SECTION II - PROJECT APPROACH

This section summarizes the overall project approach, main engineering tasks and workflows, project assumptions, and other necessary efforts in developing the Master Drainage Plan.

PROJECT CONSIDERATIONS

Several factors were considered in evaluating the existing drainage systems (open ditches and underground storm sewers), identifying flooding problem areas and inadequate drainage systems, and determining and optimizing drainage improvements. These considerations are:

- Historical flooding areas within the City of Harlingen – information of historical flooding was obtained from previous drainage reports associated with the City and adjacent areas.
- Storm sewer design criteria – the City of Harlingen's drainage design criteria requires storm sewers designed to convey runoff from the 5-year frequency storm event (20-percent annual exceedance) with the corresponding hydraulic gradient controlled to be not higher than the gutter line.
- Outfall drainage ditch capacity – 25-year minimum flood protection.

Extensive storm sewer data for this project are available from the 1983 Storm Water Management Plan and as-built plans. The available storm sewer data includes pipe size, lengths, flow line elevation, and manhole invert and rim elevations; however, the majority of the available storm sewer data does not include gutter line elevations. Based on typical roadway design parameters, a 4-lane roadway section has a gutter elevation approximately 0.5 feet below the road crown and a curb height of 6 inches. For the analysis of this project, a gutter elevation of 6-inches below the manhole rim elevation was assumed, which was used as the benchmark for the hydraulic gradient. In determining the adequacy and the required improvements of a storm sewer system, the criterion is the gutter line elevation (6 inches below the manhole rim). Also, manhole rim elevation was assumed to be about the same as the natural ground elevation in this study.

GEOGRAPHIC INFORMATION SYSTEM

Geographic Information Systems (GIS) provides a powerful workspace for storing, compiling, overlaying, integrating, updating and exchanging, analyzing, displaying, and managing geospatial data. It provides a consistent method for watershed and stream network delineations using digital elevation models (DEMs) of land-surface terrain. GIS provided a vital role, from GIS data manipulation to geospatial data layers creation in developing this project.

GIS Applications

ESRI ArcGIS 9.2 was used to facilitate the various engineering workflows required for the project development, including base mapping, topographic data development, drainage system delineations, drainage and hydrologic parameter extraction, hydrologic and hydraulic model development and analyses, and drainage system inventory. The suite of ArcGIS tools utilized for the purpose of this project include Spatial Analyst, 3D Analyst, ArcHydro, HEC-GeoHMS, HEC-GeoRAS, and other extensions.

Digital base maps were complied from various sources. These datasets were manipulated using conversion, geographical referencing, and integration based on standard GIS shape file format. GIS base map data layers for this project included raster aerial photographs, vector street maps, political boundaries (cities, counties, districts), and stream and drainage ditch centerlines.

Automated extraction of topographic parameters from DEMs has been recognized as a viable alternative to traditional surveys and manual evaluation of topographic maps, particularly as the quality and coverage of DEM data increase. A continuous DEM dataset was developed based on 506 tiled LiDAR datasets obtained from Texas Natural Resource Information System (TNRIS). The original LiDAR has a resolution of 3.3 feet in LAS format. These tiled LiDAR datasets were converted to ESRI Arc GRID format through a series of spatial data processing procedures including LAS to Multipoint , Multipoint to TIN, and TIN to GRID. The resulting tiled grid datasets were then merged into one single grid dataset. With consideration of the very large size of LiDAR dataset, a 5-foot grid dataset was created as the base DEM dataset for this project. Large data voids were identified and were filled using USGS DEM data. Small data gaps were identified and filled by interpolating surrounding data points.

By examining the high resolution LiDAR DEM imagery, natural streams and drainage ditches are generally well defined by the LIDAR data. However at many stream crossings (culverts and bridges), elevations on top of the structures were included in the DEM. If the original DEM data was used for automated drainage delineations, the natural drainage patterns could be erroneously modeled without removing these structures from the DEM.

To remove the dam-like hydraulic structures in the DEM, stream and drainage ditch lines and aerial photographs were used to identify stream crossing structure locations. Stream crossing breaklines were created at each hydraulic structure where the top elevation of the structure was included in the DEM. ArcHydro reconditioning tool was used to burn the resulting stream crossing breaklines into the DEM with a certain depth based on the adjacent flowline elevations at the structure. A hydro-enforced DEM was created.

In developing the HEC-HMS and HEC-RAS models, the ArcHydro, HEC-GeoHMS, HEC-GeoRAS tools were extensively used to facilitate the delineation of drainage areas, extraction of drainage parameters, and extraction of channel cross section geometric data.

Storm sewer system GIS layers were created by digitizing paper maps from the 1983 Storm Water Management Plan and as-built plans. Two storm sewer system GIS layers were created, storm pipe and manholes. For storm drainage catchment delineations, the storm sewer line layer was first integrated into the 5-foot DEM dataset. This process was necessary since the LiDAR data does not reflect the underground storm sewer alignments. The resulting drainage catchment delineations were verified based on street patterns, aerial photographs, and field visits.

GIS Feature Layers

As part of the engineering efforts in developing the master drainage plan, a set of GIS data layers were created, which provide a digital version of the master drainage plan. The GIS data layers can be updated by incorporating future drainage projects and provide an easy way to locate drainage features such as:

- Locations of storm sewer and manholes
- Locations of drainage ditches and hydraulic structures (bridges and culverts)
- Characteristics of drainage systems (pipe shape, sizes, material, sizes, flowline, channel sizes, and structural sizes)
- Flood prone areas
- Soil and land use data

The common datum, projection, and coordinate system for this project are:

- Horizontal Datum - North American Datum 1983 (NAD83) (Feet)
- Vertical Datum - North American Vertical Datum 1988 (NAVD88)
- Projection - Lambert Conformal Conic
- Coordinate System - State Plane Texas South 4205

PROJECT DATA DEVELOPMENT

Pertinent project data, previous drainage studies, construction plans, field survey, aerial mapping, soil and land use maps, were collected from various sources, including local, state and Federal agencies. Compilation of GIS datasets required conversion of file formats and coordinates systems, as well as geographical referencing of nonspatial datasets.

Storm Water Management Plan - 1983

The City of Harlingen's 1983 Storm Water Management Plan contains information on the existing storm sewer systems such as storm sewer locations, shapes, sizes, lengths, materials, and flowlines elevations. The existing storm sewer system maps from the 1983 Plan were scanned and georeferenced to the digital aerials and street maps in the project GIS workspace. The storm sewer alignments and manholes from the plan were then digitized as line and point GIS feature layers.

Construction Plans

Digital and hard copy as-built plans for various drainage improvement projects within the study area were obtained from the City's Public Works Department. The digital data was in CAD format and was converted to GIS format and was overlaid with the base maps. Hardcopies of storm sewer plans were digitized and combined with the storm sewer data from the 1983 Plan.

LiDAR Data

Tiled LiDAR datasets in LAS format for all of Cameron County were obtained from Texas Natural Resources Systems (TNRIS). The tiled LiDAR data for the study area was converted to ESRI grid format and merged into a single continuous dataset using ArcGIS tools, as discussed in detail in the Geographic Information System (GIS) Section.

Field Survey Data

Field data was collected by Brown Leal & Associates for this project using GPS. Field surveying was performed to collect channel and hydraulic geometric data for the studied drainage ditches. The field data was collected in accordance to the Appendix N: Data Capture Guidelines, Guidelines and Specifications for flood hazard Mapping Partners. Hydraulic structures include bridges, culverts, levees, and other structures within each drainage ditch corridor. The field survey data is referenced to NAD 83 State Plane Coordinate System – Texas South and the vertical datum of NAVD 88. These datum references are consistent with the coordinate system and vertical datum of the available LiDAR data for countywide FEMA Map Modernization Project. The surveyed data was used in combination with the LiDAR dataset for the drainage ditch HEC-RAS hydraulic modeling analysis. Field survey data and notes are provided in ***Appendix A***.

Soil Map

Hydrologic soil group type is an important hydrologic parameter for NRCS dimensionless unit hydrograph rainfall-runoff hydrologic modeling. The NRCS Soil Data Mart provides over 2000 soil surveys with spatial and tabular information. Geospatial soil data for Cameron County was downloaded from the NRCS Soil Data

Mart. The dataset with both spatial and tabular data were spatially referenced to the UTM NAD83.

Land Use Map

Land use within a drainage basin generally has the greatest impact on the rainfall – runoff relationship. Land use and land cover are important physical characteristics in estimating the percent impervious cover of a drainage basin. Parcel maps for Cameron County were obtained from the City of Harlingen Public Works Department. The land use and land cover types were assigned to each polygon based on parcel map and aerial photos. Corresponding percent impervious cover was assigned to the land use map. For the purpose of this project, land use and land cover are classified into eight (8) groups:

TABLE 1. LAND USE TYPE VS PERCENT IMPERVIOUS COVER

LAND USE TYPE	LAND USE DESCRIPTION	IMPERVIOUS (%)
U	Undeveloped areas	0
RS	Residential < 1 acre	40
RL	Residential Large > 1 and < 5 acres	20
RR	Residential Rural > 5 acres	5
T	Transportation corridors and streets, etc.	90
W	Water bodies, streams, lakes, reservoirs, etc	100
GA	Green Areas	10
HD	Commercial, Industrial, Schools	85

The land use polygon layer was converted to a raster format based on percent impervious cover value. Composite percent impervious cover for each drainage area was then computed using ArcGIS Zonal Statistics of Spatial Analyst based on the percent impervious cover raster dataset.

Aerial Photographs

Aerial photographs, which cover the entire Rio Grande Valley was acquired from Aerials Express. These aerial photographs were flown in 2006 by Aerials Express. The aerial photographs have a ground resolution of 1.5 feet. The images are provided in an ECW compression format and are referenced to UTM NAD83.

DRAINAGE BASIN MODELING

Drainage basin modeling analysis included drainage area delineations, watershed and hydrologic parameters estimation, development of HEC-HMS hydrologic models, computation of peak flows and hydrographs at various locations for various storm events.

Drainage Area Delineation

Drainage areas were delineated using the LiDAR DEM dataset developed for the project area. LiDAR-based DEM provides a more detailed terrain representation than traditional USGS DEM. ArcGIS tools including Spatial Analyst, 3D Analyst, ArcHydro and GeoHMS were used for automation of drainage basin delineations and extraction of drainage parameters for the hydrologic models for this study. ArcHydro and GeoHMS have the capability to process DEM datasets for automated drainage and stream delineations and generation of spatially based hydrologic input files for HEC-HMS. Automated drainage delineation included a series of spatial data processing steps to derive the drainage networks from the DEM dataset. Specific steps consisted of computing the flow direction, flow accumulation, stream definition, watershed delineation, watershed polygon processing, stream processing, and watershed aggregation. The resulting delineations were verified by consulting with city engineering staff, aerials photographs, and field visits.

HEC-HMS Modeling

The Hydrologic Modeling Systems (HEC-HMS) modeling program was used to compute peak flows and hydrographs at various locations within the drainage systems for various storm events. The HEC-HMS program is the most widely used software package for hydrologic modeling analysis and was developed by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers (USACE). The program is designed to simulate the precipitation-runoff-routing processes of watershed systems. HEC-HMS requires three input components: basin component, precipitation component, and control component. The basin model was generated using GIS based on the LiDAR DEM dataset. The basin file contains the schematic, the hydrologic methods, and watershed and hydrologic parameters. The basin schematic represents drainage patterns with the connectivity of the hydrologic elements (subbasin, reach, junctions, detentions, and diversions).

HEC-GeoHMS was used to estimate hydrologic parameters by providing tables of physical characteristics of the streams and watersheds. When the streams and subbasins delineation were finalized, stream physical characteristics, such as longest flow lengths, upstream and downstream elevations, and slopes, were extracted from the terrain data.

These physical characteristics were then exported and used externally to estimate other drainage and hydrologic parameters.

Precipitation Data

The frequency-based hypothetical storm method was selected within the HEC-HMS meteorologic model. This method defines a storm event that the precipitation depths that have a consistent exceedance probability for various storm durations. Using this method requires input of depths of precipitation for various durations and frequencies. The precipitation depths for various durations for a specified exceedance probability are usually obtained by consulting locally-developed depth-duration-frequency functions. For this project, the precipitation depths were taken from the USGS publication – Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas, 2004 by Asquith and Roussel. The newly developed Atlas was developed based on the generalized logistic distribution method (GLO) which is discussed in detailed in the Water-Resources Investigation Report by USGS in cooperation with Texas Department of Transportation. This method provides more accurate estimates of rainfall depths than the traditional TP-40 and HYDRO-35.

The CCDD5 Flood Protection Plan report recommends a 48-hour storm duration to be used for hydrologic modeling within the Cameron County area. For consistency of analysis between the CCDD5 report and this study, the 48-hour storm duration was adopted for this project. **Table 2** below summarizes the precipitation depths for various durations and storm events.

TABLE 2: RAINFALL DEPTHS-DURATION-FREQUENCY

STORMS	15-MIN	1-HR	2-HR	3-HR	6-HR	12-HR	24-HR	48-HR
2-year	1.0	1.9	2.4	2.5	2.8	3.2	3.5	4.1
5-year	1.4	2.5	3.1	3.3	3.8	4.4	5.1	6.0
10-year	1.3	3.0	3.7	3.9	4.6	5.3.	6.0	7.0
25-year	1.9	3.5	4.5	4.6	5.7	6.5	7.5	9.0
100-year	2.3	4.5	5.8	6.2	8.0	9.0	10.0	12.0

The frequency-based hypothetical storm meteorologic model was used for this study. This model employs a balanced and nested rainfall distribution, which positions the block of maximum incremental depth at the middle of the required duration. The remaining blocks are arranged then in descending order, alternately before and after the central block.

Unit Hydrograph

A rainfall/runoff transformation is required to convert rainfall excess (total rainfall minus infiltration losses) into runoff from a particular drainage area. The unit hydrograph is a well-known, commonly-used empirical model of the relationship of direct runoff to excess precipitation. The unit hydrograph method represents a hydrograph generated from one unit (inch) of direct runoff and is a nationally accepted, standard engineering practice approach. The underlying concept of the unit hydrograph is that the runoff process is linear, so the runoff from greater or less than one unit of rainfall is simply a multiple of the unit runoff hydrograph.

The NRCS Dimensionless Unit Hydrograph method was used for this study. This method was used in previous FEMA floodplain studies and other drainage projects within Cameron County. The standard dimensionless unit hydrograph was an "averaged" hydrograph derived from a large number of natural unit hydrographs from watersheds ranging in sizes and geographical locations. Using the standard NRCS UH, it is assumed that 37.5 percent of the total runoff volume occurring in the rising side of the curvilinear hydrograph (or the receding time from the peak is 1.67 times of the time to peak). Based on this assumption and with consideration of unit conversion factors in the equation (such as square miles to acres, acre to cubic feet, and hour to seconds) in peak discharge computation, a peaking rate factor (PRF) of 484 was derived as shown in the following equation:

$$Q_p = \frac{484A}{T_p}$$

Equation 1

in which Tp is time of peak and is related to the duration of the unit of excess precipitation as:

$$T_p = \frac{\Delta t}{2} + t_{lag}$$

Equation 2

in which Δt is the excess precipitation duration and t_{lag} is the subbasin lag.

Peaking Rate Factor (PRF)

Recent studies have shown that peaking rate factor (PRF) has a range of value from below 100 to more than 600. The University of Florida has found that a peak rate factor of 75-100 is appropriate for Flatwoods watersheds. A study done by University of Lamar, University of Houston, and Texas Tech University has shown a mean PRF of 370 from watersheds including Trinity River, Brazos River, Colorado River, San Antonio River in Texas. Since City of Harlingen is located in the Gulf Coastal Plain, it is obvious that the constant peaking rate factor of 484 does not represent the runoff behavior for the study area.

The U.S. Bureau of Reclamation (USBR) developed a dimensionless unit hydrograph for drainage design purposes for a project entitled "Valley Gravity Project" in 1948. The dimensionless unit hydrograph was obtained by averaging two study areas with 48.56 and 22.83 square miles respectively within the Valley. By examining the dimensionless unit hydrograph, a PRF value of 130 was derived.

The Flood Protection Plan report prepared by Espey Consultants, Inc. for CCDD5 recommends a PRF value of 200 for the study area. To be consistent, a 200 PRF value was used for this project. Based on previous project experience with the Hidalgo County Flood Map Modernization Project, a 200 PRF is a conservative value.

Initial Abstraction

The initial loss parameter represents the rainfall volumes that do not contribute to the excess runoff. Typical representative rainfall losses include land-cover interception, depression storage, and infiltration. The NRCS Runoff Curve Number (CN) method was used for this project. This method relates accumulated runoff to accumulated rainfall with an empirical curve number (CN). CN is a function of land use and cover, soil classification, hydrologic conditions, and antecedent soil moisture. The initial abstraction is approximated by the following empirical equation:

$$Ia = 0.2S \qquad \textit{Equation 3}$$

Where S is the potential maximum retention and defined as:

$$S = (1000/CN) - 10 \qquad \textit{Equation 4}$$

Curve Number (CN)

The CN for a drainage basin can be estimated as a function of land use, soil type, and antecedent watershed moisture, using tables published by the NRCS. With these tables and knowledge of the soil type and land use, a single-valued CN can be found. For a watershed that consists of several soil types and land uses, a composite CN is calculated as:

$$CN_{composite} = \frac{\sum A_i CN_i}{\sum A_i} \qquad \textit{Equation 5}$$

Since GIS digital geospatial datasets (land use and soil) are available, the composite CN values for drainage basins were computed using land use data, hydrologic soil group, and a lookup table that relates land use and soil group with curve number from the following procedures:

- A lookup table was developed between land use/soil types and runoff curve number (CN) in excel format, as shown in **Table 3**. The lookup table included two fields: (1) A composite code developed based on land use and hydrologic soil group, (2) curve number values from TR55 and other publications.
- The soil map was intersected with the land use map to obtain a combined layer containing both soil and land use data.
- Using the CN lookup table, CN values were assigned to each intersection polygon.
- The intersected polygon was converted to an Arc grid raster with each cell value corresponding to the CN values obtained in Step 3.
- Using HEC-GeoHMS for ArcGIS 9 and the drainage basin polygons, composite CN values were computed from the CN grid for each drainage area.

The NRCS runoff curve numbers are published for three types of Antecedent Moisture Conditions (AMC): I, II and III. AMC I represents dry soil conditions and AMC III represents saturated soil conditions. AMC I is used for areas that have the lowest runoff potential. In general, AMC II is considered to be the typical soil condition; however, studies have indicated that AMC II is not appropriate in all parts of Texas. Investigations have shown that the average condition ranges from AMC I in west Texas to between AMC II and III for east Texas. Runoff curve numbers vary from 0 to 100, with the smaller values representing soils with lower runoff potential and the larger values representing soils with higher runoff potential. As discussed in the Cameron County Drainage District No.5 Flood Protection Plan, an AMC I is more appropriate to represent average condition for the study area with dry soil conditions.

TABLE 3. CURVE NUMBER (CN) LOOKUP TABLE

HYDROLOGIC SOIL TYPE	LAND USE TYPE	COMBINED CODE	CURVE NUMBER
A	GA	GAA	49
B	GA	GAB	69
C	GA	GAC	79
D	GA	GAD	84
W	GA	GAW	100
A	HD	HDA	89
B	HD	HDB	92
C	HD	HDC	94
D	HD	HDD	95
W	HD	HDW	100
A	RL	RLA	51
B	RL	RLB	68
C	RL	RLC	79
D	RL	RLD	84
W	RL	RLW	100
A	RR	RRA	45
B	RR	RRB	62
C	RR	RRC	75
D	RR	RRD	82
W	RR	RRW	100
A	RS	RSA	62
B	RS	RSB	76
C	RS	RSC	84
D	RS	RAD	88
W	RS	RSW	100
A	T	TA	82
B	T	TB	88
C	T	TC	91
D	T	TD	93
W	T	TW	100
A	U	UA	39
B	U	UB	61
C	U	UC	74
D	U	UD	80
W	U	UW	100
A	W	WA	100
B	W	WB	100
C	W	WC	100
D	W	WD	100
W	W	WW	100

Lag Time

The lag time represents the time difference between hyetograph center of mass to the peak of the hydrograph for a subbasin. The following NRCS Lag Time equation was used for this study:

$$L_p = \frac{L_W{}^{0.8}\left\lfloor\left(1000/CN\right)-9\right\rfloor^{0.7}}{31.67S^{0.5}} \qquad \textit{Equation 6}$$

Where Lp is the lag time in minutes, Lw is the length of the longest flow path in feet, S (%) is the slope of the longest flow path, and CN is the composite curve number.

Since there is no provision in HEC-HMS for changing the 484 constant, in order to reflect the lower PRF value of 200 for this project, the time of peak Tp parameter for each subbasin was modified by using a multiplier factor of 484/PRFnew to the resulting Tp computed based on the above NRCS lag time equation. With this modification, the unit hydrograph definition is preserved.

A comparative analysis was performed for the aforementioned approach for modifying the PRF value. Based on the comparative analysis, the modified lag approach results in similar peak flow rates to those from an user-defined unit hydrograph based on a gamma distribution.

Percent of Imperviousness

Percent impervious cover values were estimated by performing similar procedures as for estimating the composite curve numbers. The land use classifications with impervious values were converted to a grid with cell values corresponding to the percent imperviousness values. Using HEC-GeoHMS for ArcGIS 9 and the subbasin polygons, area-weighted composite percent imperviousness values were computed using Zonal Statistics function of Spatial Analyst. It should be noted that land use classifications (percent impervious cover) are reflected within the curve number within all HEC-HMS models for this study. There are no separate impervious cover parameters for HEC-HMS modeling.

Reach Routing

Three channel reach routing methods were used in this study: Modified Puls, Muskingum-Cunge 8-Point, and Lag methods. The Modified Puls routing method is a routing technique that relates storage, outflow, and water surface slope in a stream reach. It is also known as storage routing or level-pool routing based upon a finite difference approximation of the continuity equation, coupled with an empirical representation of the momentum equation. The number of routing steps is defined as the wave travel time divided by the time step (HMS computation interval). For a channel reach with less than 0.5 feet per second, the reach is assumed to be functioned as a linear reservoir with a routing step of one. In flat areas, such as the Texas coast, channel and overbank storage have a significant influence on watershed hydrology.

The Muskingum-Cunge 8-Point method is used to modify hydrographs to reflect the effects of translation and attenuation within a channel reach. The input parameters of this method are physical channel properties: channel length, channel slope, Manning's roughness coefficients, and the friction slope or channel bed slope. An 8-point cross-section configuration is used to represent a typical channel section through each stream routing reach. Storage is a function of inflow and outflow, weighted by travel time through the reach. Hydrograph diffusion is based on channel properties and the inflowing hydrograph. This method does not take into account backwater effects

Lag time is the simplest routing model. The outflow hydrograph is simply the inflow hydrograph, but with all ordinates translated (lagged in time) by a specified duration. The flows are not attenuated, so the shape is not changed.

All resulting HEC-HMS models are contained on the attached **CD-ROM**.

DRAINAGE DITCH MODELING

Drainage ditches that are contained in the nine drainage systems were analyzed using hydraulic computer models. Each system was evaluated to determine its existing conveyance capacities, identify overbank flooding risk areas and system constraints, and the hydraulic impacts of the system to underground storm sewer systems.

HEC-RAS Modeling

The U.S. Army Corps of Engineers' HEC-RAS hydraulic software package was used for open ditch modeling analysis for this project. The HEC-RAS program is the most widely used software package for open channel hydraulic modeling analysis. The program was designed to perform one-dimensional steady and unsteady flow hydraulic calculations for a full network of natural and constructed channels. The HEC-RAS model creation requires definition of the land surface to be modeled and flow data for hydrologic events analyzed. The geometric and flow data is used to calculate water surface profiles from energy loss computations.

Channel geometric data is used to characterize the flow-carrying capacity off the stream and adjacent floodplain. Reach lengths are used to define the distance between cross sections and for energy loss calculations in HEC-RAS and indicated the path of flow between cross sections. Various energy loss coefficients are required for HEC-RAS hydraulic models, including channel and overbank friction, contraction-expansion losses, bridge losses and miscellaneous losses.

The HEC-RAS model was developed by importing the geometric import file generated from HEC-GeoRAS as discussed below, entering the existing hydraulic structure data such as bridges and culverts, and flow data into the geometry model data. The overbank cross section data were checked and the data points were filtered to reduce the number of points for each cross section. Two tailwater conditions were considered in this study: normal depth and known water surface elevations. Existing channel capacities were determined based on the existing HEC-RAS models. Alternative HEC-RAS geometry models were created using the Channel Modification tool to evaluate alternative channel improvements, and using Bridge/Culvert module to analyze hydraulic structure modifications.

HEC-GeoRAS

HEC-GeoRAS, an extension for use with ArcGIS, is a set of ArcGIS tools specifically designed to process DEM data to extract channel geometric data for HEC-RAS model development. Using HEC-GeoRAS requires a digital terrain model (DTM) represented by ESRI ArcGRID or triangulated irregular network (TIN). The current version of HEC-GeoRAS creates an import file, referred to as the RAS GIS Import File, which contains the channel network, channel cross-section geometry, and reach length data from a set of RAS themes and a DTM (collectively referred as the RAS Layers). The RAS themes, including stream centerlines, flow path centerlines, channel banks, and cross section cut lines, are created based on LiDAR data, channel cross section field survey data, and aerial photograph.

Roughness Coefficients

Various energy loss coefficients are required for HEC-RAS hydraulic models, including channel and overbank friction, contraction-expansion losses, bridge losses and miscellaneous losses.

Typically, channel roughness is indicated by Manning's n-values. It is the most important of the hydraulic loss coefficients. The variation of water surface elevation along a stream is largely a function of the boundary roughness and the stream energy required to overcome friction losses. The N value varies with flow depth, alignment, amount and type of vegetation, and flow obstructions. For all hydraulic models this study a Manning's roughness coefficient of 0.04 was assumed for drainage ditches, and 0.08 for overbanks.

Although water surface profiles are mostly influenced by friction forces, changes in the energy grade line, and the corresponding water surface elevations, can result from significant changes in stream velocity between cross sections. This is most apparent in the vicinity of bridges, which tend to force the discharge through an opening smaller than the upstream and downstream channels. Therefore, contraction and an expansion at a bridge results in eddy energy losses. These losses are usually quantified with coefficients of contraction or expansion. Bridge that causes relatively small changes in the energy grade and water surface profiles can be adequately modeled using appropriate values of Manning's and contraction and expansion coefficients. Contraction and expansion coefficients are applied upstream and downstream, respectively, of culverts and bridges to model the contraction and expansion of flow. In this study, contraction and expansion coefficients of cross sections bounding bridges and culverts is 0.3 and 0.5, respectively. All other cross sections use the default contraction and expansion coefficients of 0.1 and 0.3.

Bridges that cause the profile to become rapidly varied near and within the bridge require detailed bridge analysis with more detailed methods and bridge geometry data such as number, location, and shape of bridge piers, roadway profiles, weir flow coefficients, and upstream and downstream road overtopping elevations.

Ineffective flow limits are added to cross sections to accurately model any given section's inability to convey flow, such as cross sections that bound bridges and culverts. Ineffective limits were also set at the top of the channel banks to account for storage in overbanks that do not contribute to channel conveyance.

All resulting HEC-RAS models are contained on the Attached **CD-ROM**.

STORM SEWER MODELING

EPA SWMM Program

The EPA Storm Water Management Model (SWMM) (version 5) was used to evaluate and analyze hydraulic performance of storm sewer systems. SWMM is a dynamic rainfall-runoff simulation model which can be used for single event or long-term (continuous) simulation of runoff quantity from primarily urban areas. The runoff component of SWMM operates on a collection of catchment areas that receive precipitation and generate runoff. The routing portion of SWMM simulates runoff through a system of pipes, channels, storage, pumps, and regulators. SWMM tracks the runoff volume and flow rate generated from each subcatchment, and the flow rate and flow depth in each pipe and channel during a simulation period.

SWMM is a dynamic unsteady flow model rather than a steady state. It has the capability to simulate the effects of storage and backwater in conduits and floodplains and the timing of runoff. It generates a closer representation of the hydraulic grade line at any point within the drainage system.

SWMM computes hydraulic grade line elevation using the EXTRAN routing block. The EXTRAN routing block uses a link-node representation of the storm sewer/channel system. The storm sewer system is idealized as a series of sewer reaches or links connected by nodes or manholes. Each link transmits flow from node to node. The link-node representation of a system makes it possible to numerically solve the gradually varied unsteady flow equations that are the mathematical basis of the model.

Rational Method

The Rational Method is the most widely used method for estimating peak flows for storm sewer design and analysis. The peak storm runoff rate is estimated from *Equation 7*. Peak rate of runoff for a catchment is a function of the drainage area, runoff coefficient and mean rainfall intensity for a duration equal to the time of concentration (T_c).

$$Q = CIA \qquad \textit{Equation 7}$$

where Q is the maximum rate of runoff, C is runoff coefficient representing a ration of runoff to rainfall, I is the average rainfall intensity for a specific return period and time of concentration, and A is the contributing drainage area to the study location.

The runoff coefficient C is parameter that is least susceptible to precise determination and requires judgment and understanding. While engineering judgment will always be required in the selection of runoff coefficients, a typical coefficient is available to represent the integrated effects of many drainage basin parameters. The type of land use can greatly affect the amount of runoff. The quantity of runoff and peak flow rates are increased when the land is developed because the impervious surface area increases with the addition of roads, driveways, roofs, etc. The table below lists the recommended runoff coefficients for each land use from publications:

TABLE 4. RUNOFF COEFFICIENT VALUES

LAND USE TYPE	LAND USE DESCRIPTION	IMPERVIOUS (%)	C
U	Undeveloped areas	0	0.20
RS	Residential < 1 acre	40	0.45
RL	Residential Large > 1 but < 5 acres	20	0.25
RR	Residential Rural > 5 acres	5	0.25
T	Transportation corridors and streets, etc.	90	0.74
W	Water bodies, streams, lakes, reservoirs, etc	100	1.00
GA	Green Areas	10	0.20
HD	Commercial, Industrial, Schools	85	0.80

Drainage areas usually consist of several different catchments that have different runoff coefficients. A composite runoff coefficient "C" must be computed to take into account the various surface characteristics of the catchments. Based on the geospatial land use polygon dataset, area-weighted composite runoff coefficient values were estimated using the following procedures:

- Join the runoff coefficient table to the land use polygon coverage based on land use types.
- Covert the land use polygon dataset into raster dataset with C as the pixel value based on the runoff coefficient C value field.
- Compute area-weighted C values for each catchment using the Zonal Statistics tool of Spatial Analyst of ArcGIS.

Rational Method requires the rainfall intensity (I) parameter, which a function of the time of concentration (T_c). The rainfall intensity as a function of the time of concentration was estimated by the Steel Equation:

$$I = \frac{b}{(d + T_c)^e}$$

Equation 8

Where:

I = Rainfall intensity (inches/hour)
b, d, e = Numeric constants dependent on rainfall frequency
T_c = Time of concentration (minutes)

The numeric constants are summarized in the following table.

TABLE 5. NUMERIC CONSTANTS FOR RAINFALL INTENSITY ESTIMATION

RAINFALL FREQUENCY	b	d	e
2-year	74.0	9.2	0.826
5-year	78.0	8.8	0.786

The time of concentration is the time required for water to flow from the hydraulically most remote point of the drainage area to a point under investigation (a manhole in this project). There are many methods available for estimating the time of concentration. The following equation adopted by the City of Houston Drainage Criteria Manual was used for the purpose of this project:

$$T_c(\min utes) = 10A^{0.1761} + 15$$ *Equation 9*

where A is drainage area in acres.

Triangular Hydrograph

As discussed earlier, SWMM is a dynamic model and requires hydrographs at nodes within the system. EPA-SWMM 5 does not have the function to generate hydrographs using Rational Method. For this reason, hydrographs were approximated by a triangular shape with peak flow estimated from the Rational Method and a time base of 2 times T_c. It was assumed that the rainfall duration is equal to the time of concentration for each drainage catchment. The triangular hydrograph applied to the corresponding manhole node as user defined hydrograph and routed through the storm sewer network to compute hydraulic grade line elevations at various locations within the analyzed system.

All resulting EPA-SWMM models are contained in the attached **CD-ROM**.

CAPITAL IMPROVEMENT PLAN (CIP)

The purpose of the CIP is to give the City a tool to plan its future capital improvement expenditures for drainage systems within the City. The CIP enables the City and its stakeholders to identify when projects can be implemented and to coordinate the schedule and funding. The CIP identifies the general characteristics of proposed drainage modifications or improvements required for the drainage systems that are currently considered to be inadequate relative to City's drainage design criteria and other factors.

Proposed improvements considered within the Capital Improvement Plan include:

- Storm sewer enlargements
- Drainage ditch channel modifications
- Drainage ditch structure replacements/ modifications
- Detention basins
- Levees
- Buyouts

Proposed Structural Measures

Proposed storm sewer improvements include the replacement and upsizing of the existing system, either portions or the entire system. Drainage ditch proposed improvements include channel modification including channel excavation and concrete riprap lining. Drainage ditch structure improvements include the replacement of existing culvert crossings or the addition of culvert barrels to the existing structure. Detention basins include the excavation of tracts to provide storage volumes for the reduction of flow rate within the channel or to provide a lower tailwater condition for storm sewer systems.

Modifications to the existing drainage systems, both open ditches and underground storm sewers, were proposed based on extensive hydrologic and hydraulic modeling analyses with considerations of City's drainage criteria, flooding complaints, desired flood protection levels, outfall tailwater effects, and development conditions.

Existing drainage ditches were evaluated to identify system capacities, overbank flooding areas, system constraints, and improvement opportunities. Existing HEC-RAS hydraulic models were constructed for each drainage ditch using LiDAR and field survey topographic data and as-built plans. Peak flows for the hydrologic event studied were obtained from HEC-HMS models.

Improvement plans were optimized by formulating and evaluating various alternative measures (modifications to channels and hydraulic structures, and construction of detention basins). Channel Design/Modification Editor of HEC-RAS was used for evaluating the various channel modification options. Bridge Culvert Data Editor was used to evaluate hydraulic structure modification options.

Detention basin measures were investigated with consideration of open land availability adjacent to the drainage ditches, tailwater conditions, development conditions, and the effectiveness of detention to reduce peak flows within the channels. Storage volume requirements were evaluated using the storage routing routine within the HEC-HMS model.

Storm sewer systems were evaluated using EPA-SWMM to identify existing capacities and inadequacies. EPA-SWMM modeling analysis was performed to determine existing conditions hydraulic grade lines and identify flooding problem areas by visualizing graphically hydraulic grade line and examining the modeling output results. According to the existing City's drainage design criteria, storm sewers should be designed for the 5-year rainfall event.

The CIP proposed modifications are for conceptual designs only, and do not include sufficient required details for final construction. Prior to final design of the proposed improvements, a more detailed analysis will need to be conducted by future design engineers or consultants to verify conditions used for the CIP.

<u>Proposed Non-Structural Measures</u>

One of the most effective tools for flood damage reduction involves buyout and demolishing of structures that were built deep in flood prone areas where structural flood reduction projects are impractical. Structures in this situation were typically built years ago before detailed floodplain maps and studies were available and prior to the adoption of floodplain management regulations. The purchase or "buyout" of houses that are flood-prone has proven to be one of the most cost-effective tools in many communities, when used under the correct conditions. Buying and removing flood-prone structures is the best way to ensure that they are not damaged by floods in the future. When compared to the high costs of larger channels, stormwater detention basins or other flood damage reduction options, a buyout can be a more cost-effective method of addressing the risk to people and their property.

Flooding in the City of Harlingen is relatively shallow, and as for other communities, much of the time the appropriate flood damage reduction tool is to construct a structural project to reduce expected flood levels.

Unit Cost

Probable construction costs associated with the proposed improvements were estimated based on unit cost information obtained from previous storm sewer projects constructed within the City of Harlingen and unit cost rate from Texas Department of Transportation Department (TxDOT). **Table 7** summarizes the unit cost data for this project.

Unit costs for channel modifications include:

- Channel excavation
- Bridge/culvert replacement
- Detention excavation
- Replacement of pavement
- Dewatering
- Traffic control
- Engineering contingency

Unit costs for storm sewer system improvements include:

- Removal of existing storm sewer pipe and pavement
- Placement of new pipe, manholes, and inlets
- Replacement of pavement
- Dewatering
- Trench safety
- Traffic control
- Engineering contingency

The unit costs do not include any cost allowance for major utility relocations, additional pavement replacement beyond the alignment of the proposed system, or acquisition of additional rights-of-way. Furthermore, the cost of any hydraulic impact mitigation on receiving streams or ditches is not included.

Proposed Project Costs

The costs for proposed improvements for drainage system were estimated based on the unit cost rates. The CIP project costs and project priority are discussed in detail in ***Section VII – Capital Improvement***. **Tables 19** and **20** summarize CIP project costs and priority for drainage ditches, and **Table 21** summarizes storm sewer systems CIP project costs and priority. Proposed improvements for drainage ditches and storm sewers along with costs and priority are summarized in ***Drainage Ditch Maps*** and ***Storm Sewer Maps*** respectively.

Prioritization

The CIP prioritizes future projects and provides a plan for funding of those projects. However, a project's position in the CIP can change based on a number of factors, such as lack of funding and other resources, reduction in priority and lack of public support. The City should attempt to support and recommend flood control projects that, whenever possible, will provide a regional flood control benefit. In addition, the CIP projects should contribute to the improvement of the community's well-being, support the development of the community, maintain the benefits of existing drainage systems, help protect natural habitat and landscapes, and provide multiple-use opportunities for flood control facilities.

To assist in prioritizing the potential CIP projects, a ranking matrix was developed based on three criteria. Relative weight values are assigned for each criterion. The proposed CIP projects were prioritized according to the recommended prioritization criteria ranking values. **Table 6** presents the prioritization criteria and respective relative weight values.

TABLE 6. CIP PRIORITIZATION CRITERIA

CRITERION	CRITERION DESCRIPTIONS	RELATIVE WEIGHT VALUE
1	Is the system hydraulically inadequate?	2
2	Have there been any flooding complaints?	3
3	Are project benefits independent of downstream improvements?	1

TABLE 7. SUMMARY OF UNIT COSTS

ITEM	UNIT	COST
PIPE SEWERS		
18" RCP	LF	$188
24" RCP	LF	$203
30" RCP	LF	$225
36" RCP	LF	$248
42" RCP	LF	$270
48" RCP	LF	$300
54" RCP	LF	$345
60" RCP	LF	$375
72" RCP	LF	$405
INLETS		
TYPE A CURB INLETS	EA	$2,835
TYPE A PRECAST CURB INLETS	EA	$2,235
TYPE F STORM INLETS (24" TO 36") RECEIVING PIPE	EA	$4,635
TYPE C-C INLET	EA	$3,135
MANHOLES		
DRAINAGE PRECAST CONCRETE MANHOLES	EA	$4,635
STORM MANHOLE 5' DIAMETER	EA	$4,935
STORM MANHOLE 7' DIAMETER	EA	$6,435
BOX CULVERTS		
CONC BOX CULV (6 FT X 6 FT)	LF	$416
CONC BOX CULV (7 FT X 7 FT)	LF	$495
CONC BOX CULV (8 FT X 6 FT)	LF	$686
CONC BOX CULV (8 FT X 7 FT)	LF	$712
CONC BOX CULV (8 FT X 8 FT)	LF	$740
CONC BOX CULV (10 FT X 5 FT)	LF	$812
CONC BOX CULV (10 FT X 6 FT)	LF	$670
CONC BOX CULV (10 FT X 8 FT)	LF	$832
CONC BOX CULV (10 FT X 9 FT)	LF	$1,016
CONC BOX CULV (10 FT X 10 FT)	LF	$828
WING WALLS		
WINGWALL (FW-0)(HW=6 FT)	EA	$10,228
WINGWALL (FW-0)(HW=7 FT)	EA	$13,988
WINGWALL (FW-0)(HW=8 FT)	EA	$16,190
WINGWALL (FW-0)(HW=9 FT)	EA	$21,615
WINGWALL (FW-0)(HW=10 FT)	EA	$18,656
WINGWALL (FW-0)(HW=11 FT)	EA	$19,929
CONCRETE PAVING		
RIPRAP (CONC) (5IN)	CY	$513
EXCAVATION		
EXCAVATION (Detention)	CY	$3
EXCAVATION (Channel)	CY	$3