COWART CREEK WATERSHED MASTERPLAN

TABLE OF CONTENTS

1.0 INTRODUCTION 1
1.1 Background 1
1.2 Project Scope 1
1.2.1 Previous Studies 2
1.3 Datum Adjustments and Projections 2

2.0 HYDROLOGIC METHODOLOGY 3
2.1 Rainfall Hyetograph 3
2.2 Loss Rates 5
2.3 Sub-watershed Parameters 5
2.3.1 Drainage Area 6
2.3.2 Watershed Length 6
2.3.3 Watershed Length to Centroid 7
2.3.4 Channel Slope 7
2.3.5 Watershed Slope 7
2.3.6 Percent Land Urbanization 8
2.3.7 Percent DLU Affected by On-Site Detention 8
2.3.8 Percent Land Urbanization (Minimum) 9
2.3.9 Percent Land Urbanization (Detention) 10
2.3.10 Percent Channel Improvement 10
2.3.11 Percent Channel Conveyance 10
2.3.12 Percent Ponding 15
2.4 Unit Hydrograph Parameters 16
2.5 Stream Reach Routing 17

3.0 EXISTING CONDITIONS 19
3.1 Hydrology 19
3.1.1 Model Parameters 19
3.1.2 Results 23
3.2 Hydraulics 23
3.2.1 Model Parameters 23
3.2.2 Results 29
3.3 Conclusions 30

4.0 PROPOSED CONDITIONS 32
4.1 Flood Improvement Components 32
4.1.1 Bailey Road Drainage Improvements 32
4.1.2 CW100-00-00 Improvements 33
4.1.3 CW101-00-00 Improvements 34
4.1.4 Channel Improvement (CW103-00-00) 35
4.1.5 Channel Improvement (CW103-01-00) 36
4.1.6 Channel Improvement (CW105-00-00) 36
4.2 Hydrology 36
4.2.1 Model Parameters 37
4.2.2 Results 37
4.3 Hydraulics 38
4.3.1 Model Development 38
4.3.2 Results 38
4.4 Benefit-Cost Analysis 39
4.5 Conclusions 40

5.0 REFERENCES 42

TABLE OF CONTENTS (CONT'D)

TABLES

1.1 Datum Adjustments

3.1 Watershed Parameters – Existing Conditions
3.2 Landuse Summary – Existing & Proposed Conditions
3.3 Green & Ampt Loss Rates Summary – Existing & Proposed Conditions
3.4 Channel Improvement Summary – Existing Conditions
3.5 Onsite Detention Summary – Existing & Proposed Conditions
3.6 Clark Unit Hydrograph – Existing Conditions
3.7 Routing Relationships – Existing Conditions
3.8 Overflow Summary – Existing Conditions
3.9 Node Peak Flow Comparison – Previous vs. Existing Conditions
3.10 Flood Stage Comparison – Previous vs. Existing Conditions
3.11 Inundation Analysis – Existing Conditions
3.12 Bridge / Culvert Headlosses – Proposed Conditions

4.1 Watershed Parameters – Proposed Conditions
4.2 Clark Unit Hydrograph – Proposed Conditions
4.3 Channel Improvement Summary – Proposed Conditions
4.4 Routing Relationships – Proposed Conditions
4.5 Overflow Summary – Proposed Conditions
4.6 Node Peak Flow Comparison – Existing vs. Proposed Conditions
4.7 Flood Stage Comparison – Existing vs. Proposed Conditions
4.8 Inundation Analysis – Proposed Conditions

EXHIBITS

1.1 Studied Watersheds

3.1 Subwatershed Boundaries
3.2 L & Lca – Existing Conditions
3.3 Land Use Map – Existing Conditions
3.4 Channel Improvements – Existing Conditions
3.5 Overflows – Existing Conditions
3.6 CW100-00-00 Profile – Existing Conditions
3.7 CW101-00-00 Profile – Existing Conditions
3.8 CW102-00-00 Profile – Existing Conditions
3.9 CW103-00-00 Profile – Existing Conditions
3.10 CW103-01-00 Profile – Existing Conditions
3.11 CW104-00-00 Profile – Existing Conditions
3.12 CW105-00-00 Profile – Existing Conditions
3.13 Level of Service – Existing Conditions
3.14 Approximate Floodplains – Existing Conditions
3.15 Inundation Analysis – Existing Conditions
3.16 Problem Areas

4.1 Proposed Improvements
4.2 Proposed Improvements Area #1
4.3 Proposed Improvements Area #2
4.4 Overflows – Proposed Conditions
4.5 CW100-00-00 Profile – Proposed Conditions
4.6 CW101-00-00 Profile – Proposed Conditions

TABLE OF CONTENTS (CONT'D)

4.7 CW102-00-00 Profile – Proposed Conditions
4.8 CW103-00-00 Profile – Proposed Conditions
4.9 CW103-01-00 Profile – Proposed Conditions
4.10 CW104-00-00 Profile – Proposed Conditions
4.11 CW105-00-00 Profile – Proposed Conditions
4.12 Level of Service – Proposed Conditions
4.13 Approximate Floodplains – Proposed Conditions
4.14 Inundation Analysis – Proposed Conditions
4.15 Phase 1
4.16 Phase 2
4.17 Phase 3
4.18 Phase 4

APPENDICES
Appendix A – HEC-HMS Output
Appendix B – HEC-RAS Output
Appendix C – Preliminary Opinion of construction Cost Estimate by Dannenbaum Engineering
Appendix D – Benefit-Cost Analysis by Jeffrey S. Ward & Associates, Inc.
Appendix E – Digital Back-up DVD

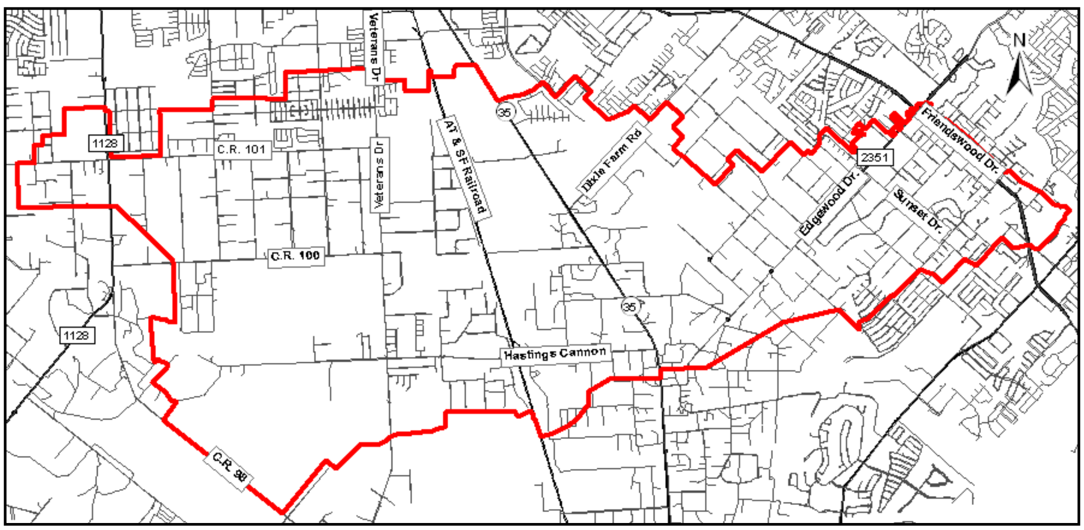
1.0 INTRODUCTION

1.1 Background

The Cowart Creek Watershed, which lies both within Brazoria and Galveston Counties, is a tributary to Clear Creek. The watershed has a total drainage area of 21.2 mi^2, with 2.7 mi^2 lying within Galveston County (all in the City of Friendswood) and the remaining 18.5 mi^2 falling within Brazoria County (approximately 5.6 mi^2 of this within the City of Pearland).

The residents in the region of the study area have experienced major flooding in the past. Through evaluation of the National Flood Insurance Paid Claims database for the City of Friendswood, the City of Pearland, Brazoria County, and Galveston County, it was determined there have been ten major floods and several small events that have caused damage within the study area within the past 34 years. The area has had 10 Presidential Disaster Declarations since 1973. Each of these declarations was as a result of flooding.

Figure 1.1
Vicinity Map



1.2 Project Scope

This purpose of this study is to develop an implementable and cost effective comprehensive watershed master plan that will result in the elimination or significant reduction of flood damages within the watershed. Reduction in the watersheds flood damages will be achieved without negatively impacting downstream communities. In addition, a limited environmental investigation will be performed to evaluate environmental issues, needs and opportunities in the watershed. The basic environmental considerations are to protect, preserve, and maintain existing environmental values and to minimize unavoidable damages to the environment. It is the intent of environmental investigation to identify environmental constraints through the use of existing resources to facilitate the assessment of alternative flood reduction plans for the planning area.

The channels that will be evaluated as part of the Cowart Creek Watershed Master Plan include:

Figure 1.2
Studied Streams and Subwatersheds

Stream ID	Stream Name	Subwatershed
CW 100-00-00	Cowart Creek	CW100
CW 101-00-00	Cowart Creek Tributary #1	CW101
CW 102-00-00	Cowart Creek Tributary #2	CW102
CW 103-00-00	Cowart Creek Tributary #3	CW103
CW 103-01-00	Cowart Creek Tributary #3A	CW103
CW 103-02-00	Cowart Creek Tributary #3B	CW103
CW 104-00-00	Cowart Creek Tributary #4	CW104
CW 105-00-00	Cowart Creek Tributary #5	CW105

A map showing the studied watersheds and their channels is available in Exhibit 1.1.

1.2.1 Previous Studies

Dannenbaum Engineering collected the effective FEMA hydrologic models (10%, 2% and 1% exceedence events), effective hydraulic models, and effective drainage boundaries for the studied streams. In addition to that, several drainage studies were collected in order to refine the drainage boundaries and watershed parameters of Cowart Creek watershed. The collected drainage studies include:

- Tropical Storm Allison Recovery Project - Clear Creek and Armand Bayou – Final Hydrology (Dannenbaum Engineering – 2004)
- Clear Creek General Reevaluation Report Hydrologic Analysis – Without Project Conditions (Dannenbaum Engineering Corporation – July 2003)
- Clear Creek Watershed Modeling Update – Hickory Slough, Mary’s Creek and Cowart Creek (Dannenbaum Engineering Corp. – 2006)
- Clear Creek Watershed Modeling Update – Chigger Creek and Magnolia Creek (Dannenbaum Engineering Corp. – 2006)
- Clear Creek Regional Flood Control Plan – Hydraulic Baseline Report (Dannenbaum Engineering Corporation – 1991)

1.3 Datum Adjustments and Projections

All mapping and GIS data are referenced to North American Datum (NAD) 1983 State Plane Coordinate System, Texas South Central Zone. Vertical elevations are referenced to the North American Vertical Datum (NAVD) 1988 (2001 Adjustment).

Vertical datum conversions were made through observed and published ties between National Geodetic Survey (NGS), Harris Galveston County Subsidence District (HGCSD), City of Pearland and Harris County benchmarks. Adjustments between the vertical datum adjustment of the effective studies (1978) and the datum adjustment of this analysis (2001) may be found in Table 1.1.

2.0 HYDROLOGIC METHODOLOGY

Although the Cowart Creek watershed lies outside of Harris County, this study utilized Harris County hydrologic methodology in order to remain consistent with the hydrologic modeling for the Clear Creek watershed developed as part of the Tropical Storm Allison Recovery Project (TSARP).

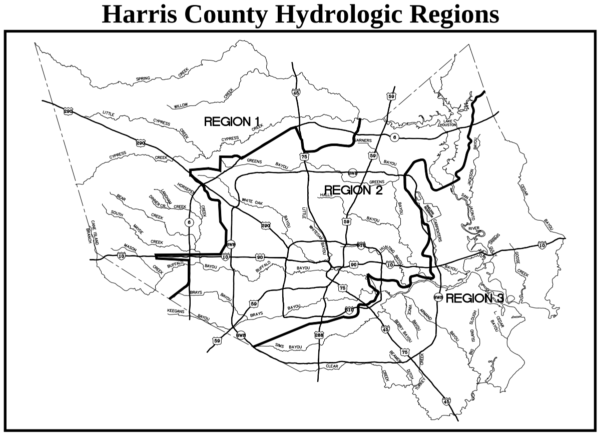
2.1 Rainfall Hyetograph

Flood hazard flows were developed assuming a uniform area rainfall distribution over the entire modeled watershed. The distribution of the rainfall is represented by a succession of incremental rainfall intensities over a finite storm duration. The incremental rainfall pattern is a frequency-based rainfall pattern assigned by HEC-HMS and is dependent upon the following user supplied parameters:[1]

- **Exceedence Probability –** A storm event can drop rainfall totals that have a probability of occurrence at that location within a year. A 50% exceedence event means the rainfall total has a 50% chance of occurring once in every two years. Similarly, a 1% exceedence event could occur once every hundred years. The latter is often called the 100-yr event and can occur at any moment.
- **Max Intensity Duration** – A 15-minute maximum intensity duration is used, unless any of the modeled subbasins have a time of concentration less than 15 minutes. In that case, a maximum intensity duration of 5-minutes should be used.
- **Storm Duration** – Harris County uses a 24-hour storm duration.
- **Peak Center** – The storm peak should be 67% of the storm duration.
- **Storm Area (mi^2)** – A storm area of 0.01 mi^2 should be used. This forces HEC-HMS to use point rainfall without depth-area reduction.

In addition to the above user-supplied parameters, partial-duration point precipitation depths that correspond to the selected exceedence frequency are needed for input into HEC-HMS. The partial-duration point precipitation depths are based upon USGS values for each of Harris County's three (3) hydrologic regions (see Figure 2.1).[2] Point precipitation depths for these three (3) regions may be found in Figures 2.2, 2.3 and 2.4.[3] The Clear Creek watershed lies in Hydrologic Region 3.

Figure 2.1
Harris County Hydrologic Regions



[1] US Army Corps of Engineers – Hydrologic Engineering Center "Hydrologic Modeling System HEC-HMS Users Manual" (2001): Page 102.
[2] Source: TSARP Project Team. "Recommendations for Rainfall Amounts in Harris County" TSARP White Papers (2002).
[3] Ibid

Figure 2.2
Harris County Hydrologic Region 1 Rainfall (inches)

Duration	Return Period (by Exceedence Event)							
	50%	20%	10%	4%	2%	1%	0.4%	0.2%
5-Min	0.7	0.9	1.0	1.1	1.2	1.3	1.4	1.5
15-Min	1.1	1.4	1.5	1.8	2.0	2.2	2.5	2.7
30-Min	1.4	1.8	2.1	2.4	2.7	3.0	3.5	3.9
60-Min	1.9	2.5	2.8	3.4	3.8	4.2	4.9	5.5
2-Hour	2.2	3.0	3.5	4.2	4.9	5.5	6.6	7.5
3-Hour	2.5	3.3	3.9	4.8	5.6	6.5	7.8	9.0
6-Hour	2.9	4.0	4.9	6.1	7.2	8.5	10.4	12.2
12-Hour	3.4	4.8	5.9	7.4	8.7	10.2	12.6	14.7
24-Hour	4.1	5.8	7.1	9.0	10.6	12.4	15.2	17.7
2-Day	4.7	6.6	8.1	10.1	11.8	13.6	16.4	18.7
4-Day	5.4	7.6	9.2	11.3	13.1	14.9	17.6	19.8

Figure 2.3
Harris County Hydrologic Region 2 Rainfall (inches)

Duration	Return Period (by Exceedence Event)							
	50%	20%	10%	4%	2%	1%	0.4%	0.2%
5-Min	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
15-Min	1.1	1.4	1.5	1.7	1.9	2.1	2.4	2.6
30-Min	1.5	1.8	2.1	2.4	2.7	3.0	3.4	3.8
60-Min	2.0	2.5	2.9	3.4	3.8	4.3	4.9	5.5
2-Hour	2.3	3.1	3.6	4.3	5.0	5.7	6.7	7.6
3-Hour	2.6	3.5	4.1	5.0	5.8	6.7	8.0	9.2
6-Hour	3.1	4.3	5.1	6.4	7.6	8.9	10.9	12.8
12-Hour	3.7	5.1	6.2	7.8	9.2	10.8	13.3	15.5
24-Hour	4.4	6.2	7.6	9.6	11.3	13.2	16.2	18.9
2-Day	5.0	7.1	8.6	10.8	12.5	14.5	17.4	20.0
4-Day	5.8	8.1	9.8	12.1	14.0	15.9	18.8	21.1

Figure 2.4
Harris County Hydrologic Region 3 Rainfall (inches)

Duration	Return Period (by Exceedence Event)							
	50%	20%	10%	4%	2%	1%	0.4%	0.2%
5-Min	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
15-Min	1.1	1.4	1.5	1.7	1.9	2.1	2.3	2.5
30-Min	1.5	1.9	2.1	2.4	2.7	3.0	3.4	3.7
60-Min	2.0	2.5	2.9	3.4	3.8	4.3	5.0	5.5
2-Hour	2.4	3.1	3.7	4.4	5.0	5.7	6.8	7.7
3-Hour	2.7	3.5	4.2	5.1	5.9	6.8	8.2	9.4
6-Hour	3.2	4.4	5.3	6.6	7.7	9.1	11.2	13.1
12-Hour	3.8	5.3	6.4	8.0	9.5	11.1	13.6	15.9
24-Hour	4.5	6.4	7.8	9.8	11.6	13.5	16.6	19.3
2-Day	5.3	7.5	9.0	11.2	13.1	15.1	18.1	20.7
4-Day	6.2	8.7	10.5	12.9	14.8	16.9	19.8	22.3

2.2 Loss Rates

Harris County uses the Green & Ampt method to approximate runoff losses in HEC-HMS. The Green & Ampt method is physically-based and estimates losses based on a function of soil texture and the capacity of the given soil type to convey water.[4] Green & Ampt watershed parameters were developed for Harris County on for which the Clear Creek watersheds use loss rates found in Group 4 as listed below[5]:

The following values should be used:
Initial Loss = 0.1 inches
Volume Moisture Deficit = 0.385 unit less
Wetting Front Suction = 12.45 inches
Hydraulic Conductivity = 0.024 in/hr

The Clear Creek watershed use loss rates found in Group 4.

2.3 Sub-watershed Parameters

"Hydrology for Harris County" defines watershed parameters as "the physical characteristics that define the hydrologic properties of a watershed. They are measured and computed from topographic maps, aerial photographs, survey notes, construction drawings, etc." [6] Harris County's Hydrologic Methodology uses watershed parameters to compute Clark's unit graph time of concentration (T_C) and storage coefficient (R) values. The Clark unit graph parameters, drainage area, and Green & Ampt rainfall loss rates of a subbasin are used by HEC-HMS to develop the runoff hydrograph for a particular subbasin.

This section will define each of Harris County's watershed parameters, which will be used in this study to remain consistant with the hydrologic modeling for the Clear Creek watershed developed as part of TSARP even thought the study area is in Brazoria and Galveston Counties, and detail how each parameter should be measured.

[4] TSARP Project Team "Recommendation for Replacing HEC-1 Exponential Loss Function in HEC-HMS" TSARP White Papers (2002).
[5] Ibid
[6] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-1.

2.3.1 Drainage Area

The following excerpt was taken from the "Hydrology for Harris County" discussion of drainage area (A):

> The watershed is subdivided into drainage areas, or subbasins, that allow the total watershed to be studied in greater detail. When subdividing a watershed into subbasins, two factors must be considered. The first factor is the purpose of the study, which defines the areas of interest in the watershed and hence the locations where subbasin boundaries and analysis points should occur. The second factor is the hydrometeorological processes and physical basin characteristics. Each watershed is intended to have uniform hydrometeorological processes and each subbasin should be of adequate size and proper shape so as not to compromise the validity of the watershed parameters.[7]

Subbasin drainage area is measured in square miles. Drainage area should not be less than one square mile since the Flood Hazard Study may not be valid for subbasins with areas less than this limit.[8] If it is necessary to have a subbasin with a drainage area less than one square mile, the subbasin's resultant peak flows should be checked for reasonableness.

The shape of a subbasin has a direct affect on the subbasin's watershed length (L) and watershed length to centroid (Lca). The Flood Hazard Study derived a relationship between A, L and L_{ca}, as seen in Equation 2.1 and 2.2.[9] If the relationship between L, L_{ca}, and A for any subbasin vary substantially for Equations 2.1 and 2.2, then the subbasin's boundaries should be modified.[10]

$$L = 2.08\,A^{0.53} \qquad \text{(Eq. 2.1)}$$
$$L_{ca} = 1.02\,A^{0.55} \qquad \text{(Eq. 2.2)}$$

In lightly developed areas, topographic maps or LiDAR data may be used to delineate drainage boundaries. In areas of higher development, roads, railroads or lot grading typically forms drainage boundaries. Storm sewer systems do not usually define drainage boundaries, as they only carry a fraction of the 100-year storm event.[11]

2.3.2 Watershed Length

"Hydrology for Harris County" gives the following definition of watershed length:

> The watershed length (L) is the length of the longest watercourse for the sub-area. It is defined from the outflow point to the upstream sub-area watershed boundary, and is measured in miles. The watershed length is a factor in determining the value of T_C+R, but only affects Clark's storage coefficient (R) of a subbasin.[12]
>
> For an undeveloped watershed, the watershed length typically follows the longest definable channel and overland flow path.[13] This path can be measured from topographic maps, LiDAR data, and aerial photos. However, in developed subbasins, the watershed length often follows roadside ditches and major streets.

[7] Ibid
[8] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-3.
[9] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page C-14.
[10] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-2.
[11] Ibid
[12] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-2.
[13] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-3.

2.3.3 Watershed Length to Centroid

Watershed length to centroid (L_{ca}) is defined in "Hydrology for Harris County" as the length along the longest watercourse (L) from the outflow point to a point perpendicular to the computed centroid of the drainage area and is measured in miles. The length to centroid represents the average distance a particle of runoff water will travel before reaching the outflow point and is used in determining the Clark's time of concentration (T_C) of the subbasin.[14]

Since watershed length to centroid is dependent upon shape, it is important that subbasins are properly shaped so as to not provide unrealistic L_{ca} values. If unreasonable values of L_{ca} are produced, the subbasin boundaries can be altered or L_{ca} can be artificially adjusted by separately considering different areas of the subbasin.[15] In addition, if two or more points along L are the same distance from a subbasin's centroid, the point that best represents the average watercourse length should be used.[16]

2.3.4 Channel Slope

"Hydrology for Harris County" defines channel slope (S) as,

> …the weighted average slope of the longest watercourse of a watershed. It is representative of how fast the runoff moves through a subbasin watercourse. The average channel slope is the divisor in the hydrologic equations that calculate the time of concentration (T_C) and storage coefficient (R) of a subbasin. It is measured from stream profile plots, construction drawings and topographic maps, and is computed in feet per mile.[17]

The average channel slope must neglect all abrupt changes in flowline, such as drop structures. In addition, the first 10% and last 15% of the channel reach should be ignored since channel slopes typically vary at the upstream and downstream limits of the reach.[18]

2.3.5 Watershed Slope

The following definition of watershed was taken from the "Hydrology for Harris County" Seminar Notebook's discussion of watershed slope (S_o):

> The watershed slope (S_o) is the average overland slope of a subbasin. It is measured from topographic maps at several representative overland flow paths, averaged, and computed in feet per mile. Sudden changes in overland slope should be excluded.
>
> Similar to S, the watershed slope helps represent the speed that runoff drains overland from the drainage boundary to a subbasin watercourse. It is used in the calculation of a subbasin's time of concentration (T_C), and only requires the following degree of accuracy:[19]
>
> $S_o < 20$ feet/mile
> $20 < S_o < 40$ feet/mile
> $S_o > 40$ feet/mile

[14] Ibid
[15] Ibid
[16] Ibid
[17] Ibid
[18] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-4.
[19] Ibid

2.3.6 Percent Land Urbanization

"Hydrology for Harris County" states the following concerning percent land urbanization (DLU):

> Percent land urbanization (DLU) or development percentage is the portion of a drainage area that is used for residential, industrial, commercial, and institutional purposes. Urban development reduces the infiltration area of a watershed thereby creating more excess runoff and increasing the speed that overland runoff will travel to a watercourse. It is used in the interpolation between undeveloped and fully developed values for the time of concentration (T_C) and storage coefficient (R) of a subbasin and is expressed as a percent of the total drainage area. The land urbanization also is a factor in the rainfall loss rates.[20]

Percent land urbanization (DLU) is determined by measuring the amount of each land use type within a subbasin. By measuring each land use shown in Figure 2.5[21] and assigning their appropriate DLU value, a weighted DLU value can be determined for each subbasin.

Percent impervious is calculated in the same manner as DLU. Using the land use area measurements, a weighted impervious percentage can be computed for each sub-watershed using the land use – impervious percentage relationship shown in Figure 2.5.

Figure 2.5
Impervious and Development Values

Landuse	Code	% Land Urbanization	% Impervious
High Density	HD	100%	85%
Undeveloped	U	0%	0%
Developed Green Areas	GA	50%	15%
Residential – Small Lot	RS	100%	40%
Residential - Large Lot	RL	50%	20%
Residential - Rural Lot	RR	0%	5%
Isolated Transportation	T	100%	90%
Water	W	0%	100%
Light Industrial	IC	100%	60%
Unknown		0%	0%
Airport	Air	100%	50%

2.3.7 Percent DLU Affected by On-Site Detention

Starting in 1984, HCFCD began to require that all new development mitigate peak flow impacts through detention. Typically, mitigation is provided through on-site detention or, in some cases, regional detention capacity may be purchased to mitigate a development's flow impacts. The effects of large regional detention ponds can be modeled directly within HEC-HMS; however, under HCFCD methodology, small on-site detention ponds cannot be directly modeled within HEC-HMS. A simplification must be applied to show the benefits of on-site detention.

[20] Ibid
[21] TSARP Project Team "Recommendation for Replacing HEC-1 Exponential Loss Function in HEC-HMS" TSARP White Papers (2002).

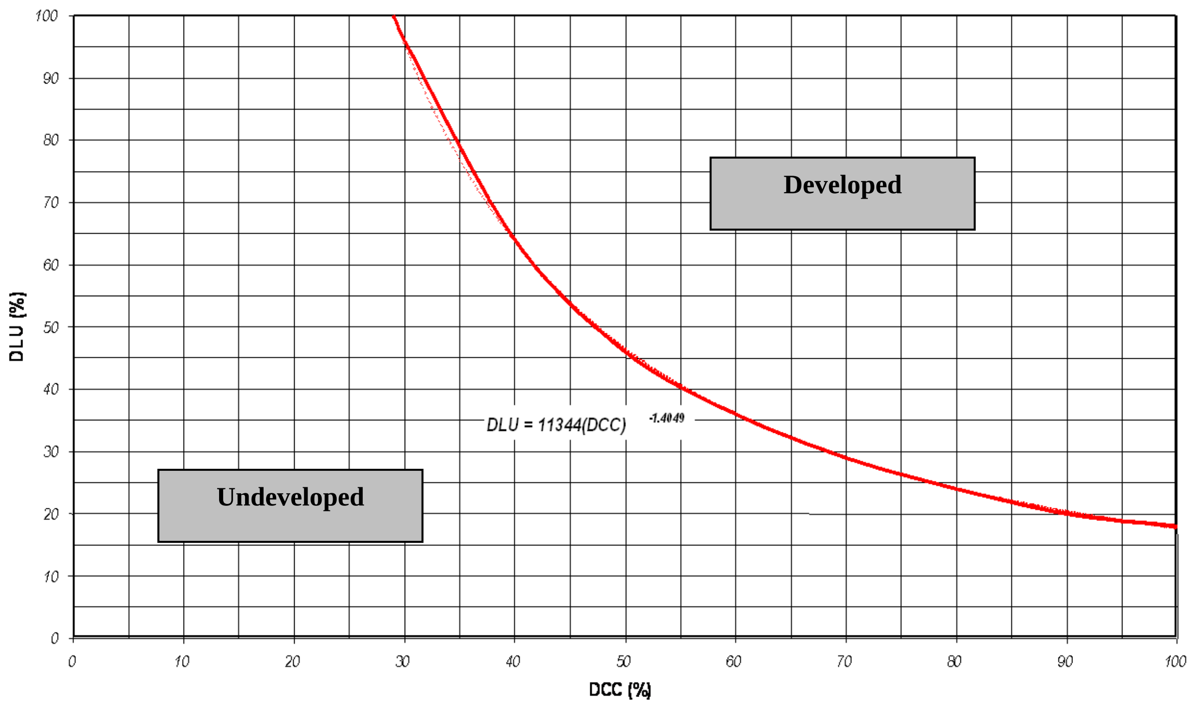
To reflect the effects of Harris County's on-site detention requirements, the percentage of each subbasin that is affected by on-site detention is measured. The percentage of the subbasin affected by detention is identified as DET in T_c & R calculation sheets. DET is used to adjust DLU to reflect the benefits of on-site detention. This adjustment will be explained in more detail in Section 2.3.9.

2.3.8 Percent Land Urbanization (Minimum)

As seen in Section 2.4, the T_c+R equation varies depending on whether a subbasin is developed or undeveloped. In the previous HCFCD methodology a subbasin was considered undeveloped if its DLU was less than 18%[22]; however, inconsistencies in flows would sometimes occur around the 18% threshold. Peak flows would often decrease as DLU increased in the immediate vicinity of 18% DLU.

To remedy the flow inconsistency around 18% DLU, the DLU threshold between the undeveloped and developed conditions was no longer fixed at 18%. Based upon definitions and equations from "Hydrology for Harris County," (HCFCD-March 3, 1988), a threshold (DLU_{MIN}) between undeveloped and developed subbasin conditions was defined by the TSARP Hydrology Committee (see Figure 2.6).[23]

Figure 2.6
Undeveloped/Developed Threshold



Each subbasin will have this threshold, or DLU_{MIN}, defined based upon its Percent Channel Conveyance (DCC) value. DCC is discussed in detail in Section 2.3.11. The equation for DLU_{MIN} is as follows: [24]

$$DLU_{MIN} = 11344(DCC)^{-1.4048} \qquad \text{(Eq. 2.3)}$$

Explanation of the use of DLU_{MIN} in the T_c+R equation will be discussed in more detail in Section 2.4.

[22] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page C-11.
[23] TSARP Project Team "Recommendation for Replacing HEC-1 Exponential Loss Function in HEC-HMS" TSARP White Papers (2002).
[24] TSARP Project Team "Recommendation for Replacing HEC-1 Exponential Loss Function in HEC-HMS" TSARP White Papers (2002).

2.3.9 Percent Land Urbanization (Detention)

As previously discussed, Percent Land Urbanization (DLU) is adjusted to reflect the presence of on-site detention. The Percent Land Urbanization (Detention), or DLU_{DET} value is used in the T_c+R equations (see Section 2.4) to reflect on-site detention and is dependent upon DET and DLU_{MIN}. The equations for Percent DLU_{DET} are shown below:[25]

$$DLU_{DET} = DLU - DET \quad (\text{if } DLU_{DET} >= DLU_{MIN}) \quad \text{(Eq. 2.4)}$$

$$DLU_{DET} = DLU_{MIN} \quad (\text{if } DLU_{DET} < DLU_{MIN}) \quad \text{(Eq. 2.5)}$$

$$DLU_{DET} = DLU \quad (\text{if } DLU < DLU_{MIN}) \quad \text{(Eq. 2.6)}$$

Please note that impervious percentage should remain unadjusted and should account for all impervious cover, regardless of the existence of on-site detention.

2.3.10 Percent Channel Improvement

"Hydrology for Harris County" gives the following definition of Percent Channel Improvement (DCI):

> Percent Channel Improvement (DCI) is the portion of the longest watercourse which has an improved channel. It is expressed as a percent of the longest definable channel. An improved channel section is defined as a section that has been significantly altered from its natural state by a construction project for the purpose of providing storm flow capacity for existing or proposed urban development. It is used in the interpolation between undeveloped and fully developed values of the Time of Concentration (T_C) for a subbasin. Aerial photographs, construction plans, and field investigations are used to determine the extent of channel improvements.[26]

2.3.11 Percent Channel Conveyance

Percent Channel Conveyance (DCC) is the ratio of discharge carried between channel banks to the 1% exceedence event discharge that would be anticipated if the channel had full conveyance. The conveyance of a channel is interpreted to be its ability to carry runoff in an area of uniform high velocity.[27]

The 1% exceedence event full conveyance discharge can be determined by estimating the total drainage area upstream of the computation point, its weighted urban development, and average channel slope then reading the discharge from Figures 2.7 through 2.11.[28]

DCC is measured from a HEC-RAS model in which the 1% exceedence event full conveyance discharge for a subbasin is held constant through the basin's channel reach. DCC, or the percentage of flow conveyed within channel banks, is measured at all cross sections along the channel reach. A weighted average DCC value, based upon channel reach length, is determined for the main channel of the subbasin. DCC is then rounded to the nearest 10 percent for the subbasin under consideration.

By definition, an undeveloped watershed has a percent channel conveyance equal to 100% (i.e., the natural flood plain carries the water it is expected to in order to accommodate the undeveloped watershed).[291] Assuming no channel improvements, a basin's DCC will decrease as DLU increases.[30]

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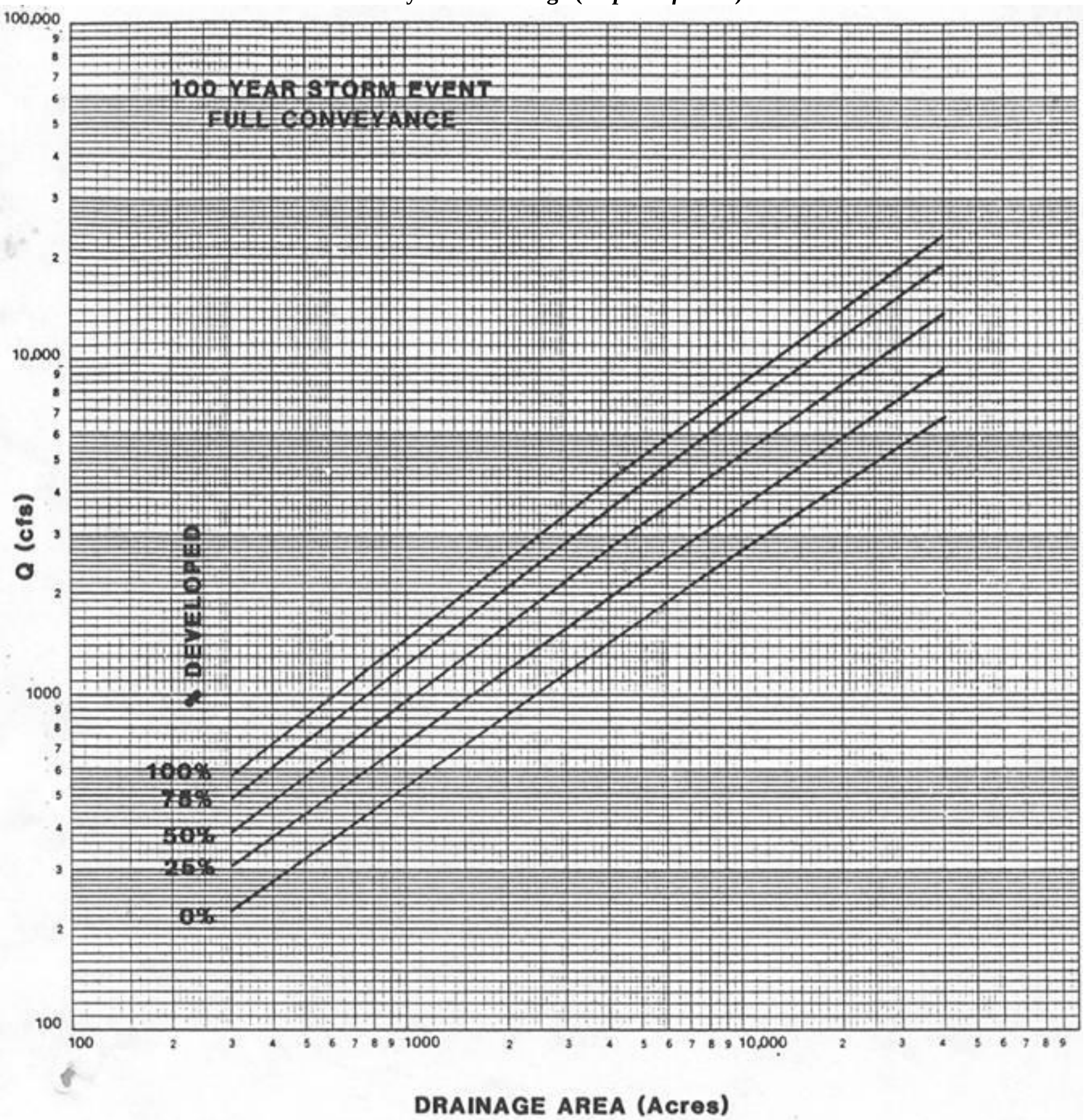
[25] Ibid
[26] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-5.
[27] Ibid
[28] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Exhibit C-4 through C-9.
[29] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-6.

Figure 2.7
Conveyance Discharge (*Slope = 1ft/mile*)



30 Ibid

Figure 2.8
Conveyance Discharge (*Slope = 3ft/mile*)

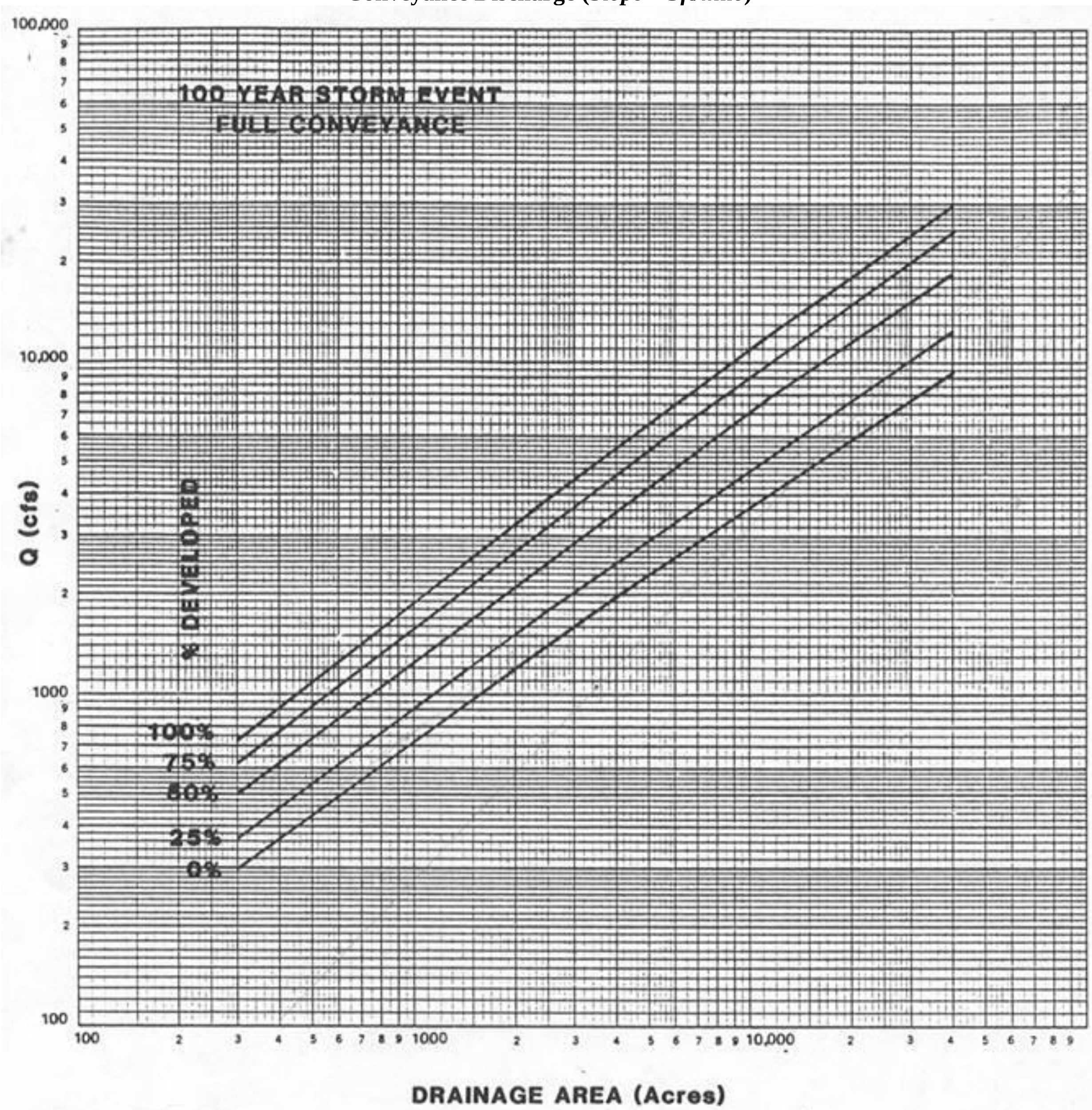


Figure 2.9
Conveyance Discharge (*Slope = 5ft/mile*)

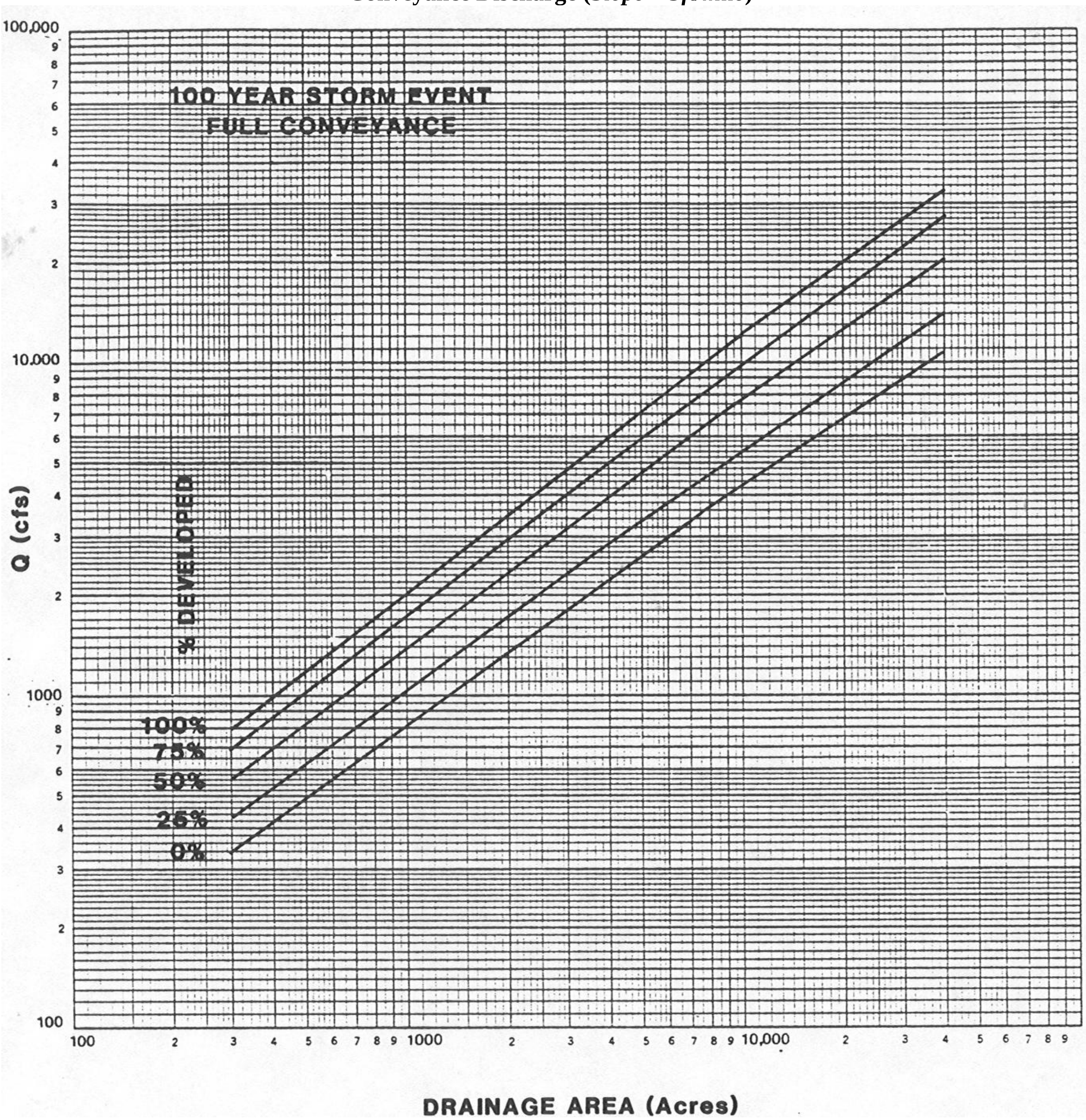


Figure 2.10
Conveyance Discharge (*Slope = 7ft/mile*)

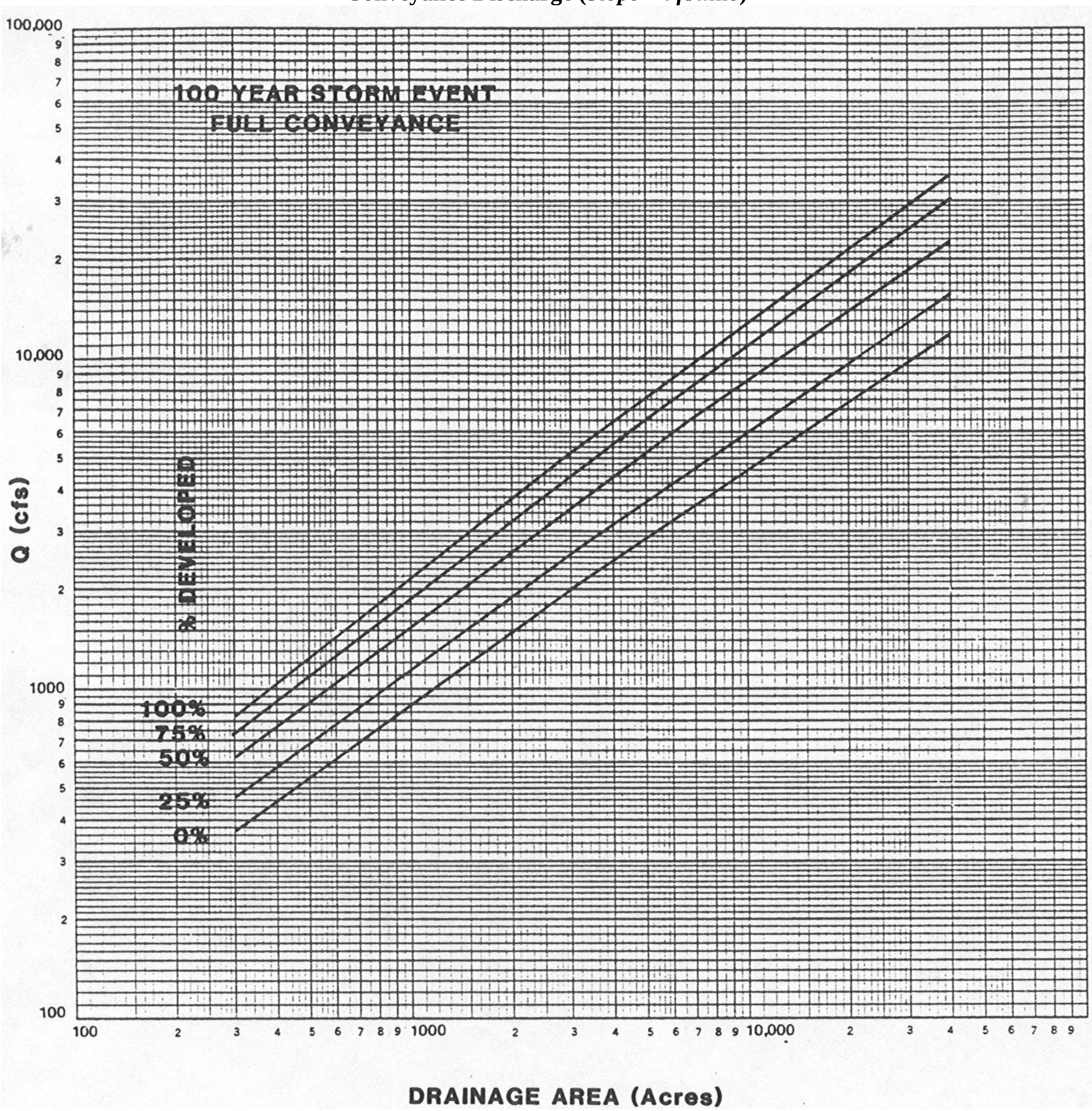
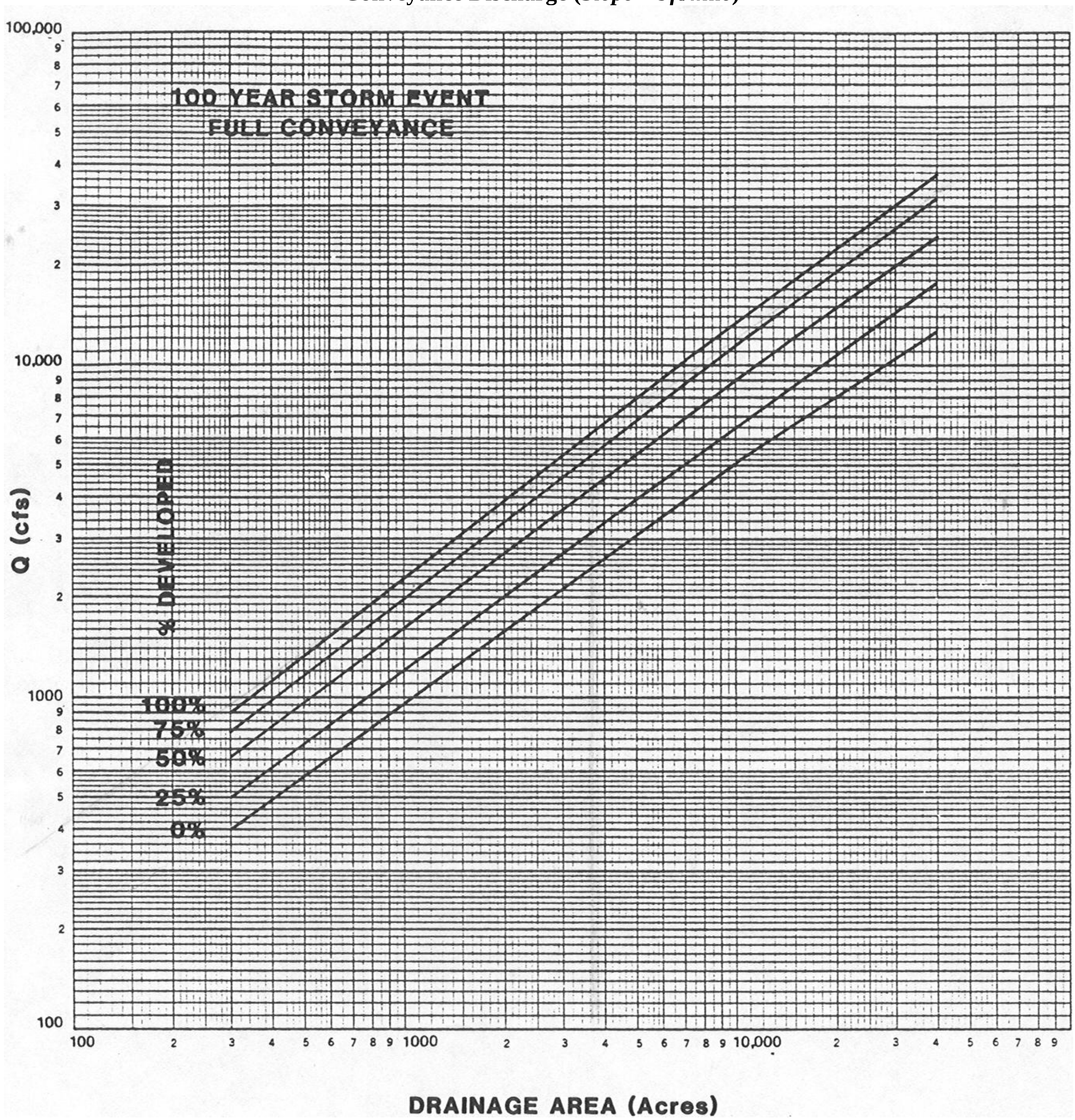


Figure 2.11
Conveyance Discharge (*Slope = 9ft/mile*)



2.3.12 Percent Ponding

Percent Ponding (DPP) may be defined as, "the portion of a sub-area where runoff is retarded from reaching a watercourse due to obstructions or natural storage. Such obstructions include leveed fields (rice farms), swamps, etc. It is expressed as a percent of the total drainage area." [31]

Percent ponding is used to increase Clark's storage coefficient (R) after its value has been calculated through the unit graph parameter equations and after the on-site detention adjustment factor has been applied.[32] The adjustment of R due to the percent ponding should only be used when the ponded areas

[31] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-7.
[32] TSARP Project Team "Recommendation for Replacing HEC-1 Exponential Loss Function in HEC-HMS" TSARP White Papers (2002).

cover at least 20% of the watershed and is dependent upon the storm frequency being analyzed, as shown below:[33]

Exceedence Event	Pond Adjustment Factor (RM)	
50%	$RM = 1.36\ DPP^{0.24}$	(Eq. 2.7)
20%	$RM = 1.31\ DPP^{0.214}$	(Eq. 2.8)
10%	$RM = 1.28\ DPP^{0.199}$	(Eq. 2.9)
4%	$RM = 1.25\ DPP^{0.171}$	(Eq. 2.10)
2%	$RM = 1.23\ DPP^{0.153}$	(Eq. 2.11)
1%	$RM = 1.21\ DPP^{0.132}$	(Eq. 2.12)
0.4%	$RM = 1.19\ DPP^{0.106}$	(Eq. 2.13)
0.2%	$RM = 1.17\ DPP^{0.086}$	(Eq. 2.14)

The flooded portion of a reservoir should not be considered a ponding area since its runoff will not be delayed from reaching a watercourse.[34] Reservoir attenuation should be accounted for in storage routing computations or should be modeled directly in HEC-HMS.

2.4 Unit Hydrograph Parameters

Unit hydrograph parameters are calculated from the subbasin characteristics outlined in Section 2.3. Utilizing the calculated unit hydrograph parameters in the Clark's Unit Hydrograph method allows for development of an estimated runoff hydrograph for a subbasin. Harris County utilizes the Clark's Unit Hydrograph technique due to its wide acceptance and the large number of storm hydrographs that have already been correlated to Clark's Unit Hydrograph parameters.[35]

The HEC-HMS model requires three (3) parameters to predict runoff hydrographs using Clark's methodology:[36]

1. Time of Concentration (T_C) - The time required for rainfall excess to travel the entire length of the longest watercourse (L).

2. Storage Coefficient (R) - Attenuates the hydrograph at the outflow point to account for storage in the subbasin.

3. Time-Area Curve - Defines the cumulative area of the subbasin as a function of time. The default curve in HEC-HMS is used.

The Harris County Flood Hazard Study developed equations for T_C & R. "Hydrology for Harris County" states the following concerning the equations:

Experience has shown that the optimized individual values of T_C and R are a function of the calibration procedure used but that the sum of the two parameters, T_C+R, is relatively independent of the procedure. As a result, the Flood Hazard Study developed one equation that computes T_C directly and another which computes the sum of TC+R. The storage coefficient (R) is simply the difference between the two computed values.[37]

[33] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Exhibit C-10.
[34] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-7.
[35] Ibid
[36] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-8.
[37] Ibid

The HCFCD unit hydrograph equations are as follows:[38]

$T_C = D[1 - (0.0062)(0.7\ DCI + 0.3\ DLU_{DET})](L_{ca}/\ S^{1/2})^{1.06}$ (Eq. 2.15)

$C=7.25$ (if $DLU < DLU_{MIN}$) (Eq. 2.16)

or

$C=4295\ (DLU_{DET})^{-0.678}\ (DCC)^{-0.967}$ (if $DLU >= DLU_{MIN}$) (Eq. 2.17)

$T_C+R = C(L/\ S^{1/2})^{0.706}$ (Eq. 2.18)

where:

L	=	watershed length, in miles
L_{ca}	=	length to centroid, in miles
S	=	channel slope, in feet per mile
DLU	=	percent urban development*
DLU_{MIN}	=	percent land urbanization (minimum)*
DLU_{DET}	=	percent land urbanization (detention)*
DCI	=	percent channel improvement*
DCC	=	percent channel conveyance*
D	=	2.46 (if $S_o \leq 20$ feet/mile)
D	=	3.79 (if 20 feet/mile $< S_o \leq 40$ feet/mile)
D	=	5.12 (if $S_o > 40$ feet/mile)
S_o	=	watershed slope, in feet per mile

*Note: The values for DLU, DLU_{MIN}, DLU_{DET}, DCI, and DCC should be whole numbers (i.e., 50% would be represented by the number 50).

As outlined in Section 2.3.12, R must be adjusted for subbasins with ponding greater than 20%.

2.5 Stream Reach Routing

The TSARP flood hazard update uses the Modified Puls Method of routing. This flood routing method is based on the continuity equation and a relationship between flow and storage or stage. The routing is modeled on an independent-reach basis from upstream to downstream.[39]

"Hydrology for Harris County" gives the following evaluation of the Modified Puls Routing Method:

> The Modified Puls Routing Method is based on the continuity equations and is applicable to both channel and reservoir routing. The Modified Puls Method is usually referred to as a reservoir routing technique because it assumes an invariable storage-outflow relationship. The method neglects the variable slope of the water surface that occurs during the passage of a flood wave down a channel; however, the Method's limitations can be partially overcome by making successive routings through a number of relatively short stream reaches. In effect, this procedure reduces the relative importance of the wedge storage and simulates the stream flow through small contiguous reservoirs. Also, wedge storage is generally not as significant a factor in the sluggish gulf coast systems because of the relatively flat and wide flood plains.

[38] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page E-8.
[39] US Army Corps of Engineers Hydrologic Engineering Center "HEC-1 Flood Hydrograph Package User's Manual" September 1990, Page 39.

> Many of the other methods of flood routing utilize coefficients that are calibrated on the original configuration of the channel from historic gage information. The effects of channel improvements negate gage data and can make adjustments to routing parameters difficult. An advantage of the Modified Puls Method is that it is more amenable to simulations of varying degrees of channel improvements. The effects of channel improvements can be measured directly by the storage-outflow relationship used in the Modified Puls Method. A good correlation between computed and historic hydrographs was obtained using the Modified Puls Method for the calibration effort of the Flood Hazard Study.[40]

The Modified Puls method of routing requires three (3) parameters[41] to function:

- Storage-Outflow Relationship
- Number of Subreaches
- Initial Conditions

The storage-outflow relationship for a reach is determined from HEC-RAS by executing a multiple profile run of predetermined flow rates. The flow rates should encompass the expected 0.2% exceedence event discharge. Flows in the storage-outflow HEC-RAS model should be kept constant between HEC-HMS routing reaches.

The number of subreaches for a routing reach is calculated from the multiple profile run used to develop the reach's storage-outflow relationship. The average of all the profiles' travel time through a routing reach should be determined. Dividing the average travel time by the HEC-HMS model's time increment yields the number of subreaches for a given routing reach. The number of subreaches should be rounded to the nearest whole number.

If during the travel time calculations, the average velocity in the reach is found to be less than 1.0 feet per second and the reach's energy grade is relatively flat, it may be reasonable to assume that the reach is functioning as a linear reservoir. Therefore, instead of a high number of routing steps produced by the low velocity, the number of routing steps should be set to one (i.e. reservoir routing).[42]

Initial conditions for all routing reaches should be set to "Inflow = Outflow".

[40] Harris County Flood Control District "Hydrology for Harris County" March 3, 1988, Page D-5.
[41] US Army Corps of Engineers – Hydrologic Engineering Center "Hydrologic Modeling System HEC-HMS Users Manual" (2001): Page 74.
[42] TSARP Project Team "Recommendation for Replacing HEC-1 Exponential Loss Function in HEC-HMS" TSARP White Papers (2002).

3.0 EXISTING CONDITIONS

In order to effectively quantify the extent and frequency of flooding within the watershed and also to measure benefits of proposed drainage improvements, the existing conditions modeling of the watershed needed to be evaluated. A watershed hydrologic model existed from previous studies of Cowart Creek. However, this hydrologic model would require additional subbasin delineations, hydrologic nodes, routing reaches, etc. to adequately quantify benefits of drainage improvements. Similarly, hydraulic models existed for most of the streams included in the study. However, several streams did not have an existing hydraulic model, had an existing hydraulic model in an outdated model format, or had a hydraulic model that was not on the correct vertical datum adjustment. Therefore, a complete reevaluation of the existing condition hydrologic and hydraulic models of the Cowart Creek watershed was warranted. This section will discuss the reevaluation of existing conditions.

3.1 Hydrology

An existing conditions HEC-HMS model was available from Dannenbaum's 2006 update of the Cowart Creek watershed. This model was used as a base for the creation of updated HEC-HMS models for this study. This section will discuss the creation and the results from the updated existing conditions HEC-HMS models for the Cowart Creek watershed.

3.1.1 Model Parameters

The following is an application of the methodology outlined in Section 2.0.

3.1.1.1 Sub-watershed Delineation

Watershed delineation for this update was performed using aerial photography (2005), LiDAR data, storm sewer data, and limited field reconnaissance. Dannenbaum utilized the TSARP drainage boundaries as a first draft. Newly acquired LiDAR data for the tributaries was used to update the TSARP boundaries. These boundaries were further modified to reflect changes in drainage patterns due to development.

LiDAR data and the Arc Hydro program were used to determine drainage patterns within the studied watersheds. The Arc Hydro Terrain Preprocessor utility uses LiDAR DEM (Digital Elevation Model) data to define overland flow paths (streams) and the catchments that contribute to the Arc Hydro streams. The user can control the size of the catchments and the number of streams developed by Arc Hydro. It should be noted that the LiDAR DEM data reflects overland flow patterns and does not reflect storm sewers or bridge/culvert crossings.

The last step in the development of the updated watershed boundaries was to finalize the watershed boundaries utilizing the Arc Hydro catchments, aerial photographs, field investigations, storm sewer plans, and previous studies. In undeveloped areas, the Arc Hydro catchment data was the primary source used to update the subbasins. Limited field reconnaissance was used to check certain areas of the watershed and to make changes, if necessary. These efforts yielded the final sub-watershed boundaries found Exhibit 3.1.

3.1.1.2 Sub-watershed Parameters

In order to develop TC & R values (Table 3.2) for use in the Clark's Unit Hydrograph method, the various watershed parameters discussed in Section 2.3 were estimated for each subbasin. This section will briefly discuss the development of each parameter, as well as any unique assumptions or methodologies that were utilized in their calculation.

3.1.1.2.a Watershed Length

Watershed length (L) was updated for each subbasin of the studied watersheds, as per the methodology outlined in Section 2.3.2. LiDAR data, aerial photos, and storm sewer plans were used to define the longest watershed length for each subbasin. The watershed lengths were drawn in a GIS format to allow automatic measurement of lengths. Exhibit 3.2 displays the longest watershed length for each subbasin.

3.1.1.2.b Watershed Length to Centroid

Watershed length to centroid (L_{ca}) was updated as per the methodology outlined in Section 2.3.3. The calculation of L_{ca} was automated by use of the watershed length shapefile, subbasin shapefile, and a GIS script that determined the centroid of the subbasin. Table 3.1 shows the measured values while Exhibit 3.2 also displays the L_{ca} paths.

3.1.1.2.c Channel Slope

Channel slope (S) was calculated as per the methodology outlined in Section 2.3.4. Using HEC-RAS, a line of best fit was placed through the minimum channel elevation of each section within a subbasin. The slope of this line was used as the channel slope of the concerned subbasin. This methodology was modified for subbasins that contained sudden drops. A slope was taken either upstream or downstream of the drop so that a channel's slope (without influence of the drop) was estimated. In cases where the subbasin's primary outfall was not studied, there were two options for slope measurement. The first choice was to use the effective channel slope or estimate the channel slope from an existing hydraulic model. If this first option was not possible, then the channel slope was estimated from LiDAR data. Table 3.1 lists the channel slopes.

3.1.1.2.d Watershed Slope

Watershed slope (So) was calculated for each subbasin using LiDAR data, as per the methodology outlined in Section 2.3.5. Several representative overland flow paths were overlaid on a DEM (Digital Elevation Model). The profiles of the flow paths were extracted from the DEM, allowing the slope of these flow paths to be calculated from a line of best fit through their inverts. Taking the best fit of these points neglects any sharp peaks or drops. An average of the representative So values was used for each subbasin. Subbasins outside of the limits of the LiDAR coverage utilized effective watershed slope values. Watershed slopes are also found in Table 3.1.

3.1.1.2.e Percent Land Urbanization

A land use theme that reflects 2005 conditions was created from the TSARP land use theme for the Clear Creek watershed. Percent Land Urbanization (DLU) was calculated for the studied watersheds by assigning one of eleven land use categories outlined by the TSARP Hydrology Committee seen in Section 2.3.6. Exhibit 3.3 shows the land use map used in the hydrologic update. The amount and type of land use within each subbasin was determined using a GIS analysis of this information.

This land use information was used to develop a weighted land urbanization percentage (DLU) and impervious percentage for each subbasin based upon values found in Exhibit 3.3. Calculated DLU and impervious percentage values may be found in Table 3.1, for which Table 3.2 contains the back-up data for DLU and impervious values. Loss rates, which are dependent upon land use are shown in Table 3.3.

3.1.1.2.f Percent Channel Improvement

Percent channel improvement (DCI) was updated as per the methodology outlined in Section 2.3.10. Two GIS shapefiles were created to calculate DCI values: longest definable channel and improved channel. The longest definable channel was delineated as the "longest definable channel" along each subbasin's watershed length (L) path. A portion of the longest definable channel was identified as "improved" if channel improvements were present. Each subbasin's improved channel length was divided by its longest definable channel length to determine the subbasin's DCI value. Table 3.1 displays the calculated DCI values, and Table 3.4 shows the backup calculations for deriving those values. Exhibit 3.4 illustrates the longest definable channel and the channel improvements for each subbasin in the watershed.

3.1.1.2.g Percent Channel Conveyance

As outlined in Section 2.3.11, HEC-RAS was used to determine the percent channel conveyance (DCC) for subbasins that contained sufficient HEC-RAS data. A 100% conveyance value was assumed for subbasins without the necessary hydraulic data, unless an effective DCC value was known or a lesser value was identified as being more reasonable during the calibration process.

3.1.1.2.h On-Site Detention Pond Adjustment to T_C & R

As part of TSARP a shapefile that identified the amount of development with on-site detention located within each Keymap grid of the Clear Creek Watershed was created. By overlaying the Keymap grid and the Clear Creek watershed, a GIS script was used to calculate the amount of development with on-site detention for each subbasin. The area of development with on-site detention was divided by the total area of the subbasin to determine the percentage of the subbasin that is affected by detention (DET).

On-site detention information was not supplied for areas outside of Harris County, so Dannenbaum developed a methodology to identify areas serviced by on-site detention outside of Harris County. During a project for the Corps of Engineers, Dannenbaum delineated (in GIS) development limits within the Clear Creek watershed for 1984 and 2000 development conditions. Since on-site detention was required after 1984, the amount of development outside of Harris County that should have on-site detention could be determined by comparing the two development shape files. The detention data outside of Harris County was incorporated into the Key Map grid detention shapefile. This allowed a DET value for subbasins outside of Harris County to be determined. In addition, it was assumed that all post-TSARP development (2001) provided on-site detention. The DET value summary can be found in Table 3.1 and the derivation may be found in Table 3.5.

3.1.1.3 Unit Hydrograph Parameters

Clark's unit hydrograph parameters were calculated using methodology outlined in Section 2.4. The resulting Clark unit hydrograph parameters for the studied watersheds may be found in Table 3.6.

3.1.1.4 Stream Reach Routing (SV/SQ)

The Modified Puls routing method was utilized for all of the studied streams. The updated HEC-RAS models were used to create a multiple profile run in which flows were kept constant between routing nodes. Flows between nodes were a percentage of the maximum anticipated 1% exceedence event flow for that reach. Profiles for 5, 10, 20, 40, 60, 80, 100, 120, 160, and 200% of the maximum anticipated 1% exceedence event were run. Note that some profiles were ignored if they were found to cross other routing profiles.

The time step for a routing reach was the average travel time between routing nodes for all routing profiles. The average travel time was divided by the HEC-HMS model's time interval to calculate a time step for the reach. A check was performed for each reach to verify that the reservoir routing method was appropriate. If the average velocity in a reach was less than 1 ft/s and the energy grade of the reach was generally flat, then reservoir routing (1 time step) was used. The updated routing data that was used in the HEC-HMS models may be found in Table 3.7.

3.1.1.5 Diversion Rating Curves

Due to the flat topography in the Cowart Creek watershed and man-made channels that do not follow natural drainage patterns, there are many overflows that alter the watershed's hydrology. Figure 3.1 lists the seven (7) overflows that are included in the Cowart Creek hydrology:

Figure 3.1
Overflows

No.	HEC-HMS ID	From		To
1	CW100B_OVFL	CW100B	–	CW103A
2	CW100C_OVFL	CW100C	–	CW103C
3	CW103C_OVFL	CW103C	–	CW103E
4	CW103D_OVFL	CW103D	–	CW102B
5	CW103E_OVFL	CW103E	–	CW102B
6	CW103F_OVFL	CW103F	–	CW102D
7	CW102A_OVFL	CW102A	–	CH100A

The location of each overflow is shown in Exhibit 3.5, while Table 3.8 displays the inflow/diversion relationship used to represent each overflow. It should be noted that each overflow was estimated by placing lateral weirs at the location of the overflow in the appropriate HEC-RAS model. Various profiles were run through the HEC-RAS model and their associated diversions to CW103-00-00 were noted. This section briefly discusses each overflow

<u>CW100-00-00 to CW103-00-00 (CW100B_OVFL)</u>
An overflow from CW100-00-00 to CW103-00-00 exists during extreme storm events. This overflow occurs between C.R. 831 and C.R. 143 and flows south to CW103-00-00 via the roadside ditches of C.R. 827, C.R. 831, C.R. 103, C.R. 879C, C.R. 104, C.R. 829 and C.R. 143.

CW100-00-00 to CW103-00-00 (CW100C_OVFL)
A second overflow exists between CW100-00-00 and CW103-00-00 during extreme storm events. This overflow is located between C.R. 143 and the AT&SF Railroad. Once storm flows exceed the south bank of CW100-00-00 it will travel south to CW103-00-00.

CW103-02-00 to CW103-01-00 (CW103C_OVFL)
An overflow from CW103-02-00 to CW103-01-00 exists during extreme storm events. This overflow occurs between Veterans Dr. and the AT& SF Railroad and flows south to CW103-01-00 via direct contact with CW103-01-00 on the west of the overflow to the roadside ditches of Veterans Dr. and McKeever Rd. and Carrie Ln.

CW103-01-00 to CW102-00-00 (CW103D_OVFL, CW103E_OVFL, & CW103F_OVFL)
An overflow from CW103-01-00 to CW102-00-00 exists during extreme storm events. This large overflow occurs between Kristi Ln and S.H. 35 and flows south to CW102-00-00 via the roadside ditches of C.R. 128, C.R. 175, C.R. 927 and Amoco Dr.

CW102-00-00 to CH104-00-00 (CW102A_OVFL)
An overflow from CW102-00-00 to CH104-00-00 exists during extreme storm events. This overflow occurs between C.R. 175 and AT&SF Railroad and flows south to CH100-00-00. LiDAR shows that because CW102-00-00 intersects a natural channel that drains south to CH100-00-00, an overflow occurs once flood stages exceed the elevation of the southern channel bank of CW102-00-00.

3.1.2 Results

With the necessary hydrologic parameters at hand, an existing conditions HEC-HMS model was created for the 50%, 20%, 10%, 4%, 2%, 1% and 0.2% exceedence events. HEC-HMS results from the updated existing conditions HEC-HMS model were compared against the results of previous studies. Table 3.9 summarizes the comparison based on available data from the previous study to the current (existing) study for the 10%, 2%, 1%, and 0.2% exceedence events. Appendix A contains the HEC-HMS output from the existing conditions model.

The previous 1% exceedence event peak flow at the mouth of Cowart Creek (CW100-00-00) was 5,933 cfs or 10.5% lower than the 6,555 cfs peak flow developed from the existing conditions model. CW102-00-00 and CW103-00-00 experienced the largest changes in peak flow rates from the previous study. The existing peak flow at the mouth of CW102-00-00 decreased 25.7% from the previous study, while the peak flow at the mouth of CW103-00-00 increased by 19.4%. These peak flow rate changes can be primarily attributed to modification of overflow relationships. In addition, a refinement of drainage areas and updated routing data affected the results of the existing conditions model.

3.2 Hydraulics

An existing conditions hydraulic method was developed to quantify existing peak flood stages along the studied streams. The resultant peak flood stages would help identify problem areas that would be the focus of the proposed drainage improvement modeling.

3.2.1 Model Parameters

3.2.1.1 Hydraulic Model Development

Hydraulic simulations were carried out for the nine (9) streams identified in Section 1.2. Dannenbaum recently developed HEC-RAS models for CW100-00-00 and CW104-00-00 that could be utilized for this study. In addition, Civil Tech created HEC-RAS models for CW101-00-00, CW101-01-00 and CW105-00-00 that could be used for this study with slight modifications. The remaining four (4) streams had to have hydraulic models created through use of HEC-RAS (version 3.1.3), ArcGIS (version 9.2) and HEC-GeoRAS (version 4.1). HEC-GeoRAS is an extension to ArcGIS that processes geospatial data for use with the Hydrologic Engineering Center's River Analysis System (HEC-RAS). The extension permits GIS users to develop a HEC-RAS GIS Import File from a digital terrain model (DTM) represented by a triangulated irregular network (TIN) and user developed GIS themes. HEC-GeoRAS was used to assemble the majority of the hydraulic models' geometric data.

When imported into HEC-RAS, the RAS GIS Import File creates a nearly complete HEC-RAS geometry file. The completeness of the HEC-RAS geometry file depends on the number of user developed GIS themes; however, HEC-GeoRAS has the ability to create a HEC-RAS geometry file with the following attributes:

- Stream Centerline
- Stream/Reach Names
- Cross Sections
- Cross Section Stationing
- Cross Section Bank Stations
- Reach Lengths
- Elevations
- Manning's n-values
- Levees
- Ineffective Flow Areas
- Storage Areas

The geometry file created by HEC-GeoRAS is intended to be a basic framework for the final HEC-RAS geometry file. The geometry file created by HEC-GeoRAS does not include bridge data. Attributes such as bank stations, flow paths, n-value locations, ineffective flow areas, and expansion/contraction coefficients must be refined before the geometry file can be utilized in a flood hazard model.

To mirror Dannenbaum's process for developing the HEC-RAS models, the hydraulic model development will be discussed in two sections: Basic Geometry Assembly and Model Refinement.

3.2.1.2 Basic Geometry Assembly

HEC-GeoRAS requires the following sets of data to create a RAS GIS Import file:[43]

- Terrain TIN/LiDAR Grid
- Stream Centerline Theme
- Cross Section Cut Line Theme

In addition, there are several optional GIS themes that the HEC-GeoRAS program can use to enhance the RAS GIS Import file:[44]

- Main Channel Banks Theme
- Flow Path Centerlines Theme
- Levees Theme
- Land Use Theme
- Ineffective Flow Area Theme

[43] HEC-GeoRAS Users Manual
[44] HEC-GeoRAS Users Manual

- Storage Area Theme

All of the required GIS themes, as well as the main channel banks, flow path centerline, ineffective flow areas, and land use (Manning's n-value) themes were created. The next section will discuss the development of these themes and the creation of the initial geometry file for Cowart Creek and its tributaries. Refinement of the final HEC-RAS geometry file will be discussed in Section 3.2.1.3.

Terrain LiDAR Grid

LiDAR data supplied by the City of Pearland and the City of Friendswood was used in this modeling update. The LiDAR data, which is contained on the attached CD, was supplied in the form of an ArcInfo export interchange file (.e00) and was imported into ArcGIS as raster grid file that reflected the watershed's topography.

The LiDAR raster grid has a cellsize (X,Y) of 6X6 and is in the North American Datum (NAD) 1983 State Plane Coordinate System, Texas South Central Zone. It should be noted that the LiDAR data has some limitations, since LiDAR data does not always represent true ground elevations for areas with dense vegetation. This is especially true for areas adjacent to streams, since channel areas typically contain dense vegetation. In addition, LiDAR data cannot supply elevation data below water. Channel data is needed below water to get a complete representation of a channel's configuration.

To compensate for LiDAR's shortcomings, channel data from the effective hydraulic models, which are based on field surveyed sections, were added to the LiDAR developed cross sections. Based upon bridge locations, each HEC-RAS cross section had the channel data of the nearest effective HEC-2 cross section added. The effective HEC-2 channel data was vertically adjusted (seen in Section 1.2) to match the datum adjustment of the LiDAR data. For portions of CW103-00-00, CW103-01-00 and CW103-02-00, survey data was available from C L Davis & Company. This survey data was used to supplement the HEC-RAS cross section data taken from LiDAR since these streams were not modeled as part of the effective study.

Stream Centerline

HEC-GeoRAS requires a stream centerline shapefile to be used for assigning river stationing to cross sections and to display as a schematic in the HEC-RAS Geometric editor.[45] The initial stream centerlines were taken from the TSARP stream file that contains all the significant streams, bayous, and ditches in the Clear Creek watershed. The stream centerlines for studied streams were modified to match aerial photographs, as well as flow paths and contours created from the LiDAR data. For the purposes of HEC-GeoRAS, the creek centerline was drawn from upstream to downstream. Using HEC-GeoRAS, the River Name and Reach Name were added to the shapefile.

Cross Section Cut Lines

A cross section cut line shapefile is required by HEC-GeoRAS in order to extract cross section data from the DTM. HEC-GeoRAS overlays the cross section cut line shapefile on top of the terrain data and user-defined shapefiles to extract cross section elevation data and other attributes. Cross section locations are user-defined and their layout follow guidelines set in the HEC-RAS Hydraulic Reference Manual. For instance, the models require cross sections at the upstream and downstream faces of a bridge, as well as an upstream section based on a 1:1 contraction of flow and a downstream section based on a 2:1 expansion of flow.

Main Channel Banks

The HEC-RAS main channel banks locate the divide between main channel flow and flow in the overbanks. Although these bank stations do not affect hydraulic calculations (assuming horizontal variations in n-values), they do affect the hydrologic parameter, Percent Channel Conveyance (DCC).

[45] Ibid

Bank stations should be placed based upon the "Hydrology for Harris County" definition of channel conveyance, which states, "the conveyance of a channel is interpreted to be the capability to carry runoff in an area of uniform high velocity."[46]

The GIS shapefile for the main channel banks was created for use with HEC-GeoRAS. The channel banks were created as two lines representing the right and left channel banks. The banks were drawn from upstream to downstream and followed on either side of the region that was identified as the main channel. Using the main channel banks and channel cross section cut lines, HEC-GeoRAS was able to define the left and right bank station for each HEC-RAS cross section. The bank stations defined by HEC-GeoRAS were manually refined within HEC-RAS, as necessary.

Flow Path Centerlines
The purpose of the flow path centerlines is to locate the hydraulic flow path of the left overbank, main channel, and right overbank. Just like the stream centerline, the flow path centerlines were drawn in a GIS line shapefile from upstream to downstream in the direction of flow. The channel flow path followed the stream centerline, while the left and right overbank flow paths followed an assumed overbank center of mass.

Levees
HEC-GeoRAS uses the Levee Alignment option to create linear features that obstruct the lateral flow of water in the floodplain. This is an optional component in HEC-RAS modeling. The studied streams do not have any levees.

Land Use (Manning's "n" Values)
HEC-GeoRAS is able to assign Manning's n-values to cross sections using a cross section cut line and land use shapefile. A land use polygon shapefile was created to map the various overbank Manning's n-values within the limits of the HEC-RAS model. Channel n-values were assigned manually within HEC-RAS. Manning's n-values were assigned utilizing aerial photos and field reconnaissance.

Ineffective Flow Areas
Areas of a cross section that do not actively convey flow are called ineffective flow areas. HEC-GeoRAS can extract ineffective flow region limits from an ineffective GIS theme. Ineffective flow regions were initially mapped in GIS and refined within HEC-RAS to reflect the effects of bridges and overbank areas that do not convey flow. Ineffective flow areas around bridges assume a 1:1 contraction and 2:1 expansion of flow.

Storage Areas
Storage areas, which are typically used during unsteady-flow analysis to model flood plain storage, or off-line detention ponds can also be created through HEC-GeoRAS. Since any off-line or online detention was modeled directly in HEC-HMS, no storage areas were required for hydraulic modeling.

3.2.1.3 Model Refinement

The previous section detailed Dannenbaum's effort to generate basic HEC-RAS geometry files through HEC-GeoRAS. The geometry files generated by HEC-GeoRAS did not contain bridge data and required adjustment of attributes such as bank station locations, flow paths, n-value locations, ineffective flow areas, and expansion/contraction coefficients. This section will describe some of the HEC-RAS geometry file attributes that were modified in an effort to finalize the HEC-RAS geometry files of the studied streams.

[46] American Society of Civil Engineers and others "Hydrology for Harris County" (March 1988): C-9.

Main Channel Banks

HEC-GeoRAS was utilized to generate bank station locations for each of the HEC-RAS cross sections. HEC-GeoRAS often did not place bank stations in the desired location, even though the bank station shapefile appeared to be correct. When this occurred, the HEC-RAS bank stations were manually adjusted in HEC-RAS.

Flow Path Centerlines

Flow path centerlines were initially defined as explained in the previous section; however, the left and right overbank flow paths were refined after developing preliminary flood boundaries.

Manning's "n" Values

As with bank stations, the n-values generated by HEC-GeoRAS did not always fall at the correct location within the HEC-RAS cross sections. Minor adjustments were made within HEC-RAS to correct n-values. In addition, channel n-values were modified within HEC-RAS to match field conditions.

Bridges

Bridge data was taken from two sources: the effective hydraulic model and field reconnaissance (which included limited survey to verify/replace what was in the previous models). Since many of the bridges within the study reaches have been replaced since the effective hydraulic studies, a comparison of the effective HEC-2 bridge data was made against field reconnaissance and LiDAR extracted bridge data. The comparisons included bridge deck profiles, deck widths, deck thickness, abutment widths, pier widths and pier shapes. In cases that the effective bridge data closely matched measurements collected in the field, the effective bridge data (after adjustment to the 2001 datum) was used in the updated models.

It should be noted that the roadway surface of the bridge was not always considered the "bridge deck" in HEC-RAS. If some other feature, such as the bridge railing or a roadway barrier, provided a significant blockage to flow, the tops of these barriers were considered the top of the bridge deck.

Skew

In some cases, bridge and culvert crossings are not perpendicular to the channel or overbank flow. Often the bridge sections and in some cases, the bounding cross sections for these structures are not cut perpendicular to the channel but cut parallel to the structure. In this case a skew adjustment was applied to the bridge opening, bounding cross sections or both.

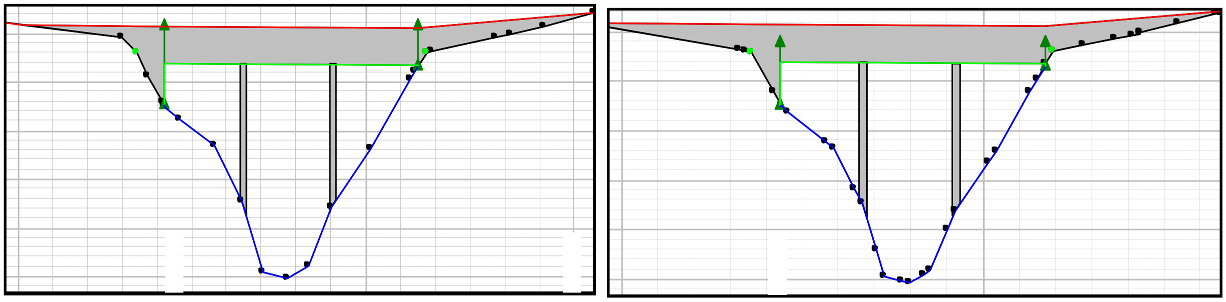
It should be noted that bridges with skew angles less than 20 degrees did not have a skew angle applied. Based upon recommendations in the HEC-RAS Hydraulic Reference Manual, skew factors do not need to be calculated for cross sections with skew angles less than 20 degrees.[47]

Ineffective Flow Areas

As previously mentioned ineffective flow areas extracted from HEC-GeoRAS were manually refined in HEC-RAS. The majority of the ineffective flow areas were a result of bridge or culvert crossings. Ineffective flow areas were placed at the upstream bridge cross sections and defined areas outside the bridge opening as ineffective. Figure 3.2 shows that flow outside the bridge opening was made ineffective until flood stages reached the minimum deck elevation of the bridge. The downstream bridge section also had areas outside the bridge opening made ineffective; however, these areas became effective once flood stages reached an elevation halfway between the bridge's low chord and the deck elevation, as seen in Figure 3.2(b). The horizontal limits of the ineffective areas were based upon a 1:1 contraction and 2:1 expansion of effective flow limits.

[47] US Army Corps of Engineers – "HEC-RAS River Analysis System Hydraulic Reference Manual" (2002): Page 3-19.

Figure 3.2
Bridge Ineffective Flow Areas



(a) Upstream Bridge Face (b) Downstream Bridge Face

Expansion and Contraction Coefficients
Expansion and contraction coefficients are used by HEC-RAS to determine the energy losses due to transitions in a cross section area[48]. The transition losses result from bridges, drop structures, channel modifications or any other changes in cross section area. The Harris County Flood Control District Design Manual (May 1994) recommends the following coefficients to approximate transition losses[49]:

Type of Transition	Contraction	Expansion
No loss	0.0	0.0
Gradual	0.1	0.3
Bridge Section	0.3	0.5
Abrupt	0.6	0.8

HEC-RAS uses the default contraction and expansion coefficients of 0.1 and 0.3, respectively. The HEC-GeoRAS generated geometry file does not modify expansion and contraction coefficients to reflect bridge crossings or other sudden transitions in cross sectional area. These coefficients were manually modified to reflect bridges and other transitions during the refinement of the geometry file.

Typically, the default contraction coefficient was changed to 0.3 and the default expansion coefficient was changed to 0.5 at two sections immediately upstream and one section immediately downstream of a bridge. However, if a structure proved to be only a minor impedance to flow, the expansion/contraction coefficients were set at 0.2 and 0.4, respectively.

3.2.1.4 Flood Hazard

Once the HEC-RAS geometry file was checked and refined, a HEC-RAS flood hazard models were developed for the studied streams. In order to run a HEC-RAS flood hazard model, boundary conditions were set and flows were extracted from each watershed's HEC-HMS model. The following section describes this process.

Boundary Conditions
The downstream reach boundary condition for Cowart Creek and its tributaries were defined utilizing a normal depth slope. Each channel's normal depth slope was determined from the average channel slope at the mouth of the channel.

Peak Flows

[48] US Army Corps of Engineers – "HEC-RAS River Analysis System Hydraulic Reference Manual" (2002): Page 3-19.
[49] Harris County Flood Control District "HEC-2 Modeling and Submittal Guidelines (An Insight to HCFCD Review Procedures)" May 1994: Chapter 3, Section III.E.

The peak flows used for the flood hazard model were taken from the watershed hydrologic (HEC-HMS) model created by Dannenbaum. The model created flows for the 50%, 20%, 10%, 2%, 4%, 1%, and 0.2% exceedence events, as discussed in Section 3.1.2.

HEC-HMS node peak flows were associated to a HEC-RAS cross section. Flows at cross sections between HEC-HMS nodes were interpolated from based upon stationing and a semi-log relationship. In cases that the HEC-RAS model extended upstream of most upstream HEC-HMS node, peak flows were extrapolated for these portions of the model. The flows were extrapolated from the overall basin flow based on contributing area, development, and channel slope for the overall basin.

3.2.2 Results

The results of the existing conditions hydraulic analysis were reviewed for several purposes. First, the updated existing conditions flood stages were compared against the results of previous studies[50] to verify the reasonableness of the updated peak flood stages (see Section 3.2.2.1). Secondly, updated floodplains, flood profiles and estimated structural flooding were reviewed to better identify problem areas within the watershed. This section will discuss these reviews.

Results Verification

Table 3.10 compares the 10%, 2%, 1% and 0.2% peak flood stages of CW100-00-00 and CW104-00-00 between a previous study and the updated existing hydraulic model. This table shows that the updated hydraulic model increases Cowart Creek 1% event peak flood stages by an average of 0.23 ft from the previous hydraulic study. CW104-00-00 reflects less dramatic changes between the updated and previous hydraulic studies. The updated hydraulic model decreases 1% peak flood stages by an average of only 0.02 feet when compared to the previous hydraulic study for CW104-00-00.

Significant changes in peak flood stages were not unexpected for CW100-00-00, as the previous hydraulic study for CW100-00-00 reflects overflow data this is in many cases over 15 years old. In addition, hydraulic and hydrologic studies of several previously unstudied tributaries affect peak flows on the main channel of CW100-00-00. Conversely, only minor changes were expected in the peak flood stages of CW104-00-00, as very few changes were made to the updated hydraulic model when compared against the previous hydraulic model.

Existing conditions HEC-RAS output may be found in Appendix B.

Water Surface Elevation Profiles

Frequent overbank flooding is not necessarily a problem, assuming that structural flooding does not occur. Therefore, channel capacity should not be the sole criteria used to identify existing flooding problems. Other methods, such as review of flood profiles, should be employed. Review of flood profiles allow stream crossings that cause significant headloss to be identified. When structures (buildings) are plotted against flood stage profiles, it allows for the identification of flow restrictions in channels that can potentially cause structural flooding. Exhibits 3.6 though 3.12 show the existing 10% and 1% exceedence event flood profiles for the studied streams within the Cowart Creek watershed. The exhibits also include approximate structure (buildings) locations that were used in the development of the inundation analysis, which is discussed later in more detail.

Level of Service

A Level of Service Analysis identifies, by stream segment, the largest storm event that the stream is able to convey without flooding overbank areas. Exhibit 3.13 shows the various levels of service that may be expected within the Cowart Creek watershed. The stream segments that show a "< 50% Exceedence"

[50] Dannenbaum Engineering Corporation "Clear Creek Watershed Modeling Update – Hickory Slough, Mary's Creek and Cowart Creek" 2006.

level of service can expect to see overbank flooding (not necessarily structural flooding) during a 50% exceedence event. Almost the entire length of Cowart Creek (CW100-00-00) upstream of the AT&SF Railroad has a level of service less than 50% exceedence. This low channel capacity is also shown at several locations along CW103-00-00. CW100-00-00, CW102-00-00, CW103-00-00, CW103-01-00, CW103-02-00, CW104-00-00 and CW105-00-00 all have at least one channel segment with an existing 50-20% exceedence capacity.

Approximate Floodplain

The updated flood stages for the studied streams of the Cowart Creek watershed were used to develop approximate flood plains for the 10% and 1% exceedence events. The updated exiting flood plains for the 10- and 100-year events may be found in Exhibit 3.14 and show, as in the effective FIRM panels, that the 100-year event flood plain is very large upstream of the AT&SF Railroad. It should be noted that flood plains shown in the exhibit are not intended to be FEMA quality flood plain delineations. However, they are intended to help identify deficiencies in existing drainage systems.

Structural Inventory and Inundation Analysis

Dannenbaum created a structural inventory and performed an inundation analysis for the watershed. The structural inventory is database that was created to include a point in GIS for each home within the 0.2% event flood plain of each studied stream of the Cowart Creek watershed. The database relates each structure to a HEC-RAS stream station of a studied stream. A slab elevation was associated to each structure based upon the available LiDAR data and an assumed slab adjustment of 0.5 feet. An inundation analysis was performed using these structural inventory databases and the HEC-RAS model results for the Cowart Creek watershed. The inundation analysis identified the event at which each structure could potentially be inundated by floodwater by comparing assumed slab elevations with HEC-RAS flood stages.

The results of the inundation analysis, which identify structures that are potentially flooded during the 50%, 20%, 10%, 2%, 4% and 1% exceedence events, may be seen in Exhibit 3.15. As seen in Table 3.11, there are 41, 234 and 530 structures that could potentially be inundated by the 50%, 10% and 1% exceedence events, respectively. The Exhibit 3.15 shows that a majority of the potentially inundated structures are located upstream of the AT&SF Railroad on CW100-00-00, CW103-00-00 and CW103-01-00. Severe structural flooding is also expected on CW105-00-00.

3.3 Conclusions

Based upon the results of the existing conditions hydrologic and hydraulic analysis, problem areas within the Cowart Creek watershed were identified. These problems areas will form the focus of the proposed improvement plan (Section 4.0). Areas needing improvement or that presented challenges to flood mitigation were recognized through analysis of the results outlined in Sections 3.1.2 and 3.2.2. In addition, input from team members and from stakeholders through public meeting produced additional information about localized flooding conditions in the watershed not identified through macro-level modeling. Note that focus of this study is to propose drainage improvements to reduce the occurrence of structural flooding. Therefore, Exhibit 3.14 may overlook drainage problems that do not induce structural flooding.

Exhibit 3.16 displays (in green) the prevalence of overflows in the upstream reaches of CW100-00-00 and CW102-00-00, CW103-00-00, CW103-01-00 and CW103-02-00. These overflows are caused by the fact that these channels are generally man-made upstream of the AT&SF Railroad and often do not follow natural drainage patterns. A general lack of channel capacity results in overflows between tributaries that cascades to the southeast.

As noted in Exhibit 3.13, there are channel reaches that suffer low level of service that do not have capacity to convey the 1% exceedence event. Existing conditions capacity limitations were identified by evaluating hydraulic models as well as reviewing level of service. Exhibit 3.16 identifies the channel segments that have the lowest channel capacities to potentially induce structural flooding. Generally, the main channel of Cowart Creek (CW100-00-00) has its lowest capacities upstream of SH 35 and near the Brazoria/Galveston county line. Significant channel capacity deficiencies are also present on CW103-00-00 upstream of SH 35. Finally, CW101-00-00 has channel capacity issues that are primarily the result of poor channel grading.

Drainage structures, such as bridge and culvert crossings, can be the cause of the low drainage system capacities shown in Exhibit 3.13. Table 3.12 shows the bridge/culvert crossings that create a 1% exceedence event head loss greater than 0.5 feet. However, structures that do not induce structural flooding may not be considered a problem. To identify problem structures, approximate structure locations were plotted against flood profiles (Exhibits 3.6 to 3.12) to identify bridge/culvert crossings that cause both significant headloss and structural flooding. Depending on circumstances, these crossings may prove to be good candidates for replacement. It should be noted that even though the 1% exceedence event flood plain is dramatically larger upstream of the AT&SF Railroad than downstream, this flood plain increase is not caused the AT&SF Railroad crossings, but by a lack of channel capacity upstream of the railroad.

In addition to the previously mentioned drainage problems, the existing conditions analysis also identified potential limitations to proposed drainage improvements. As seen in Exhibit 3.16, the oil and gas field located between SH 35 and the AT&SF Railroad will hinder any channel improvements in the area due to the number of pipelines that cross through the existing CW103-00-00 and CW102-00-00 channel alignments. Channel improvements in this area would be expensive due to the high cost of relocating these pipelines. Construction of channel improvements along CW100-00-00, CW101-00-00, CW101-01-00 and CW105-00-00 will also be challenging due to dense development within the Friendswood area.

The following section will propose drainage improvements to address the flooding problems identified in this section.

4.0 PROPOSED CONDITIONS

As discussed in Section 3.0, the existing conditions analysis of this study identified areas within the Cowart Creek watershed that should be considered problem areas. It also identified specific causes (deficient bridge/culvert crossings, overflows, undersized channels, etc.) of flooding. This section will discuss the conveyance improvements and regional detention facilities that are proposed to alleviate flooding within these problems areas. Reduced flood stages, the reduced occurrence of structural flooding, as well as the associated construction cost estimate for the proposed flood reduction plan will also be discussed.

4.1 Flood Improvement Components

In general, flooding concerns are greatest in areas upstream of the AT&SF Railroad where low capacity man-made channels go against natural flow patterns. Flooding concerns along these channels are exacerbated by extreme event overflows that cascade from the Cowart Creek to some of Cowart Creek's tributaries to the south. This plan will improve the conveyance capacity of Cowart Creek to reduce or remove existing overflows from the main channel of Cowart Creek. Regional detention will be part of the plan in order to make sure that the removal of the existing overflows does not impact downstream areas. Regional detention will also be proposed to provide flood reduction benefits for the Friendswood area, since conveyance improvements in the Friendswood area may not be feasible due to limited right-of-way or the environmental sensitivity of certain stream reaches. The Cowart Creek Watershed Master Plan will also incorporate the drainage improvements of the Bailey Road Drainage Improvement Project (Section 4.1.1). Finally, flood improvements will be proposed for the Clover Acres area to relieve flooding concerns on CW101-00-00.

This section will discuss each of the individual drainage improvements that comprise the proposed improvement plan of the Cowart Creek Watershed Master Plan (see Exhibit 4.1).

4.1.1 <u>Bailey Road Drainage Improvements</u>

Dannenbaum was tasked with including a future drainage improvement project that will be constructed by the City of Pearland in the proposed drainage improvements of this Master Plan. The Bailey Road Drainage Improvement Project, as seen in Exhibit 4.2, is currently in development by JKC Associates. This project will convey runoff from the upper reach of Cowart Creek (from Manvel Rd to a point downstream of Harkey Road) within a box culvert system. In addition to the conveyance improvement, areas upstream of the improvement will have its runoff diverted to CW103-01-00. This diversion would undoubtedly cause flow impacts on CW103-01-00 due to the increase in contributing drainage area. However, Detention A (Bailey Road Detention) and Detention D, will be constructed to mitigate any potential flow increases on CW100-00-00 and CW103-01-00.

Please note that the box culvert component of the Bailey Road Drainage Improvement Project was not included in Dannenbaum's proposed conditions modeling, as the final configuration of the box culvert system was not determined at the point Dannenbaum was simulating proposed conditions. Cowart Creek was assumed to have its existing conveyance capacity upstream of Veteran's Road, since the majority of the flood reduction benefits and downstream flow increases would be developed by the proposed diversion channel. The hydrologic effects of the diversion channel were simulated as part of this study.

4.1.2 CW100-00-00 Improvements

A trapezoidal channel improvement is proposed for the main channel of Cowart Creek between SH 35 and Veteran's Drive. The channel improvement will consist of a 6 ft bottom width earthen channel (4:1 side slopes) from SH 35 to the AT&SF R.R., as well as a 15 ft bottom width lined channel (3:1 side slopes) from the AT&SF R.R. to upstream of Wells Drive. Upstream of Wells Drive the channel improvement is a 15 ft bottom width earthen channel (4:1 side slopes) until Veteran's Drive. The proposed flowline of the channel improvement is shown in Exhibit 4.5. This improvement is intended to produce flood stage reductions on the main channel of Cowart Creek and, as a result, remove Overflow #2, as seen in Table 4.5. The reduction in overflow to CW103-00-00 during extreme events will produce direct flood stage reductions on CW103-00-00 and its tributaries.

In addition to the channel improvements on CW100-00-00 Dannenbaum proposes either the removal or replacement of several stream crossings, as listed in Figure 4.1. As seen in Exhibit 4.1, removal of the oil trap upstream of Greenbriar Avenue, as well as two small culvert crossings upstream of SH 35 is included in the proposed drainage improvement plan. These structures were removed by the local drainage districts prior to this study.

Figure 4.1
Crossing Improvements for CW100-00-00

Name	Existing Conditions	Proposed Conditions
C.R. 115	Replaced Bridge	
Unnamed Rd#2	Replaced Bridge	
A.T. & S.F. Railroad	Replaced Bridge	
Shell Road #4	Replaced Bridge with Inline Structure	
Unnamed Rd #1	Removed Bridge	
FM 2351	Expanded Bridge	
Oil Trap	Removed Trap	

Expansion of the FM 2351 bridge is also included in the proposed drainage improvement plan. Although significant headloss is not shown for this bridge during the 1% exceedence event, there is a 0.47 ft. headloss at this bridge for the 4% exceedence event. Expansion of the existing FM 2351 bridge and the subsequent upstream peak flood stage reductions are necessary to ensure that the overall drainage improvement plan has no negative impacts. It was confirmed by the Brazoria Drainage District #4 (BDD4) that the bridge at FM 2351 was designed and constructed to be expanded, if necessary.

The AT&SF Railroad and the Wells Road bridges will need to be replaced. These replacements are not because of a lack of capacity, but because of the significant changes to channel configuration in the area. The existing channel flowline is being deepened by approximately 3 feet under proposed conditions, which could potentially compromise the structural stability of the existing crossings if not replaced. Please note that for the proposed hydraulic simulations, the existing configuration of the AT&SF and Wells Road bridges were maintained. Only the channel configuration beneath the bridges was modified.

In addition to channelization, several regional detention facilities will be proposed to mitigate potential flow impacts that may be induced by the Cowart Creek channelization. The regional detention facilities will also provide flow and flood stage reductions for downstream areas. Detention A, which is also known as the Bailey Road Detention, had been partially constructed. However, there are plans to expand the detention facility in order to provide storage volume for future development and to reduce existing peak flows on Cowart Creek. From construction plans, Dannenbaum determined that the ultimate Detention A (Bailey Road) Regional Detention Facility will provide a total of 333 ac-ft of storage, with approximately 216 ac-ft of this storage available for reduction of existing flood stages. Therefore, the Bailey Road Detention Facility was simulated with 216 ac-ft of storage in the proposed drainage improvement plan.

In addition to the Bailey Road Detention Facility (Detention A) Dannenbaum simulated Detention B (217 ac-ft) and Detention C - North (223 ac-ft) to utilize the land that BDD4 was in the process of acquiring. These detention facilities are needed to mitigate the increased peak flows developed by the Cowart Creek channel improvements and the removal of OVFL#2. Detention B will function as an offline detention facility and will outfall back into CW100-00-00. However, the north cell of Detention C will divert flow from CW100-00-00 and ultimately release it into CW103-00-00. This diversion of flow is required to prevent downstream flow impacts on CW100-00-00 and to mimic the effects of the existing overflow OVFL#2.

An expanded version of the existing Bailey Road Detention Facility (Detention E) was incorporated into the proposed conditions modeling. As shown in Exhibit 4.3 Detention E has a total of 805 ac-ft of storage, which is required to prevent downstream flood stage impacts and to provide additional flood stage benefits to the Friendswood area.

4.1.3 CW101-00-00 Improvements

As outlined in Section 3.0, CW101-00-00 has conveyance capacity deficiencies, primarily upstream of Tall Pines Drive. These capacity issues have been confirmed by residents, as they have stated that storm runoff tends to pond excessively upstream of FM 2351. This ponding, which in the past has impacted structures, is due to poor channel grading upstream of Greenbriar Drive. Even though not shown in the existing stream profile (Exhibit 3.7), the flowline of existing culverts under FM 2351 is approximately 22.9 feet. This is approximately 0.4 feet lower than the flowline of the Pine Hollow culvert crossing that is 560 feet downstream. The poor channel grading reduces the effective opening area of the 4-6x4 RCB under FM 2351.

It is proposed that a channel improvement be constructed in order to improve drainage upstream of FM 2351. The channel improvement, which is detailed in Exhibit 4.3, is needed to improve channel grading so that the FM 2351 culvert crossing will function more efficiently, especially under more frequent storm events. In addition to regrading CW101-00-00, it is proposed that the Cowards Creek Drive, Tall Pines Drive, Greenbriar Drive and Pine Hollow Drive crossings be revised as shown in Figure 4.2. The profile of the channel improvements is shown in Exhibit 4.6.

Figure 4.2
Crossing Improvements for CW101-00-00

Name	Existing Conditions	Proposed Conditions
Pine Hollow	1 - 12' X 4' Box Culvert	2 - 10' X 6' Box Culverts
Greenbriar	1 - 9' X 4' Box Culvert	2 - 10' X 6' Box Culverts
Tall Pines	2 - 9' X 4' Box Culverts	3 - 10' X 6' Box Culverts
Cowards Creek Dr	2 - 10' X 6' Box Culverts	3 - 10' X 6' Box Culverts

Approximately 0.47 mi^2 drains to the culvert crossing under FM 2351. This is a large drainage area, especially considering that upstream of FM 2351 CW101-00-00 is essentially a roadside ditch. A similar situation exists for CW101-01-00 where approximately 0.35 mi^2 drains to its culvert crossing of FM

2351. To increase the capacity of the CW101-00-00 and CW101-01-00 crossings under FM 2351 would be expensive and would likely require significant downstream channel improvements in areas were right-of-way is limited. Therefore, Dannenbaum is proposing a diversion culvert, as seen in Exhibit 4.3. The diversion culvert system, which have 2-7x7 RCB at its downstream limit, will divert approximately 0.23 mi^2 and 0.12 mi^2 of contributing drainage area away from CW101-00-00 and CW101-01-00, respectively. In addition to reducing downstream flows on CW101-00-00 and CW101-01-00, the diversion culvert will provide additional outfall depth for roadside ditches along County Road 125, Westfield Lane and Rustic Lane. Because the culvert will divert runoff to Cowart Creek upstream of its original outfall location, peak flow impacts could potentially developed on Cowart Creek. Therefore, Detention F (121 ac-ft) is proposed to mitigate any impacts developed because of the diversion. The proposed diversion culvert will outfall directly into Detention F, which will outfall into Cowart Creek just downstream of CR 130.

4.1.4 Channel Improvement (CW103-00-00)

Previous analysis showed that the upstream portion of CW103-00-00 receives overflows from CW100-00-00 during extreme storm events. As discussed in Section 3.2, CW103-00-00 can't efficiently convey its runoff, as well as the overflows from CW100-00-00. Therefore, an earthen channel improvement with varying bottom widths was proposed for CW103-00-00 from Harkey Rd to Veterans Drive. Along this path five (5) bridge and culvert structures will have to be replaced(as shown in Figure 4.3) to accommodate the increased channel capacity. This channel improvement will have the added benefit of reducing the magnitude of the existing overflow to CW103-01-00. A profile of the improved CW103-00-00 may be seen in Exhibit 4.8.

Figure 4.3
Crossing Improvements for CW103-00-00

Name	Existing Conditions	Proposed Conditions
Harkey Rd	Replaced Bridge	
Unnamed Rd #7	Replaced Bridge	
Moore Rd	Replaced Bridge	
A.T. & S.F. Railroad	Replaced Bridge	
Unnamed Rd #3	Replaced Bridge	

As mentioned in Section 4.1.1, Detention D will provide mitigation for the Bailey Road drainage improvements and as a result, was included in the proposed conditions modeling. The facility, which will have a wet bottom and will utilize a pump, is comprised of two (2) cells with a total storage volume of 1003 ac-ft. Detention D will divert flow from CW103-00-00 and outfall to CW103-01-00. As discussing in Section 4.1.5, Detention D will also divert flow from CW103-01-00.

The north and south cells of Detention C will be utilized as inline detention for CW103-00-00. As seen in Exhibit 4.2, the detention configuration allows the portion of CW103-00-00 that lies adjacent to the AT&SF railroad and the reach that intersects the oilfields to be bypassed. This bypass provides additional depth for upstream reaches of CW103-00-00 and avoids channel improvements within the oilfield area. Detention C (South Cell) will provide 531 ac-ft of storage and will require an 800 ft long earthen outfall channel (6 ft bottom width and 4:1 side slopes) to outfall into the existing CW103-00-00 channel.

4.1.5 Channel Improvement (CW103-01-00)

The Bailey Road improvements, as simulated by JKC & Associates and discussed in detail in Section 4.1.1, improves CW103-01-00 to just upstream of Wells Drive and utilizes Detention E as mitigation. Dannenbaum proposed an earthen channel improvement (15 ft bottom width and 4:1 side slopes) to CW103-01-00 from the AT & SF Railroad to Veteran's Drive (see Exhibit 4.9). As seen in Exhibit 4.2, Dannenbaum's modeling assumes that conveyance improvements were made to CW103-01-00 upstream of Veteran's Drive, even though their hydraulic effects were not simulated as part of this analysis. Within the channel improvement reach four (4) culvert crossings (listed in Figure 4.4) are proposed to be replaced with larger structures to accommodate the increased flows from upstream.

Figure 4.4
Crossing Improvements for CW103-01-00

Name	Existing Conditions	Proposed Conditions
Unnamed Rd #9	Replaced Bridge	
Bounds Dr	Replaced Bridge	
Frazier Ln	Replaced Bridge	
Amoco Dr	Replaced Bridge	

As mentioned in Section 4.1.4, CW103-01-00 will divert part of its flow into Detention D. Detention D will outfall back into CW103-01-00 just upstream of Wells Dr. The combination of channelization and detention will in turn provide ancillary benefits to CW102-00-00 in the form of reduced overflows. Improvements on channels north of CW102-00-00 were the only feasible way to mitigate flooding in this extremely low-lying region where detention was impractical.

4.1.6 Channel Improvement (CW105-00-00)

This tributary has limited channel right of way, and as a result, there were few options for channel improvements. However, as shown in Table 3.12, existing culvert crossings were shown to be the largest restriction upon CW105-00-00's conveyance capacity. Therefore, as shown on Exhibit 4.3 and Figure 4.5, Dannenbaum proposes that the culvert crossing at Shadowbend and Spreading Oaks be replaced. The proposed flood profiles for CW105-00-00 may be seen in Exhibit 4.11.

Figure 4.5
Crossing Improvements for CW105-00-00

Name	Existing Conditions	Proposed Conditions
Shadow Bend	2 – 36" RCP's	2 - 7' X 3' RCB's
Spreading Oaks	3 – 42" RCP's	3 - 6' X 3' RCB's

4.2 Hydrology

This section outlines the proposed conditions hydrology that were needed to reflect the drainage improvements covered in Section 4.1.

4.2.1 Model Parameters

The various drainage improvements proposed for the Cowart Creek watershed required that a proposed conditions hydrologic model be developed from the existing conditions hydrologic model. The creation of the proposed conditions model required an update to Clark Unit Hydrograph parameters, Modified Puls storage-routing relationships, overflow relationships, as well as the addition of regional detention ponds to the hydrologic model.

Watershed parameters were changed to develop proposed conditions Clark Unit Hydrograph parameters. The changed watershed parameters included channel slope (S), channel improvement percentage (DCI) and channel conveyance percentage (DCC). The proposed channel slope and channel conveyance percentage values may be found in Table 4.1. The proposed channel improvement percentages may be found in Table 4.2. Table 4.3 details the proposed Clark Unit Hydrograph parameters that were applied to the proposed conditions hydrologic model.

The proposed channel improvements not only affected the Clark Unit Hydrograph parameters of the model, but also change storage routing relationships. The proposed conditions storage routing relationships may be found in Table 4.4.

Exhibit 4.4 demonstrates the influence that the proposed conveyance improvements have on reducing overflows. Compared to existing condition overflows shown in Exhibit 3.5, the proposed overflows are greatly decreased or removed altogether. Overflows will also occur less frequently and with less intensity under proposed conditions. For example, Overflow #3 sees a 40% decrease in the 1% exceedence event peak overflow when compared to existing conditions. The reductions or removal of overflows (shown in Table 4.5) lead to lower peak flows along CW102-00-00, even though no drainage improvements were proposed specifically for this channel.

4.2.2 Results

As shown in Table 4.6, proposed conditions 1% exceedence event node peak flows on Cowart Creek (CW100-00-00) are generally lower than those of existing conditions. The largest peak flow reduction for the 1% exceedence event occurs downstream of the proposed JKC diversion. At node CW100#2 there is a 209 cfs (50.8%) reduction in the 1% exceedence event peak flow. There are several locations where the 1% exceedance event peak flow is increased. For instance, at node CW100#3d, the 1% exceedence event peak flow is increased by 75.6%; however, as discussed later, this flow increase is offset by the reach's channel improvements. Ultimately, the 1% exceedence peak flow is reduced by 13.1% (857 cfs) at the mouth of Cowart Creek.

Results are similar for the tributaries of Cowart Creek. The 1% exceedence event peak flow is decreased by a maximum of 42.5% on CW101-00-00. CW102-00-00 has a maximum decrease of 23.6%, due to the reduction in overflows from CW103-01-00 and CW103-00-00. Outside of the diversion reach, CW103-00-00 has a maximum decrease in the 1% exceedence event peak flow of 65.7%, while CW103-01-00 has a maximum decrease of 19.2%.

There are some locations where the 1% exceedence event peak flow increases. However, at these locations the peak flow increases were typically offset by channel conveyance improvements. In the case of the peak flow increase at the mouth of CW103-00-00 the 1%, the increased flood stages were contained within channel banks.

Proposed conditions HEC-HMS output may be found in Appendix A.

4.3 Hydraulics

The conveyance improvements laid out in the proposed plan were reflected in the HEC-HMS hydrologic model by updating watershed parameters connected to storage routing (SV/SQ) and conveyance (DCC) as mentioned in Section 4.2.1. The process resulted in new hazard flows that became the input for the proposed hydraulics model which reflected improvements to the channel geometry and crossings.

4.3.1 Model Development

The proposed conditions hydraulic model was created from the existing conditions HEC-RAS model. First, the channel geometry was modified to reflect the channel improvements, bridge improvements and culvert improvement that were discussed in Section 4.1. The proposed hydrologic model (HEC-HMS) results were used to develop proposed condition flood hazard flows for the 50%, 20%, 10%, 4%, 1% and 0.2% exceedence events. The proposed conditions flood hazard flows were applied to the proposed conditions HEC-RAS geometry for the studied streams to develop proposed conditions flood stages.

4.3.2 Results

This section will discuss the benefits of the proposed drainage improvements for the Cowart Creek watershed in terms of flood stage reduction, level of service, flood plain limits and in terms of numbers of flooded structures. Section 4.4 will describe the benefits of the proposed plan in terms of damages avoided.

Tabular Flood Stage Comparison

As seen in Table 4.7, the proposed drainage improvement plan reduces 1% exceedence event flood stages by an average of 0.82 ft on Cowart Creek. The largest flood reductions are located just downstream of the proposed Bailey Road diversion where the 1% exceedence event is reduced by 3.54 ft and just upstream of Wells Road, where the 1% exceedence event peak flood stage is reduced by 2.49 ft. No flood stage increases on Cowart Creek are induced by the proposed improvements.

CW101-00-00 and CW101-01-00 both experience significant reductions in 1% exceedence event peak flood stages. The average reduction is 1.73 ft and 1.5 ft for CW101-00-00 and CW101-01-00., respectively. The maximum peak 1% exceedence event flood stage reduction is 3.2 ft and 1.86 feet for CW101-00-00 and CW101-01-00, respectively. No flood stage increases are induced by the proposed improvements on either CW101-00-00 or CW101-01-00.

CW102-00-00 has an average 1% exceedence event peak flood stage reduction of 0.21 ft and a maximum reduction of 0.47 ft upstream of the AT&SF Railroad. These flood stage reductions will occur with no channel or bridge improvements to CW102-00-00.

Peak 1% exceedence event flood stages on CW103-00-00, CW103-01-00 and CW103-02-00 are reduced by the proposed drainage improvement plan by an average of 1.07 ft, 0.08 ft and 0.24 ft, respectively. These streams experience maximum flood stage reductions of 2.4 ft, 0.86 ft and 0.8 ft for CW103-00-00, CW103-01-00 and CW103-02-00, respectively. A maximum peak flood stage increase of 0.13 feet can be expected near the mouth of CW103-00-00; however, this increase is contained within channel banks.

CW104-00-00 experiences an average reduction of 0.15 ft in the 1% exceedence event peak flood stage. The maximum reduction in the peak 1% exceedence event flood stage is 1.66 feet and occurs at the mouth of CW104-00-00. It should be noted that no improvements were proposed on CW104-00-00. However, due to the CW104-00-00 hydraulic model being merged with the CW100-00-00 hydraulic model, CW100-00-00 backwater reductions are reflected in CW104-00-00 flood stages.

CW105-00-00 experiences a maximum reduction in the 1% exceedence event peak flood stage of 1.07 feet and an average reduction of 0.18 feet.

Proposed conditions HEC-RAS output may be found in Appendix B.

Water Surface Elevation Profiles and Level of Service

Table 4.7 shows a significant drop in peak flood stages that can be expected from the proposed Master Plan improvements. Exhibits 4.5 through 4.11 display the flood stage reductions in a profile view. These exhibits provide more detail about the relative change in flood stage due to channelization or structural (i.e. culvert) replacement. In addition, other important information is displayed, such as the location of structures (homes) and the limits of channel improvements.

The proposed condition level of service (LOS) for the studied streams within the Cowart Creek watershed are shown in Exhibit 4.12. By comparing the proposed LOS results against the existing LOS shown in Exhibit 3.13, the benefits of the proposed drainage improvement plan are evident. The majority of the benefits to the watershed are located upstream of the AT&SF Railroad where significant reaches of CW100-00-00, CW103-00-00, CW103-01-00 and CW102-00-00 had an existing LOS between the 50% and 20% exceedence event. A large reach of CW100-00-00 and CW103-00-00 had an existing LOS less than the 50% exceedence event. As seen in Exhibit 4.12, the proposed drainage improvement plan improved LOS so that the minimum LOS upstream of the AT&SF Railroad for all the Cowart Creek tributaries is between the 20% and 10% exceedence event.

Approximate Floodplain

Exhibit 4.13 displays the 10% and 1% exceedence event floodplains for proposed conditions. While this remains an approximate floodplain delineation, it is clear that the 10% exceedence event floodplain (compared to existing conditions) is reduced to within the banks along most of CW100-00-00, CW103-00-00 and CW103-01-00. This leads to the near elimination of overbank flooding for the 10% exceedence event on these streams upstream of the Brazoria-Galveston County line. The expansive 1% exceedence event floodplain is substantially smaller for proposed conditions upstream of the AT&SF Railroad. However, it should be noted that the proposed conditions floodplains were not created for CW100-00-00 upstream of Veteran's Drive. In addition, the 1% and 10% exceedence event floodplains limits for CW100-00-00 within Friendswood were reduced only slightly because the existing floodplains were already contained within channel banks.

Inundation Analysis

Exhibit 4.14 shows the results of the proposed conditions inundation analysis. This exhibit identifies structures that are potentially flooded during the 50%, 20%, 10%, 2%, 4% and 1% exceedence events. As seen in Table 4.8, there are 18, 25 and 202 structures that could potentially be inundated by the 50%, 10% and 1% exceedence events, respectively. This represents a 56%, 89% and 62% reduction in inundated structures from existing conditions for the 50%, 10% and 1% events.

4.4 Benefit-Cost Analysis

The benefits of the proposed infrastructure recommendations laid out in this report need to be quantified not only in terms of flood stage reductions but also in terms of economic benefits. An economic analysis was performed to compare the benefits of the proposed plan, in terms of damages avoided, against the estimated construction cost of the proposed plan. This section outlines the economic evaluation of the proposed Cowart Creek watershed master plan.

The initial step in the economic analysis was to develop an opinion of the proposed plan's construction cost. Dannenbaum developed the construction cost estimate for the proposed plan; however, the estimated construction cost for the Bailey Road improvements was taken by a report developed by JKC &

Associates. The Bailey Road improvements include: the diversion channel and channel improvements, Detention D, and the conveyance improvements to Cowart Creek upstream of the diversion channel. Appendix C outlines the preliminary opinion of construction cost (by stream) developed by Dannenbaum. It represents the land acquisition, survey, engineering and construction costs for the channel improvements and detention facilities proposed in Section 4.1.

Figure 4.6
Dannenbaum Preliminary Opinion of Construction Cost

Item	Cost
Land Acquisition	$ 5,241,379
Channel Improvement Construction	$ 11,178,504
Detention Construction	$ 37,590,816
Engineering & Surveying (15%)	$ 6,096,165
Grand Total (Land & Construction)	**$ 60,106,864**

The preliminary project cost estimate developed by Dannenbaum is $60,106,894.00. As seen in Figure 4.6, approximately 60% of the total cost is due to the construction of detention facilities and 19% is developed by channel improvements and bridge replacements. The JKC & Associates estimated construction cost for the Bailey Road improvements is $22,526,782.00, which leads to a grand total of $82,633,646.00 for improving drainage within the Cowart Creek watershed.

Appendix D contains an in-depth benefit-cost analysis (BCA) by Jeffrey S. Ward & Associates, Inc. The memorandum covers the methodology and datasets utilized in the effort. The structural inventory and HEC-RAS results were used within the FEMA Full-Data Riverine Benefit-Cost Analysis module to determine a benefit-cost ratio. And while Appendix C dealt with the cost of construction, Appendix D compares the cost of improving drainage in the Cowart Creek Watershed to the benefit of avoided future damages resulting from implementation the master plan. The overall ratio of the master plan benefits (avoided damages) to the project's construction cost is 0.26.

4.5 Environmental Concerns

An Environmental Baseline Overview was performed by Berg Oliver Associates, Inc before the Master Plan analysis was done so that environmentally sensitive areas could be avoided when planning improvements. Once the Master Plan was completed, the proposed improvements were compared to the Environmental Baseline Overview to verify that there would be no environmental concerns. The comparison concluded that there would be no conflicts.

4.6 Conclusions

The drainage improvements proposed in the Cowart Creek Watershed Master Plan significantly reduce the amount of overbank floodings during the 10% exceedence event. As a result, the number of structures below 10% exceedence event flood stages is reduce from 234 to 25, which is an 89% reduction in the number of flooded structures. Even though the 1% exceedance event is not as dramatic, the number of structures floodedis reduced by 62% (530 to 202). These flood reduction benefits are created without inducing downstream flooding.

The overall estimated construction cost for the project is $82.63 million dollars and the expected annual benefit (i.e., avoided flood damages) is $21.88 million dollars. The cost and benefit of the alternatives are considered as one package because they were designed to work together and so the separate pieces would not have the same affect on the overall watershed as the entire plan does. The project yields a benefit-cost ratio of 0.26, which may seem low at first glance. However, the benefit-cost analysis was not able to quantify all benefits. A reduced flood plain will allow for more development and, as a result, will expand the local tax base.

The improvements outlined in this master plan are in the process of being implemented. Specifically, the City of Pearland is expected to enter the final design phase for the Bailey Road improvements in June 2008 and complete final design by December 2009. Right-of-way acquisition for the Bailey Road improvements is expected to be completed by December 2009. Construction is anticipated to be complete in December 2011.

It is recommended that the entities within the watershed begin obtaining right-of-way for the remainder of the proposed plan components. Emphasis should be placed on acquiring right-of-way for the proposed regional detention facilities, as the proposed detention will need to be constructed before conveyance improvements. Construction of the proposed plan components in this order will not induce downstream flood stage increases.

Due to the magnitude of the improvements evaluated, they will need to be split up in to phases. The first step in any of the improvements will be to acquire the Right Of Way needed. The improvements have been group in to four different phases. The first phase includes the diversion channel, Detention A, both cells of Detention D, and the channel improvements directly upstream and downstream of the diversion channel. These improvements can be seen in Exhibit 4.15. The second phase will include the channel improvements on CW100-00-00, Detention B and all cells of Detention E. These improvements can be seen in Exhibit 4.16. The third phase will include the remainder of the channel improvements on CW103-02-00, channel improvements on CW103-00-00, and both cells of Detention C and the diversion that will be used in conjunction with Detention C. These improvements can be seen in Exhibit 4.17. The fourth and final phase will include the diversion culvert near Detention F, Detention F and the channel improvements on CW101-00-00. The improvements can be seen in Exhibit 4.18. Each phase will be implemented when funding for that phase becomes available.

5.0 REFERENCES

1. "ARC HYDRO: GIS for Water Resources," David R. Maidment, 2002.

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