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GIS FILES

<u>Filename</u>	<u>Description</u>
A100-BASINS.SHP	Clear Creek Watershed Boundaries
A100-L.SHP	Longest Watershed Path Shapefile
A100-LCA.SHP	Length to Centroid Path Shapefile
A100-LONGEST-1980.SHP	Longest Definable Channel Shapefile (1980 Conditions)
A100-CI-1980.SHP	Channel Improvement Shapefile (1980 Conditions)
A100-LONGEST-2000.SHP	Longest Definable Channel Shapefile (2000 Conditions)
A100-CI-2000.SHP	Channel Improvement Shapefile (2000 Conditions)
A100-DEVELOP-1980.SHP	Development Shapefile (1980 Conditions)
A100-DEVELOP-2000.SHP	Development Shapefile (2000 Conditions)
A100-CENSUS.SHP	Census Tract Shapefile
A100-POLITICAL.SHP	Political Entity Boundaries
A100-PONDING-1980.SHP	Ponding Shapefile (1980 Conditions)
A100-PONDING-2000.SHP	Ponding Shapefile (2000 Conditions)
A100-SOILS.SHP	Hydrologic Soil Group Shapefile
A100-CREEK.SHP	Creek Centerlines
A100-HEC-NODES.SHP	HEC-1 Routing Notes

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Development.xls	Development (DLU) projections
L-lca.xls	Longest watershed length and length to centroid summary
1980 Channel Improvement Summary.xls	Channel Improvement calculations (1980 conditions)
Ponding.xls	Ponding calculations
2000 Channel Improvement.xls	Channel Improvement calculations (2000 conditions)
2060 Main Channel Conveyance.xls	Conveyance projections
Tc&R Sheets.xls	Clark Unit Hydrograph parameter calculations

HEC-1 Batch Files

1980 Conditions
2010 "Controlled" Conditions
2060 "Controlled" Conditions
2060 "Uncontrolled" Conditions

Note: The HEC-1 batch files are organized by directory. All directories will contain the following files.

MKNA - COED directive file.
MKQ - COED directive file.
MKQ.BAT - COED directive file.
QDRIVE.BAT - Executable file for running COED script.
SEED1.DAT - HEC-1 seed file used by coed.
STRIP - COED directive file.
P2 -Text file that contains the 2-year event TC&R values to be input into the seed file.
P5 -Text file that contains the 5-year event TC&R values to be input into the seed file.
P10 -Text file that contains the 10-year event TC&R values to be input into the seed file.
P25 -Text file that contains the 25-year event TC&R values to be input into the seed file.
P50 -Text file that contains the 50-year event TC&R values to be input into the seed file.
P100 -Text file that contains the 100-year event TC&R values to be input into the seed file.
P250 -Text file that contains the 250-year event TC&R values to be input into the seed file.
P500 -Text file that contains the 500-year event TC&R values to be input into the seed file.

HEC-1 Files

1994Int.dat	1994 historical event simulation
2001Int.dat	2001 historical event simulation

Data

Nexrain_Oct1994.txt	October 1994 Historical Rainfall Data (HEC-1 Format)
Nexrain_Jun2001.txt	June 2001 Historical Rainfall Data (HEC-1 Format)

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HEC-RAS Files

Plan Files:

CC_GRR_WO.p01	Clear Creek Without Project - FEMA Flows
CC_GRR_WO.p02	Clear Creek Without Project - 1980 Conditions
CC_GRR_WO.p03	Clear Creek Without Project - Storage-Routing
CC_GRR_WO.p04	Clear Creek Without Project - 1994 Historical Event
CC_GRR_WO.p09	Clear Creek Without Project - 2001 Historical Event
CC_GRR_WO.p10	Clear Creek Without Project - 2010 Conditions
CC_GRR_WO.p11	Clear Creek Without Project - 2060 Conditions
CC_GRR_WO.p12	Clear Creek Without Project – 2060 (Uncontrolled) Conditions

Geometry Files:

CC_GRR_WO.g05	Clear Creek Without Project Geometry
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Flow Files:

CC_GRR_WO.f02	FEMA Flows
CC_GRR_WO.f03	Without Project 1980 Conditions Flows
CC_GRR_WO.f04	Without Project 2001 Historical Flows
CC_GRR_WO.f05	Without Project 1994 Historical Flows
CC_GRR_WO.f07	Without Project Storage Routing Flows
CC_GRR_WO.f08	Without Project 2010 Conditions Flows
CC_GRR_WO.f09	Without Project 2060 Conditions Flows
CC_GRR_WO.f10	Without Project 2060 (Uncontrolled) Conditions Flows

CLEAR CREEK GENERAL REEVALUATION REPORT HYDROLOGIC ANALYSIS WITHOUT-PROJECT CONDITIONS

I. INTRODUCTION AND SCOPE

Purpose - This write-up documents hydrologic modeling studies conducted for the Clear Creek General Reevaluation Report (GRR). The write-up describes the development and calibration of a rainfall-runoff model for the Clear Creek watershed and a hydraulic model for the mainstream of Clear Creek. Simulations with the new models to generate flood frequency are also presented. The models and simulations represent without-project conditions, i.e. flood conditions that are expected to occur in the absence of any major flood damage reduction projects. Hydrologic modeling representing a slate of proposed project alternatives will be documented in the future upon completion.

Models - The rainfall-runoff model (HEC-1) and hydraulic model (HEC-RAS) were created using newly acquired data. The HEC-1 model covers the entire watershed of Clear Creek. The HEC-RAS model includes the mainstream of Clear Creek from the Fort Bend County line to the outlet at Galveston Bay. Similar models have been developed in the past for other purposes, but the new models benefit from modern technologies such as Geographic Information Systems software (GIS), Global Positioning System surveying (GPS), and improved modeling software.

Model Simulations – Simulations with the new models were conducted for historical events and also for hypothetical flood frequency events. Two historical floods were simulated including the October 1994 flood and Tropical Storm Allison (June 2001). The results were compared to historical records to verify the accuracy of the new models.

Hypothetical flood frequency events were simulated to develop the stage and flow frequency results needed in the GRR for flood damage computations. The GRR will address flood damages for a 50-year analysis period (2010 to 2060) representing the approximate economic service life of any project that might be proposed. The flood frequency simulations were made for watershed conditions representing both 2010 and 2060 to capture the range of flooding expected over that period.

Flood Sources - Flood damages along Clear Creek and Clear Lake can result from stream flooding events and also from storm tides. The GRR study authority only addresses stream flooding, so the hydrologic analysis is limited to that flood source.

Tributary Modeling - The write-up only documents flood flows for the main stem of Clear Creek. The GRR will consider flood damages for the main stem and also for six major tributaries including Hickory Slough, Marys Creek, Cowart Creek, Chigger Creek, Mud Gully, and Turkey Creek. The original study scope did not include tributaries, so they were not included in the startup surveys. Later, the GRR scope was broadened; so tributary flood frequency results will be required. Flood flows for the tributaries are a byproduct of the

mainstream HEC-1 simulations. The corresponding flood stages will be computed with hydraulic models developed in previous flood insurance studies. Neither the flows nor stages for tributaries are presented at this time.

Related Studies and Models – Computer models for the Clear Creek watershed have been developed and updated several times in the last few decades. Some of the earliest models were created for developing the old Authorized Federal Project, for watershed master planning, and for the first flood insurance mapping studies. These models date back to the early 1980's. Dannenbaum Engineering Corporation (DEC) performed most of the Clear Creek hydrologic modeling studies to date working with funding from agencies including Harris County Flood Control District and the Texas Water Development Board.

The Clear Creek Hydraulic Baseline Report was completed in September 1991. Hydrologic and hydraulic models of Clear Creek and its tributaries were updated from the earlier flood insurance models. The computed flood plain and floodway boundaries were incorporated into the updated FEMA FIRM panels (September 22, 1999).

The Clear Creek Regional Flood Control Plan was completed in December 1992. This report analyzed structural and non-structural flood control alternatives for the projected flood control needs of the Clear Creek watershed. A regional plan was selected that would reduce or eliminate flood damages and accommodate continued watershed development. The plan relied on mainstream flood capacity that was to be provided by the old Authorized Federal Project, which is no longer slated for construction.

In 1997 modeling studies were conducted to identify a Sponsor Proposed Alternative that would substitute for the Authorized Federal Project but with less environmental impacts. A plan was identified and presented at a series of public meetings. The Corps of Engineers decided that the new sponsor plan was not sufficiently similar to the original plan to allow construction under the original project authorization.

More recently, following Tropical Storm Allison in June 2001, FEMA and Harris County decided to modernize the flood insurance maps within Harris County. The new Clear Creek GRR HEC-RAS model was transferred to the FEMA contractor along with all the supporting maps and terrain data. Thus, the new FEMA model will be a derivative of the GRR HEC-RAS model. The FEMA analysis will utilize HEC-HMS for rainfall-runoff modeling instead of HEC-1.

Projections and Datum - Mapping and GIS related data for the GRR were referenced to NAD 83 and the State Plane Coordinate System, South Central Zone. Vertical elevations were referenced to NAVD 88 (2000 epoch) where epoch refers to the date of the surveys.

Watershed Description - The Clear Creek watershed is located in the San Jacinto-Brazos Coastal Basin and lies within the counties of Harris, Galveston, Fort Bend, and Brazoria (see Exhibit I.1). As seen in Exhibit I.2, the watershed includes 16 cities and covers approximately 260 square miles. The Clear Creek watershed is about 45 miles long in an east-west direction and varies in width from 2.5 miles at its upstream end to 13.5 miles at its midpoint. The

watershed is a flat coastal plain with a maximum ground surface elevation of approximately 70 feet and a minimum elevation of about 5 feet at the outlet.

Stream Gages and Records - Stream gages have been operated along Clear Creek since at least 1946. Gage measurements are important to the GRR hydrologic analysis because they provide a historical record of specific flood flows and stages that can be used to calibrate the hydrologic models. Table I.1 lists locations and record periods for each gage along the mainstream of Clear Creek. The table also shows how each gage was utilized in the hydrologic analysis.

**TABLE I.1
STREAM GAGES ALONG CLEAR CREEK**

Gage ID	Location	Operated By	Record Type	Record Period	How Gage Was Used in the GRR Modeling Analysis			
					Annual Peaks Used for Statistical Frequency Analysis	Recorded Hydrographs Used for HEC-1 Historical Event Simulations	Stage Discharge Measurements Used for HEC-RAS Roughness Calibration	Recorded Stage Used for HEC-RAS Historical Event Flood Profile Simulations
08077000 Clear Creek Near Pearland, TX	Downstream side of State Highway 35 at X/S 185547	USGS	Stage and flow	WY1946-1994	X		X	
08077540 Clear Creek At Friendswood, TX	Downstream side of FM 2351 at X/S 112393	USGS	Stage and flow	WY1995-1997		X (Oct 1994)	X	X (Oct 1994)
08077600 Clear Creek Near Friendswood, TX	Downstream side FM528 at X/S 90072	USGS	Stage only Stage and flow	WY1966-1994 WY1998-2001	X	X (TS Allison - June 2001)	X	X (TS Allison - June 2001)
HCOEM Gages	Eight gages along the Harris Co reach	Harris Co. Office of Emergency Manag.	Stage only	Beginning in 1985				X

II. SPECIAL MODELING ASSUMPTIONS AND OVERVIEW

GRR Approach Versus Master Plan Approach - A conventional drainage master plan would target some future development scenario and then determine the size of drainage features, secondary laterals, and primary laterals necessary to accommodate that design condition. The GRR analysis is fundamentally different. Its objective is to identify the most efficient measures that can be taken to reduce flood damage. It does not presume or attempt to accommodate future, locally constructed channel rectifications since that would be forcing a solution that has not been demonstrated. Thus, the new without-project models do not assume a future condition where primary laterals are rectified. If the opposite assumption were made, mainstream flood damage estimates for future condition would be greater due to the higher flows that would result from widened tributary channels. The HEC-1 models were coded to reflect development trends in the watershed. The impervious percentage of each subbasin was increased and the runoff coefficients were adjusted to show conversion to urban drainage systems. However, the models were not coded to reflect future channel rectification of primary laterals.

Second Outlet Modeling - The Clear Lake Second Outlet is a dredged outlet channel connecting Clear Lake to Galveston Bay. It is technically a project feature (a portion of the old Authorized Federal Project). Thus, the Second Outlet is deliberately excluded from the GRR without-project models. It will be modeled later for the with-project modeling phase. Its effectiveness will be measured both as a stand-alone feature dedicated to flood damage reduction for property around Clear Lake and alternatively as a flow mitigation feature as was originally intended. A goal of the GRR will be to determine which function is the most efficient for reducing flood damage.

Detention Basin Modeling - The new models treat detention basins in one of two ways depending on their function. Detention basins built to mitigate development are represented in the HEC-1 model by adjusting subbasin runoff coefficients to show that the full effects of development are at least partially mitigated. Detention basins built to *reduce* the existing flood risk (not merely mitigate development) are intentionally omitted from the GRR modeling. This approach prevents these locally constructed flood damage reduction features from reducing computed benefits for any future Federal project. This strategy is expressly directed by Federal legislation (Water Resources Development Act of 1996, Section 575). The intent is to avoid penalizing local governments for taking action to reduce existing flood damage.

Predicting the Effects of Detention Policies – Many communities within the Clear Creek watershed have ordinances that are designed to prevent new urban development from creating or worsening flood problems. The ordinances require detention basins to detain increased runoff. The current and future effectiveness of these policies is an important factor in predicting flooding along Clear Creek. A method was developed to reflect detention policies in the GRR HEC-1 modeling. This is described in detail in later sections of this write-up.

Overview of Analysis Procedures - Hydrologic models were developed, calibrated, and used to compute flood frequency along Clear Creek. The individual steps are outlined in Table II.1.

TABLE II.1 OUTLINE OF WITHOUT-PROJECT MODELING ANALYSIS

Initial Model Development

- Obtain basic data (channel cross-section surveys, bridge surveys, digital terrain data, digital orthophotos, population projections, rainfall atlas data, historical rainfall data, stream gage records, etc.)
- Develop HEC-RAS and HEC-1 models. Develop the needed HEC-1 model versions (1994 and 2001 for historical flood simulations and 1980, 2010, and 2060 for flood frequency simulations). Use population projections to estimate urban development conditions for the 2010 and 2060 models.

Calibrate Models

- Calibrate HEC-RAS by adjusting hydraulic roughness values until computed stage-discharge agrees with gage measurements.
- Calibrate/verify HEC-1 by simulating historical events (October 1994 and June 2001 floods). Compare simulated flood hydrographs with hydrographs recorded at stream gages. Also, compute water surface profiles with HEC-RAS using simulated flood peaks from HEC-1 and compare with observed high-water marks.

Flood Frequency Simulations

- Simulate flood frequency events and compare resultant flow frequency to values from independent sources (regression equations and statistical analysis of gage records). Adjust assumed HEC-1 rainfall loss rates if necessary so that simulated peaks are judged to be reasonable in comparison to the other estimates. (This verification process was made with the HEC-1 model representing 1980 conditions. The regression estimates were also based on 1980 conditions, and the statistical analysis results roughly approximate 1980 conditions.)
- Simulate HEC-1 flood frequency events for 2010 and 2060 watershed conditions (eight frequencies each). Compute corresponding flood stages with HEC-RAS. This step defines flood frequency for the beginning and ending of the GRR economic analysis period.

Export 2010 and 2060 Flood Frequency to Economics Model

- Determine uncertainty parameters for the HEC-1 flow frequency results.
- Determine uncertainty parameters for the HEC-RAS stage discharge results.
- Export computed flood flows, stages, and uncertainty parameters into the FDA economics models for computation of expected flood damage over the analysis period (2010-2060).

(End of Analysis)

III. HEC-1 WATERSHED MODELING

Watershed and Subbasin Delineation – A watershed delineation was created representing the current (year 2000) watershed and subbasin boundaries for the Clear Creek watershed. This delineation was used to represent all modeling conditions. Boundaries tend to change slightly as development alters natural drainage patterns, but the changes are rarely significant at the mainstream and major tributary level. The delineation was patterned similar to versions used for previous modeling studies of Clear Creek and reflects both surface topography and also subsurface storm drain patterns. The delineation was not computer generated from digital terrain as is possible with GIS computer processing. The watershed and subbasin boundaries used in this study may be found in Exhibit III.1.

HEC-1 Subbasin Unit Hydrograph Method – The Clark unit hydrograph method was used in the Clear Creek HEC-1 model to compute runoff from individual subbasins. Harris County Flood Control District equations were used for estimating the required subbasin runoff coefficients TC and R. The equations are described in Appendix A. The equations utilize the following subbasin characteristics:

- Drainage Area (A)
- Watershed Length (L)
- Watershed Length to Centroid (L_{ca})
- Channel Slope (S)
- Watershed Slope (S_o)
- Percent Land Urbanization (DLU)
- Percent Channel Improvement (DCI)
- Percent Channel Conveyance (DCC)
- Percent Ponding (DPP)

Flood Hydrograph Routing and Routing Steps - Hydrograph routing in the HEC-1 model was performed using the Modified Puls routing method. The required storage-outflow data for each reach were determined with HEC-RAS using multi-pass analysis so that the final storage-outflow routing data and final computed flood profiles were consistent. The number of routing steps for each routing reach was based on reach travel time divided by the HEC-1 computation interval (15-minutes). The storage-outflow routing data derived from the calibrated HEC-RAS model represent current (year 2000) conditions. However, the same routing data were judged to be sufficiently representative for all without-project conditions.

Storage-outflow data for routing reaches along tributaries were borrowed from previous flood insurance models since new HEC-RAS modeling was only available for the mainstream. It should be noted that the storage-routing data for all tributaries including Armand Bayou were not changed for the future condition models i.e., no future channel rectifications were assumed.

Subbasin Rainfall Loss Potential - The initial/constant loss rate methodology was utilized for this study. This method assumes that the first rainfall increments are absorbed or otherwise abstracted up to a certain “initial rainfall loss” value specified in inches. All additional rainfall in

excess of this initial loss will become direct runoff except for a “constant loss” specified in inches per hour.

Loss rates are dependent upon the predominant soil types within the watershed. To determine the soil types within the Clear Creek watershed, Soil Conservation Service (SCS) soil data for Harris, Galveston, Brazoria, and Fort Bend counties were compiled. Exhibit III.2 displays the various SCS hydrologic soil group categories for the Clear Creek watershed. As seen in the exhibit, the predominant hydrologic soil group is Type “D”. This soil group has the highest runoff potential of the four SCS soil groups (A, B, C or D).

Relatively small loss rates were assumed for the HEC-1 flood flow frequency simulations to reflect the impermeable nature of the soils. An initial loss of 0.5 inches and a constant loss rate of 0.1 inches per hour were used. The suitability of these values for flood flow simulations was verified by comparing the HEC-1 results with flood flow frequency from independent methods. Loss rates for historical simulations were set at values that most closely matched runoff volumes recorded at flow gages during the actual events.

HEC-1 Modeling Versions - Versions of the HEC-1 model were developed to represent different watershed conditions. A primary product of the modeling analysis was flood flow frequency for 2010 and 2060. Therefore HEC-1 model versions were needed representing those two conditions. Other versions were needed for calibration purposes. Table III.1 lists all of the HEC-1 model versions created and describes their purpose in the analysis.

Creating HEC-1 Modeling Versions - The HEC-1 model versions (1980, 1994, 2001, 2010, and 2060) were created by changing the subbasin runoff coefficients TC and R (and imperviousness) to represent each watershed condition. The values needed for each condition were computed with subbasin parameters representing each year. The subbasin parameters in the TC and R equations can be divided into two groups as follows:

- Development-Independent Parameters:
 - Drainage Area (A)
 - Watershed Length (L)
 - Watershed Length to Centroid (L_{ca})
 - Channel Slope (S)
 - Watershed Slope (S_o)
- Development-Dependent Parameters:
 - Percent Land Urbanization (DLU)
 - Percent Channel Improvement (DCI)
 - Percent Channel Conveyance (DCC)
 - Percent Ponding (DPP)

The development-independent parameters were set based on aerial photos, previous studies, and survey data. Their values are the same for all model versions (see Appendix B).

The development-dependent parameters were estimated using the methods outlined in Table III.2 and described below. Their values are different for each model version.

TABLE III.1
HEC-1 MODELING VERSIONS
(Without-Project Conditions)

HEC-1 Version (Watershed Conditions Representing):	Purpose in Analysis
1994	Historical simulations to simulate the October 1994 flood event. Comparisons of recorded and simulated hydrographs confirm the accuracy of the HEC-1 modeling. Computed peak flows also used in HEC-RAS to simulate historical flood profile for comparison with historical high water marks.
2001	Historical simulations to simulate the June 2001 (T.S. Allison) flood event. Comparisons of recorded and simulated hydrographs confirm the accuracy of the HEC-1 modeling. Computed peak flows also used in HEC-RAS to simulate historical flood profile for comparison with historical high water marks.
1980	Flood frequency simulations to compute 1980 flow frequency for comparison with independent estimates (statistical frequency analysis and regression equation methods). The comparison reveals the accuracy of the HEC-1 flood frequency modeling and the assumed rainfall loss rate.
2010	Flood frequency simulations to compute flow frequency for flood damage calculations. (Represents the beginning of the economic analysis period.)
2060	Flood frequency simulations to compute flow frequency for flood damage calculations. (Represents the end of the economic analysis period.)
2060 Uncontrolled	Flood frequency simulations to compute flow frequency for 2060 conditions assuming no watershed development detention requirements. These results compared to the other 2060 case show the impact of existing detention policies on flow frequency.

TABLE III.2
DERIVATION OF DEVELOPMENT-DEPENDENT SUBBASIN PARAMETERS
(Without-Project Conditions)

HEC-1 Model Version:	DLU (Percent Land Urbanization)	DCI (Percent Channel Improvement)	DCC (Percent Channel Conveyance)	DPP (Percent Ponding)
1980	Measured from 1980 aerial photos	Measured from 1980 aerial photos	Taken from 1991 Hydraulic Baseline Report	Measured from 1984 aerial photos
1994	Interpolated between 1980 and 2000 measured values	Interpolated between 1980 and 2000 measured values	Interpolated between 1980 and 2010	Interpolated between 1980 and 2000 measured values
2001	Interpolated between 2000 measured values and 2010	Interpolated between 2000 measured values and 2010	Interpolated between 1980 and 2010	Interpolated between 2000 measured values and 2010
2010	Based on census tract population projections for 2010	Interpolated between 2000 and 2060 values based on subbasin DLU	Interpolated between 2000 and 2060 based on subbasin DLU. (DCC for 2000 assumed same as 1980.)	Based on measured 2000 ponding values reduced proportionally by additional development
2060	Based on census tract population projections for 2060	DCI set equal to DLU. Values for subbasins adjacent to mainstream were weighted based on lateral and mainstream length.	Weighted average between the lateral (improved) and the main channel (unimproved). Lateral value is based on DLU table.	Based on measured 2000 ponding values reduced proportionally by additional development

Projecting 2010 and 2060 Urbanization (DLU) Using Population Trends – The first development-dependent subbasin parameter, DLU, was measured for 1980 and 2000. Values for 2010 and 2060 were projected based on population trends using the following steps:

- Map the urban development for year 2000:
Development for year 2000 was mapped (see Exhibit III.3) using aerial photos from that year. Developed area within the watershed was delineating as 100% developed or 50% developed. Areas designated as 100% development include dense commercial/industrial areas, and dense residential subdivisions. Areas designated as 50% developed included residential subdivisions with large lots or light industrial areas.
- Determine year-2000 development acreage for census tracts:
The amount of developed area within each census tract covering the watershed was determined by overlaying the development map onto the census tract map and computing the weighted developed area falling within each tract (see Exhibit III.4).
- Determine year-2000 “population/development area” ratio for census tracts:
Year-2000 population counts were coupled with the development area acreage within each census tract to compute the population/developed area ratio.
- Project development for each census tract for 2010 and 2060:
It was assumed that the “population/developed area” ratio for each census tract would remain constant in the future. Thus, the amount of development for each tract could be projected for 2010 and 2060 conditions based upon census projections for those years.
- Determine 2010 and 2060 development for each subbasin:
The development levels for each subbasin were then obtained by overlaying the census tract map with the subbasin map to compute the weighted development in each subbasin.

Results of Development Projections – The urban development projections provided development percentages for each subbasin (see Appendix C). The development percentages for the entire basin (based on the weighted subbasin values) are shown in Table III.3.

**TABLE III.3
CLEAR CREEK URBAN DEVELOPMENT PERCENTAGES**

Year	Corresponding Clear Creek Watershed Urban Development	Source
1980	25%	Measured from 1980 aerial photos
2000	35%	Measured from 2000 aerial photos
2010	42%	Based on census tract population projections for 2010
2060	69%	Based on census tract population projections for 2060

Determining Remaining Development-Dependent Subbasin Parameters – The remaining development-dependent subbasin parameters were determined as described in Table III.2 and detailed below for each parameter. (See Appendix C for listings of the derived 2010 and 2060 parameters for each subbasin.)

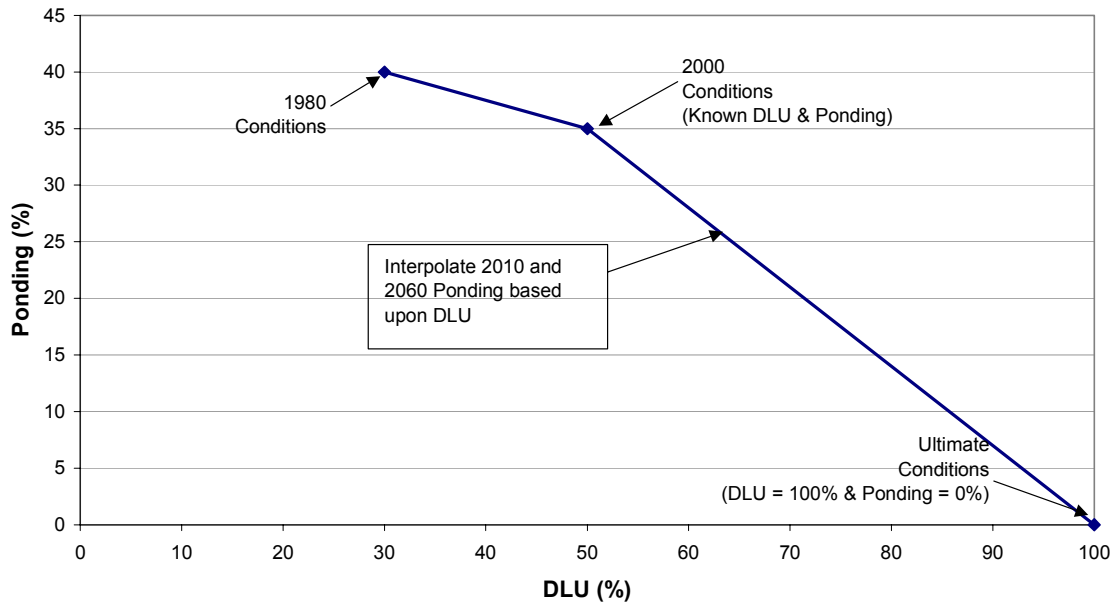
- **Percent Channel Improvement (DCI)** - The main channel of Clear Creek, Armand Bayou, and their major tributaries were assumed to not have future channel improvements beyond current (year 2000) conditions. However, the minor lateral channels of the watershed were assumed to have drainage improvements that pace development. Thus, 2060 DCI values for subbasins on minor lateral channels were set equal to the subbasin’s development percentage (DLU). Subbasins along the mainstream of Clear Creek had their 2060 DCI values predicted by taking a weighted average between the predicted lateral channel improvements (equal to subbasin DLU) and the existing main channel DCI. The future condition DCI values were constrained to never be lower than the 2000 value. Year-2010 values for DCI were interpolated between year-2000 values and 2060 projected values based upon each subbasin’s DLU.
- **Channel Conveyance (DCC)** – Year-2060 channel conveyance was also predicted based upon each subbasin’s urbanization percentage. The conveyance percentage of Clear Creek and its tributaries was known for current conditions and assumed to be static. Year-2060 channel conveyance on minor lateral channels was assumed to increase by the following relationship.

<u>Urbanization %</u>	<u>Conveyance % (Never Less Than Current (2000) Conveyance)</u>
< 18%	Year-1980 Conveyance
18%	35% Conveyance (Approximate 2-year capacity)
25%	37% Conveyance
50%	66% Conveyance (Approximate 10-year capacity)
100%	100% Conveyance

A weighted average between the lateral conveyance and the main channel conveyance was determined for all subbasins. Year-2010 values for DCC were interpolated between year-2000 values and 2060 projected values based upon each subbasin's DLU.

- Ponding Percentage (DPP) - Ponding percentages of each subbasin were reduced according to the amount of development that is projected for future conditions. Since a subbasin that is fully developed (DLU = 100%) will have 0% ponding, and the year-2000 ponding and DLU percentage are known for each subbasin, the 2010 and 2060 ponding percentage can be interpolated between these two points.

Figure III.1
Example of the Ponding vs. Development Relationship



Subbasin Control Factors - Beginning in the 1980's some jurisdictional entities began to require storm water detention for new development projects. Thus, development after that period will have less flow impacts than indicated by the development-dependent subbasin parameters. The parameters discussed in the previous sections are "uncontrolled", meaning they do not reflect the effects of detention ordinances. A method was developed to adjust the uncontrolled parameters to incorporate detention ordinances.

A Control Factor (CF) was derived for each of the four development-dependent subbasin parameters. The Control Factors allow only a fraction of the maximum increase in a parameter to occur. For example, a subbasin with a DCI control factor of 0.2 would allow only 20% of the "uncontrolled" increase in DCI between 1980 and 2060 conditions to occur, as seen below:

**FIGURE III.2
CONTROL FACTOR EXAMPLE**

2000 DCI	2060 "Uncontrolled" DCI	Control Factor	2060 "Controlled" DCI
20%	80%	0.2	32%

$$\begin{aligned}
 \text{2060 "Controlled" DCI} &= ((2060 \text{ "Uncontrolled" DCI} - 2000 \text{ DCI}) \times \text{Control Factor}) + 2000 \text{ DCI} \\
 &= ((80\% - 20\%) \times 0.2) + 20\% \\
 &= 32\%
 \end{aligned}$$

Control factors were applied to each subbasin in the Clear Creek watershed. These factors were estimated by considering the relative stringency of the detention ordinances of each entity within the Clear Creek watershed. This ranking was used to postulate a Control Factor for DLU, DCI, DCC and DPP by entity. Using GIS, weighted Control Factors were determined for each subbasin based upon the amount of each entity falling within the subbasin.

Many residents along Clear Creek feel that runoff control ordinances have been ineffective and that rapid development in the watershed has led to a noticeable increase in flooding. On the other hand, plots of the recorded annual flood peaks for stream gages along the creek do not reveal a pattern of dramatic flow increases. Annual flood peak trends in the most recent historic period are not much different from those earlier in the record. Thus, impacts to date have been relatively moderate so as to be obscured by normal climactic fluctuations. Test modeling runs using the Control Factor approach produced results consistent with this trend, i.e. flows increase with development but runoff controls buffer impacts so that residual effects are moderate.

It should be noted that Control Factors to account for residual impacts are a planning tool and not a measurement of an entity's runoff control criteria effectiveness. Control Factors were based upon conjecture and not upon fact. Modeling tests and experiments were conducted to help determine the Control Factors but the values were mostly based on judgment. Potential limitations for current policies in at least some areas are listed in Table III.4. Control Factors for each subbasin are listed in Appendix C.

TABLE III.4
POTENTIAL LIMITATIONS FOR LOCAL DETENTION POLICIES

- a. Flow increases can occur from land use changes other than residential, commercial, or industrial development. Some examples include conversions from forest to pasture, from rice farming acreage to pasture, or simply grading an area to improve drainage. Some vacant sites may have been graded to remove natural prairie pothole features to avoid possible future wetland regulation issues. Flow increases from these conversions may not be covered by detention policies.
- b. Small developments (like single homes) are frequently exempted from detention requirements. Given sufficient numbers, this could result in an impact. Some jurisdictions are already substantially developed so detention is not required for the few remaining vacant lots.
- c. Some areas of the Clear Creek watershed that were formerly used for rice farming have unusually low runoff rates due to the ponding effects of agricultural levees. As these areas develop, conventional engineering computations for detention requirements may not recognize this and accordingly overestimate the pre-development flow. Thus, the detention requirements are underestimated since a much larger detention basin would be needed to control runoff to the original (unusually low) rates.
- d. The correction of drainage problems in established, developed areas may be considered a maintenance issue rather than new construction. Thus, detention is not required. An example of this would be the emergency deepening and widening of a lateral following a severe flood. This would result in increased flow for downstream areas.
- e. Requirements for some jurisdictions are not codified because it was envisioned that a basin-wide drainage district would be formed to unify and strengthen requirements. Also, in the absence of a basin-wide authority, the administration and interpretation of existing detention policies is vulnerable to local politics.
- f. Parking lots or recreation areas may be accepted for meeting detention requirements. However, the nuisance effects might eventually lead to drainage modifications that would diminish the detention function.
- g. Required detention volume may sometimes be created by “leveeing off“ flood plain area so that, in effect, the detention volume is robbed from the floodplain volume. Also, placement of fill in the floodplain is not regulated in some jurisdictions. These actions deplete floodplain storage volume, which generally increases downstream flows.
- h. Maintenance of private detention facilities (i.e. sediment removal, outlet works repair, etc.) over the long term (decades) may be uncertain. Also, some private facilities may not be protected from future conversion to other uses.

HEC-1 Historical Flood Simulations - A historical event simulation is generated by applying a measured or estimated rainfall pattern for a particular rainfall event to a watershed's hydrologic model. The resultant runoff hydrographs are compared to measured stream gage data and/or compared to measured highwater marks by placing the resultant peak flows in HEC-RAS. This verification process was completed for Clear Creek using the June 2001 and October 1994 events.

Rainfall for Historical Simulations - Nexrain Corporation was contracted to develop rainfall data, in a HEC-1 format, for Clear Creek. Nexrain Corporation developed data for the June 2001 (Tropical Storm Allison) and the October 1994 events. Nexrain based their rainfall upon 15-minute NEXRAD radar rainfall estimates that have a data resolution of approximately 2 km x 2 km. The raw radar rainfall data was calibrated to measured values using rainfall gages from three sources: the Harris County Office of Emergency Management (HCOEM), the City of Houston, and the National Weather Service (NWS). Sixty (60) gages were used to calibrate the 1994 storm event, while one hundred fifty six (156) gages were used to calibrate the 2001 event. Calibration of the raw NEXRAD rainfall data was required to mitigate the effects of ground-based objects and other factors that might skew the results of the raw radar rainfall data.

As a final product, Nexrain developed a basin average rainfall distribution for each of the drainage basins in the Without-Project conditions HEC-1 models for Clear Creek. The October 1994 event data has a time increment of 30 minutes and spans from October 15, 1994 (12:00 AM) to October 19, 1994 (12:00 AM). The June 2001 event data has a time increment of 30 minutes and spans from June 5, 2001 (12:00 AM) to June 10, 2001 (12:00 AM). The rainfall data for the 1994 and 2001 historical events may be found in Appendix D.

Comparison of Nexrain Rainfall with Gage Data - Nexrain's 1994 event rainfall data was compared with other rainfall data collected by the City of Pearland (COP) for this event. Table III.5 compares the COP gages with the Nexrain adjusted basin rainfall data (basin average) that corresponds to each gage. It should be noted that the COP gages were not used in the Nexrain calibration since they are not a uniform increment temporal gages. The COP gage data was collected by hand at irregular intervals, making it unusable in Nexrain's calibration process; however, the overall depth of rainfall collected by these gages should be comparable with the Nexrain values.

As seen in Table III.5, some significant differences exist between the Nexrain rainfall data and the COP data. It is understood that this table is comparing point rainfall data with average basin rainfall depths; however, significant differences are present in enough locations that the accuracy of the Nexrain's 1994 rainfall data should be questioned. The accuracy is not questioned due to methods used by Nexrain, but is questioned due to the limited number of time series rainfall gages available to calibrate the NEXRAD radar rainfall data. This may have contributed to an underprediction of the 1994 event rainfall.

**Table III.5
1994 EVENT RAINFALL COMPARISON**

<i>Gage Data</i>		<i>NEXRAIN Data</i>		Difference (Inches) (2)-(1)	% Difference
Rain Gage ID	Total Rainfall (Inches) (10/17 - 10/18/94) (1)	Associated Drainage Basin	NEXRAIN Basin Total Rainfall (in) (10/15 - 10/18/94) (2)		
COP-1	27.03	A100A2	10.254	-16.776	-62.1%
COP-2	27.09	A100B5	14.808	-12.282	-45.3%
COP-3	22.08	A100C	14.732	-7.348	-33.3%
COP-4	21.02	A100E1	12.381	-8.639	-41.1%
COP-5	21.04	A100E1	12.381	-8.659	-41.2%
COP-6	22.07	A100E2	10.673	-11.397	-51.6%
COP-7	19.07	A100E2	10.673	-8.397	-44.0%
CPWW-1	13.1	A100E1	12.381	-0.719	-5.5%
CPWW-2	13.7	A100E2	10.673	-3.027	-22.1%
HCFCD #140	13.94	A119B	10.629	-3.311	-23.8%
HCFCD #180	10.71	A100D	13.33	2.62	24.5%

As seen in Table III.6, a similar comparison was made for Nexrain's 2001 event data. The 2001 Nexrain rainfall data was compared to HCFCD gages within the Clear Creek watershed. During the June 2001 event at least seventeen (17) temporal rain gages were present within the Clear Creek watershed, as opposed to approximately four (4) for the 1994 event. The increased number of rainfall gages during the 2001 event allowed for a better calibration of the raw radar rainfall data, as supported by Table III.6. Again, it should be noted that Table III.6 is comparing a point rainfall depth (HCFCD gage data) with a basin average depth (Nexrain basin total rainfall).

**Table III.6
2001 EVENT RAINFALL COMPARISON**

<i>Gage Data</i>		<i>NEXRAIN Data</i>		Difference (Inches) (2)-(1)	% Difference
Rain Gage ID	Total Rainfall (Inches) (6/5 - 6/10/01) (1)	Associated Drainage Basin	NEXRAIN Basin Total Rainfall (in) (6/5 - 6/10/01) (2)		
HCFCFCD #100	10.67	A100M	14.095	3.425	32.1%
HCFCFCD #105	19.53	MA100E	18.062	-1.468	-7.5%
HCFCFCD #110	19.21	A100J	17.566	-1.644	-8.6%
HCFCFCD #115	28.66	CW100D	18.415	-10.245	-35.7%
HCFCFCD #120	13.7	A100H	17.26	3.56	26.0%
HCFCFCD #125	19.72	CH100C1	18.706	-1.014	-5.1%
HCFCFCD #130	21.5	A100I	18.92	-2.58	-12.0%
HCFCFCD #140	18.9	A119B	18.849	-0.051	-0.3%
HCFCFCD #150	16.14	A120B	18.904	2.764	17.1%
HCFCFCD #160	17.12	A100E2	18.335	1.215	7.1%
HCFCFCD #180	22.48	A100D	20.978	-1.502	-6.7%
HCFCFCD #190	13.81	A100B4	15.299	1.489	10.8%
HCFCFCD #210	18.19	B100E	17.619	-0.571	-3.1%
HCFCFCD #220	19.65	B111A	19.91	0.26	1.3%
HCFCFCD #230	20.2	B106B	20.628	0.428	2.1%
HCFCFCD #240	19.02	B100A	21.187	2.167	11.4%
HCFCFCD #250	18.66	B104G1	19.402	0.742	4.0%

Historical Event Watershed Parameters – Watershed parameters for the 1994 and 2001 development conditions were created for the 1994 and 2001 historical simulations. Only the development-dependent watershed parameters (DLU, DCI, DCC & DPP) had to be created as outlined in Table III.2. The historical simulation hydrologic models reflect 1% exceedance event R-values.

Historical Event Simulation Verification – To verify the accuracy of the study’s hydrologic and hydraulic models, a calibration was performed. The results from the historical event simulations were compared with measured stream gage data, as well as highwater marks. If necessary, modifications would be made to the study’s hydrologic and hydraulic models until their results adequately matched measured values.

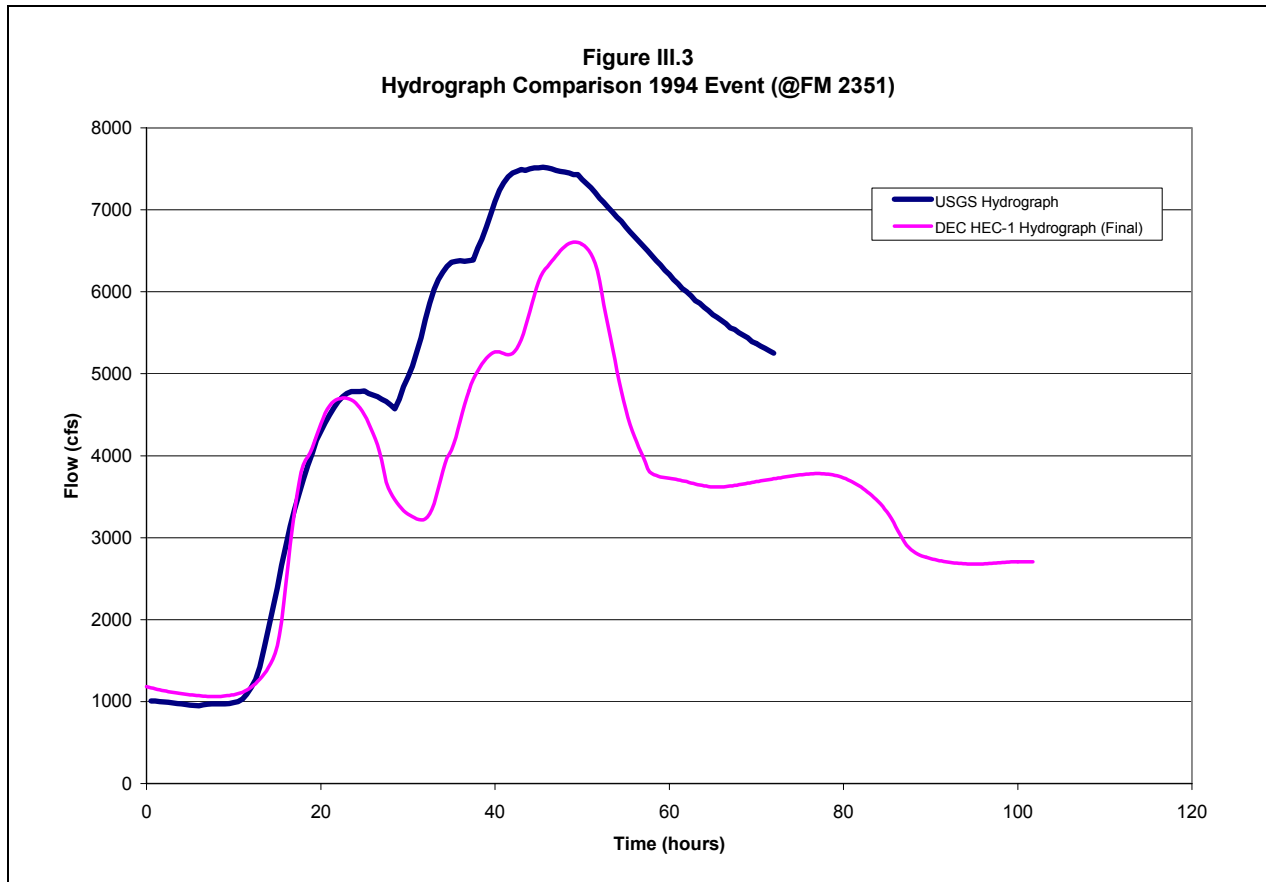
There were two steps in the historical event calibration: loss rate verification and peak flow verification. Loss rate parameters were verified based upon a comparison of the measured hydrograph volume versus the simulated hydrograph volume at a common location. Simulated loss rates would be modified, if necessary, to match measured runoff volume. The peak flow and hydrograph shape of the simulated event was verified against a measured hydrograph

(Stream Gage). This comparison was used to verify the accuracy of the HEC-RAS routing data (n-values).

The 1994 condition watershed parameters and the 1994 event rainfall were used to develop a HEC-1 simulation for the 1994 storm event. The hydrograph from a measured USGS stream gage was compared with the HEC-1 model's simulated hydrograph at the same location. As seen in Table III.7, the simulated 1994 event hydrograph at FM 2351 had approximately 24% less volume than the measured USGS hydrograph (08077540) at the same location. In addition, the peak of the simulated hydrograph (6607 cfs) was approximately 12% lower than the measured hydrograph's peak flow of 7520 cfs. Figure III.3 displays this relationship.

Table III.7
1994 EVENT HYDROGRAPH VOLUME COMPARISON

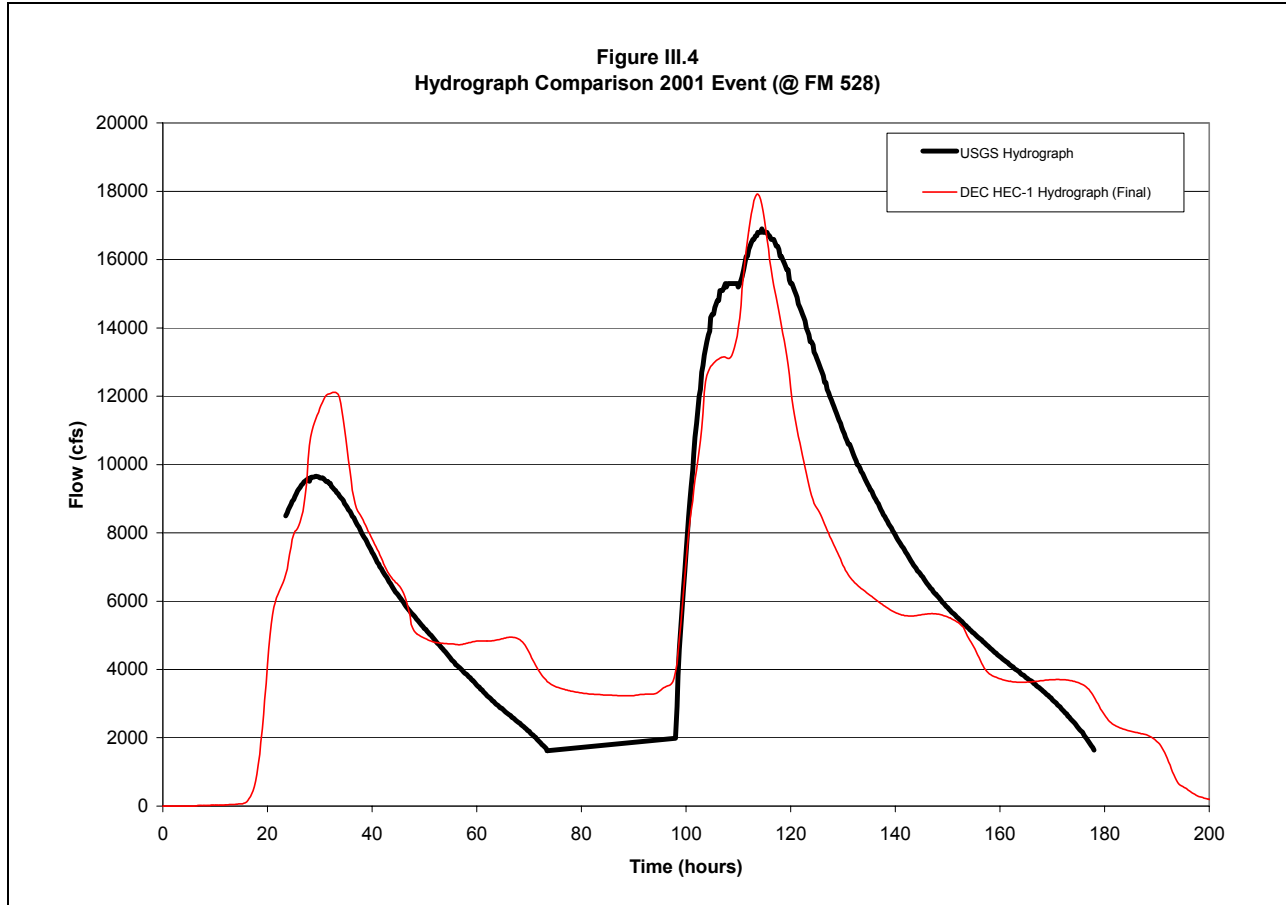
Start Time:	10/17/1994	12:00 AM
End Time:	10/20/1994	12:00 AM
1.	USGS Volume (ac-ft) =	29281
2.	DEC HEC-1 Volume w/ Nexrain Data (ac-ft) =	22184
% Difference (1-2) =		-24.2%



A similar comparison was made at FM 528 for the June 2001 rainfall event. When the simulated runoff hydrograph was compared with the measured USGS hydrograph (08077600) at this location, the two hydrographs had very similar volumes. The simulated hydrograph had a runoff volume that was only 2.4% less than the measured hydrograph at FM 528. The peak flows of the hydrographs also matched well. The simulated hydrograph peak (17781 cfs) was only 5% higher than the measured hydrograph's peak flow of 16900 cfs. The simulated and measured hydrograph may be seen in Figure III.4.

Table III.8
2001 EVENT HYDROGRAPH VOLUME COMPARISON

Start Time:	06/05/2001	11:30 PM
End Time:	06/12/2001	10:00 AM
1.	USGS Volume (ac-ft) =	86764
2.	DEC HEC-1 Volume-Final Model (ac-ft) =	84724
% Difference (1-2) =		-2.4%



Conclusions from Historical Event Simulation Verification – The results of the historical event simulation verification show that the simulated 2001 event matches much better with measured stream gage data than the simulated 1994 event. This discontinuity in the results of the historical simulations is most likely the result of inaccurate rainfall data for the 1994 event. As previously mentioned, only four (4) temporal rainfall gages located within the Clear Creek watershed were available to calibrate the NEXRAD rainfall data for the 1994 event, while at least seventeen (17) temporal gages were available for calibration of 2001 rainfall event.

Based upon the previously mentioned factors, much more weight was placed on results of the June 2001 historical simulation. The results of the June 2001 event historical event simulation verification show that the hydrologic models used in this study accurately represent watershed conditions. No modification to the models is warranted based upon the results of the historical simulation.

HEC-1 Flood Frequency Simulations - Flood flow frequency is generated with HEC-1 by simulating rainfall events associated with specific exceedance frequencies. The rainfall aerial distribution pattern is assumed uniform over the basin and a unique temporal distribution is generated by HEC-1. These synthetic rainstorms are referred to as “hypothetical events.” Peak

flood flows resulting from hypothetical event simulations are assumed to have the same frequency as the applied rainfall event. Thus, the resultant peak flows define flow frequency at each computation node in the model. In reality, rainstorms occur in an infinite array of temporal and aerial patterns and occur coincidentally with a varying range of antecedent moisture conditions in the watershed. Thus, hypothetical event simulations do not capture the true complexity of the real-world flood spectrum, and the "same frequency" assumption is empirical. The process is verified by comparing results with flow frequency estimates from independent methods. This verification process was accomplished for Clear Creek using two independent methods as will be described in a subsequent paragraph.

Rainfall for Flood Frequency Simulations - Eight hypothetical rainfall events were compiled using rainfall frequency publications TP-40 (USWB, 1961) and NWS-35 (NOAA, 1977). The adopted storm duration was 24 hours and the computation interval was set at 15 minutes. Hypothetical storm with longer durations can be simulated, but for moderate sized basins like Clear Creek the 24-hour duration is adequate. Table III.9 shows the rainfall depth values that were coded into the HEC-1 simulations for each frequency.

**TABLE III.9
POINT RAINFALL DEPTHS
FOR HEC-1 FLOOD FREQUENCY SIMULATIONS
FROM TP-40 AND NWS-35**

Percent Chance Exceedance	Depth in inches for rainfall duration of:						
	15-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
50	1.2	2.4	2.9	3.2	3.8	4.4	5.2
20	1.4	2.9	3.7	4.1	5.0	6.0	7.0
10	1.5	3.4	4.3	4.8	5.9	7.2	8.6
4	1.7	3.9	4.9	5.6	6.9	8.5	9.9
2	1.8	4.3	5.6	6.3	7.8	9.6	11.4
1	2.0	4.7	6.2	7.1	8.7	10.8	13.0
0.4	2.2	5.1	6.8	8.2	10.0	12.5	15.0
0.2	2.4	5.4	7.2	9.0	11.0	13.8	16.4

Note:

- o Values for 0.4 and 0.2 percent chance exceedance are extrapolated.
- o The durations shown are those required for the HEC-1 automatic event generation procedure with a computation interval of 15-minutes. The 15-minute values are from NWS-35.

Comparison of TP-40 Rainfall with Newer Sources - The TP-40 rainfall atlas was compiled over 40 years ago. A newer rainfall frequency atlas was recently prepared for Texas by the USGS. Rainfall from the new atlas is shown for comparison purposes in the following table.

**TABLE III.10
USGS POINT RAINFALL DEPTHS
FOR COMPARISON WITH TP-40**

Percent Chance Exceedance	Depth in inches for rainfall duration of:						
	15-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
50	1.1	2.0	2.6	2.7	3.2	3.8	4.5
20	1.4	2.6	3.4	3.7	4.5	5.3	6.3
10	1.5	3.0	3.9	4.3	5.4	6.4	7.7
4	1.7	3.5	4.7	5.3	6.9	8.2	9.7
2	1.9	3.9	5.3	6.1	8.2	9.8	11.6
1	2.0	4.4	6.0	7.1	9.8	11.7	13.2
0.4	2.3	5	7	8.5	12.4	14.7	16.1
0.2	2.5	5.6	7.9	9.8	14.7	17.5	18.4

Note: The USGS depths are based on annual series analysis which results in smaller depths for the 50% chance exceedance in comparison to TP-40.

The depths from the newer atlas are similar to the TP-40 values although depths for specific durations are sometimes larger or smaller. A decision was made to use the TP-40 values for the following reasons:

- The HEC-1 hypothetical event generation process includes an adjustment for storm area size. The HEC-1 internal adjustment factors are those specified in TP-40. The USGS has only published new reduction factors for the 24-hour depth. Consistent adjustments for the other durations are not available.
- Rainfall depths from the newer source for the Clear Creek area show no major differences overall in comparison to the TP-40 values.
- The adopted rainfall depths for the flood frequency simulations are not critical to the process since some rainfall infiltration losses must be assumed. A higher or lower assumed loss would counteract the rainfall depth differences between the two sources. The final computed flood flow frequency is compared with independent methods to insure that the adopted rainfall values and losses, together, result in reasonable flood frequency peaks.

Annual Series Adjustment to Rainfall - TP-40 rainfall values are based on partial series analysis of historic rainfall data considering all high values in the gage record. The desired flow frequency estimates were needed in annual series form, which considers only annual maximum values. The adjustment factors in Table III.11 are applied to convert the rainfall depths into annual series equivalents. No adjustment is needed for frequencies beyond the 10-percent event. The HEC-1 program automatically performs this adjustment in the hypothetical event generation process.

**TABLE III.11
PARTIAL SERIES TO ANNUAL SERIES
RAINFALL ADJUSTMENT FACTORS
FROM TP-40**

Percent Chance Exceedance	Conversion Factor
50	0.88
20	0.96
10	0.99

Depth-Area Adjustment to Rainfall - Rainfall atlas values generally represent depths that can be expected over a small area. When used to represent rain depths over large watershed areas, the depths must be reduced. For a given frequency, the appropriate depth is less for a large area than for a smaller area. The HEC-1 program automatically performs this adjustment in the

hypothetical event simulation process based on the adjustment criteria in TP-40. Area reduction factors are a function of storm area size and duration. The JD record in HEC-1 allows hypothetical event simulations corresponding to a range of area sizes to be executed in parallel. The appropriate flow at each stream node is interpolated from the resulting array of flows based on the contributing basin area at that node. An inherent assumption is that the storm size is equal to the total basin size at each node.

The USGS published area reduction factors for only the 24-hour duration. The following Table compares resultant rainfall from both sources for this duration for 4 locations. As seen in Table III.12, the USGS rainfall criteria actually results in less applied rainfall than TP-40 after the area adjustment factor is applied (when compared for the 24-hour depth for the 1 percent event).

**TABLE III.12
COMPARISON OF AREA ADJUSTED RAINFALL
FOR FOUR LOCATIONS
(24-HOUR DURATION AND 1% EXCEEDANCE EVENT)**

Location:	Basin Area Sqmi	TP-40 Atlas (Used In HEC-1 Simulations)			USGS Atlas (For Comparison Only)			Ratio Tp-40/ Usgs
		DEPTH (In.)	AREA REDUCTION FACTOR	ADJ DEPTH (In.)	DEPTH (In.)	AREA REDUCTION FACTOR	ADJ DEPTH (In.)	
Near headwaters at Fort Bend Co. Line	6.3	13.0	0.99	12.9	13.2	0.90	11.9	1.1
USGS gage 08077000 Clear Cr. near Pearland at SH35	36.0	13.0	0.96	12.5	13.2	0.83	11.0	1.1
USGS gage 08077600 Clear Cr. near Friendswood at FM528	120.2	13.0	0.93	12.1	13.2	0.76	10.0	1.2
Outlet at Galveston Bay	258.5	13.0	0.91	11.8	13.2	0.71	9.4	1.3

HEC-1 Flood Frequency Simulation Results – Tables showing the computed flood frequency results are included in Appendix E. Results are shown for 1980, 2010, 2060, and 2060 uncontrolled. A brief summary of flows is included in Table III.13.

The modeling results predict a measurable flow increase resulting from increased development. For instance, flow increases (from 1980) for the 50 percent exceedance event averaged 6 percent to 2010 and 22 percent to 2060. With no runoff controls the increase to 2060 averaged 41

percent. For the 1 percent exceedance event, the average increases were about half those described for the 50 percent event.

**TABLE III.13
COMPUTED FLOW FREQUENCY SUMMARY
FOR FOUR LOCATIONS**

Location:	Watershed Condition	Peak Flow (CFS) for Percent Chance Exceedance							
		50	20	10	4	2	1	.4	.2
Near headwaters at Fort Bend Co. Line	1980	268	475	656	793	906	1,058	1,247	1,295
	2010	269	477	659	794	907	1,060	1,248	1,296
	2060	285	497	681	818	906	1,058	1,240	1,282
	2060UC	333	554	744	884	976	1,099	1,383	1,617
USGS gage 08077000 Clear Cr. near Pearland at SH35	1980	1,036	1,506	1,909	2,439	3,022	3,681	4,561	5,271
	2010	1,198	1,686	2,224	2,834	3,407	4,008	5,005	5,628
	2060	1,219	1,818	2,360	2,940	3,531	4,087	5,138	5,727
	2060UC	1,326	2010	2,628	3,263	3,842	4,478	5,465	6,298
USGS gage 08077600 Clear Cr. near Friendswood at FM528	1980	5,352	8,347	10,775	12,679	14,365	16,328	18,837	20,503
	2010	5,604	8,570	11,006	12,858	14,554	16,532	18,996	20,629
	2060	6,383	9,632	11,817	13,507	15,350	17,394	19,918	22,053
	2060UC	7,081	10,579	12,802	14,487	16,705	19,106	22,372	24,925
Outlet at Galveston Bay	1980	8,220	13,824	20,576	25,340	31,665	37,546	44,482	49,805
	2010	8,509	14,508	21,317	26,544	32,628	38,313	45,253	50,753
	2060	8,673	17,617	24,523	31,406	36,542	41,978	49,584	54,692
	2060UC	9,671	20,732	27,314	34,199	39,582	45,393	53,493	58,782

Notes: This summary is excerpted from Appendix E.

Computed flows are not directly comparable to other studies due to the unique assumptions of the GRR.

HEC-1 Flood Frequency Verification with Independent Methods - The HEC-1 flood frequency results were compared with values determined with other methods to insure that the modeling results were reasonable. The comparisons were made for 1980 watershed conditions, so HEC-1 simulations were conducted to generate flood flow frequency for that condition. The independent methods used were 1) USGS regression equations and 2) statistical analysis of recorded flood peaks. These are described in the following paragraphs and the resultant comparisons are illustrated in Figures III.5 through III.8. The comparisons were based on 1980 conditions because development patterns for that period are well documented and because that period is more representative of the stream gage records used in the statistical analysis. Values were computed at four specific locations along Clear Creek including: the upstream limits of the study near the Fort Bend County line, at State Highway 35, at FM528, and the mouth of Clear Creek at Galveston Bay.

Flood Frequency Computed with Regression Equations - The USGS publishes regression equations for computing flood flow frequency for streams as a function of independent variables

that characterize the contributing basin. They provide a simple method to estimate flood peaks, but only within certain statistical limits of accuracy. There are two sets of equations that are relevant to the Clear Creek study area. Both methods were utilized so two sets of values were generated. The methods used were as follows:

- WRI 80-17, "Technique for Estimating the Magnitude and Frequency of Floods in the Houston, Texas Metropolitan Area"
- WRIR 96-4307, "Regional Equations for Estimation of Peak-Streamflow Frequency for Natural Basins in Texas"

The latter method is for undeveloped basins, so the resultant flood frequency values must be adjusted for urbanization effects. The adjustment was made using the regression formulas in the following USGS publication.

- Water-Supply Paper 2207, "Flood Characteristics of Urban Watersheds in the United States"

Backup data for the regression computations are included in Appendix F along with tables showing computed flood peaks. Resultant flood frequency is shown graphically on Figures III.5 through III.8 for four locations along Clear Creek.

Flood Frequency Computed with Statistical Analysis - Flood frequency estimates were made using statistical analysis of annual maximum flood peaks at two gage sites along Clear Creek. The gages at State Highway 35 and at FM 528 are the only two gages that have significant records to justify statistical analysis. The gage records are not strictly homogenous since there have been development changes in the watershed that influence the magnitude of flood flows. Theoretically, the recorded flood peaks used in the statistical analysis should represent consistent hydrologic conditions. It is sometimes helpful to use only a segment of the gage record before or after development or to otherwise adjust the record to remove the effects. However, analysis results using segments of the gage records for the Clear Creek gages did not show a clear pattern of development effects. Thus, a decision was made to include the entire, available records for both gages and qualify the results as being only estimates. This was deemed adequate since the results were only needed for comparison purposes. Gage records for the two gages are shown in Appendix F. Resultant flood frequency is shown graphically on Figures III.6 and III.7.

Gage records used in the analysis are listed in Appendix F. Peak flows for the Friendswood gage were estimated from stages for water years 1966 – 94 since only stages were reported for that period. The flows were estimated using the latest available gage rating from the USGS. Ideally historic ratings would have been used, but these were not generally available, and there have been no significant channel modifications affecting this gage.

FIGURE III.5

**FLOW FREQUENCY FOR CLEAR CREEK AT FORT BEND CO. LINE ($DA_{HEC-1} = 6.3$ SQMI)
1980 WATERSHED CONDITIONS**

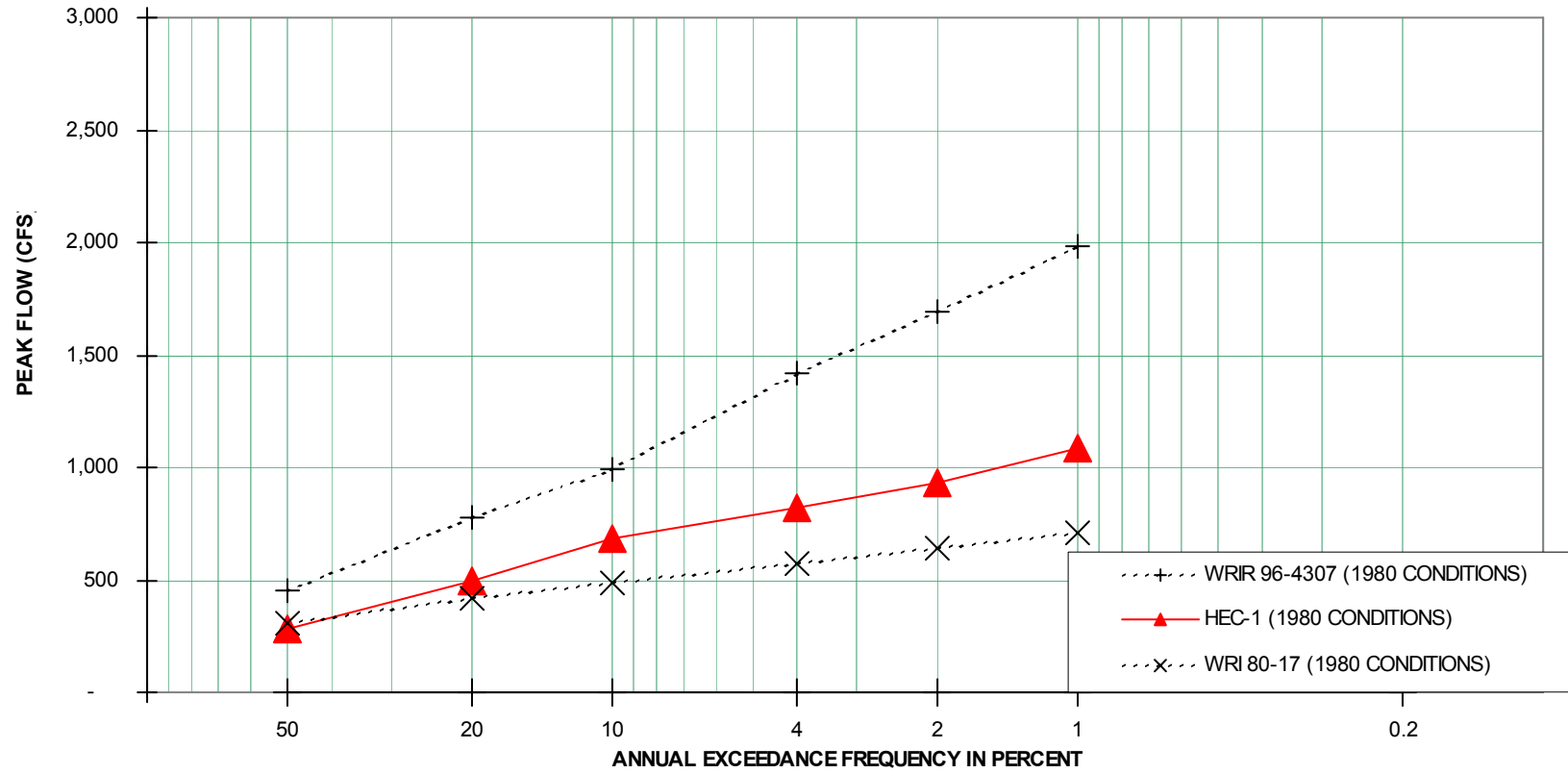


FIGURE III.6

FLOW FREQUENCY FOR CLEAR CREEK AT STATE HW35 ($DA_{HEC-1} = 36.0$ SQMI)
GAGE = CLEAR CREEK NR PEARLAND 08077000
1980 WATERSHED CONDITIONS

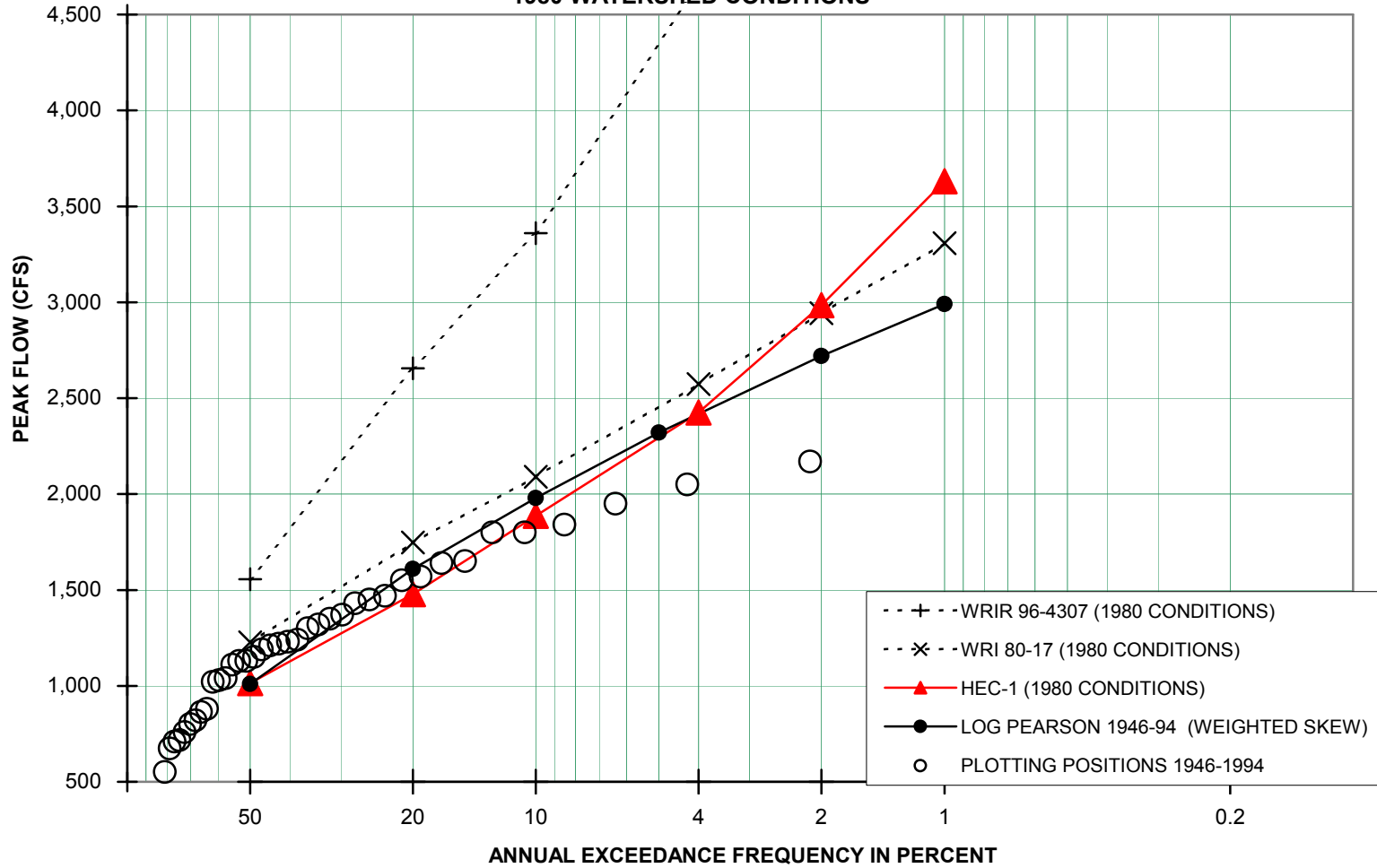


FIGURE III.7

**FLOW FREQUENCY FOR CLEAR CREEK AT FM 528 ($DA_{HEC-1} = 120.2$ SQMI)
GAGE = CLEAR CREEK NR FRIENDWOOD 08077600
1980 WATERSHED CONDITIONS**

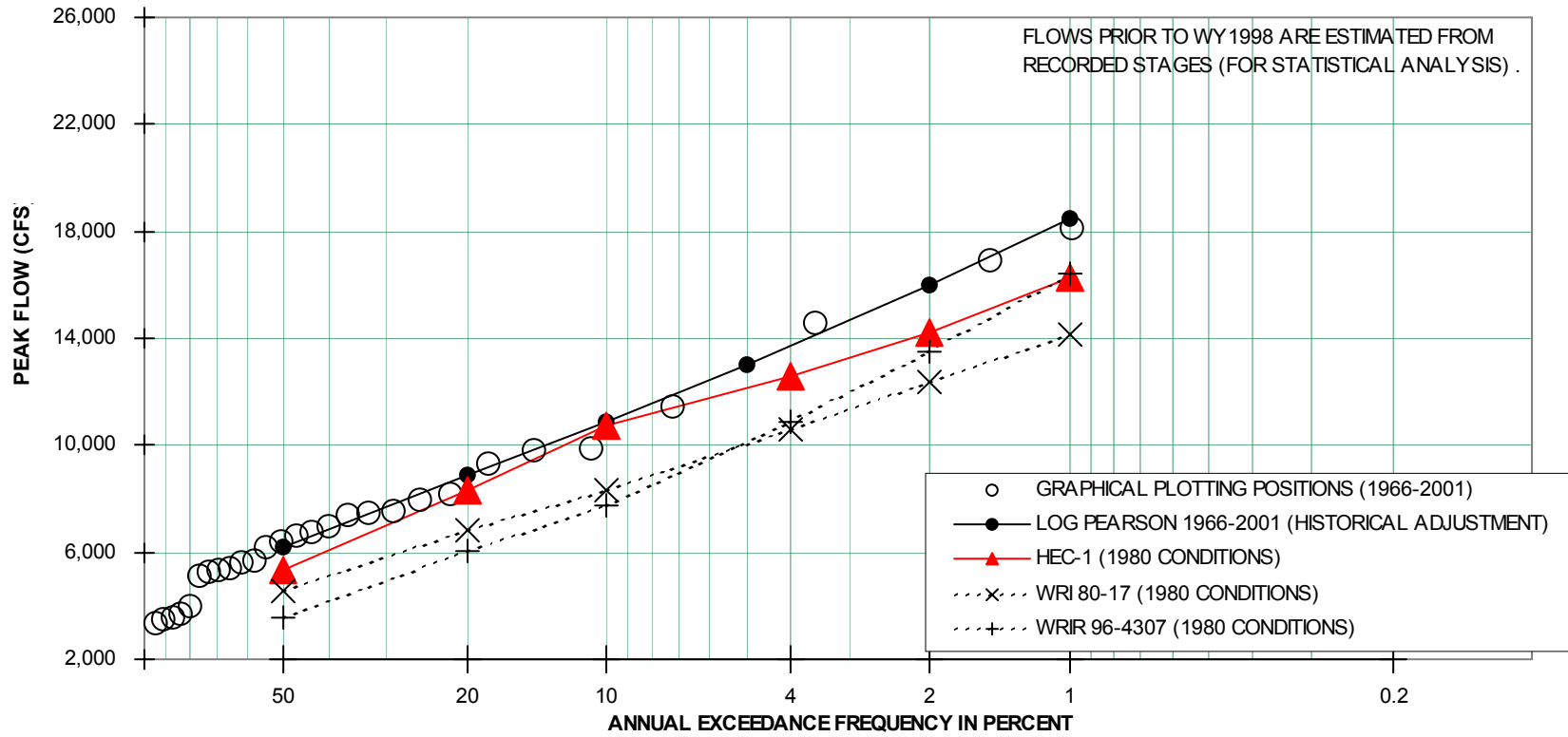
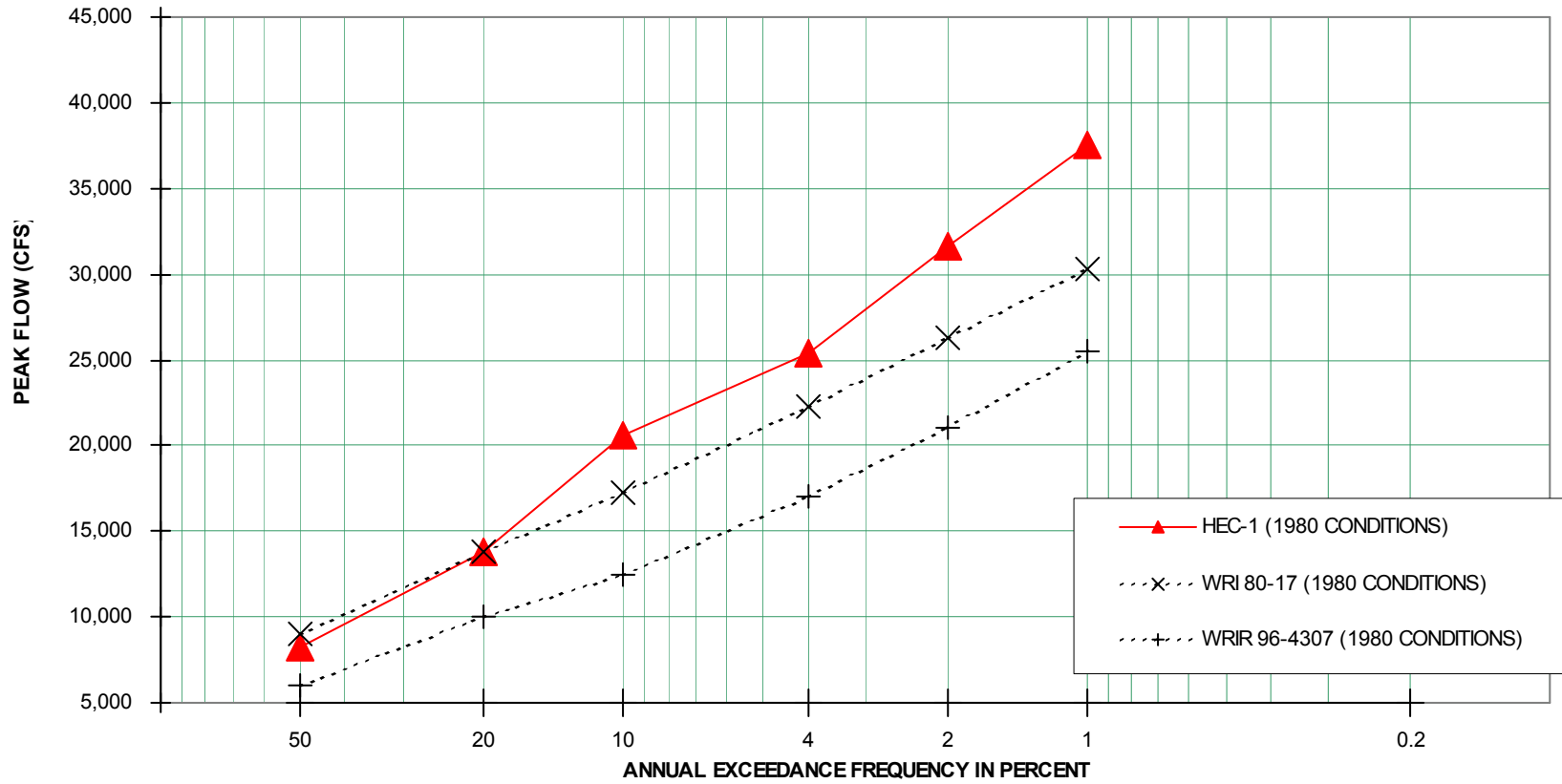


FIGURE III.8

**FLOW FREQUENCY FOR CLEAR CREEK AT GALVESTON BAY OUTLET (DA_{HEC-1} = 258.5 SQMI)
1980 WATERSHED CONDITIONS**



The statistical analysis was conducted using the computer program HEC-FFA which employs analysis procedures recommended in WRC Bulletin 17B. A generalized skew value of 0.06 with a mean squared error of 0.350 was used based on map values presented in USGS WRI 96-4117.

A historic period of 201 years was specified for the largest three floods in the record for the Friendswood gage. There is no specific observer accounts to justify this, however the adjustment was judged to be appropriate. Rainfall for these three events is known to have been extreme (greater than 1% exceedance). Without the adjustment the 16,900 cfs flood peak for Tropical storm Allison would equate to about a 10% exceedance event. With the adjustment the Allison flood computes to be closer to a 2% to 1% exceedance event, which is more consistent with the observed rainfall severity (11.5 inches in 24-hours in the Friendswood area).

Conclusions from HEC-1 Flood Flow Frequency Verification With Independent Methods –

The flow frequency comparisons (Figures III-5 through III.8) provide an independent measure of the accuracy of the HEC-1 flood frequency simulations. All of the methods tend to agree for the more frequent events like the 50 percent exceedance event. The methods diverge for the rare events. This is to be expected since the larger events require a long historical period to become statistically represented in the gage record. The HEC-1 results fit the statistical estimates well and also agree closely with the WRI 80-17 regression method. That regression equation is likely the better of the two used since it was developed specifically for the Houston area and because it considers urban development effects. The other regression method is newer, but it was developed regionally and it requires additional adjustment for urbanization.

IV. HEC-RAS HYDRAULIC MODELING

Data Sources, Projections, and Datum - Topographic data for hydraulic modeling were obtained from several sources as shown in Table IV.1.

**TABLE IV.1
SOURCE OF TOPOGRAPHIC DATA
FOR HYDRAULIC MODELING**

Data Type	Coverage/Description	Spacing/ Resolution (Feet)	Date of Flight/ Survey
Field surveyed channel cross sections	Clear Lake Outlet Reach	700	2000
	Clear Lake Reach	2,000	2000
	Galveston County – Harris County Reach	400	1985 & 2000
	Brazoria County – Harris County Reach	700	2000
Photogrammetric digital terrain data	Entire watershed	100	Feb 2000
	Floodplain area	50	Feb 2000
Digital orthophotos	Entire watershed (monochrome)	1	Feb 2000
	Entire watershed –DOQQ’s (color IR)	3	1995

Horizontal projections were referenced to NAD 83 and the State Plane Coordinate system, South Central Zone. Vertical elevations were referenced to NAVD 88. Cross sections surveyed in 1985 were originally surveyed to a different datum but were converted to NAD 83 / NAVD 88 and also adjusted for subsidence.

The surveyed channel cross sections and the digital terrain data covering the Clear Creek floodplain area were the source of all hydraulic modeling sections. The digital terrain data were prepared by the mapping contractor (Atlantic Technology) to Class 1 1990 ASPRS Standards for a 2-foot contour map. Terrain elevations were generated conventionally with aerial photogrammetry. The elevations of over fifty field-surveyed points were compared to the digital terrain values as an independent quality control test. The resultant root mean square error was 1.16 feet.

Ground control points, bridge data (opening geometry, pier dimensions, low chord and top of roadway elevations), and channel cross sections were obtained by field surveys by John Chance Land Surveyors, Inc. Nineteen survey monuments were established along the creek and are documented in a digital report that show a location map and photo for each monument.

Creating the ARC-VIEW TIN - The GIS software ARC-VIEW was used for creating the HEC-RAS model using a software extension known as GEO-RAS developed by the Corps' Hydrologic Engineering Center (HEC). GEO-RAS enables HEC-RAS cross sections, reach lengths, and roughness values to be extracted from GIS data. Cross sections were extracted from a TIN, which is a digital representation of terrain. The TIN was created within ARC-VIEW from Microstation files provided by the mapping contractor containing mass point and break line layers.

Cross Sections - The cross section alignment layout was created by drawing sections in ARC-VIEW. Sections were drawn left to right looking downstream and extended the full width of the TIN, which covered a preliminary floodplain plus a 2,000-foot buffer. Sections were generally drawn through each surveyed channel section so that the Clear Creek channel detail would be captured accurately. A total of 357 sections were included in the final layout. Exhibit IV.1 shows the cross section layout theme from ARC-VIEW.

Filtering Cross Section Coordinates - After the initial creation of the HEC-RAS file, it was necessary to reduce the number of coordinates at each cross section. The GEO-RAS creation process generally resulted in sections exceeding the 500-point limit. A filter tool in HEC-RAS eliminates unnecessary coordinates from the section according to user specified tolerances. Filtering was minimized so that the section would maintain a high density of coordinates. The filtering tool in version 3.2 of HEC-RAS did not specifically preserve roughness boundaries, therefore it was desirable to maintain as many coordinates as possible so that roughness boundaries (described below) were not distorted excessively.

Bridge Crossings - There are nineteen bridge crossings coded in the HEC-RAS model as listed in Table IV.2.

**TABLE IV.2
BRIDGES CODED IN THE CLEAR CREEK
HEC-RAS MODEL**

Bridge Crossing	Downstream Cross Section	Comment
Almeda School Rd.	234420.7	
SH288	223445.1	
Cullen Blvd.	211227.4	
Stone Rd.	205888.3	Timber bridge
Mykawa Rd.	189432.4	
AT&SF RR	189373.4	
SH35	185547.5	08077000 Clear Creek nr Pearland gage (WY1946 -1994)
Bennie Kate	170703.4	Timber bridge
Country Club Dr.	160052.5	
Dixie Farm Rd	143346.3	
FM2351	112393.5	08077540 Clear Creek at Friendswood gage (WY1995-1997)
Whispering Pines	95406.35	
FM528	90072.02	08077600 Clear Creek nr Friendswood gage (WY1966-94, 1998-2000)
W Bay Area Blvd.	73892.70	
Interstate 45	55615.42	
SH3	46279.31	
MKT-RR	46214.15	
FM270	37212.22	
SH146	3054.182	

Manning’s Overbank Roughness Values - Hydraulic roughness values (Manning’s n) were based on field observations of channel and overbank vegetation cover and also based on aerial photography. A special GIS map was created to define roughness patterns (Exhibit IV.2). GEO-RAS captures roughness boundary stations at each cross section and imports the corresponding values and boundaries into the HEC-RAS file during the creation process. Three main overbank roughness classes were delineated as shown in Table IV.3. For the reach from section 54018.04 near Interstate 45 to section 152591.1 upstream of Dixie Farm Road, Overbank roughness values were coded to vary with stage. Overbank roughness values were all coded with two decimals and channel and oxbow values were coded with three decimals so that they could be distinguished in the model more easily.

**TABLE IV.3
MANNING’S “n” ROUGHNESS**

Overbank Class	Initial Roughness	Roughness from Calibration
Dense vegetation	0.12	0.12 - 0.24
Dense urban development	0.15	0.15
Sparse vegetation or development	0.07	0.07 – 0.10

Manning's Roughness for Channel Areas - Channel roughness classes were also delineated on the GIS roughness map so that the appropriate values would be transferred to the HEC-RAS model. The channel polygons were delineated to capture just the immediate low bank areas that are either submerged with water or barren of vegetation due to frequent submergence. These areas were the easiest to distinguish on the digital orthophotos. Roughness values assigned to these channel polygons varied from 0.025 to 0.041. All oxbow cutoff polygons were coded with 0.042 roughness values.

Calibration of Roughness Values - Figures IV.1 through IV.3 show comparisons of gage rating curves with HEC-RAS computed values. The model calibrates closely to the gage data, but it was necessary to vary overbank roughness values with stage to match the gage data over the full range of flows. The gage measurements and gage ratings were adjusted to the GRR datum (NAVD88, 2000 epoch). The required datum adjustment was determined by running an instrument level from the gage reference monument to monuments established for the GRR surveys. Datum adjustments are shown in the following table.

**TABLE IV.4
DATUM ADJUSTMENTS FOR USGS GAGE DATA**

CLEAR CREEK NR FRIENDSWOOD (08077600)

Location: Downstream side of bridge at FM528 (HEC-RAS Section: 90072)
 Distance to gage datum below BM "CCDD GPS No 30": 25.245 Feet
 Elevation of CCDD GPS No 30 NAVD88 (2000 epoch): 25.130 Feet
 Gage datum NAVD88 (2000 epoch): **-0.115** Feet

CLEAR CREEK AT FRIENDSWOOD (08077540)

Location: Downstream side of bridge at FM2351 (HEC-RAS Section: 112393)

	<Oct 1996	>Oct 1996
Distance to gage datum below BM "CCDD GPS No 3":	25.937	29.037 Feet
Elevation of CCDD GPS No 3 NAVD88 (2000 epoch):	28.97	28.97 Feet
Gage datum NAVD88 (2000 epoch):	3.033	-0.067 Feet

CLEAR CREEK NR PEARLAND (08077000)

Location: Downstream side State Highway 35 (HEC-RAS Section: 185547)
 Distance to gage datum below benchmark "RM4" 19.44 Feet
 Elevation of RM4 NAVD88 (2000 epoch): 44.29 Feet
 Gage datum NAVD88 (2000 epoch): **24.85** Feet

FIGURE IV.1

**Stage versus Discharge
Clear Creek Near Pearland 08077000
HEC-RAS Results Compared to Gage Data**

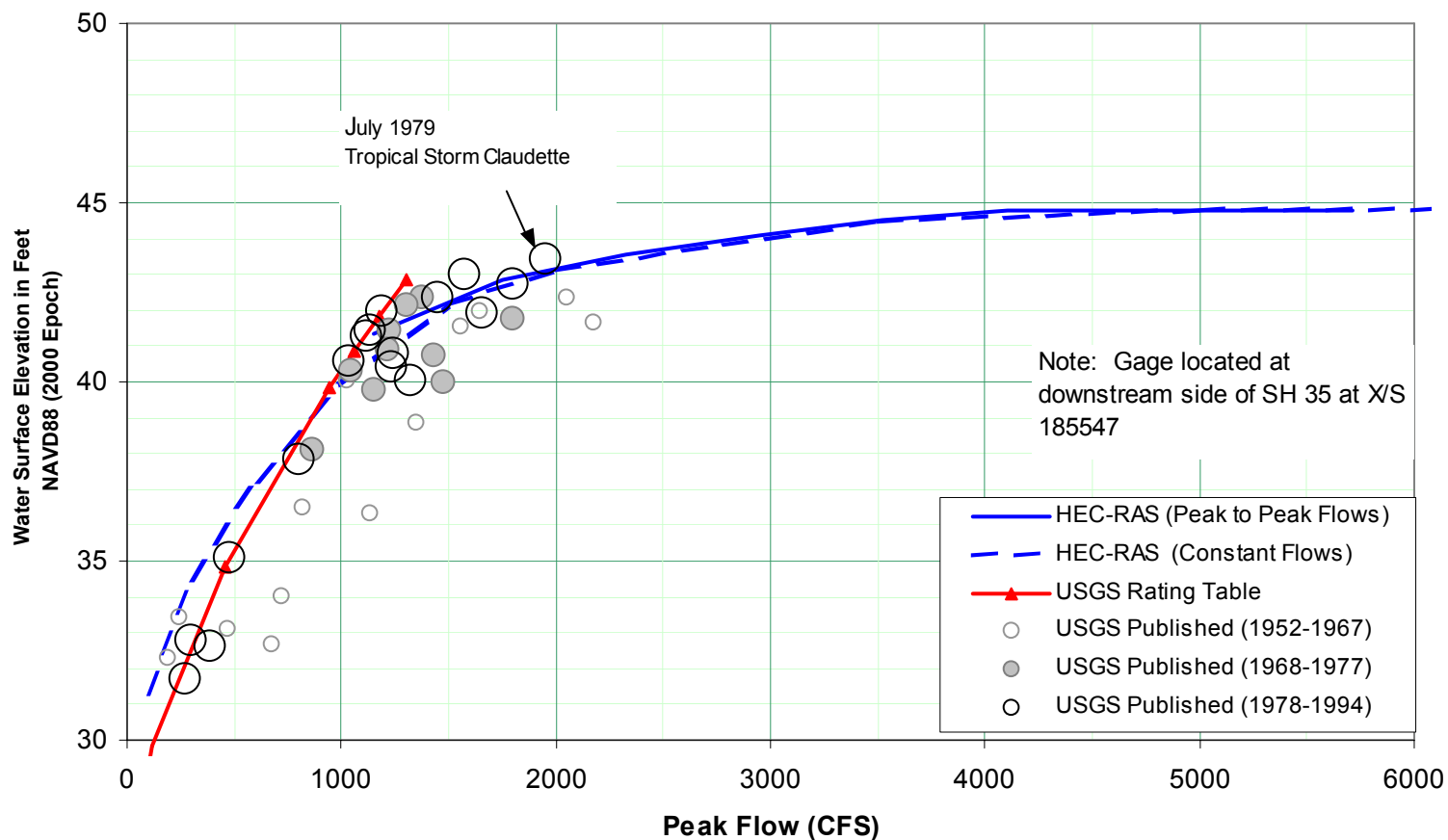
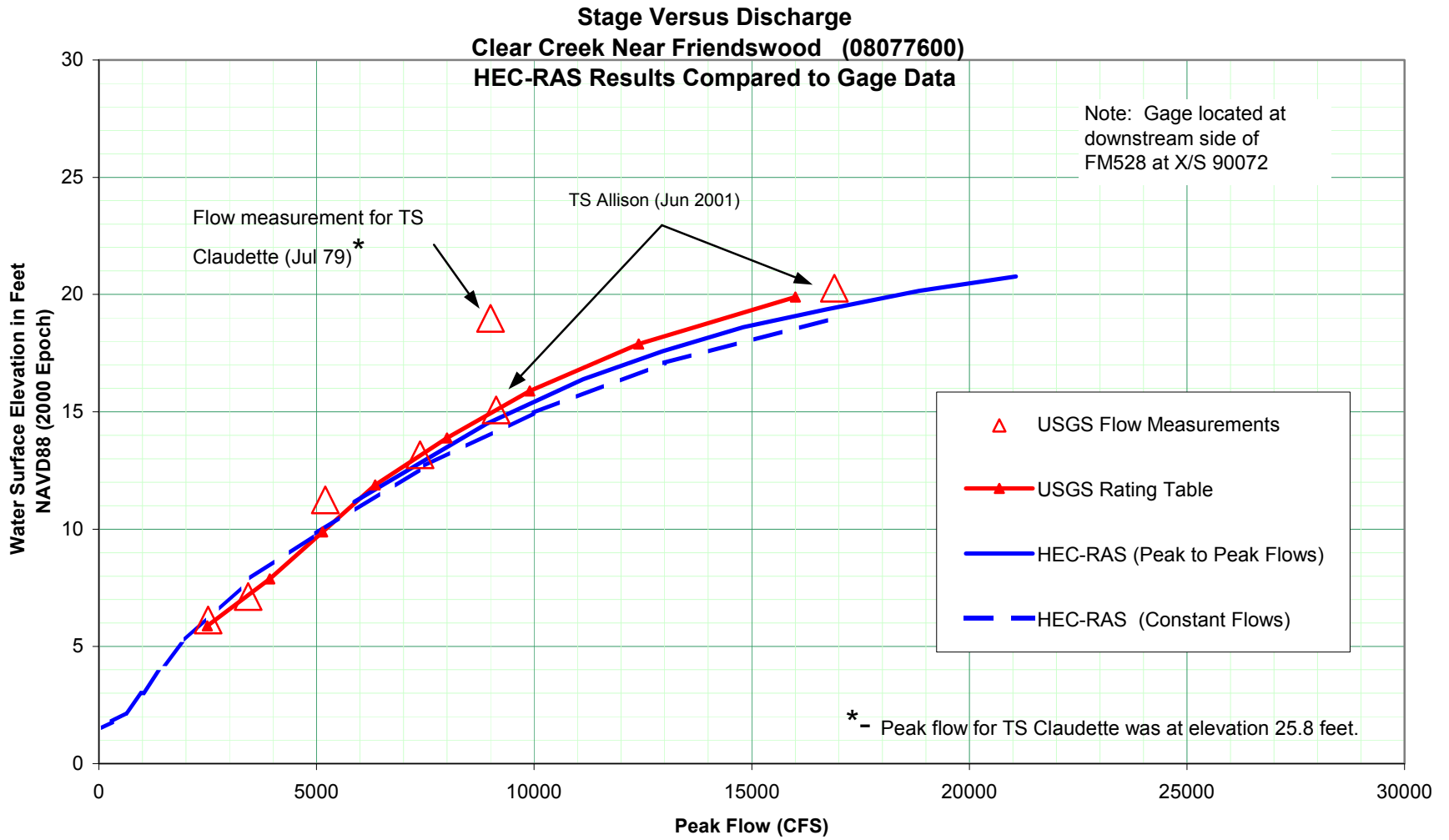


FIGURE IV.2



FIGURE IV.3



Starting Conditions – The flood frequency profiles for all conditions were computed assuming a starting water surface elevation of +1.45 feet at Galveston Bay. Conditions for real flood events depend on astronomical tides and wind or storm induced variations, all of which vary through the duration of a runoff event. Coincident probability studies show that a +1.45 feet starting condition yields accurate stage-frequency estimates in Clear Lake for rainfall-runoff events with nominal coincident tide conditions.

Insertion of HEC-1 Flows – Exhibit IV.3 shows the flood flow frequency flow values in profile view along Clear Creek. The cross sections from the HEC-RAS steady flow data file are shown along with tributary locations. This exhibit is helpful in illustrating how flows change along the creek and the flow increments associated with each tributary.

HEC-RAS Historical Floods Simulations - Flood profiles were simulated for the October 1994 flood and the June 2001 (T.S. Allison) flood using peak flows resulting from the HEC-1 historical flood simulations. The computed flood profiles are compared to high watermarks and gage data in Exhibits IV.4 and IV.5. High watermarks for the T.S. Allison flood are documented in Table IV.5.

HEC-RAS Flood Flow Frequency Simulations – Exhibit IV.6 shows a comparison of 2010, 2060, and 2060 uncontrolled flood profiles for the 1 percent chance exceedance event. Exhibit IV.7 shows a digitally plotted floodplain for the 2010 flood (1 percent chance exceedance).

Development Impacts on Hydraulics - Residential and commercial development along Clear Creek tends to restrict flood capacity as property on the fringe of the floodplain is raised with fill for new construction. Federal flood insurance regulations generally permit this process allowing up to a one-foot increase in the one-percent exceedance flood elevation. Fill in the floodplain also tends to increase peak flow rates by reducing the buffering effects of floodplain storage volume. Some communities have more strict regulations, which reduce these impacts.

The modeling for the GRR takes into effect the hydrologic impacts of future development but does not include the hydraulic impacts described in the preceding paragraph. These hydraulic impacts are realized mostly for rare events and flood damages are compounded more heavily by frequent events. Still, there are some future condition flood damage increases which have not been captured in the analysis.

**TABLE IV.5
HIGH WATER MARKS FOR TROPICAL STORM ALLISON (JUNE 2001)**

RAS Sec	Location	Elev (ft)		Comments	Marks from Dannenbaum		
		NAVD88 2000epoch			Location	Elev	Comment
		Jun 6	Jun 9				
53599	Private road d/s of I-45 on south side		10.5	Surveyed by HCFCD 10.88 /78adj CC04 9.97/78adj 9.55 NAVD88/2000 Correction= 9.97-9.55=0.42	14000	4.76	
					30139	5.74	
					42050	8.21	League City
					57171	10.02	League City
					65855	13.71	League City
55615	North bank/Downstream	8.7	10.8	Surveyed by HCFCD 11.24 /78adj 16.5" below low chord *elev. 9.93ft	89314	19.60	League City
					95406	22.40	Friendswood
					100234	23.20	Friendswood
					101082	21.15	Friendswood
					107933	25.02	Friendswood
55861	Midspan/Upstream (HCFCD Gage #109)	7.78	11.1	Orange paint on side of embankment HCFCD readings were 8.2 and 11.5	112796	25.70	Friendswood
					143346	32.98	Friendswood
					150467	33.88	Friendswood
					151242	34.34	Pearland
					151941	34.86	Pearland
73998	South bank/Upstream		16.2	7" above low chord *elev.15.6 ft*	153192	35.78	Pearland
					158159	37.37	Houston
					160052	38.37	Houston
					167361	40.59	Pearland
					178408	42.35	Pearland
87139	Residence - Leisure		19.9	Approx. 36" above slab.	185547	44.96	Houston
					189373	44.88	Houston
					192943	45.90	Brookside Village
					197996	47.45	Brookside Village
					199968	50.27	Brookside Village
90082	Residence - Royal Parkway		21.8	12" above floor. Floor=20.8'	211227	51.89	Brookside Village
					223445	54.69	Pearland
					95406		
					99945		
					101605		
103109	Residence - Clearview *Friendswood*		21.9				
104949	Residence - Whittier Oaks Residence - Pennystone Ct.		24.3	6" above floor. Equal to threshold.			
108589	Off 2351 Residence - Wandering Trail Residence - Wandering Trail Residence - Wandering Trail		23.4	Approx. 12" above floor.			
110331	Imperial Estates Residence - Imperial Dr.	22.0	25.2	On 6/6/01: 9" above floor, On 6/9/01 48" above floor			
110479	Residence - Imperial Drive		25.1	27" above floor			
112394	FM2351 (1776 Memorial Park) South bank-Downstream North bank/Downstream	22.1	25.1	24" above low chord *elev. 23.1 ft* 12" below low chord *elev. 23.1ft*			
112703	Residence - Cherry Tree Ln. Dixie Farm Rd. South bank/Downstream	32.0	31.3	6/6/01: 6" over oxbow 6/9/01: 9" below low chord *elev. 32 ft*			
143346	Sleepy Hollow Residence - Rip Van Winkle		34.2	12" above floor.			
152375	Green Tee Terrace Residence - Green Tee Residence - Country Club Rd.	36.8	38.5	Lift Plant, 6/6/01: water was 12" over oxbow			
159022	Residence - Robinson Dr. SH 35 (Pearland) South side/Downstream	43.0	42.9	6/6/01:14"below low chord *elev. 44.2 ft* 6/9/01:16" below low chord *elev. 44.2 ft*			
160053	South side/Upstream	43.0	43.0	6/6/01:15" below low chord *elev. 44.2 ft* 6/9/01:14" below low chord *elev. 44.2 ft* Commercial Businesses on southside/downstream:flooded on 6/6/01			
183604	Mykawa South bank/Upstream	44.9	45.8	6/6/01:18" below low chord *elev. 46.4 ft* 6/9/01:7" below low chord *elev.46.4 ft*			
185548	Stone Rd.	47.6		6" below low chord *elev. 48.1 ft*			
185606	Cullen Blvd. (FM 865)	49.3		21" below low chord *elev. 51 ft*			
189526	SH 288	53.4		peak equal to low chord *elev. 53.4 ft*			
205888	Old Airline Rd.(Almeda School)	56.9	59.5	1st pk 18" below low chord *elev. 58.4 ft* 2nd pk reported by dd4 to be 18" over lowspot on road 58.0+1.5=59.5			

V. RISK AND UNCERTAINTY PARAMETERS

Export of Flood Frequency Results for Flood Damage Computations - The HEC-RAS flood frequency profiles for 2010 and 2060 were exported to the Flood Damage Analysis (FDA) program for computation of flood damages. FDA extracts both flow frequency and stage discharge data from the HEC-RAS export file. The export file is a standard table option on the summary profile table menu in HEC-RAS. The table must have a WSP extension and the cross sections in the table must appear in order starting from the downstream end. Additional keyboard input is required to define the error functions for each of these data sets so that damages can be computed using “risk and uncertainty” methods. The following paragraphs describe the derivation of the error functions.

Derivation of Discharge Uncertainty - The uncertainty of flow frequency results can be derived using two approaches. When the flow frequency values are thought to fit a log Pearson III distribution, the uncertainty can be derived analytically from the mean, standard deviation, skew, and representative record length. Conversely, the order statistics approach is preferred for deriving uncertainty when the log Pearson distribution is not applicable. The Clear Creek flow frequency values are influenced by development, so the order statistics method was adopted. FDA performs the derivations, but an equivalent record length is required. Equivalent record length was selected using guidance from Table 4-5 of EM 1110-2-1619. A value of 30 years was selected for 2010 conditions since the flow data were estimated with rainfall-runoff modeling calibrated at short-record gages within the watershed. A shorter length of 25 years was used for 2060 conditions since development projections and detention policy effectiveness introduce additional uncertainty for the distant future.

Derivation of Stage Uncertainty - The uncertainty of computed flood stages can be attributed to the natural variability of the stream and to hydraulic modeling inaccuracies. Guidance is provided in EM 1110-2-1619 for estimating and combining both components.

Natural variations include such factors as seasonal vegetation changes, debris constrictions, and unsteady flow effects. Equation 5-5 from EM 1110-2-1619 was used to compute the standard deviation of stage uncertainty due these natural effects. Values were computed for several reaches along the creek with results ranging from 0.3 to 0.5 feet as shown in Table V.1. Figure 5-3 of the EM was used to estimate upper bounds. Upper bound values and adopted values for natural variations are also shown in Table V.1.

Hydraulic modeling inaccuracies include errors in estimating roughness values, errors in cross section topography, and errors in defining effective flow area. Minimum values were estimated from Table 5-2 of the EM. The cross sections for the Clear Creek hydraulic model were based on field surveys for the mainstream channel and on digital terrain data (equivalent to a 2-foot contour map) for the overbank portions. Manning’s reliability were judged to be good since both stream gages and high-water marks were used to set roughness values (as described in other sections of this report). As an additional measure of modeling uncertainty, a series of tests were conducted to determine the sensitivity of the model to the roughness coefficient, Manning’s n. The adopted roughness values were multiplied by 1.25 and by 0.75 and the resultant profile differences were tabulated. Taking the stage difference between the upper and lower roughness

values to be reasonable bounds, the standard deviation was then estimated as the difference divided by 4. Table V.2 shows the resultant modeling uncertainty values and the adopted values.

Combined stage uncertainty was determined by combining the natural variability and the modeling uncertainty into one value using equation 5-6 from the EM. Final values ranged from 1.0 to 1.1 feet so for simplification a standard deviation value of 1 foot was used for the entire study reach as shown on Table V.3.

**TABLE V.1
STAGE UNCERTAINTY DUE TO NATURAL VARIATIONS**

Computed with Equation 5-5 EM1110-2-1619

Location:	I bed	A basin (mi ²)	H range (ft)	Q _{1%} (cfs)	S _{natural} (ft)
Outlet at Galveston Bay	4	258.5	5	37,884	0.3
FM 528	4	120.2	20	16,172	0.5
SH 35	4	36.0	16	3,568	0.5
Almeda School Road	4	6.3	9	1,082	0.4

Upper Bound From Figure 5-3 EM1110-2-1619

Location:	Stream Slope (ft/ft)	Upper Bound S _{natural} (ft)
Outlet at Galveston Bay	0.00018	2.3
FM 528	0.00024	2.2
SH 35	0.00022	2.1
Almeda School Road	0.00034	2.0

Adopted Values (Natural Variation)

Location:	Adopted S _{natural} (ft)
Outlet at Galveston Bay	0.3
FM 528	0.5
SH 35	0.5
Almeda School Road	0.4

**TABLE V.2
STAGE UNCERTAINTY
DUE TO MODELING LIMITATIONS (TABLE 5-2 EM1110-2-1619)
AND FROM ROUGHNESS SENSITIVITY TESTING**

Location:	Model Limitations From EM	Roughness Sensitivity From HEC-RAS Testing		Adopted S_{model} (ft)
	S_{model} Min (ft)	Prof Diff (ft)	S_{rough} (ft)	
Outlet at Galveston Bay	0.5	1.7	0.4	0.5
FM 528	0.4	3.0	0.8	0.5
SH 35	0.5	1.5	0.4	0.5
Alameda School Road	0.5	0.5	0.1	0.4

Notes:

1. Prof Diff is the HEC-RAS profile difference that results when Manning's n multiplied by 1.25 and 0.75
2. S_{rough} is profile difference divided by 4.

**TABLE V.3
STAGE UNCERTAINTY
COMBINED TOTAL
FROM EQUATION 5-6 EM1110-2-1619**

Location:	$S_{natural}$ (ft)	S_{model} (ft)	S_{total} (ft)
Outlet at Galveston Bay	0.3	0.5	1.0
FM 528	0.5	0.5	1.1
SH 35	0.5	0.5	1.1
Alameda School Road	0.4	0.4	1.0

Notes:

1. An S_{total} of 1.0 foot was adopted for the entire study reach to simplify input.

VI. CONCLUSIONS

HEC-RAS and HEC-1 models representing Clear Creek and the Clear Creek watershed were developed, calibrated, and used to compute flood frequency for the Clear Creek GRR. The models were verified with available data in the form of gage rating curves, observed historical hydrographs, and observed high water marks. The flood flow frequency results were compared with flow frequency estimates from statistical analysis and also regression equation methods. In all cases the model results were judged to be reasonable.

VII. LIST OF REFERENCES

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