop spillways are usually constructed of steel sheet piling with concrete aprons. Steel sheet pile drop structures can sometimes be considered a temporary structure. The Standard DISTRICT Details define the design features of a straight drop spillway.

The distance erosion protection aprons extend upstream and downstream of the drop is determined using hydraulic analysis. The DISTRICT recommends using concrete paving upstream and immediately downstream of the drop. This pavement must extend upstream and downstream to a point where the velocity is reduced to the levels contained in Table 4-2 and must also contain the hydraulic jump. Because of the additional impact load on the downstream slope paving, a six (6) inch thick pad is recommended. Concrete energy dissipaters should be used at the ends of the concrete paving to decrease flow velocities and protect the concrete toe.

The drop structure should be designed for active and passive soil forces. Design calculations are required for each drop structure along with a copy of a geotechnical report defining soil characteristics of the site.

3.5.4.3. BAFFLE CHUTES

See Reference 15 for hydraulic and structural criteria regarding baffle block chutes.

3.5.4.4. SLOPED DROP STRUCTURES

Sloped drop structures can be made of monolithic poured-in-place reinforced concrete slope paving. The same design principles hold true for sloped drop structures as do for straight drop structures (i.e., the draw down curve and hydraulic jump must be contained within the structures).

The sloped drop structure shall have minimum forty eight (48) inch toe walls on the upstream and downstream ends. The sides of the structure shall have minimum eighteen (18) inch toe walls.

3.6. DETENTION – HYDROLOGIC DESIGN

The hydrologic methods for detention design should be in accordance with Section 2.0 of this Appendix. The hydrologic design criteria for the DISTRICT is divided into three design categories based on the size of the contributing drainage area.

Small Projects	For drainage areas between zero (0) acre and two (2) acres
Medium Projects	For drainage areas between two (2) acres and two hundred (200) acres.
Large Projects	For drainage areas greater than two hundred (200) acres.

When detention is proposed in any amount that is less than the amount required for ultimate build out, a note must be provided on the drainage plan stating that "future development on this site must be approved by Brazoria Drainage DISTRICT No. 4". Detention for that additional (future) development would be as per the current (potentially more stringent) requirements at the time that ultimate development is proposed.

3.7. ON-SITE FACILITIES

3.7.1. SMALL PROJECTS

Small Projects are defined as those projects that are two (2) acres or smaller. If a project causes change in runoff coefficient (existing vs. developed) times the area of the development equal to or less than 0.7, the project may be eligible for purchasing regional detention. Mitigation of such facilities will be incorporated within the DISTRICT regional detention facilities, provided capacity is available and the development is within the detention facility service area. If regional capacity is not possible, on-site detention will be required based 0.65 ac-ft. per acre and the outlet will be sized based on the procedure presented in Section 3.7.2 (below). In this case the volume that is calculated using 0.65 ac-ft. per acre will be considered to be the 100-year volume. The 10-year and 3-year volumes will be considered to be 57% and 39% of the 100-year volume, respectively. The generation of runoff hydrographs and the routing of flood flows are not required for Small Projects.

3.7.2. MEDIUM PROJECTS

Medium Projects in the DISTRICT shall include those more than two (2) acres to two hundred (200) acres in size. Medium projects will have their mitigation detention volumes calculated using the methodology presented in Section 2.2.1. All calculations shall be presented to the DISTRICT Engineer, including maps of suitable scale showing the flow paths used to calculate the existing and developed time of concentration. Hydrograph routing through the detention basin is not required. The outflow will be sized as follows:

a) Determine the storage elevation in the basin for the 3-year, 10-year, and 100-year storms.

b) Determine water surface elevation in the receiving system (if reasonably able to) for the 3-year, 10-year, and 100-year storms.

c) Use the orifice equation to compute the opening size(s) as follows:

$$Q = CA\sqrt{2gH},$$

Where:

	Q	=	Basin Outflow (cfs)
	C	=	Pipe Coefficient
	A	=	Restrictor cross-sectional area
	g	=	Acceleration due to gravity (32.2 ft/s^2)
	H	=	The elevation difference between the water surface in the detention pond and the receiving system for a given storm. The design engineer shall assume two (2) feet unless the engineer can

substantiate a known elevation in the receiving system.

Round up to the next half-foot diameter for restrictor pipes above eighteen (18) inch diameter. Some additional blockage of the pipe may be necessary to obtain the correct restrictor area. No restrictor pipes shall be less than six (6) inches in diameter. Restrictors shall always be placed at the upstream end of a pipe to enable cleaning.

For ponds discharging into creeks or ditches, the outfall structure shall be designed to ensure that the allowable flow is not exceeded during a 3-year, 10-year and 100-year event. This may be achieved using a combination of pipes and or weirs. The flowline of each pipe or weir level shall be set based upon the 3-year, 10-year, and 100-year water surface elevations in the detention pond.

Storm events in excess of the 100-year event must be considered in the design of detention facilities from the standpoint of overtopping. For a detention facility that is an excavated pond and has no dam associated with it, the outflow structure must be designed with an overflow structure or swale as per section 3.4.5. This will allow the passage of extreme events with no adverse impacts to adjacent structures. For detention facilities with a dam, the possibility of dam failure must be considered as part of the design. Specific dam criteria for storm events in excess of the 100-year design storm shall be established by the DISTRICT Engineer on a case-by-case basis.

The detention requirements shall never be less than 0.65 acre feet per acre.

3.7.3. LARGE PROJECTS

Large projects shall include those greater than two hundred (200) acres in size.

For projects in excess of two hundred (200) acres, HES-RAS HEC-HMS modeling shall be performed. The HEC-HMS modeling shall include analysis of existing and developed runoff. This analysis must demonstrate that no increase in runoff for the 3-year, 10-year, and 100-year event storms. Similarly a HEC-RAS model shall that no increase in the water surface elevation of the receiving system will occur during the 3-year, 10-year, and 100-year events. See section 2.0 for the specific requirements.

The minimum rate of detention for all large projects shall be 0.65 ac-ft/ac.

3.8. OFF-SITE FACILITIES

Off-site detention facilities will generally be regional in nature. The facility may be sized for one development, but will be designed to serve the entire watershed by reducing the flood potential of a stream. Most of these facilities are envisioned to be adjacent to a channel to receive flood water from the main drainage artery through a system of multistage inlet pipes and high level weirs.

For the design of an off-site detention basin, the hydraulics of the stream and flood damage relationship of the watershed must be evaluated. This will be performed under the direction and advice of the DISTRICT Engineer. This evaluation will result in flood frequency/stage-damage estimates of the stream.

Upstream discharge of unmitigated runoff into a stream resulting in increases in flow or water surface elevation for the 3-year, 10-year, and 100-year events are not permitted. For this reason, careful planning must be used to insure that all property within a detention pond service area is able to discharge directly to the pond without causing these disallowed increases in flow or water surface elevation.

An off-site facility shall be considered a small, medium or large project based upon its service area.

Therefore, the appropriate part of Section 3.7 shall be used to determine the design parameters of the proposed off-site facility.

The construction plans for any proposed off-site facility shall include a detention service area map and a detention service table. The table shall specify the existing, proposed and future detention amount allocated to each sub area within the pond service area.

Subsequent plan submissions by others utilizing detention in these regional facilities shall include a current and updated version of the detention service table.

Any project proposing to utilize off site or regional detention must be accompanied by an analysis prepared by an Engineer which demonstrates how the 100-year developed flows will be conveyed to the detention area during an extreme event. This analysis must demonstrate that this can be achieved without adversely affecting property between the proposed development and the detention pond.

No property outside the approved service area for off-site facility will be allowed until a revised construction plan, service map, and service table is approved by the DISTRICT.

4. HYDRAULIC DESIGN CRITERIA

4.1. GENERAL

4.1.1. INTRODUCTION

The hydraulic design of a channel or structure is of primary importance to insure that flooding and erosion problems are not aggravated or created. This section summarizes methodologies, procedures, and criteria which should be used in the hydraulic analysis of the most common design problems in Brazoria County, Texas. In some instances, methodologies and parameters not discussed in this Appendix may be required. When an approach not presented herein is required, it should be first reviewed with the DISTRICT'S Engineer.

4.1.2. DESIGN FREQUENCIES

All of the DISTRICT'S open channels will be designed to contain the runoff from the 100-year frequency storm within the easement or fee strip, except where channel improvements are necessary to offset increased flows from a proposed development. In those cases, the 100-year flood profile under existing conditions of development should not be increased.

In areas served by closed systems, storm water runoff should be removed during the 100-year frequency storm without flooding of structures. This is accomplished through the design of the street system, the storm sewer system, and other drainage/detention systems.

4.1.3. REQUIRED ANALYSIS

In designing a facility for flood control purposes, a hydraulic analysis must be conducted which includes all the factors significantly affecting the water-surface profile or the hydraulic grade line of the proposed facility. For open channels, the primary factors are losses due to friction, constrictions, bridges, culverts, confluences, transitions and bends. The design of channels or conduits should generally minimize the energy losses caused by these factors which impede or disrupt the flow. Factors affecting the hydraulic grade line in closed conduits are entrance losses, friction losses, exit losses and bend losses.

4.1.4. ACCEPTABLE METHODOLOGIES

Several methods exist which can be used to compute water-surface profiles in open channels. The methodology selected depends on the complexity of the hydraulic design and the level of accuracy desired. Peak discharges and discharge hydrographs developed using one of the methodologies described in Section 2.0 of this Appendix must be incorporated into the existing effective HEC-RAS model in order to determine the impact of any proposed development flood control facility on the entire channel system.

For the design of proposed channel with flow confined to uniform cross-sections, either a hand calculated normal depth or direct step computation is sufficient. Manning's Equation should be used for computing normal depth. For designing a non-uniform proposed channel with flow in the overbanks, the use of HEC-RAS is recommended. Any proposed channel improvements to an existing ditch or creek within the jurisdiction of the DISTRICT must be modeled using HEC-RAS and incorporated into the model used in the Flood Protection Plan (Reference 29). The DISTRICT'S Engineer shall approve the use of HEC-2 models. This applies to existing, revised and proposed models.

Bridges, culverts and expansion and contraction losses are taken into account in the HEC-RAS computer program. If these losses are significant and the normal depth or direct step method is employed, the losses must be included in the backwater calculations. Design criteria for bridges, culverts, transitions, bends and drop structures are presented in the remainder of this section.

4.2. OPEN CHANNEL DESIGN

4.2.1. LOCATION AND ALIGNMENT

The first step in designing or improving an open channel drainage system is to specify its location and alignment. Good engineering judgment must be incorporated to insure the proposed channel location provides maximum service to an area while minimizing construction and maintenance costs. General factors and the DISTRICT criteria which should be taken into account in locating improved channels are as follows:

a) Follow existing channels, ditches, swales or other low areas in undeveloped watersheds. This will minimize the cost of the channel itself and the underground storm sewer system, and will allow for overland flow to follow its natural drainage pattern.

b) For safety reasons, channels and roads must not be located adjacent to one another. Should such a conflict appear unavoidable, the design must be approved by the DISTRICT Engineer.

c) The angle at which two channels intersect must be ninety (90) degrees or less (angle measured between channel centerlines on upstream side of point of intersection).

d) The minimum radius of curvature for unlined and lined channel bends shall be as per the latest DISTRICT'S details. Bend losses and super elevation must be included in the hydraulic analysis of sever curves.

4.2.2. EXISTING CROSS SECTIONS

For determining existing flood profiles, both the channel section and overbank areas must be used in the hydraulic calculations. Channel sections must be based on a recent field survey.

In some cases, the DISTRICT may have recent survey information, which can be utilized. Plans of previous channel improvements should only be used for very preliminary analysis. Overbank areas are best defined by field surveys, but this is not always practical or economically justified. Elevations in the floodplain beyond the limits of the channel can be obtained from the best topographic information available for the study reach.

When designing a channel improvement, the channel sections used should extend beyond the DISTRICTS easement or fee strip a reasonable distance. The distance shall be determined and agreed upon by the DISTRICT Engineer and the developer on a case by case analysis. The purpose of including elevations beyond the easement and fee strip is to avoid a design which creates ponding adjacent to the easement or fee strip. A reasonable distance depends on the adjacent terrain, but in no case shall it be less than twenty (20) feet.

4.2.3. TYPICAL DESIGN SECTIONS

Typical channel sections have been established which should be used in designing improved channels. Minimum dimensions are based on experience of constructing and maintaining channels.

Four typical cross sections are given in the Standard DISTRICT Details. For some applications, other cross section configurations may be necessary. A proposed cross section different from the typical sections presented should be reviewed with the DISTRICT for approval before proceeding with design or analysis.

4.2.3.1. EARTHEN CHANNELS

The most common flood control channel is a totally earthen channel. This is generally the most economical design except in the already developed areas where land costs are extremely high. The initial construction cost for a concrete lined channel is generally three to four times that of an earthen channel.

In the design of an earthen channel, consideration of long-term maintenance has a very strong influence on design parameters. The following are minimum requirements to be used in the design of all earthen channels:

a) Maximum earthen side slopes should be four (4) horizontal to one (1) vertical. Slopes flatter than four (4) to one (1) may be necessary in some areas due to local soil conditions. For channels and detention reservoirs six (6) feet deep or greater, side slopes selection shall be supported by geotechnical investigations and calculations.

b) Minimum bottom width is ten (10) feet unless approved by the DISTRICT Engineer.

c) A minimum maintenance berm is required on either side of the channel of between ten (10) to thirty (30) feet depending on channel size as follows:

TABLE 4-1A
KEY TO EASEMENT AND FEE STRIP REQUIREMENTS

		CHANNEL BOTTOM WIDTH							
		6	8	10	12	15	20	30	40
CHANNEL DEPTH	4	A	A	B	B	B	B	C	C
	6	A	A	B	B	B	B	C	C
	8	B	B	B	C	C	C	C	C
	10	D	D	D	D	D	D	D	D
	12	D	D	D	D	D	D	D	D
	14	D	D	D	D	D	D	D	D
	16	D	D	D	D	D	D	D	D
	18	D	D	D	D	D	D	D	D

TABLE 4-1B
ULTIMATE MAINTENANCE REQUIREMENTS FOR CHANNELS

KEY VALUE	TOTAL	EACH SIDE		
A	30	15		
B	40	20		
C	50	25		
D	60	30		

Larger maintenance berms may be required due to the future needs of an ultimate channel. Easement and fee strip requirements for all main channels are included in the Flood Prevention Plan. (References 28 and 29).

d) Backslope drains or interceptor structures are necessary at a maximum of one thousand (1,000) feet intervals to prevent sheet flow over the ditch slopes. Refer to the Standard DISTRICT Details.

e) Channel slopes must be re-vegetated immediately after construction to minimize bank erosion.

f) Flow from roadside ditches must be conveyed to the channel through a roadside ditch interceptor and pipe. Refer to the Standard District Details.

g) Maintenance berm shall not be encumbered by any permanent improvements, easements or right-of-way.

4.2.3.2. CONCRETE-LINED TRAPEZOIDAL CHANNELS

In instances where flow velocities are excessive, channel confluences create a significant erosion potential or easements and fee strip is limited, fully or partially concrete lined channels may be necessary. The degree of structural analysis required varies significantly depending on the intended purpose and the steepness of the slope on which paving is being placed. Slope paving steeper than 3:1 shall be designed based on a geotechnical analysis that addresses slope stability and groundwater pressure behind the paving.

Following are minimum requirements for partially or fully concrete lined trapezoidal channels (Standard DISTRICT Details):

a) All slope paving shall include a minimum twenty-four (24) inch toe wall at the top and sides and a minimum forty eight (48) inch toe wall across or along the channel bottom for clay soils.

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

c) Concrete slope protection placed on 3:1 slopes should have a minimum thickness of four (4) inches and be reinforced with #3 bars on eighteen (18) inch centers both ways.

d) Concrete slope protection placed on 2:1 slopes should have a minimum thickness of five (5) inches and be reinforced with #3 bars on fifteen (15) inch centers both ways.

e) Concrete slope protection placed on 1.5:1 slopes should have a minimum thickness of six (6) inches and be reinforced with #4 bars on eighteen (18) inch centers both ways. Poured in place concrete side slopes should not be steeper than 1.5:1.

f) In instances where the channel is fully lined, no backslope drainage structures are required. Partially lined channels will require backslope drainage structures as outlined in Item 4 of Section 4.2.3.1.

g) Weep holes should be used to relieve hydrostatic head behind lined channel sections. Refer to the Standard DISTRICT Details.

h) 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. Refer to the Standard DISTRICT Details.

i) For bottom widths of twenty (20) feet and greater, transverse grade beams shall be installed at twenty (20) feet spacing on center. Grade beams shall be one (1) foot wide, one (1) foot-six inches deep, and run the width of the channel bottom. Refer to the Standard DISTRICT Details.

4.2.3.3. RECTANGULAR CONCRETE PILOT CHANNELS (LOW FLOW SECTIONS)

In limited easement and fee strip, it is sometimes necessary to have a vertical walled rectangular section. A standard section was developed some years ago in Harris County, which consists of four (4) foot vertical walls and variable bottom widths. Above the vertical walled section, a trapezoidal section is used varying from earthen to concrete lined depending on the design requirements. Most contractors in the area have had significant experience in the construction of this section.

Presented below are minimum requirements for rectangular concrete pilot channels:

a) Typical structural requirements are shown in the Standard DISTRICT Details. The structural steel design is based on Grade 60 steel. This should be confirmed by a design check based on local soil conditions.

b) Minimum bottom width should be eight (8) feet.

c) For bottom widths twelve (12) feet or greater, a center depression is required. Refer to the Standard DISTRICT Details.

d) For bottom widths twenty (20) feet or greater, transverse grade beams shall be installed at twenty (20) feet spacing on center. Grade beams shall be one (1) foot wide, one (1) foot six (6) inches deep, and run the width of the channel bottom. Refer to the Standard DISTRICT Details.

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

f) Escape stairways shall be constructed in accordance with the Standard DISTRICT Details. Escape stairway shall be located at the upstream side of all street crossings, but not to exceed fourteen hundred (1400) feet intervals.

g) For rectangular concrete pilot channels with earthen side slopes, the top of the vertical wall should be constructed in accordance with the Standard DISTRICT Details to allow for future placement of concrete slope paving.

h) Weep holes should be used to relieve hydrostatic pressure as shown in the Standard DISTRICT Details.

i) 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. Refer to the Standard DISTRICT Details.

j) Concrete pilot channels may be used in combination with slope paving or a maintenance shelf as shown in the Standard DISTRICT Details. Horizontal paving sections should be analyzed as one way paving capable of supporting maintenance equipment.

k) A geotechnical investigation and report shall be performed. Soil boring shall be obtained at a minimum of every one thousand (1000) feet to a depth of 1.5 times the proposed channel depth.

4.3. WATER-SURFACE PROFILES

4.3.1. GENERAL

For steady, gradually varied flow conditions in natural or improved open channels, the computational procedure known as the standard step method is recommended for computing water-surface profiles. The one-dimensional energy equation is solved by using an iterative procedure to calculate a water-surface elevation at a cross section (References 5 and 10).

Manning's Equation is used to compute energy losses due to friction (Section 4.2.4.2), while losses due to obstructions and transitions are calculated using the appropriate equations discussed in this chapter. For cases where the flow is strictly uniform, as determined by the DISTRICT, the standard step method can be reduced to a direct step method or to a uniform flow computation.

The recommended computer program available for computing water-surface profiles when using the standard step method is the Corps of Engineers' program entitled HEC-RAS, Water Surface Profiles. As indicated previously the DISTRICT prefers this program primarily because it is widely accepted and the program readily facilitates the design of channel improvements.

Good judgment must be exercised when determining cross-section locations for water-surface profile calculations. Cross sections should divide the channel into reaches which approximate uniform flow conditions. For example, closely spaced cross sections are required at an abrupt transition such as a bridge, while relatively uniform channel reaches with no significant changes in conveyance require fewer cross sections. As a general guideline, the spacing should not exceed about one thousand (1000) feet.

4.3.2 MANNING'S EQUATION

Manning's Equation is an empirical formula used to evaluate the effects of friction and resistance in open channels. For uniform flow conditions where the channel bottom and energy line are essentially parallel, Manning's Equation should be used to compute the normal depth. For gradually varied flow conditions, the slope of the energy line at a given channel section should be computed using Manning's Equation.

The equation is:

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

where Q	=	total flow in cubic feet per second
n	=	coefficient of roughness
A	=	cross-sectional area of channel in square feet
R	=	hydraulic radius of channel in feet
and S	=	slope of energy line in feet per foot (same as channel bottom slope for uniform flow)

Channel and overbank sections may have to be subdivided to represent differences in roughness across the section. Subdividing may also be helpful in computing Manning's Equation for natural, compound or non-prismatic sections (References 5 and 10).

4.3.3. MANNING'S "n" VALUES

Manning's "n" Values for design purposes should conform to Table 4-2. A "n" value of 0.045 for unlined channels represents a moderate vegetal growth. For unlined channels, with a design flow larger than ten thousand 10,000 cubic feet per second, "n" value of 0.040 may be used. For existing, unimproved channels and overbank areas, "n" values should be determined in accordance with References 10, 11, 12 and 13.

TABLE 4-2
MANNING'S "n" VALUES AND ALLOWABLE 25-YEAR VELOCITIES FOR CHANNEL DESIGN

Channel Description	Roughness Coefficient or Manning's "n" Value	Average Velocity (Feet per Second)	Maximum Velocity (Feet per Second)
Unmaintained Earthen	0.05	3.0	5.0
Grass Lined			
Predominately Clay	0.045	3.0	5.0
Predominately Sand	0.045	2.0	4.0
Concrete Lined	0.015	6.0	10.0
Articulated Block	0.045	5.0	8.0
Overbanks and Existing Unimproved Channels	See References 3, 4 and 5	Not Applicable	Not Applicable

4.3.4 VELOCITIES

Average and maximum allowable velocities based on twenty-five (25) year flows are given in Table 4-2. In the portion of Brazoria County where sandy soils are known to exist, soils information may be needed to determine the predominant type of soil and the corresponding allowable velocities for unlined channels. Maximum velocities also apply to bridges, culverts, transitions, etc. Where velocities exceed the maximum allowed, erosion protection must be provided.

4.3.5. FLOWLINE SLOPE

Maximum slopes are generally controlled by the maximum allowable velocity. Channel slopes shall not be less than 0.1%.

4.3.6. STARTING WATER-SURFACE ELEVATIONS

For design of open channels, starting water-surface elevations at the channel mouth will generally be based on the normal depth (slope-area method in HEC-RAS) in the design channel.

In determining actual flood profiles or flood plain delineation, the water-surface elevation from the outfall channel should be projected horizontally upstream until it intersects the flood profile on the design channel. An assumption that the peaks occur at the same time will generally produce a conservative flood profile. Otherwise, an analysis of coincident flow may be conducted to determine the flow in the outfall channel at the time the peak flow occurs on the design channel.

4.3.7. HEAD LOSSES

Manning's Equation is used to estimate energy or head losses due to channel friction and resistance. Other sources of losses in open channels include confluences, transitions, bends, bridges, culverts and drop structures. When computing water surface profiles either by hand or with the help of a computer program, the engineer must include the significant sources of head loss.

4.3.8. CONFLUENCES

The alignment of confluences is critical to channel erosion and energy losses caused by turbulence and eddies. The principle variables used in designing channel junctions are angle of intersection, shape and dimensions of the channel, flow rates and flow velocities. Definitions of the variables are given in the Standard DISTRICT Details.

The angle of intersection between the main channel and tributary channels or storm sewers shall be thirty (30) degrees as shown in the Standard DISTRICT Details. Outfalls or junctions perpendicular to the receiving channel will create severe hydraulic problems, and therefore will not be allowed without approval by the DISTRICT Engineer.

Any protective lining must extend far enough upstream and downstream on both channels to prevent serious erosion. The slope protection must be carried up to at least the ten (10) year flood level in both channels. A good grass cover must be established and maintained from the edge of the protection to the top of bank.

If the main channel flowline is lower than the side channel flowline, an erosion control structure must be used in the side channel.

4.3.9. TRANSITIONS

4.3.9.1. DESIGN

Transitions in channels should be designed to create a minimum of flow disturbance and thus minimal energy loss. Transitions generally occur at bridges or culverts, and where cross-sections change due to hydraulic reasons or easements and fee strip restrictions. The transition can consist of either a change in cross-section size or geometry.

All angles of transition should be less than twelve (12) degrees twenty (20) feet in one hundred (100) feet. When connecting trapezoidal and rectangular channels, the warped or wedge type transition is recommended (Reference 10). If supercritical flow conditions are encountered, standing waves, superelevation and hydraulic jumps must be considered. See References 10 and 14 for discussions of transitions and supercritical flow.

4.3.9.2. ANALYSIS

Expansion and contraction losses must be accounted for in backwater computations. Transition losses are usually computed using the energy equation and are expressed in terms of the change in velocity head from downstream to upstream of the transition. The head loss between cross sections is expressed by:

$$h_l = c \left[\frac{(V_2^2 - V_1^2)}{2g} \right]$$

Where:

h_L	=	head loss (feet)
c	=	expansion or contraction coefficient
V_2	=	average channel velocity of downstream section (feet per second)
V_1	=	average channel velocity of upstream section (feet per second)
g	=	acceleration of gravity (32.2 ft/sec^2)

Typical transition loss coefficients are given below:

Transition Type	Coefficient	
	Contraction	Expansion
Gradual or Warped	0.10	0.30
Bridge Sections, Wedge, or Straight Lined	0.30	0.50
Abrupt or Squared End	0.60	0.80

When computing the backwater profile through a transition, engineering judgment must be used in selecting the reach lengths. As a guideline, the velocity should not change more than fifty (50) percent between two cross sections. Smooth transitions require fewer computation steps than the abrupt transitions.

If the HEC-2 or HEC-RAS computer program is used to compute the backwater profile, expansion and contraction losses are included in the energy equation. The user must incorporate the loss coefficients given above or as described in the user's manual (Reference 5).

4.4. BENDS

4.4.1. DESIGN

Channel bends or curves should be as gradual as possible to reduce erosion and deposition tendencies. For channel bends with a radius of curvature measured from the channel centerline of less than three (3) times the top width of the ultimate channel, slope protection is required. For both lined and unlined channels, a ninety (90) degree bend is the maximum curve allowed. Erosion protection on bends must extend at least along and twenty (20) feet downstream of the curved section on the outside bank. Additional protection may be required on the channel bottom and inside bank or further downstream than twenty 20 feet, if the channel geometry and velocities indicates a potential erosion problem.

4.4.2. ANALYSIS

Head losses should be incorporated into the backwater computations for bends with a radius of curvature less than three (3) times the channel top width. Energy loss due to curve resistance can be expressed as:

$$h_L = c_f V^2/2g$$

Where:

h_L	=	head loss (feet)
c_f	=	coefficient of resistance
V	=	average channel velocity (feet per second)
g	=	coefficient of gravity (32.3 feet/second)

Guidelines for selecting c_f can be found in Reference 10.

The HEC-2 computer program does not incorporate a bend loss computation. Therefore the DISTRICT'S Engineer must approve the use of HEC-2.

4.5. UTILITY CROSSINGS

Approval must be obtained from the DISTRICT for all utility lines which cross a flood control facility. The utility crossing should be designed to minimize obstruction of the channel flow and conform with the channel cross-section. Contact the DISTRICT for information regarding the channel section and channel easement or fee strip at a proposed crossing prior to design.

All utility lines under channels should be constructed with the top portion of the conduit a minimum of five (5) feet below the projected flowline of the ultimate channel as shown in the Standard District Details. Pipelines shall have a minimum of ten (10) feet of cover depth. When appropriate, facilities may be constructed on special utility bridges or trestles in accordance with standard bridge design criteria. Pipes or conduits spanning the channel should be located above the top of banks for hydraulic and maintenance reasons. These overhead crossings shall be approved by the DISTRICT prior to design and construction. For utility crossings on street bridges, contact the appropriate government body for approval.

All manholes required for the utility conduit shall be located outside of the DISTRICT'S easement and fee strip. Backfill within the DISTRICT'S easement and fee strip shall be in accordance with the backfill requirements specified by the respective district, county, or utility company.

Crossings must be clearly marked in the field with a sign on either side of the DISTRICT facility, which shall be placed immediately outside the DISTRICT easement or right-of-way.

5. STORM SEWER DESIGN

Storm sewers shall be designed and constructed in accordance with the applicable City Ordinance and/or criteria. In locations within the DISTRICT where no City has jurisdiction, Brazoria County storm sewer regulations shall prevail.

6. SHEETFLOW DRAINAGE

The extreme event sheet flow drainage from all developed areas must be directed to the detention facility. The entrance of this drainage must be placed in a protected (slope paved) area. It cannot be discharged over an area that is unpaved or protected. See section 3.4.5 for additional requirements.

REFERENCES

1. Turner Collie & Braden, Inc. Comprehensive Study of Drainage for Metropolitan Houston, Sections I-VI, 1969 through 1971.

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3. Hydrology for Harris County, 1988, Seminar presented by ASCE and Harris County Flood Control District.

4. Johnson, S. L., and D. M. Sayre. "Effects of Urbanization on Flood in the Houston Metropolitan Area," U. S. Geological Survey, April 1973.

5. U.S. Army Corps of Engineers. HEC-2 Water Surface Profiles Users Manual, The Hydrologic Engineering Center, Davis, California, September 1990.

6. U.S. Weather Bureau. "Rainfall Frequency Atlas for the United Stated for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years," Technical Paper No. 40, January 1963.

7. U.S. Army Corps of Engineers. Civil Works Bulletin 52-8.

8. U.S. Army Corps of Engineers. HEC-1 Flood Hydrograph Package Users Manual, The Hydrologic Engineering Center, Davis, California, Revised May 1991.

9. Joint Venture of Turner Collie & Braden, Inc. and Pate Engineers, Inc. Flood Hazard Study for Harris County, Final Report, Harris County Flood Control District, September 1984.

10. Chow, V. T., Open-Channel Hydraulics, McGraw-Hill, 1959.

11. Ramser, C. E., Flow of Water in Drainage Channels, U.S. Department of Agriculture, Technical Bulletin No. 129, November 1929.