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Attachment C. Proposed Alternatives Conditions
Attachment D. Proposed Sump/Sluice Analysis

EXECUTIVE SUMMARY

This report presents the hydrologic and hydraulic analysis methodology and results for the Wharton Interim Feasibility Study. The Wharton Interim Feasibility Study is a component of the Lower Colorado River Basin Feasibility Study initiated by the U.S. Army Corps of Engineers Fort Worth District. The City of Wharton is subject to flooding from both the Colorado River and local creeks near the community. Colorado River floods impact the City of Wharton for storms more frequent than a 25-year event. Local rainfall and flooding impact portions of Wharton for storms as frequent as a 2-year event.

The purpose of the Wharton Interim Feasibility Study hydrologic and hydraulic analysis is to identify and establish existing baseline condition flooding sources, overflow locations, and frequency water surface elevations within the city. This analysis involved not only hydrologic and hydraulic modeling of the watershed and channel conditions near the City of Wharton, but also in-depth research of historical floods and previous studies. As a result of this analysis, baseline condition water surface elevations for various frequencies were established for the Colorado River, Caney Creek, Baughman Slough, and Peach Creek, considering both Colorado River flood events and localized storm events.

A range of flood reduction and flood control alternatives on each of the rivers and creeks in the vicinity of Wharton were also analyzed as part of the study and the results are summarized in this report. The recommended alternative plan incorporates levees along the Colorado River and Baughman Slough, diversion channels along Caney Creek, pipe upgrades along Caney Creek, and channelization along Baughman Slough. If the proposed alternatives outlined in this report are implemented, approximately 2,400 acres of area within the city limits of Wharton will be removed from the 100-year floodplain. This area removed from the 100-year floodplain includes hundreds of homes and businesses. Table ES-1 provides 100-year baseline conditions water surface elevations, as well as the 100-year water surface elevation with the proposed alternative plan in place at various points of interest within the City of Wharton.

Table ES-1. 100-Year Water Surface Elevation Comparison

Location	Baseline Conditions (ft)	Proposed Alternative
Colorado River Hwy. 59	106.2	106.6
Colorado River Business 59	102.7	103.0
Caney Creek Outfall to Colorado	104.5	101.8
Caney Creek Wharton	102.1	100.8
Caney Creek Crestmont	102.1	100.2
Baughman Slough Business 59	101.5	100.0
Baughman Slough Alabama Rd	98.8	96.2
Peach Creek Business 59	100.1	99.4
Peach Creek CR 135	97.8	96.7

Although the 100-year water surface elevations along the Colorado River increased as a result of the levees, areas on the left overbank (City of Wharton) will be protected by the proposed levee.

INTRODUCTION

The City of Wharton is located in Wharton County, Texas, and is subject to several sources of flooding, including overflow from the Colorado River and localized flood events. Significant floods over the last century have resulted in millions of dollars of damage to property within the City of Wharton. Significant damages have occurred within the last ten years and rising water has forced residents to evacuate on multiple occasions. This report outlines the hydrologic and hydraulic analyses associated with the Wharton Interim Feasibility Study initiated in 2002. Hydrologic and hydraulic models of existing or baseline conditions within the City of Wharton were developed following an extensive study and analysis of the area. Flood reduction alternatives were then analyzed with the models to assess the reduction in frequency water surface elevations and ultimately flood damages.

SCOPE OF WORK

The purpose of the Wharton Interim Feasibility Study is to establish existing floodplains and flood conditions within the City of Wharton and evaluate potential alternatives to reduce annual flood damages within the city. The study incorporates the data and information gathered as part of the Lower Colorado River Basinwide Flood Damage Evaluation Project (FDEP). As part of this basinwide study, detailed hydrologic and hydraulic analyses resulted in frequency flows and water surface elevations along the Colorado River, including Wharton County. The hydraulic models developed for FDEP were refined to better simulate overflows from the Colorado River near the City of Wharton. Detailed hydraulic models were also created for smaller streams within and around the City of Wharton. Both local flood events and Colorado River flood overflows were considered in determining baseline stage-frequency relationships. Several flood control alternatives were then analyzed with the models to determine the potential reduction in water surface elevations and flood damages.

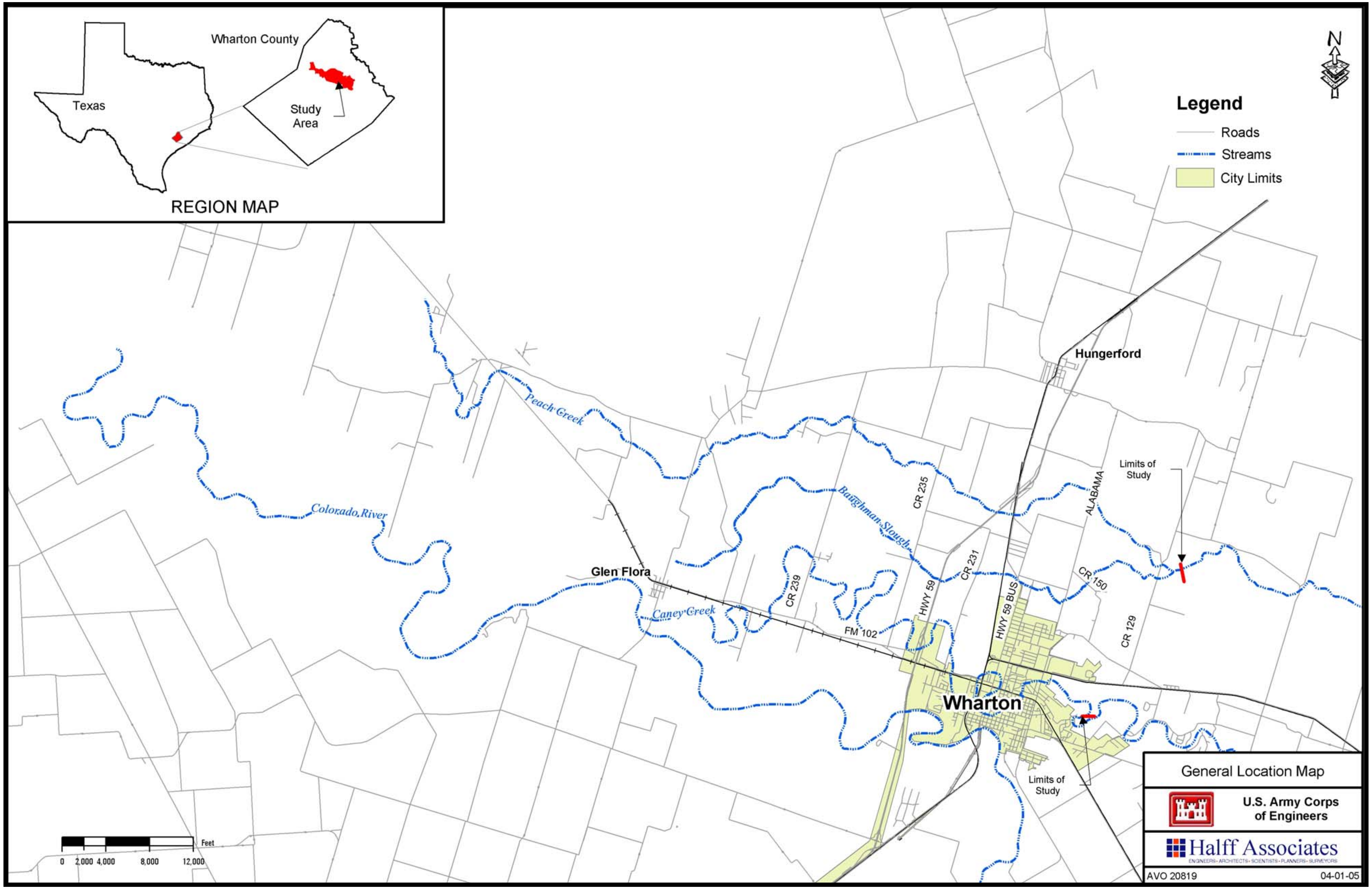
CITY OF WHARTON/WHARTON COUNTY

Wharton County is located sixty miles southwest of Houston, Texas, in the coastal prairie and encompasses an area of 1,095 square miles. Wharton County is bounded by Colorado County, Austin County, Fort Bend County, Brazoria County, Matagorda County, and Jackson County. The region is covered with subtropical vegetation consisting primarily of coarse grasses with oak, elm, and other hardwoods scattered along the stream banks. The topography of Wharton County is nearly flat with elevations ranging from 40’ to 160’. The topsoil is composed mostly of alluvial, black, sandy loam soils. The average annual rainfall is 42.3 inches. Population of the county has steadily increased over the years, and was at 41,330 according to the Texas Almanac, 2004-2005.

The City of Wharton is the county seat and has a population of 9,285 according to the Texas Almanac, 2004-2005. Wharton is located near the center of the county and is bounded by U.S. Highway 59 to the west and the Colorado River to the south. Major industries include plastics manufacturing, oil production, and agribusiness.

The City of Wharton is subject to flooding from the Colorado River, Caney Creek, Baughman Slough, and Peach Creek. Significant flooding has occurred within Wharton numerous times over the last century. A detailed description of each of these flooding sources is provided. Figure 1 provides a general location map of the Wharton area.

Figure 1. General Location Map



CITY OF WHARTON NEIGHBORHOODS & POINTS OF INTEREST

In order to understand the hydrology and hydraulics near the City of Wharton and address flooding problems, the layout of the city must be presented. Figure 2 is provided as a guide to areas within the City of Wharton. The West End Neighborhood (1) has been severely impacted by Colorado River flooding in the past. The neighborhood is bounded by the Colorado River on the south, FM 102 to the north, U.S. Highway 59 to the west, and the abandoned railroad embankment to the east. A major horseshoe shaped bend in the Colorado River (2) further aggravates flooding problems in this low lying area. The straight line distance from Highway 59 through the West End neighborhood to the abandoned railroad is approximately 6,000 feet. However, almost 14,000 feet of Colorado River flows through this same reach.

East of the railroad and Business Highway 59 is downtown Wharton and the Riverside Park area (3). Downstream of downtown (southeast of Wharton) is the wastewater treatment plant (4). An outfall channel to the Colorado River (5) also exists in this area and drains a box culvert under Alabama Road. More details concerning this storm sewer system will be provided in the *Hydraulic Modeling - Overview* section. The inlet to the Alabama Box is a low-lying park area near Santa Fe Street and Alabama Road (6).

Northern Wharton includes the Ahldag subdivision (7). Two channels in the subdivision convey flow to the Alabama/Junior College Road ditch and into Baughman Slough. These channels have overflowed in the past, most often due to local rainfall independent of the Colorado River, and created problems for residents in the Ahldag neighborhood.

There is a USGS gauging station in Wharton along the Colorado River (8). The gauge (ID# 08162000) is located on the Business Highway 59 Bridge, 1,100 feet downstream of the abandoned railroad. This location corresponds to Colorado River mile 65.0 (Station 343254.8).

FLOODING SOURCES

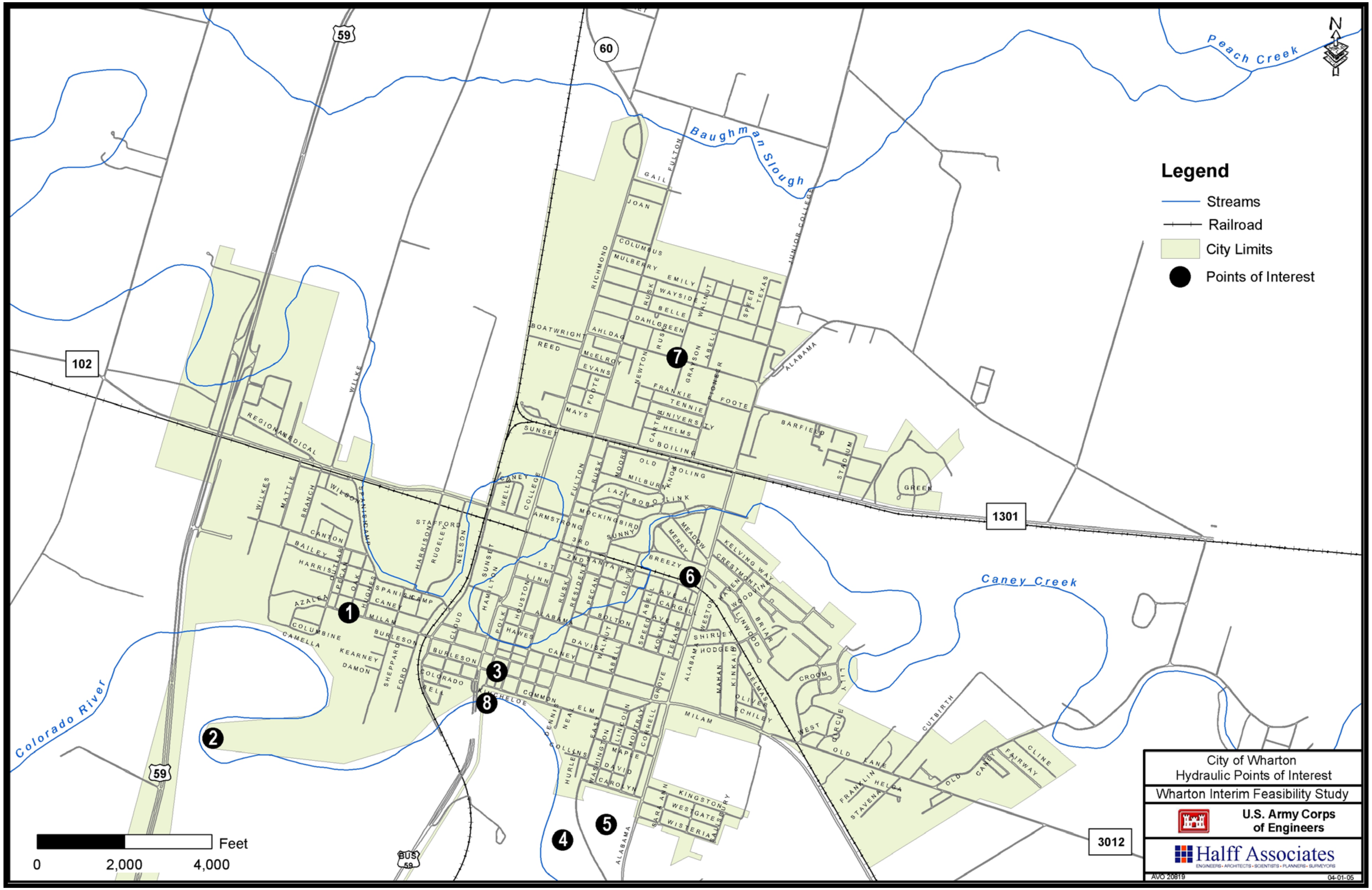
There are several sources of flooding within and near the City of Wharton. Overflows from the Colorado River have impacted the West End neighborhood, downtown Wharton, and other low lying areas in the past. Localized flooding related to Baughman Slough and Caney Creek has also resulted in problems in other neighborhoods of Wharton. Peach Creek, which flows north of the Wharton city limits, is another source of flooding for areas just outside of the City of Wharton.

COLORADO RIVER

The Colorado River is the largest river within Wharton County, and flows for over forty-seven miles through the center of the county from the Colorado/Wharton County line to the Wharton/Matagorda County line. Total drainage area of the Colorado River at the Wharton Gauge within the City of Wharton is 42,000 square miles. Lake Travis, located on the Colorado River above Austin, regulates flows for storms centered on the upper reaches of the watershed. The Lower Colorado River Basinwide study found that the 2 through 100-year frequency events on the Colorado River in Wharton are a result of storms centered below Lake Travis. The 500-year frequency event on the Colorado River in Wharton is a result of Lake Travis releases due to a storm centered above the reservoir.

The Colorado River has a mild bed slope of 0.0003 ft/ft through Wharton County. The river forms the southern boundary of the City of Wharton through much of the town. Downtown Wharton is built on the northern (left) bank of the Colorado River. The river is a major source of irrigation water within the county, and also a major source of flooding during heavy rainfall events. Six bridges cross the Colorado River in Wharton County (FM 960, U.S. Highway 59 (2), Railroad, and Business Highway 59 (2)).

Figure 2. Wharton Points of Interest



PEACH CREEK

Peach Creek is a tributary of the San Bernard River and flows north of the city limits of Wharton. The headwaters of Peach Creek are between Bonus, Texas, and Egypt, Texas, west of FM 102 (approximately 13 miles northwest of the City of Wharton). Peach Creek outfalls into the San Bernard River 11.8 miles downstream of the Business Highway 59 crossing. Peach Creek flows from its headwaters in a generally west to east direction for approximately 28 miles before its outfall into the San Bernard River on the Fort Bend/Wharton County line. The channel area of Peach Creek is overgrown with dense vegetation. The bed slope of Peach Creek is mild averaging .0005 ft/ft. The Peach Creek channel is well-defined and over twenty feet deep in the area near the City of Wharton.

For this study, Peach Creek was modeled in detail from the FM 102 crossing to just downstream of the Baughman Slough confluence, a total distance of 15.5 miles. The area from the headwaters to FM 102 was treated as a storage area and will be discussed in more detail in the *Overflow Hydraulics* section. The total drainage area of Peach Creek modeled for this study was 23.7 square miles. Numerous bridges including highways, farm-to-market (FM) roads, county roads (CR), and private drives cross Peach Creek through the study area and were modeled hydraulically.

BAUGHMAN SLOUGH

Baughman Slough is a tributary of Peach Creek and flows just north of the city limits of Wharton. The headwaters of Baughman Slough are near Glen Flora, Texas, north of FM 102 (approximately six miles west of the City of Wharton). Baughman Slough outfalls into Peach Creek just downstream of the County Road (CR) 129/Montgomery Road crossing northeast of the City of Wharton. Baughman Slough drains the northern sections of Wharton including the Ahldag subdivision. Several man-made and natural channels divert storm water runoff from the City of Wharton to Baughman Slough. The channel area of Baughman Slough is not as overgrown as Peach Creek. The Baughman Slough channel is well-defined in the area near the City of Wharton, although it does not have the capacity of Peach Creek.

For this study, Baughman Slough was modeled in its entirety from the headwaters to the confluence with Peach Creek. The total river miles studied in detail for Baughman Slough is approximately 11.3 miles. The total drainage area of Baughman Slough modeled for this study was 17.3 square miles. Numerous bridges including highways, farm-to-market roads, county roads, and private drives cross Baughman Slough and were included in the hydraulic models.

CANEY CREEK

Caney Creek and the Colorado River most likely shared portions of the same channel many years ago. Today, Caney Creek and the Colorado River still share a common channel near the City of Glen Flora, Texas. The Caney Creek headwaters are northwest of Bonus, Texas. Caney Creek outfalls into the Colorado River just west of Glen Flora, Texas. The two rivers share a common channel for approximately one mile, at which point Caney Creek splits from the Colorado River just south of Glen Flora, Texas, near FM 960. The split is actually an overflow point, and the water surface in the Colorado River must exceed elevation 114.0’ for water to spill into the Caney Creek channel. From this point Caney Creek meanders through the City of Wharton and downstream with a final outfall into Matagorda Bay near Sargent, Texas, in Matagorda County. Although Caney Creek outfalls into Matagorda Bay, through much of Wharton County and the City of Wharton, the channel is not well defined.

In other locations, small earth embankments have created a series of private ponds and dams along Caney Creek. Caney Creek throughout most areas of Wharton County does not exist in a riverine environment and resembles a series of storage areas. Within the City of

Wharton, the old Caney Creek channel has been filled and paved in most areas with development along and within the former channel. In some areas, natural flow direction has been reversed as a result of fill and grading.

For this study, Caney Creek was modeled as a series of storage areas since most locations around the City of Wharton do not exhibit the qualities of a riverine environment. Storage area divides and connections were made at major highways and other logical points of division. Details concerning the Caney Creek modeling effort will be discussed in the *Hydraulic Modeling - Overview* section.

HISTORIC FLOODS

The City of Wharton has been impacted by numerous major floods throughout its history. The construction of Mansfield Dam (Lake Travis) in 1940 decreased the Colorado River peak flows through the City of Wharton, but flooding has still occurred. Recent significant Colorado River flooding impacted the City of Wharton in 1991, 1998, and 2004. The West End neighborhood of Wharton has been most severely impacted by historic Colorado River floods. Local flooding events have also caused problems in neighborhoods such as the Ahldag subdivision.

1998 & 2004 COLORADO RIVER FLOOD

A significant Colorado River flood occurred in October of 1998. Rainfall of 8 inches to over 20 inches occurred within the Colorado River watershed along the Wharton/Colorado River county line. A minimal amount of rainfall fell within the City of Wharton. The peak flow on the Colorado River at Wharton occurred on October 23 with a rate of 74,800 cfs. The river peaked at a stage of 48.7’ (elevation 101.14’) at the Wharton gauge (Business Highway 59). Inundation areas and data related to the flood were obtained through interviews with City of Wharton officials, Wharton County officials, Wharton residents, aerial video footage, and aerial photographs. The West End neighborhood was inundated with two to four feet of water from the Colorado River. Over 500 homes in the neighborhood were infiltrated with floodwaters and residents were forced to evacuate. The Dawson Elementary School in the neighborhood was flooded with three feet of water. Figure 3 provides the approximate inundation limits for the 1998 flood based on highwater marks, aerial photographs, and video footage. Inundation areas are only shown within the limits shown on the Figure. Areas upstream of Highway 59 are outside of the limits, but were inundated in 1998.

FM 102 was overtopped west of U.S. Highway 59 and this water escaped and filled Caney Creek which began to spill north down CR 231/Wilke Road to Baughman Slough. Water did not overtop Highway 59, but passed through the bridge over the Colorado River and also through the FM 102 underpass. The estimated highwater mark at the Highway 59 bridge was 105.0’ based on photographs and known elevations of top of road and low chords of the bridge structure. FM 102 was also overtopped east of U.S. Highway 59. Water filled the Caney Creek channel and inundated the manufactured home park located northeast of the intersection of FM 102 and the abandoned railroad. The abandoned railroad embankment served as a levee preventing more extensive flooding within the City of Wharton. Water overtopped Richmond Road near the Dairy Queen (1,000 feet north of the FM 102 intersection) and old Caney Creek channel. Water rose to Elm Street along the bank of the Colorado River near downtown Wharton.

In 1998, floodwaters backed up through the Alabama Box culvert and flooded the park near Santa Fe Road and Alabama Road. The water surface elevation of the Colorado River near the Alabama Box outfall was estimated to be near 100.0’. The Caney Creek channel filled through the City of Wharton due to flow escaping over Richmond Road near the Dairy Queen and flow from the park at Santa Fe and Alabama Roads. Although a storm sewer system exists along the Caney Creek channel through Wharton, the outfall is at Rusk Street and Elm Street. The tailwater (Colorado River) elevation at this point was near 101.0'. The pipe is equipped with a flapgate and prevented Colorado River flow from backing up through the system, but interior flows along Caney Creek could not drain and the storm system was of no benefit during the 1998 event. Approximately 800 homes were damaged throughout the City of Wharton.

In November 2004, the Wharton area was again impacted by a flood of approximately the same magnitude as the 1998 event. The Colorado River crested at a stage of 48.1' with a peak flow of over 72,900 cfs. Many homes, businesses, and the elementary school in the West End neighborhood were again impacted by the November 2004 event. However, flap gates were installed on the Alabama Box after the 1998 flood event, and this prevented water from backing up through the Alabama Box into the low lying area near Santa Fe and Alabama Roads during the 2004 flood.

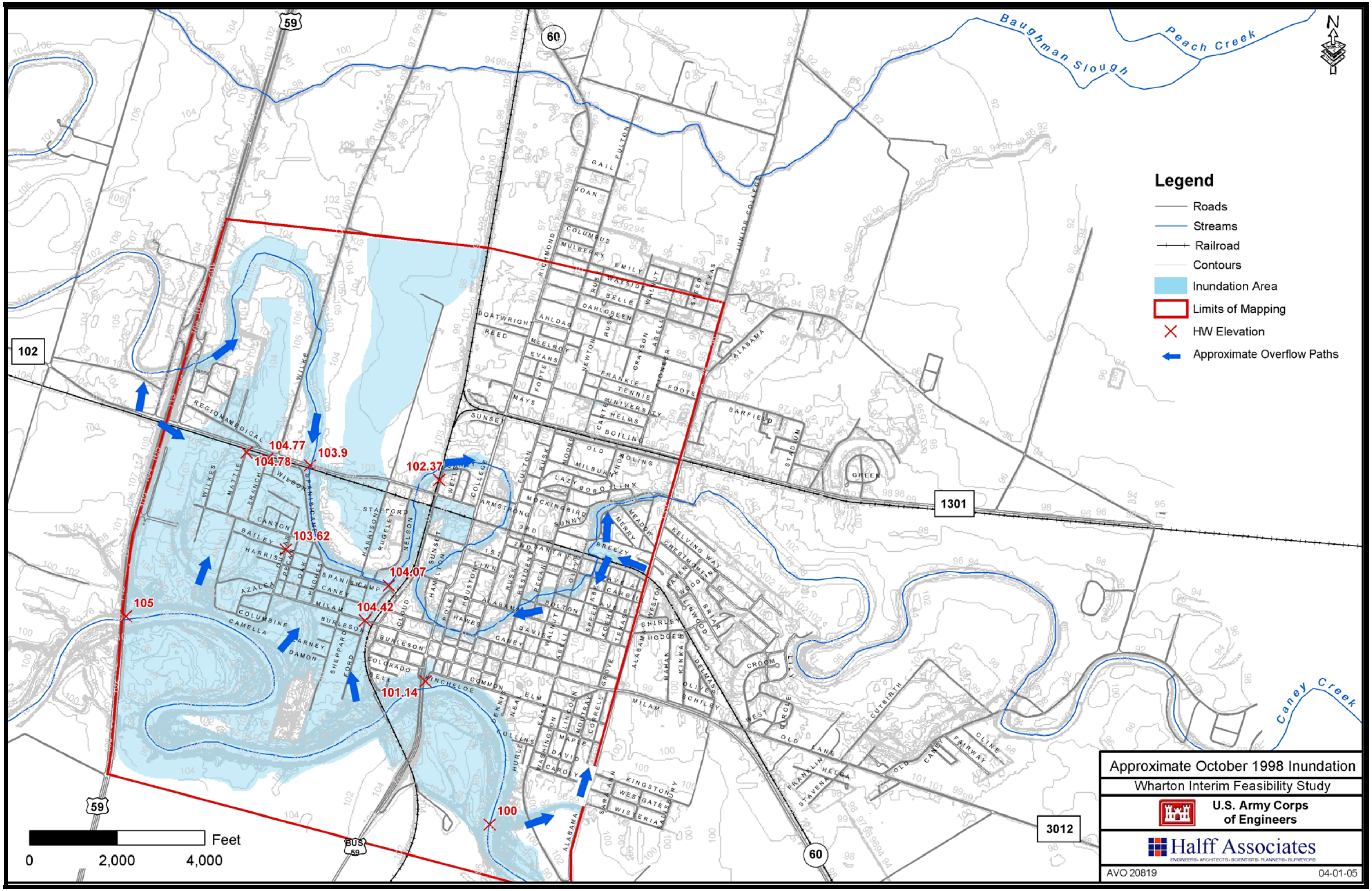
OTHER FLOOD EVENTS

Although 1998 and 2004 are the last major Colorado River floods, the City of Wharton has experienced numerous floods within the last century. Floods prior to 1940 did not experience any flood control benefits of Lake Travis and Mansfield Dam. Table 1 provides a brief summary of other Colorado River floods within the City of Wharton. The peak water surface elevations and flows are approximate. The approximation and gauge rating curve revisions over time explain the variations in estimated flows and peak water surface elevations. Table 1 is provided to give a general overview of flooding problems within the City of Wharton.

Table 1. Historic Wharton Floods

Date	Peak Q (cfs)	Peak WSEL (ft)	Comments
Dec. 1913	200,000	104.3	Every street in town: 1'-4' water. Peach Creek flooded. Colorado River water from Mackay to Hungerford (10-mile spread). Rowing only way to maneuver around courthouse square. Brazos & Colorado Rivers converged below Wharton (70-mile wide body of water).
May 1922	111,000	102.3	Centered near Smithville
June 1935	159,000	103.6	12-mile spread of water. Richmond Road Bridge overtopped at Peach Creek & Baughman Slough. Corner of Richmond Road and Milam Street flooded.
July 1938	125,000	102.8	15-mile spread of water. Richmond Road covered with 5' of water at Caney Creek. Peach Creek out of banks. 75 blocks in Wharton entirely or partially flooded. Every highway submerged 2'-6' for 5-10 miles from Wharton.
July 1940	100,000	101.4	Centered near Smithville
Nov. 1940	92,000	100.6	Centered near Columbus
Dec. 1991	61,900	97.7	Floodwaters from upstream of Lake Travis and near Austin
Oct. 1998	74,800	101.1	West End Neighborhood flooded. Flow backed-up through Alabama Box.
Nov. 2004	72,900	100.5	West End Neighborhood flooded.

Figure 3. Approximate 1998 Flood Inundation Map



Floods originating on the Colorado River are not the only events impacting Wharton. Local flooding created by Peach Creek, Baughman Slough, and Caney Creek has also caused damage throughout the City. In September 2002, Tropical Storm Fay impacted Wharton. Over 22 inches of rainfall fell over portions of Wharton County. Approximately 100 homes in Wharton were damaged. Most of the residences were in the Ahldag subdivision near Junior College Boulevard (also known as Alabama Road, Lees Lane, and CR 135). Photos of previous flood events in Wharton are shown in Attachment A.

PREVIOUS STUDIES & REPORTS

Before proceeding with the hydrologic and hydraulic study for the Wharton Interim Feasibility Study, previous reports and documents were reviewed to gather background and historic information related to flooding near the City of Wharton.

COLORADO RIVER RAFT REMOVAL

A significant collection of driftwood located near the mouth of the Colorado River grew significantly during the 1800's and early 1900's and came to be known as the "raft". The first recorded description of the raft of the Colorado River was in 1690. The raft limited navigation along the Colorado River and also increased flooding problems. In 1839, the head of the raft extended to Buckeye, Texas, near Bay City, Texas. In 1851, an attempt was made to remove the raft and open the channel to riverboat navigation. The river remained open until 1859, but driftwood collected again and clogged the channel. Following the devastating floods of 1913 and 1922 along the lower Colorado River, concerns surrounding the raft were once again raised. The Texas Legislature passed an act in 1923 to clear the raft and build levees in the hopes of mitigating future flood damages. In 1928, the head of the raft had reached forty-five miles from the mouth of the Colorado River, near Lane City, Texas. One of the best descriptions of the problem facing this section of the Colorado River Valley was written in Section 5 of the 1923 act.

> "The fact that a great raft of logs, trees, and other drift has formed in the Colorado River between the towns of Bay City in Matagorda County and Wharton in Wharton County, which completely obstructs the channel of said river for several miles, and is rapidly building upstream, and so dams up and retards the flow of water, that during every rise in said river it overflows and the country adjacent thereto and inundates large territory of fertile lands on both sides of said river in said counties, and during the great floods of recent years, and particularly those of 1913, 1914, and 1919 and 1922, the flood waters of said river owing chiefly to said raft, partly submerged the towns of Bay City and Wharton, and overflowed a large area of fertile and cultivated lands in both of said counties of Matagorda and Wharton whereby great property damage was sustained, crops and livestock destroyed, and human life lost and endangered, constituting a great public calamity."

In 1934 the raft was completely removed into the Gulf of Mexico. The effects of the raft on the streambed elevations in Wharton were addressed in a 1975 study by the Wharton Fresh Water Resources Conservation & Development Commission (WFWRCDC). Inconsistencies in water surface elevations before and after the raft removal in the 1920's and 1930's indicated that the Colorado River channel bed was deepening following the raft removal as silt was carried away and higher velocities prevailed. Table 2 indicates the difference in water surface elevations at the Wharton gauge for similar flow rates before and after the raft removal. Table 2 further supports the reason for differences in flow and stage at the Wharton Gauge presented in Table 1.

Table 2. River Raft Effects on Water Surface Elevations

Date	Wharton Gauge Height (ft)	Flow Rate (cfs)
June 1919	32.45	37,600
Oct. 1919	33.9	39,600
Sept. 1921	31.55	35,900
AVG. Prior to Raft Removal:	**32.63**	**37,700**
April 1942	22.35	38,900
April 1945	19.8	36,400
May 1946	19.5	35,600
April 1949	20.9	37,900
AVG. After Raft Removal:	**20.64**	**37,200**

1970 COE BAUGHMAN SLOUGH REPORT

In 1970, the Galveston District of the U.S. Army Corps of Engineers (COE) published a report related to the floodplain of the Colorado River and Baughman Slough in Wharton, Texas. The report documented historic floods and the dimensions/elevations of bridges crossing Baughman Slough and the Colorado River in the study area. The study indicated that the Intermediate Regional Flood (100-year) on the Colorado River at Wharton would have a peak discharge of 178,000 cfs. This value was based on analysis of historical flows from 1900 to 1968 and flows prior to 1942 were adjusted to simulate the effects of Mansfield Dam. Also included in the study are profiles and inundation surfaces for the Colorado River and Baughman Slough resulting from the Intermediate Regional Flood on the Colorado River. Table 3 provides the approximate water surface elevations for the Intermediate Regional Flood along Baughman Slough and the Colorado River as reported in this 1970 study.

Table 3. 1970 COE Baughman Slough Report Water Surface Elevations

Baughman Slough Location	WSEL (ft)	Colorado River Location	WSEL (ft)
Owen Road/CR. 235	106.0	RR	104.3
Wilke Road/CR 231	103.5	Business Highway 59 SB	103.8
RR	102.4	Business Highway 59 NB	103.8
Business Highway 59/Richmond Rd.	101.8		
Fulton Street	101.1		
Jr. College Blvd./Alabama Rd.	99.4		
Montgomery Rd./CR 129	95.0		

1977 TURK, KEHLE, & ASSOCIATES REPORT

In 1977, Turk, Kehle, & Associates prepared a report for Wharton County reviewing the 1970 Corps of Engineers' Report. The 1970 Corps' report was examined to determine if present (1977) channel conditions were considered and if flood control structures in the Colorado River drainage basin above Wharton were accounted for. The Turk, Kehle, & Associates report stated that the 1970 COE study did not take into account flood control structures on Cummins Creek. As opposed to performing a historical flow analysis along the Colorado River, Turk, Kehle, & Associates centered the 100-year rainfall event on the most critical portion of the watershed, identified as the reach from Austin to Columbus. Using this procedure, a new 100-year flow rate at Wharton was found to be 145,000 cfs, nearly twenty percent less than the 1970 COE study. This lower flow rate resulted in water levels 1.6 to 2.1 feet lower than the 1970 report.

SAN BERNARD RIVER REPORTS

Although the San Bernard River was not directly a part of this study, issues related to the tailwater effects near the Peach Creek confluence were addressed. In response to this tailwater study, two reports related to the San Bernard River were investigated. The San Bernard River watershed is approximately 130 miles long and covers an area of 1,000 square miles. The San Bernard forms the county boundary between Wharton and Fort Bend Counties. The first report studied was a 1971 Corps of Engineers Survey Report on the San Bernard River, Texas. The purpose of the report was to investigate flood control and major drainage improvements along the San Bernard River in Wharton County. A general description of the watershed was presented in this report, as well as proposed improvement alternatives. The conclusion of the study was that no improvements were economically justified at that time.

The second report, *Reconnaissance Report, San Bernard River Watershed, Texas*, was published in 1991. The report provides the results of a reconnaissance-level investigation of the feasibility of reducing flood damages in the San Bernard River watershed. The primary objective of the investigation was to determine if economically feasible measures exist to provide comprehensive flood control. The report did state that during flooding, the waters along the San Bernard River recede slowly because of dense vegetation, brush, and trees. The 1991 report also noted a 1989 study by VanSickle, Michelson, & Klein, Inc., *San Bernard Drainage Analysis Channel Clearing Project*. According to the 1991 reconnaissance report, the 1989 study identifies reaches of the San Bernard where clearing would reduce the elevation and duration of the flood flow.

WHARTON COUNTY FIS (2001)

The current effective Wharton County, Texas, Flood Insurance Study (FIS) was published in November 2001. Revisions published in the 2001 FIS did not update the hydrology and hydraulics of the Colorado River, Baughman Slough, and Caney Creek that were completed in 1982. At the request of the Federal Insurance Administration (FIA) in 1978, the Southwest Division of the U.S. Army Corps of Engineers reviewed the 1970 report related to frequency discharges along the Colorado River in Wharton. A period-of-record analysis from 1930 to 1974 was executed as part of this study. This analysis resulted in a 100-year average daily Colorado River flow rate at Wharton, Texas, of 143,000 cfs. Ten percent was added to this flow (14,300) to account for instantaneous peak and another 5,000 cfs was added to account for Mansfield Dam (Lake Travis) releases. The 100-year peak flow rate for the Wharton gauge of the Colorado River prior to any overflow escape was adopted as 162,000 cfs for the 1981 FIS work performed by Turner, Collie, and Braden. However, much of this flow was found to escape (overflow) into Caney Creek, Baughman Slough, and Peach Creek upstream of the City of Wharton. These overflows were taken into account and the published 100-year peak flow rate along the Colorado River at Business Highway 59 in Wharton, Texas, is 139,500 cfs in the 2001 FIS.

The Colorado River, Caney Creek, and Baughman Slough near the City of Wharton are studied in detail in the current effective FIS. Table 4 provides the study extents for these rivers and streams in the current effective FIS. Peach Creek is not included with the current effective FIS. Overflows from the Colorado River as well as local events on Baughman Slough and Caney Creek were modeled to determine the controlling event for the current effective FIS. Snyder's unit hydrograph method was used for the local hydrologic modeling. Table 5 provides the peak discharges published in the current effective FIS for the Colorado River, Baughman Slough, and Caney Creek.

Table 4. 2001 Wharton County FIS Study Limits

Stream	Upstream Limit	Downstream Limit	Total Distance
Colorado River	1.45 Miles U/S FM 960	10.3 Miles D/S Bus. 59	21 Miles
Baughman Slough	Owen Rd. (CR 235)	Montgomery Rd. (CR 129)	5.25 Miles
Caney Creek Upper	FM 102	Caney Creek Outfall to CR (Hughes St. & Spanish Camp Rd.)	10.2 Miles
Caney Creek Lower	Wharton Eastern City Limits	Kriegel Road	6 Miles

Table 5. Wharton County 2001 FIS Peak Discharges

Location	10-Year Q (cfs)	50-Year Q (cfs)	100-Year Q (cfs)	500-Year Q (cfs)
Colorado River (Bus. 59)	70,000	127,500	139,500	247,000
Baughman Slough (Bus. 59)	1,910	2,730	9,500	N/A[1]
Caney Creek (FM 102)	870	1,220	3,000	N/A[1]

[1] Common floodplain with the Colorado River

Table 6 provides the water surface elevations at key locations for the various frequency events as published in the current effective FIS. Note that Caney Creek was modeled in a riverine environment in the current effective FIS.

Table 6. Wharton County 2001 FIS Water Surface Elevations

Location	10-Year WSEL (ft)	50-Year WSEL (ft)	100-Year WSEL (ft)	500-Year WSEL (ft)
Colorado River – Business Highway 59	97.2	103.0	103.2	105.0
Colorado River – RR	97.4	103.0	103.5	105.4
Colorado River – U.S. Highway 59	100.0	106.5	107.0	110.5
Colorado River – FM 960	111.8	118.1	119.0	124.0
Baughman Slough – CR 129/Montgomery	89.2	90.0	91.0	102.5
Baughman Slough – CR 150/Moers Rd.	91.5	92.0	92.8	104.0
Baughman Slough – Junior College Blvd.	94.0	94.4	94.9	104.2
Baughman Slough – Fulton Street	96.9	97.0	98.2	105.0

Location	10-Year WSEL (ft)	50-Year WSEL (ft)	100-Year WSEL (ft)	500-Year WSEL (ft)
Baughman Slough – Business Highway 59	97.2	97.5	98.8	105.1
Baughman Slough – Railroad	98.5	98.6	102.0	106.5
Baughman Slough – Wilke Road/ CR 231	100.5	102.0	102.8	108.0
Baughman Slough – U.S. Highway 59	101.5	102.5	104.0	110.5
Baughman Slough – Owens Road/CR 235	104.0	104.5	106.5	112.8
Caney Creek – FM 102	105.8	106.5	107.2	109.5
Caney Creek – CR 231/Wilke Road	106.2	107.0	108.6	109.9
Caney Creek – U. S. Highway 59	107.5	108.0	109.0	110.5
Caney Creek – CR 235/Owens Road	108.0	109.2	109.2	111.8

LOWER COLORADO RIVER BASINWIDE FLOOD DAMAGE EVALUATION PROJECT

As stated previously, the Wharton Interim Feasibility Study refines the Colorado River flows and hydraulic models around the City of Wharton that were developed as part of the Lower Colorado River Basinwide Flood Damage Evaluation Project (FDEP). The FDEP involved detailed period-of-record, hydrologic, hydraulic, and reservoir simulations for over 482 river miles of the Colorado River from near San Saba, Texas, to Matagorda Bay. The watershed of the Colorado River studied during the FDEP encompassed 18,300 square miles. A product of the FDEP was water surface elevations along the Colorado River near Wharton, Texas, for the 2-year through SPF events. These models included some Colorado River overflow into Caney Creek, Baughman Slough, and Peach Creek, but further refinement was needed to better analyze flooding problems and potential solutions in the City of Wharton. Table 7 lists the computed frequency flows and water surfaces at the Wharton gauge based on a period-of-record analysis from 1930-1999 and the HEC-RAS Unsteady simulation from FDEP.

Table 7. FDEP Wharton Gauge Summary

Frequency	Period-of-Record Flow (cfs)	HEC-RAS Peak Flow (cfs)	HEC-RAS Max WSEL (ft)
2-Year	27,000	25,270	84.5
5-Year	48,000	44,070	91.9
10-Year	63,000	59,355	96.0
25-Year	88,000	78,160	100.1
50-Year	100,000	90,770	101.6
100-Year	116,000	98,315	102.4
500-Year	N/A[1]	204,795[2]	104.3
SPF	N/A[1]	237,825[2]	104.4

[1] Flow Based on Mansfield Dam Releases
[2] Flow Routed in HEC-RAS from Longhorn Dam, Town Lake, Austin, Texas

WHARTON COUNTY FIS/MAPPING UPDATE (2005)

The City of Wharton, as well as Wharton County, are participants in FEMA's National Flood Insurance Program. Local Ordinances meet the minimum standards set forth by FEMA.

The Wharton County FIS and floodplain maps are being updated in 2005. FEMA approved the modeling and mapping of the Colorado River, Baughman Slough, Peach Creek, and Caney Creek. The modeling performed for this update is similar to the modeling performed for the Wharton Interim Feasibility Study existing conditions.

LOCAL HYDROLOGIC MODELING

The City of Wharton is not only subject to flooding from Colorado River overflow, but local storm events can also produce significant flooding on Caney Creek, Baughman Slough, and Peach Creek. The coastal location of Wharton makes the city susceptible to hurricane and tropical storm events. In order to study the effects of local rainfall flooding, hydrologic models were created for Caney Creek, Baughman Slough, and Peach Creek. HEC-HMS 2.2.1 was utilized to generate runoff hydrographs for local storms. The SCS curve number and unit hydrograph methods were used to compute the runoff hydrographs from the rainfall data.

PARAMETERS/METHODS

The first step of the local hydrologic modeling process was the delineation of drainage areas and sub-basins. The Peach Creek drainage area was divided into seven sub-watersheds extending from the headwaters of Peach Creek to the Baughman Slough confluence. Baughman Slough was divided into eleven sub-watersheds extending from the headwaters to the Peach Creek confluence. The drainage areas and patterns through the Ahldag subdivision channels were included in the Baughman Slough delineation. Caney Creek was divided into six drainage areas extending from FM 102 between Glen Flora and Wharton to downstream of Old Caney Road east of Wharton. Figure 4 provides a drainage area map used for the study.

The SCS curve number method and unit hydrograph were used to generate runoff hydrographs from rainfall data. Curve numbers were generated for each sub-watershed based on land use and hydrologic soil group. Curve number calculations were performed with GIS considering both the land uses and hydrologic soil groups in Wharton County. Land uses for the area were categorized as: agriculture, heavy woods, industrial, light woods, public, residential, and water. Figure 5 shows the land use map created for the study area and Figure 6 provides the hydrologic soil group distribution.

Lag time for each sub-watershed was set equal to sixty percent of the time of concentration. The time of concentration was based on assumed velocities of flow based on overland, sheet, and channel characteristics along the longest flow path. Table 8 provides general basin parameters for each of the sub-watersheds delineated for the study.

RAINFALL

A 24-hour duration hypothetical storm was used for the various frequency event simulations in HEC-HMS. Rainfall data was obtained from Technical Paper Number 40 (TP-40) and NOAA Technical Memorandum NWS Hydro-35. The 2-year through 100-year rainfall data for each duration storm were plotted on probability paper and extrapolated to obtain 500-year frequency rainfall. Wharton area rainfall depths are summarized in Table 9.

The 24-hour duration storm data from TP-40 was compared to values generated by the report "Depth-Duration Frequency of Precipitation for Texas" (Asquith, 1998). The parameters for Wharton County developed in the report were used to generate rainfall depths for Wharton

County. Figure 7 provides a graphical comparison of the 24-hour duration rainfall as defined by TP-40 and Asquith. The graphic indicates that the TP-40 rainfall data is approximately the same as the values generated by the USGS and Asquith.

Figure 4. Drainage Area Map

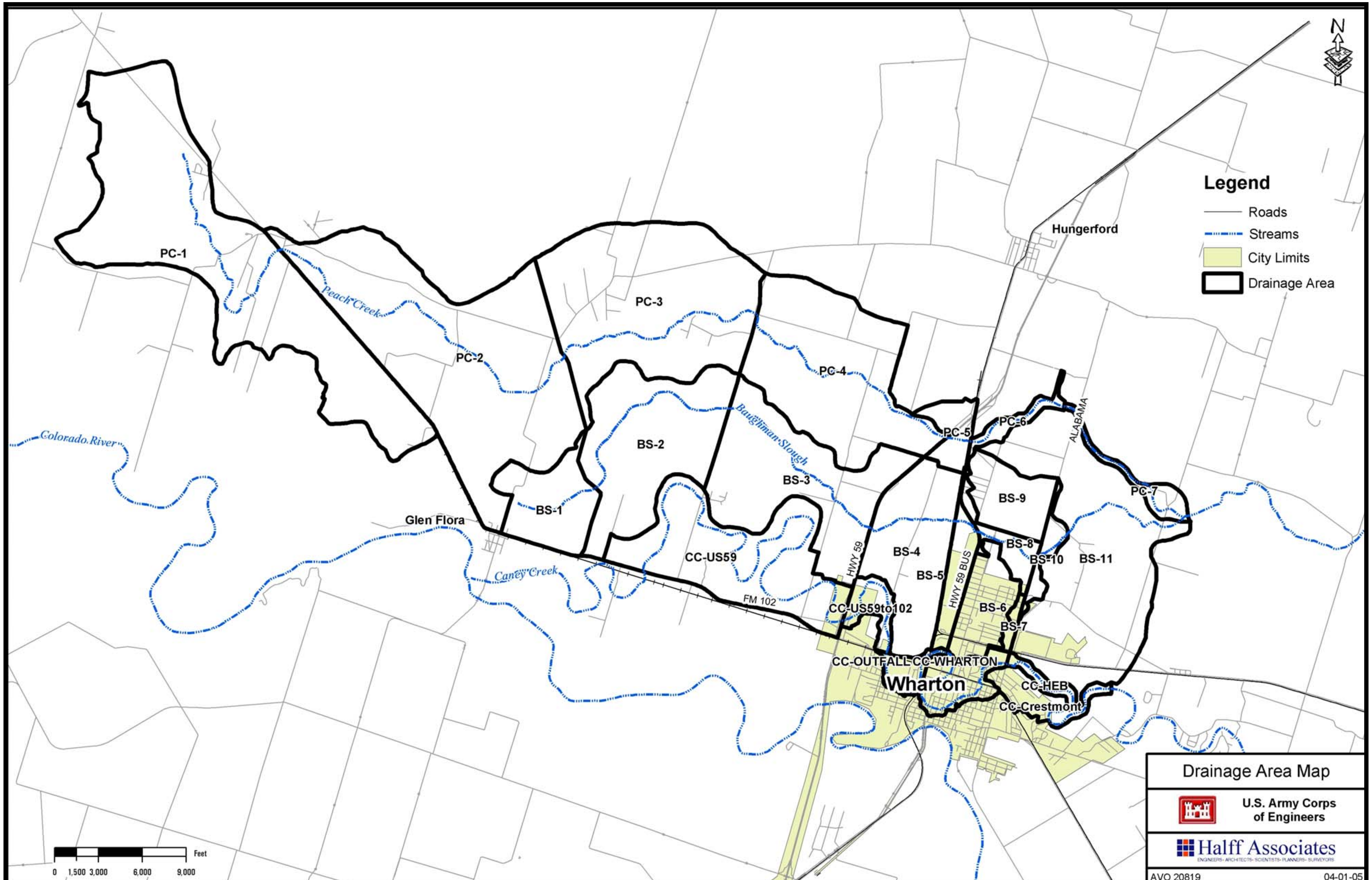


Figure 5. Land Use/ Land Cover Map

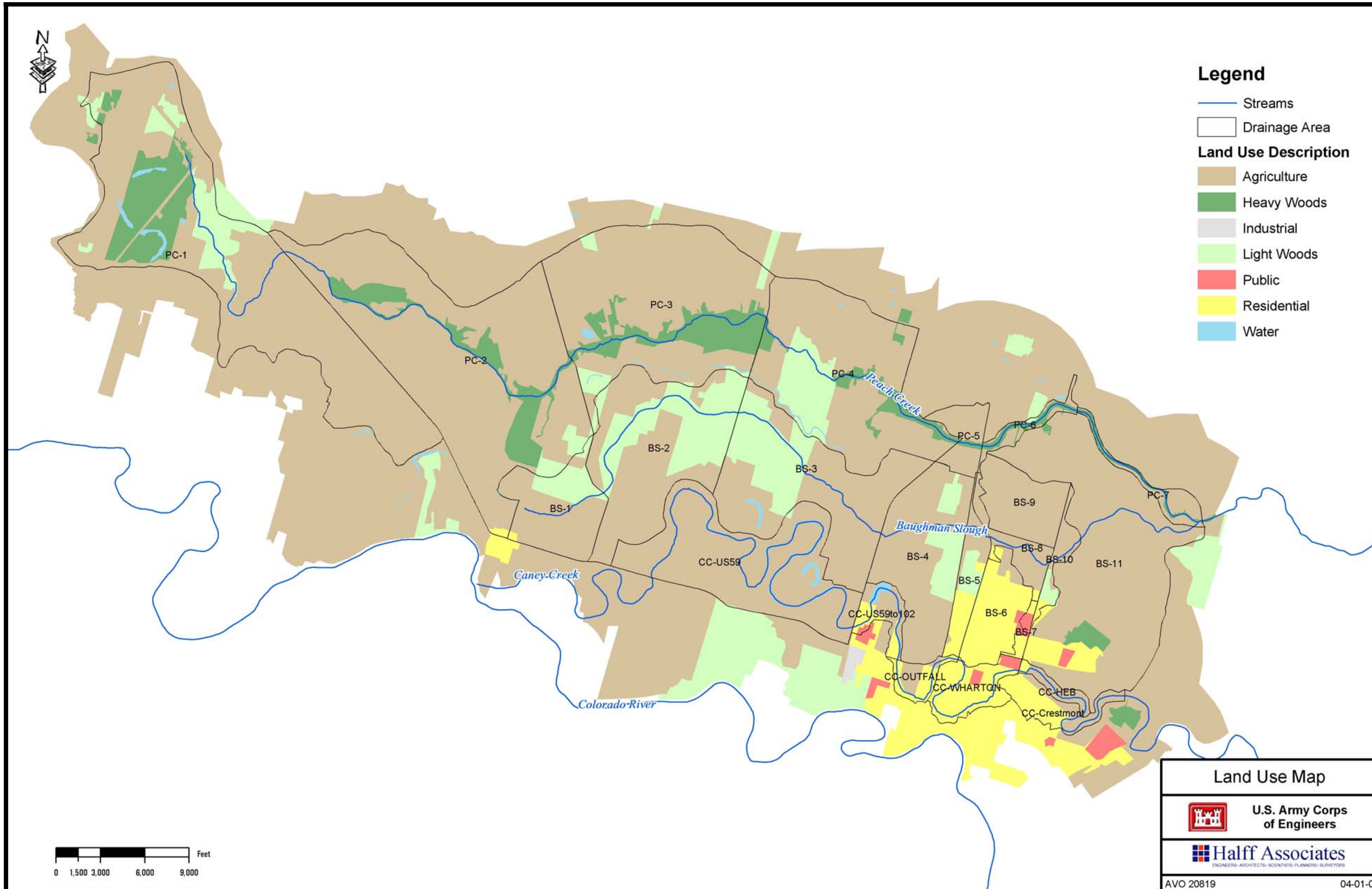


Figure 6. Hydrologic Soil Group Map

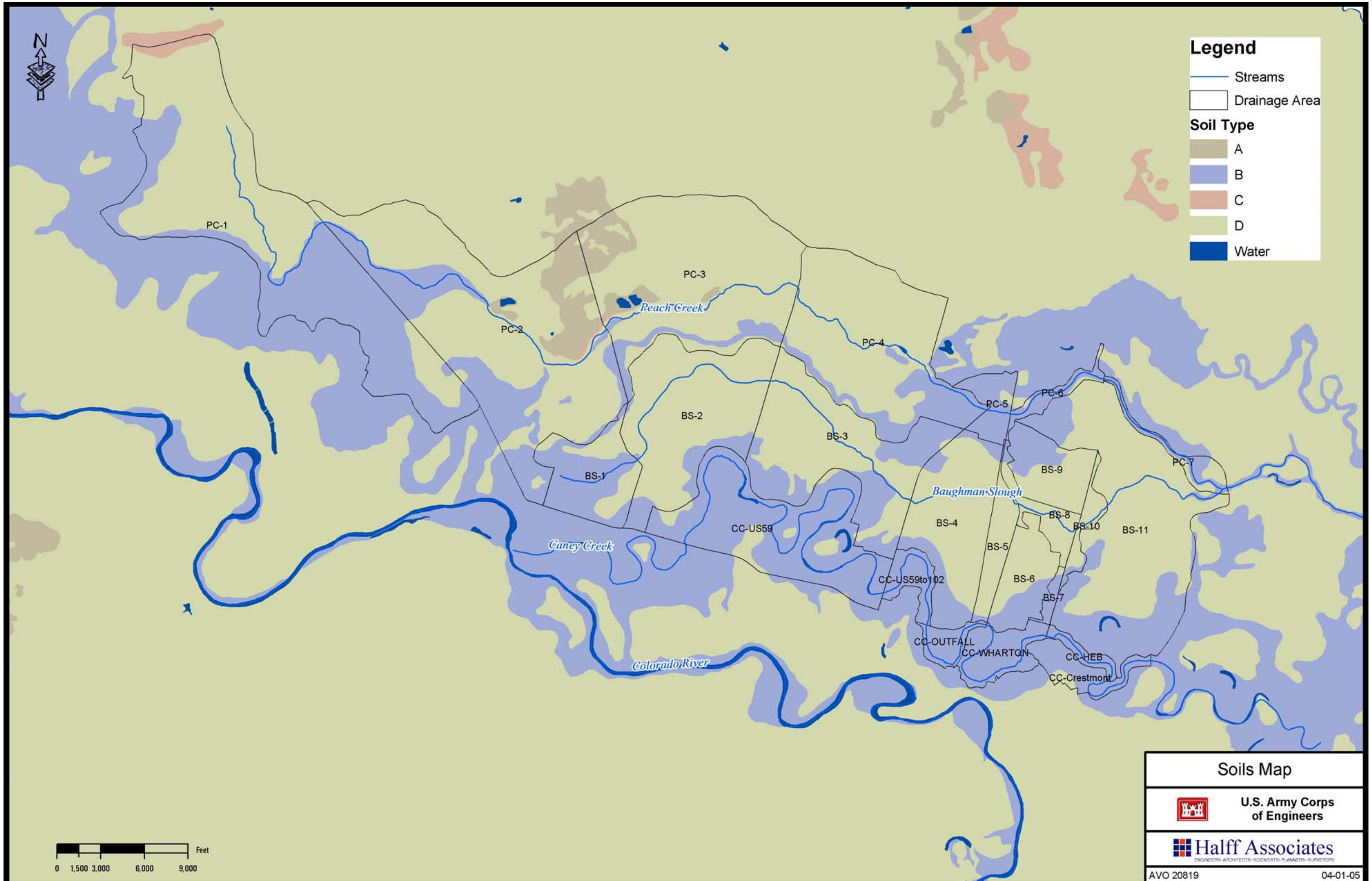


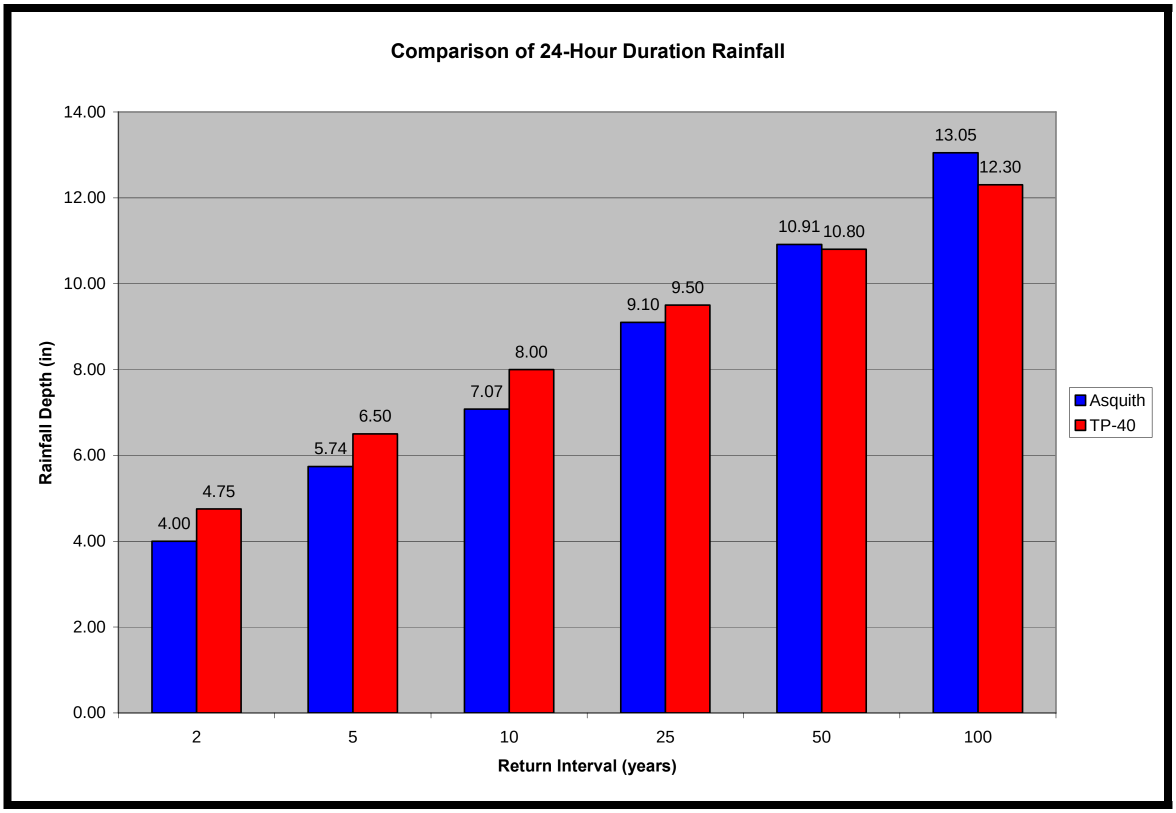
Table 8. Hydrologic Parameters

Watershed	Drainage Area (sq. mi)		Curve Number	Tc (min)	Tlag (min)
	Incremental	Cumulative			
PC-1 (U/S FM 102)	7.13	7.13	81.65	120	72
PC-2 (FM 102-FM 640)	6.73	13.86	81.74	112	67
PC-3 (FM 640-CR 239)	4.95	18.81	83.44	91	55
PC-4 (CR 239-Hwy. 59)	3.84	22.66	83.51	70	42
PC-5 (Hwy. 59-RR)	0.44	23.09	77.23	22	13
PC-6 (RR-CR 135)	0.25	23.34	68.27	39	24
PC-7 (CR 135 – BS Confluence)	0.33	23.67	75.03	52	31
BS-1 (U/S FM 640)	1.25	1.25	78.23	38	23
BS-2 (FM 640-CR 239)	3.11	4.36	83.60	83	50
BS-3 (CR 239-Hwy. 59)	3.17	7.53	82.40	76	45
BS-4 (Hwy. 59-RR)	2.10	9.63	84.55	57	34
BS-5 (RR-Richmond Rd.)	0.51	10.14	85.30	41	25
BS-6 (Ahldag Channels)	0.85	0.85	83.34	33	20
BS-7 (Alabama Ditch to Channel)	0.10	0.95	79.67	15	9
BS-8 (Richmond Rd. – Alabama)	0.40	11.49	88.21	38	23
BS-9 (To CR 150/Alabama Int.)	0.88	0.88	85.03	28	17
BS-10 (Alabama Rd. – CR 150)	0.25	12.63	84.74	26	16
BS-11 (CR 150 – PC Confluence)	4.7	17.33	85.24	69	42
CC-1 (U/S Hwy. 59)	3.12	3.12	80.03	181	109
CC-2 (Hwy. 59 – FM 102)	0.37	3.49	80.22	40	24
CC-3 (Outfall to CR)	0.28	3.76	76.12	28	17
CC-4 (Wharton)	0.64	4.40	76.01	35	21
CC-5 (CC-HEB)	0.17	4.57	77.66	31	18
CC-6 (CC-Crestmont)	0.52	5.08	77.40	51	31
CC-7 (CC-Country Club)	0.91	5.99	82.77	48	29

Table 9. Wharton Rainfall Depths

	Duration									
Frequency	**5-min**	**10-min**	**15-min**	**30-min**	**1-hr**	**2-hr**	**3-hr**	**6-hr**	**12-hr**	**24-hr**
2-Year	0.56	0.94	1.20	1.80	2.25	2.70	3.00	3.50	4.10	4.75
5-Year	0.63	1.06	1.36	2.25	2.85	3.55	3.90	4.75	5.55	6.50
10-Year	0.68	1.16	1.49	2.60	3.30	4.20	4.60	5.55	6.60	8.00
25-Year	0.77	1.31	1.69	3.00	3.75	4.75	5.30	6.60	8.00	9.50
50-Year	0.83	1.43	1.85	3.30	4.20	5.35	6.00	7.50	9.00	10.80
100-Year	0.90	1.55	2.00	3.60	4.60	6.00	6.75	8.30	10.20	12.30
500-Year	1.07	1.82	2.36	4.28	5.60	7.30	8.40	10.10	13.20	16.60

Figure 7. TP-40 Rainfall Depths vs. Asquith Rainfall Depths



FREQUENCY RESULTS

HEC-HMS simulations were made for the local storms for each sub-watershed. Although the sub-watersheds are linked with routing data in HEC-HMS, only the runoff hydrographs from each sub-watershed were input into HEC-RAS prior to any routing and combining of hydrographs. HEC-RAS routed and combined the sub-watershed hydrographs in an unsteady environment along the main stems of each stream, accounting for overflows, storage, and attenuation.

Routed and combined hydrographs were only entered into HEC-RAS for two locations not on the main stem of Baughman Slough. The Ahldag subdivision channel (BS-6) was combined with the Alabama Ditch U/S of the Ahldag channel (BS-7) and routed down the Alabama ditch to Baughman Slough. This routed hydrograph was input as a lateral inflow at the Alabama Road crossing of Baughman Slough in HEC-RAS. The same procedure was used to route the flow arriving at the intersection of Alabama Road and CR 150 down the CR 150 ditch to the confluence with Baughman Slough. Detailed discussions of the hydraulic modeling will be presented later in the report. Peak flow rates generated in HEC-HMS are presented in Table 10 for each sub-watershed for all frequency storm events modeled.

Table 10. HEC-HMS Peak Discharges

	Frequency						
Watershed	**2-YR**	**5-YR**	**10-YR**	**25-YR**	**50-YR**	**100-YR**	**500-YR**
PC-1 (U/S FM 102)	3,600	6,115	7,980	9,610	11,090	12,590	16,010
PC-2 (FM 102-FM 640)	3,595	6,075	7,910	9,515	10,970	12,435	15,795
PC-3 (FM 640-CR 239)	3,245	5,315	6,820	8,145	9,335	10,505	13,220
PC-4 (CR 239-Hwy. 59)	3,025	4,880	6,215	7,415	8,475	9,495	11,910
PC-5 (Hwy. 59-RR)	510	840	1,070	1,305	1,490	1,670	2,110
PC-6 (RR-CR 135)	140	280	385	490	580	665	890
PC-7 (CR 135 to BS Confluence)	235	410	545	670	775	880	1,130
BS-1 (U/S FM 640)	1,200	2,000	2,575	3,120	3,575	4,015	5,070
BS-2 (FM 640-CR 239)	2,245	3,640	4,645	5,545	6,340	7,120	8,930
BS-3 (CR 239-Hwy. 59)	2,350	3,845	4,925	5,900	6,760	7,595	9,560
BS-4 (Hwy. 59-RR)	2,025	3,165	3,975	4,715	5,360	5,975	7,435
BS-5 (RR-Richmond Rd.)	615	930	1,155	1,360	1,540	1,705	2,105
BS-6 (Ahldag Channels)	1,070	1,640	2,040	2,420	2,740	3,040	3,760
BS-7 (Alabama Ditch to Channel)	160	245	300	360	405	450	555
BS-6 + BS-7 (Routed to BS)	735	1,490	1,965	2,420	2,780	3,135	3,950
BS-8 (Richmond Rd. – Alabama)	555	810	980	1,145	1,290	1,420	1,735
BS-9 (To CR 150/Alabama Int.)	1,270	1,885	2,305	2,725	3,070	3,390	4,170
BS-9 Routed Down CR150 Ditch	1,230	1,855	2,285	2,700	3,045	3,365	4,135
BS-10 (Alabama Rd. – CR 150)	380	565	690	815	915	1,010	1,240
BS-11 (CR 150 to PC Confluence)	3985	6,265	7,890	9,355	10,650	11,885	14,810
CC-1 (U/S Hwy. 59)	1,120	1,965	2,605	3,185	3,700	4,245	5,470
CC-2 (Hwy. 59 – FM 102)	380	610	775	925	1,060	1,185	1,490

	Frequency						
Watershed	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR
CC-3 (Outfall to CR)	280	475	610	745	855	960	1,220
CC-4 (Wharton)	575	990	1,285	1,570	1,810	2,040	2,595
CC-5 (CC-HEB)	185	300	380	465	530	595	745
CC-6 (CC-Crestmont)	400	690	900	1,095	1,260	1,425	1,810

Although peak flow rates are reported as generated by HEC-HMS, these hydrographs were input as uniform lateral inflows into the HEC-RAS model. Combined and routed hydrographs produced by HMS are much different than those calculated by Unsteady HEC-RAS. The unsteady modeling environment takes into account attenuation, storage, and timing. Overflow losses are also accounted for with the Unsteady HEC-RAS program. The combined hydrographs computed with steady routing in HEC-HMS are steep and short with high peaks. The combined hydrographs routed with Unsteady HEC-RAS are much flatter and broader than those generated by HMS.

HYDRAULIC MODELING - OVERVIEW

Hydraulic modeling of the City of Wharton area was completed for both existing conditions and to analyze the potential impacts of various flood reduction alternatives. Since the City of Wharton is subject to both Colorado River overflow floods and localized event flooding, a probabilistic approach was utilized to determine final frequency water surface elevations both for existing conditions and proposed alternative conditions.

GENERAL

Hydraulic modeling of the Colorado River, Caney Creek, Baughman Slough, and Peach Creek in and near the City of Wharton was complex considering the flat terrain and numerous overflow points and paths. HEC-RAS Version 3.1.1 Unsteady was used for the modeling effort. The benefits of Version 3.1.1 include the ability to model lateral weirs, including those weirs that contain culverts. This feature was needed for several locations in the City of Wharton. The Colorado River was studied with Unsteady modeling techniques for the FDEP, and the lateral weirs and flow interchanges between streams created problems for a HEC-RAS steady state model.

TOPOGRAPHIC & SURVERY DATA

Topographic data for the study was based on 1998 aerial digital orthophotography at two-foot contour intervals. Digital contour mapping (2-foot contours interpolated to 1-foot contours) was used within the City of Wharton to supplement the topographic data. Detailed bridge surveys were obtained in the field. Channel cross-sections were surveyed approximately every one-half mile along the Colorado River and near the Peach Creek and Baughman Slough confluence. Cross-sections for the hydraulic modeling were generated with HEC-GeoRAS based on the 1998 topographic data. The bridge surveys were manually added to the hydraulic models and blended with the overbank topography. Cross-sections between bridges were based on the 1998 topography for the streams other than the Colorado River. However, due to dense channel vegetation and water in the channel, the channel bathymetry could not always be represented by the aerial topography alone. Channel sections were interpolated based on field surveys at the structures.

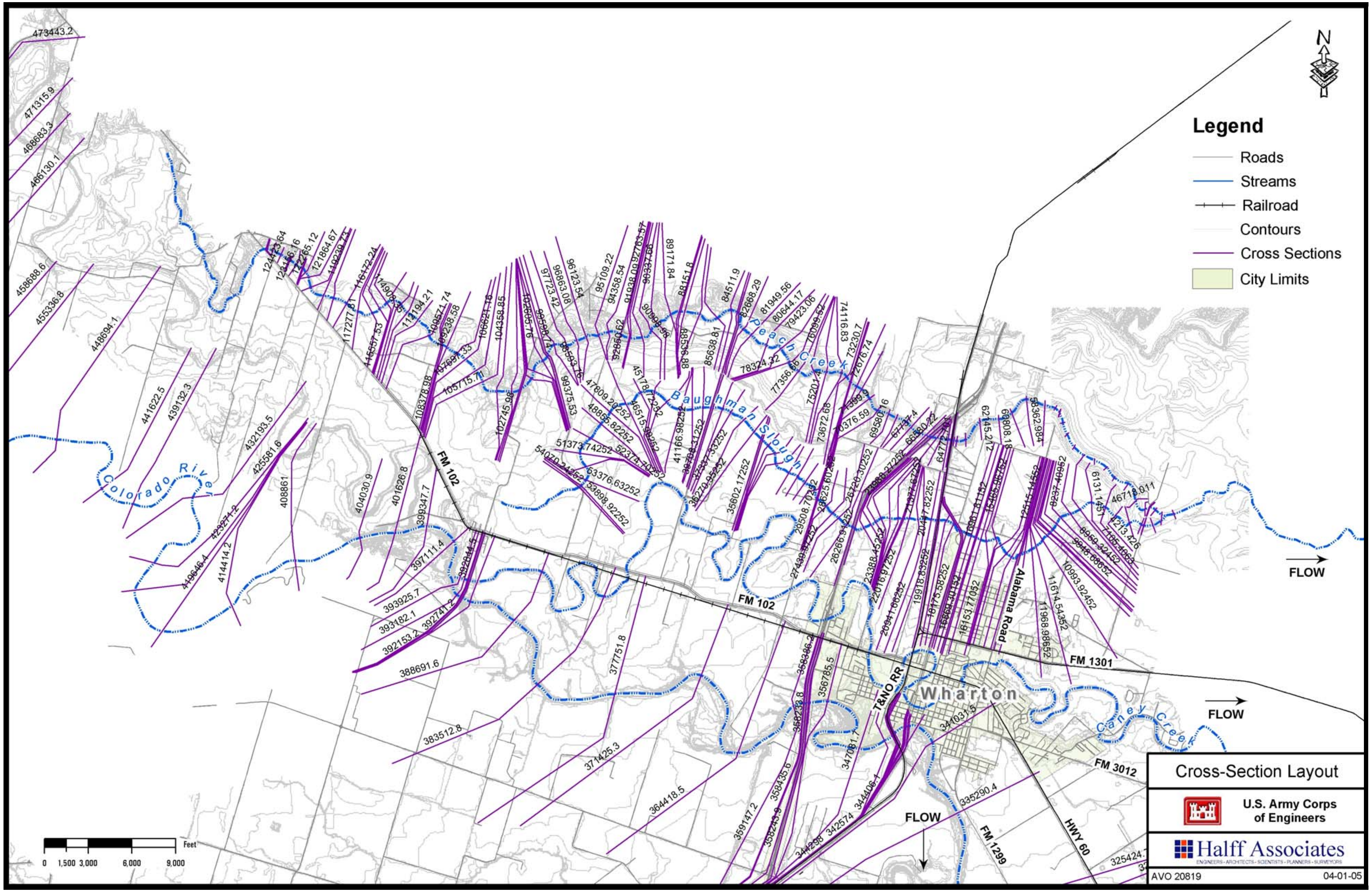
Surveyed channel cross-sections were available for Peach Creek downstream of CR 135 and Baughman Slough downstream of CR 150. Due to the narrow drainage area of Peach Creek between CR 135 and the Baughman Slough confluence, the survey data was used entirely for both the channel and the overbanks. For Baughman Slough between CR 150 and the Peach Creek confluence, the 1998 topographic data did not exist. In this area, the channel surveys were blended with 1940 Wharton County 1-foot topographic data in the overbank areas.

CROSS SECTION LAYOUT & HYDRAULIC PARAMETERS

Cross-sections already existed for the Colorado River as a result of the FDEP study, and the same cross-section locations were maintained for the Wharton Interim Feasibility Study. Cross-section locations and alignments for Peach Creek and Baughman Slough were based on topography and direction of overbank flow. Cross-sections on these two streams were placed an average of one section every 800 feet. Figure 8 provides the location and layout of hydraulic cross-sections used in this study. HEC-GeoRAS produced the elevation and station pairs for each cross-section based on the 1998 topographic and survey data. As discussed previously, channel bathymetry was manually blended with the HEC-GeoRAS cross-sections to produce the final cut lines used in the hydraulic modeling.

Photographs of all the creeks and rivers near the City of Wharton were made and these were used to establish both channel and overbank roughness values for the hydraulic modeling. Field photographs of structures were also used to verify bridge surveys and to aid in the development of the hydraulic models.

Figure 8. Cross-Section Layout



In addition to these photographs, aerial video footage of the October 1998 flood event was studied to determine overflow points and flood-prone areas within the city.

LATERAL WEIRS

The topography of Wharton results in major overflows and interchanges of flow between the Colorado River, Caney Creek, Baughman Slough, and Peach Creek during significant flood events. In order to generate accurate inundation areas, the complex interaction of these systems around Wharton had to be modeled hydraulically. Lateral weirs were included in the HEC-RAS model along the drainage divides of each of the streams at locations in which overflow could occur. The lateral weir profile was based on the natural ground profile along the stream drainage divides. A lateral weir coefficient of 2.4 was used for those locations in which overflow between watersheds occurred over natural ground. Lateral weirs were also used in locations in which structures acted as drainage divides between streams. The profile along the top of the structure was used to establish the lateral weir profile.

Along the Colorado River in Wharton County, flow can escape from the left overbank into Peach Creek, Baughman Slough, and Caney Creek. Lateral weirs were included in the HEC-RAS models along the left overbank of the Colorado River for a distance of approximately 24 miles. The extent of the Colorado River lateral weirs is river station (RS) 471315.9 (about 13 miles upstream of Glen Flora) downstream to near U.S. Business Highway 59 in the City of Wharton (RS 342574). Overflow above Glen Flora that spills into Caney Creek was ignored as Caney Creek acts as a tributary of the Colorado River above Glen Flora. Below Glen Flora, Caney Creek splits from the Colorado River into its own watershed. The lateral weir for the Colorado River overflow area above Glen Flora was positioned on the divide between the Colorado River (and Caney Creek) and Peach Creek. Overflow in this area near Egypt, Texas, passes through (via culverts at the Peach Creek crossing) or over FM 102 into Peach Creek.

FM 102 was treated as the divide between the Colorado River and Baughman Slough and Caney Creek from Glen Flora to U.S. Highway 59 just west of the City of Wharton. The few culverts under FM 102 are either small, full of sediment, or smashed, and offer little if any significant flow interchange between the Colorado River and other watersheds without overtopping of FM 102. Downstream of U.S. Highway 59 to Business Highway 59 in Wharton, the Colorado River lateral weir follows the drainage divide between the Colorado River and Caney Creek. Lateral weirs were also included along the drainage divides between Caney Creek and Baughman Slough, as well as Baughman Slough and Peach Creek. This allowed for accurate modeling of flow interchange between watersheds and streams.

STORM DRAIN OUTFALLS TO THE COLORADO RIVER

Several storm drain outfalls were considered in the modeling of the local storm events. A 48” reinforced concrete pipe extends 1,050’ from Caney Creek near the intersection of Hughes Street and West Spanish Camp Road along Hughes Street and outfalls into an open channel running parallel to Sheppard Street behind Burleson, Kearney, and Damon Streets. This channel outfalls to the Colorado River on the northwest corner of the City Landfill. No flap gates are installed at the outfall of the 48” pipe.

The City of Wharton between Richmond Road and Alabama Road is drained by a storm sewer system outfalling into the Colorado River near the intersection of Rusk Street and Elm Street. The outfall pipe is a 54” pipe with flap gates installed to prevent Colorado River water from backing up through the storm sewer system. A 42” outfall to the Colorado River near Richmond Road drains a smaller area bounded by Caney Street, the Railroad, Richmond Road, and the River.

The other major outfall to the Colorado River is a box culvert under Alabama Road on the east side of Wharton. The system originates at the low-lying park area near the intersection of

Santa Fe Road and Alabama Road with an 8' x 6' box culvert. At Milam road the box increases to a 12' x 8' box before outfall into an open channel southwest of the intersection of Wisteria and Alabama Roads. Flap gates have been installed at the outfall of the Alabama Road box culvert.

Wal-Mart Stores constructed a Super Wal-Mart Center at the northwest corner of the Intersection of FM 102 and U.S. Highway 59 in 2003. An earthen channel was constructed to drain the Wal-Mart site and flow from Caney Creek along the west side of the Wal-Mart property, under FM 102, and south to the Colorado River relief bridge channel at U.S. Highway 59. This channel will not only drain the Wal-Mart site to the Colorado River, but it will also bypass a portion of local Caney Creek flow to the Colorado River. A weir at an approximate crest elevation of 98.0' feet was constructed along Caney Creek at the upstream end of the Wal-Mart channel. The trapezoidal channel has an 18' bottom width and 3:1 side slopes for a channel depth of just under nine feet. The channel will flow at a 0.0015 ft/ft slope from Caney Creek to the relief channel of the Colorado River. Total capacity of the channel based on Manning's equation will be approximately 1,800 cfs.

The channel crossing of FM 102 consists of 2-10' x 5' box culverts, and 2-7' x 7' flap gates were installed just downstream of FM 102. The effects of the Caney Creek Outfall Ditch were simulated in the HEC-RAS models to determine impacts on water surface elevations near and within the City of Wharton. Figure 9 shows the location of the existing storm sewer system outfalls to the Colorado River.

TAILWATER EFFECTS FROM THE COLORADO RIVER

In order to consider the effects (benefits) of the storm drain outfalls during local events, it was important to establish a Colorado River water surface elevation as a tailwater condition for the outfalls. An analysis of the historic period-of-record at Wharton was analyzed to determine an appropriate Colorado River elevation during a local storm event.

Although the possibility of having a peak on the Colorado River occur simultaneously with a local storm event exists, the probability is extremely small when considering the magnitude of difference in drainage area size between the lower Colorado River basin and the local drainage areas around Wharton. In order to determine appropriate tailwater conditions at the outfalls into the Colorado River during local events, historic records were researched. Fifty years (1949-1999) of historic daily precipitation values in the Wharton area were compared to corresponding mean daily Colorado River flows. The computed local rainfall frequency is based on a 24-hour storm event using TP-40 data as discussed previously. The historic records are summarized in Table 11.

Table 11 indicates that in the Wharton fifty year period-of-record from 1949-1999, significant local rainfall has not occurred when the Colorado River flow is above a 5-year frequency (45,700 cfs – based on Basinwide FDEP Study).

A comparison was also made to determine local Wharton rainfall depths at times when the Colorado River is experiencing significant flows. The historic period of record (1949-1999) at Wharton was investigated once again to determine the dates at which recorded mean daily flows in the Colorado River exceeded 40,000 cfs. Results from this analysis further support the small chances of a Colorado River flood occurring simultaneously with a local flood. Although some large local events have occurred in the days prior to higher Colorado River flows (1960, 1961, 1994), the runoff from these storms would have passed into the Colorado River within a day due to the smaller time of concentrations for the local areas. Based on the historic local events and period-of-record data, a 5-year Colorado River water surface elevation (tailwater) was used for the local storm system outfall ratings.

CANEY CREEK STORAGE AREAS

As discussed previously, the Caney Creek channel near and through the City of Wharton has been filled and impounded in numerous locations. Caney Creek upstream and downstream of the City of Wharton is primarily a series of private ponds and dams, and does not experience true riverine flow. The Caney Creek channel throughout the City of Wharton has been filled in many locations and is undefined. Based on these observations, Caney Creek was modeled for this study as a series of storage areas.

Six distinct Caney Creek storage areas were defined and elevation versus volume relationships established. Figure 4 shows the location of the six Caney Creek areas. The storage areas extended from the FM 102 crossing of Caney Creek between Glen Flora and Wharton to a private dam upstream of Old Caney Road, southeast of the City of Wharton. Although Caney Creek splits from the Colorado River just south of Glen Flora, the culvert under FM 102 is smashed and filled with sediment making it virtually ineffective. Several channels are also present between Caney Creek and the Colorado River upstream of the FM 102 crossing. As a result, Caney Creek flow upstream of the FM 102 crossing was treated as Colorado River overflow and was required to overtop FM 102 to enter the most upstream Caney Creek storage area (CC-US59).

Lateral weirs were included in the HEC-RAS model on all sides of the Caney Creek storage areas to allow proper flow exchanges between Caney Creek, Baughman Slough, and the Colorado River. Proper culvert sizes, flowlines, and top-of-road profiles were used as the storage area connections along Caney Creek to accurately simulate flow movements and interactions. Figure 10 provides a graphical flowchart of the Caney Creek storage areas and connections. Storm sewer outfalls (Caney Creek Outfall Ditch, Hughes Street, Rusk Street, and the Alabama Road box culvert) were included in the Caney Creek lateral weirs for local storm events to allow flow to pass from Caney Creek to the Colorado River via pipes and culverts.

Figure 9. Storm Drain Outfall Locations

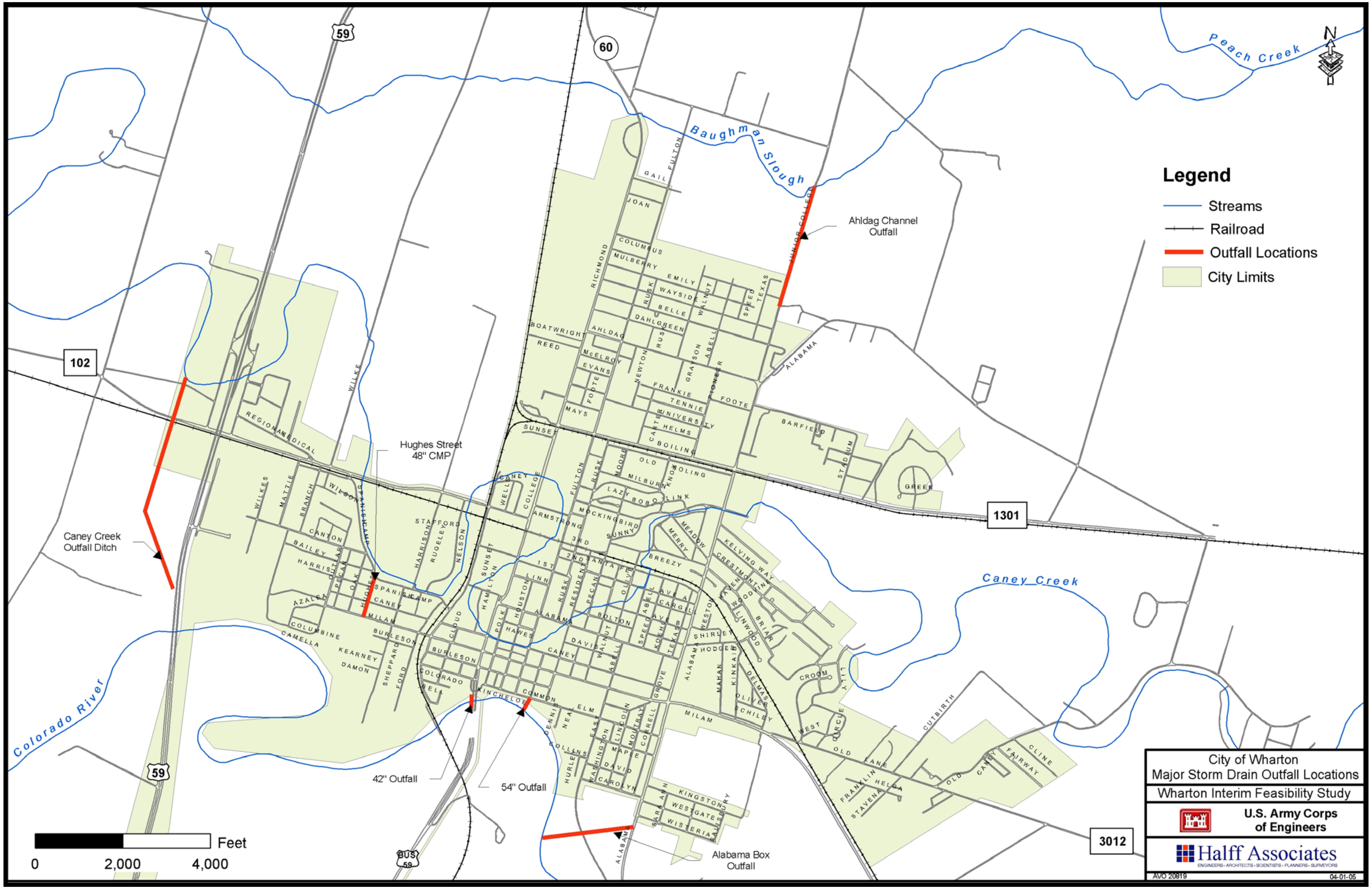


Table 11. Wharton Local Rainfall vs. Corresponding Mean Daily Colorado River Flow

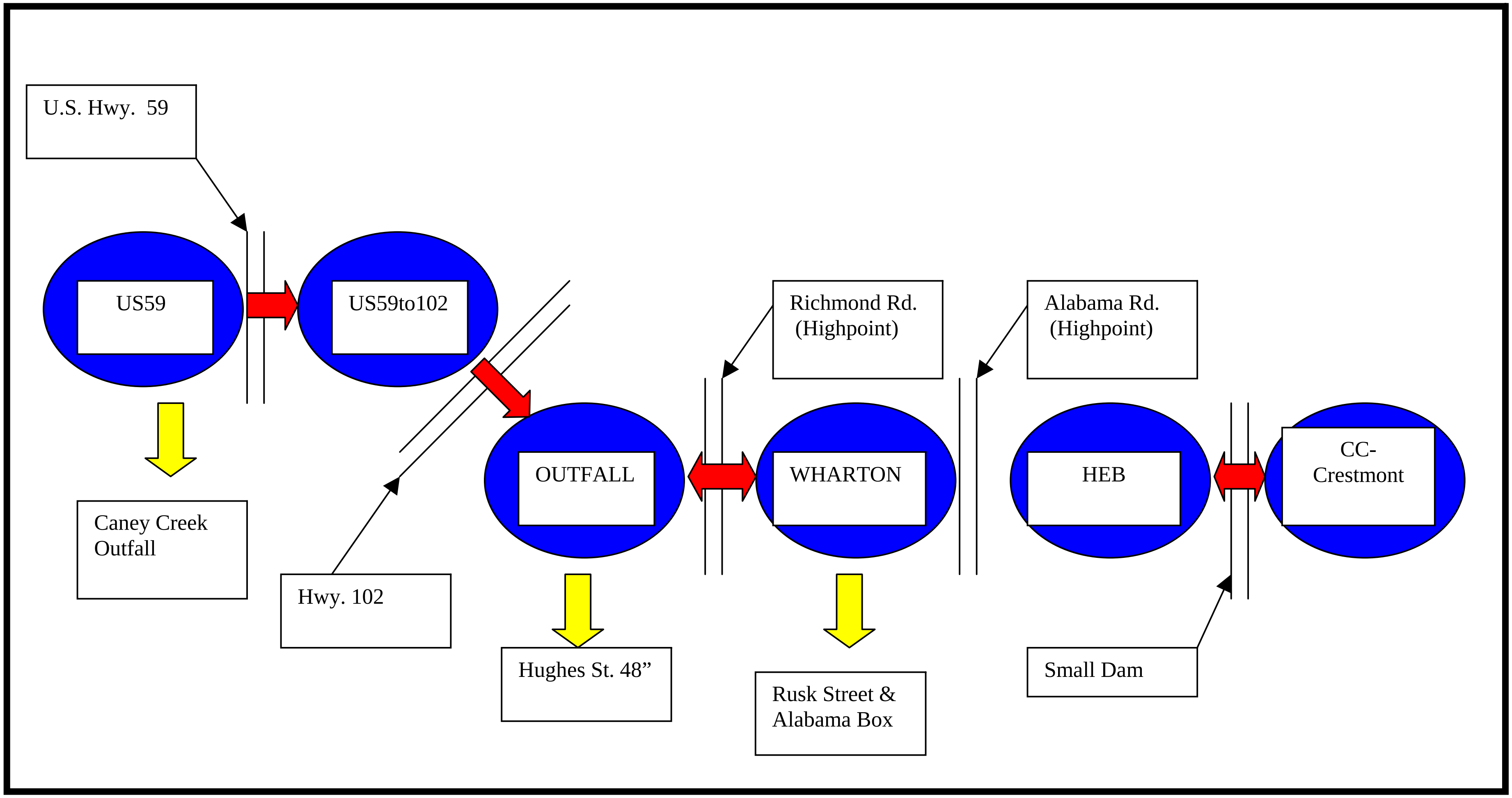
Date	24-hr Rainfall Depth (in.)[1]	Rainfall Frequency	CR Flow (cfs)[1]
Oct. 8, 1949	5.03	2.3-year	2,440
May 28, 1952	6.15	4.2-year	12,300
Aug. 30, 1953	6.90	6.25-year	3,240
Jun 26, 1960	7.48	7.7-year	41,500
Sep. 12, 1961	9.69	27.8-year	17,600
Oct. 12, 1970	6.53	5-year	5,450
May 11, 1972	6.13	4.2-year	11,800
Mar. 24, 1973	4.82	2.1-year	6,250
Oct. 12, 1973	5.36	2.8-year	4,550
Nov. 1, 1974	4.82	2.1-year	5,890
May 30, 1975	5.68	3.3-year	34,100
May 31,1975	5.13	2.4-year	31,200
Sept. 19, 1979	5.78	3.4-year	3,910
Sept. 20, 1979	4.70	2-year	11,300
May 6, 1982	5.10	2.4-year	2,730
Sept. 19, 1983	7.55	8-year	3,140
Oct. 16, 1983	5.59	3.1-year	1,650
Apr. 4, 1991	5.55	3-year	2,460
May 15, 1992	4.80	2.1-year	2,870
Oct. 18, 1994	11.58	71-year	32,000
Apr. 11, 1997	4.94	2.2-year	12,600
Sep. 11, 1998	6.63	5.4-year	3,490

[1] Obtained from EarthInfo Summary of the Day and USGS Daily Values – 2000 CD

For the overflow simulations, the Hughes Street 48" outfall was included in the HEC-RAS models. The Caney Creek Outfall Ditch, Rusk Street outfall, and Alabama Road box culvert were not included in the overflow simulations because these outfalls include flap gates that would prevent Colorado River flow from backing up into the City of Wharton. The profile of the dam on the most downstream Caney Creek storage area was included to allow weir flow to downstream portions of Caney Creek and provide the most accurate water surface elevations of the Caney Creek storage areas through the City of Wharton.

The most upstream Caney Creek storage area is denoted as CC-US59 and includes the drainage area of Caney Creek from FM 102 between Wharton and Glen Flora to U. S. Highway 59 along the west side of Wharton. FM 102 is the southern boundary of this storage area separating the Caney Creek watershed from the Colorado River watershed. The northern boundary follows the drainage divide between Caney Creek and Baughman Slough. This storage basin is the largest in area, encompassing over 3.1 square miles. From CR 239, near the Orchard subdivision, to U.S. Highway 59, Caney Creek is a continuous series of ponds and dams.

Figure 10. Caney Creek Storage Area and Connection Flow Diagram



CC-US59 is connected hydraulically to CC-59to102 via 3-7'x 4' box culverts at U.S. Highway 59. CC-59to102 includes a small lake and a portion of the Gulf Coast Medical Center property. The medical center is raised to one of the highest elevations within the City of Wharton. Ground elevations in this location range from 106'-109'. The lake was not treated independently of the storage area for this analysis. CC-59to102 is connected hydraulically to CC-OutfalltoCR via a 7'x 6' box culvert at FM 102.

The CC-OutfalltoCR storage area receives its name because of the 48" pipe extending from near the intersection of Hughes Street and Spanish Camp Road along Hughes Street to an outfall near the intersection of Milam and Hughes Streets. At the outfall, flow enters an abandoned oxbow that drains to the Colorado River. This 1,050' length of pipe, in conjunction with the oxbow channel, is designed to pass flow from Caney Creek into the Colorado River. This outfall is in the middle of the CC-OutfalltoCR storage area. Flow in Caney Creek converges at this outfall pipe from FM 102 and also from Richmond Road. Richmond Road is a highpoint along Caney Creek with no culverts allowing flow to pass from one side of Richmond Road to the other. Caney Creek is entirely filled in from the abandoned railroad south of FM 102 to the north side of FM 102. A 36" pipe carries flow between these two locations. The other abandoned railroad running north to south parallel to Richmond Road acts as a levee (boundary) between this area and the CC-Wharton storage area.

The top-of-road profile along Richmond Road acts as a hydraulic connection between water that fills the CC-OutfalltoCR storage area and spills into CC-Wharton. Caney Creek through the CC-Wharton storage area is the least defined of any location. Streets and fill have completely covered any evidence of the Caney Creek channel in many locations. A storm sewer system follows the old channel and discharges water into the Colorado River near Rusk and Elm Streets. The park with the inlet to the Alabama Road box culvert is located within this storage area. The CC-Wharton storage area is connected to CC-HEB with the top of road profile along Alabama Road near the HEB grocery store property.

The CC-HEB storage area is rather small and undeveloped. The downstream boundary of this storage area is a small dam just upstream of the lake near Kelving Way and Lily Street. The storage area immediately downstream of CC-HEB is the CC-Crestmont storage area. This area encompasses the Crestmont neighborhood and the downstream boundary is a private dam just upstream of Old Caney Road.

SAN BERNARD TAILWATER EFFECTS

During the October 22-23, 2002 Wharton Interim Feasibility Study Kickoff Meeting, issues related to the San Bernard River were discussed. Two Wharton County Commissioners felt that the San Bernard River was increasing flooding in the City of Wharton. The commissioners claimed that due to excessive sedimentation and vegetation growth in the San Bernard River channel, flooding has been increased over the years, and the San Bernard cannot properly drain runoff from Wharton County. There was concern related to the stage of the San Bernard River and its effects on Peach Creek and Baughman Slough drainage out of the City of Wharton. In order to determine the impacts of downstream conditions on flood elevations within the City of Wharton, a series of steady state HEC-RAS sensitivity analyses were performed.

The San Bernard River watershed is approximately 130 miles long and covers an area of 1,000 square miles. The river originates in Austin and Colorado Counties and flows southeasterly through or along the boundaries of Fort Bend, Wharton, and Brazoria Counties and outfalls to the Gulf of Mexico approximately eight miles southeast of Freeport, Texas. In its middle and upper reaches, the river traverses predominantly agricultural areas (including Wharton County). The banks are heavily wooded and in some areas, growths of trees and brush fill the shallow valley areas and the locations of the stream bed are often not defined. The San Bernard River is tidal and heavily wooded in the lower forty miles.

The San Bernard River flows along the boundary between Wharton and Fort Bend Counties. Peach Creek flows into the San Bernard River at River Mile 62. Baughman Slough, which drains the northern portion of the City of Wharton is a tributary of Peach Creek. Peach Creek flows approximately 11.8 miles from the Business Highway 59 crossing in Wharton to the confluence with the San Bernard River. Baughman Slough outfalls into Peach Creek 3.6 miles downstream of the Business Highway 59 crossing (8.2 miles upstream of the Peach Creek and San Bernard confluence).

The bed slope of Peach Creek is mild (<.0006 ft/ft) in the area between the City of Wharton and the San Bernard River confluence. According to the November 2001 FIS for Wharton County, the 100-year flood elevation along the San Bernard River at the Peach Creek confluence is 79'.

Although a comprehensive study of the San Bernard River is beyond the scope of work for the Wharton Interim Feasibility Study, stage on the river may affect the backwater and tailwater assumptions at the City of Wharton. Currently, the detailed HEC-RAS models developed for the Feasibility Study terminate just downstream of the Peach Creek and Baughman Slough confluence. Normal depth is assumed on Peach Creek below the confluence as a downstream boundary condition. In order to verify this normal depth assumption, a HEC-RAS sensitivity analysis was performed.

In order to determine the effects of San Bernard River water surface elevations on flooding in the City of Wharton, a rough HEC-RAS model was created. Only Peach Creek was considered in the first analysis. The Peach Creek HEC-RAS model developed for the Wharton Interim Feasibility Study has a downstream boundary just below the Baughman Slough confluence. This most downstream detailed Peach Creek cross-section was copied to the confluence of Peach Creek and the San Bernard River. The flowline of the detailed cross-section at CR 135 was 74'. From the profiles in the November 2001 FIS, the flowline of the San Bernard River at the Peach Creek confluence is 41'. The cross-section copied to the confluence was lowered by 33 feet at all points based on the difference in these two flowlines. This resulted in a flow line at the Peach Creek and San Bernard confluence of 41' (in agreement with the 2001 FIS). A linear interpolation was then used to add cross-sections every 2000 feet between CR 135 and the Peach Creek and San Bernard confluence. A total of 27 cross-sections were interpolated within HEC-RAS.

The first analysis included a downstream boundary condition of known water surface elevation for Peach Creek (at the confluence with the San Bernard River) in steady state HEC-RAS for several elevations near the 100-year San Bernard water surface elevation of 79'. Four flow rates were also selected for Peach Creek to determine the sensitivity of the model at CR 135 (City of Wharton) to varying water surface elevations on the San Bernard River. The results of this sensitivity analysis are shown in Table 12.

Table 12. San Bernard River Sensitivity Analysis

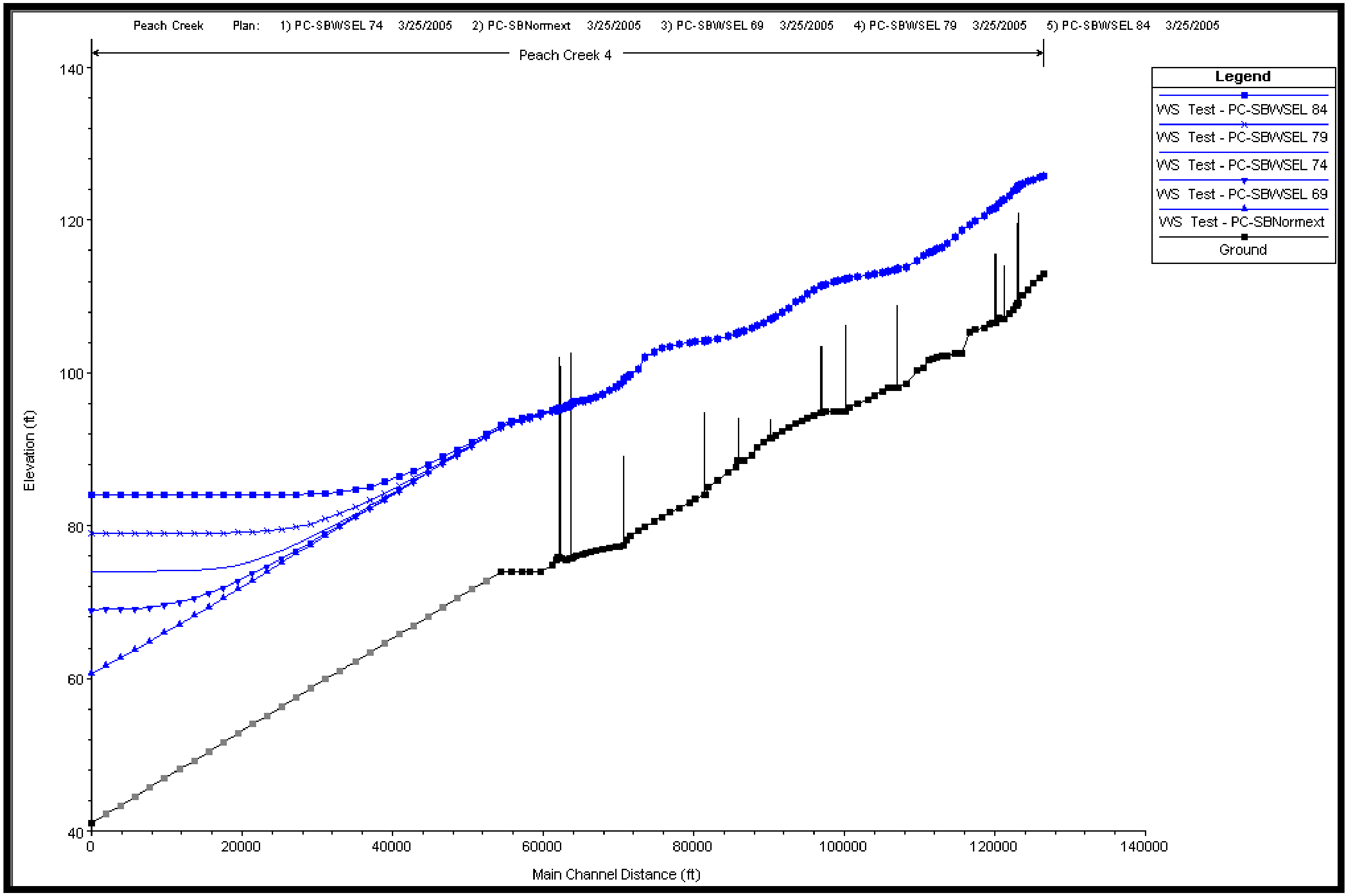
Flow Rate (cfs)	D/S WSEL Boundary (San Bernard & Peach Creek Confluence)	WSEL at CR 135 (D/S Limit of City of Wharton)
10,000	69.0'	92.8'
10,000	74.0'	92.8'
10,000	79.0'	92.9'
10,000	84.0'	93.1'
15,000	69.0'	95.7'

Flow Rate (cfs)	D/S WSEL Boundary (San Bernard & Peach Creek Confluence)	WSEL at CR 135 (D/S Limit of City of Wharton)
15,000	74.0'	95.7'
15,000	79.0'	95.7'
15,000	84.0'	95.7'
20,000	69.0'	96.9'
20,000	74.0'	96.9'
20,000	79.0'	96.9'
20,000	84.0'	96.9'
25,000	69.0'	97.9'
25,000	74.0'	97.9'
25,000	79.0'	97.9'
25,000	84.0'	97.9'

Results of Table 12 indicate that the water surface elevation on Peach Creek in the City of Wharton is not affected by the water surface elevation on the San Bernard River for higher flow rates along Peach Creek. The water surface profiles for Peach Creek with varying downstream boundary conditions (San Bernard tailwater) converge to a single elevation between the confluence and the City of Wharton for each flow. Figure 11 shows the profiles for a flow rate of 20,000 cfs with varying downstream water surface elevation boundary conditions.

The analysis of the San Bernard tailwater impacts indicate that the stage of the San Bernard does not impact water surface elevations along Peach Creek and Baughman Slough within the City of Wharton. The San Bernard stage could affect the water surface elevations of the lower reaches of Peach Creek, but will not affect the Wharton Interim Feasibility Study within the City of Wharton. The normal depth assumption just downstream of the Baughman Slough and Peach Creek confluence is acceptable for the current City of Wharton analysis.

Figure 11. Peach Creek Profiles with San Bernard River Tailwater Sensitivity



LOCAL HYDRAULICS

The local storm hydrographs computed for each sub-basin in HEC-HMS were used as input data to HEC-RAS for unsteady local hydraulic modeling. Flow hydrographs were input as uniform lateral inflows in HEC-RAS except for portions of Baughman Slough where lateral flow enters the creek via a ditch. Peach Creek was modeled from FM 102 to below the Baughman Slough confluence. Normal depth was chosen as the downstream boundary condition of Peach Creek below the Baughman Slough confluence.

Baughman Slough was modeled hydraulically from FM 640 to the confluence with Peach Creek. The Colorado River was modeled from downstream of Glen Flora to Bay City. A 5-year constant flow rate of 43,000 cfs was used in the Colorado River to generate appropriate tailwater elevations for the local storm drain outfalls (See *Tailwater Effects from the Colorado River* section). A normal depth downstream boundary condition was input at the Bay City gauge of the Colorado River. The Caney Creek outfall channel, Hughes Street pipe, Rusk Street pipe, and Alabama box culverts were all included in the hydraulic model to allow the transfer of local flows from Caney Creek to the Colorado River.

Over 4.75 miles of lateral weirs between Peach Creek and Baughman Slough were included in the local hydraulic analysis to allow for interchange of flow between these creeks and watersheds. Over 3.2 miles of lateral weirs were included along the drainage divide between the Caney Creek storage areas and Baughman Slough. Over 6.05 miles of lateral weirs were included between Caney Creek and the Colorado River from upstream of Wharton to below the Alabama box outfall.

FREQUENCY RESULTS

No highwater marks were obtained for previous local storm events on Peach Creek, Baughman Slough, or Caney Creek. Therefore, no data was available for model calibration. Water surface elevations computed at the upstream face of selected bridges along Peach Creek and Baughman Slough for various local frequency storms are shown in Table 13. Top-of-road elevations over the streams are also provided for reference. A local Wharton County official confirmed that CR 239 and CR 235 over Peach Creek are typically closed 1-2 times per year due to highwater.

Table 13. Local Event Water Surface Elevations (ft)

	Frequency							
Location	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year	Top-of-Road Elevation
Peach Creek								
FM 640	105.8	107.4	108.4	109.2	109.8	110.4	111.5	103.4
CR 239	98.2	100.4	101.4	102.2	102.8	103.3	104.6	94.7
CR 235	91.2	93.5	94.9	96.1	97.2	98.3	100.6	89.6
Hwy. 59	87.7	90.8	92.6	94.0	95.2	96.5	98.8	103.5
RR	87.4	90.6	92.3	93.8	94.9	96.2	98.5	102.5
Bus. 59	87.4	90.5	92.3	93.7	94.9	96.2	98.5	100.7
CR 135	86.2	89.1	90.7	92.1	93.2	94.4	96.6	95.2
CR 129	81.7	84.3	86.0	87.1	87.9	89.0	90.7	91.7
BS Confluence	81.5	83.9	85.6	86.6	87.3	88.1	89.7	-

Location	Frequency							Top-of-Road Elevation
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year	
Baughman Slough								
FM 640	112.1	112.5	112.5	112.5	112.5	112.5	112.6	112.5
CR 239	108.1	108.9	109.3	109.6	109.9	110.1	110.6	109.1
CR 235	104.5	105.4	105.8	106.1	106.3	106.5	107.1	107.2
Hwy. 59	102.1	102.6	103.0	103.3	103.5	103.7	104.0	104.5
CR 231	100.2	100.8	101.1	101.5	101.7	102.0	102.6	101.5
RR	99.0	99.6	99.9	100.3	100.7	101.0	101.4	102.4
Bus. 59	98.4	99.0	99.3	99.5	99.6	99.7	100.0	101.2
Fulton Rd.	96.8	97.6	97.8	97.8	97.9	97.9	98.0	98.7
Alabama Rd.	93.6	93.9	94.1	94.2	94.3	94.5	95.7	96.2
CR 150	92.3	92.8	93.0	93.2	93.4	93.5	94.4	91.8
CR 129	84.7	85.9	86.4	87.2	87.8	88.4	89.9	91.3
PC Confluence	81.5	83.9	85.6	86.6	87.3	88.1	89.7	-
Caney Creek								
CC-US59	103.1	104.4	104.9	105.2	105.4	105.6	106.0	-
CC-59 to 102	102.3	103.1	103.4	103.7	103.9	104.0	104.2	-
CC-Outfall	101.2	102.4	103.0	103.3	103.4	103.4	103.5	-
CC-Wharton	99.6	100.3	100.8	101.1	101.4	101.6	101.8	-
CC-HEB	100.2	100.6	101.0	101.1	101.4	101.6	101.8	-
CC-Crestmont	100.2	100.6	101.0	101.1	101.4	101.6	101.8	-

Several areas of overflow between streams occur as a result of local storm events independent of Colorado River flows. Table 14 provides locations where overflow occurs between drainage basins for the various synthetic local storm events.

Table 14. Local Event Overflow Locations

Frequency	Locations of Overflow*
2-year	None
5-year	Caney Creek spills to Colorado River just upstream of Highway 59
10-year	No Additional Overflows
25-year	Caney Creek spills to Baughman Slough downstream of Highway 59 Caney Creek spills to Colorado River just upstream of RR
50-year	No Additional Overflows
100-year	Baughman Slough spills to Peach Creek between Highway 59 and RR

*Overflow Locations are listed for the smallest event (most frequent). Less frequent storms include the overflow areas listed for smaller storms.

OVERFLOW HYDRAULICS

Due to the close proximity of Wharton to the Colorado River and the flat topography in the area, the city is also subject to overflows from large events on the River. The FDEP study was a comprehensive basinwide hydrologic, reservoir, and hydraulic analysis. Results of this study were used as the foundation for the Wharton Interim Feasibility Study.

Although the Feasibility Study was concerned with the City of Wharton, hydraulic models were needed upstream to account for overflows and interchanges above Wharton in order to accurately model the hydraulics within the city. The logical upstream boundary for the Colorado River overflow analysis was the Garwood gauge in Colorado County. This was a model boundary point in the FDEP study, so frequency stage and flow hydrographs along the Colorado River existed at this location. Bay City in Matagorda County was selected as the downstream boundary for the same reason. The total distance of the Colorado River modeled for the Wharton Interim Feasibility Study was over 75 river miles.

Initially, the overflow hydraulics were to be analyzed with one complex HEC-RAS model. However, due to the complexity of the overflows and system near Wharton, the model was unstable without using a small, impractical time step. To overcome this modeling limitation, the Colorado River overflows (2-100 year) were analyzed with three HEC-RAS Unsteady models. Each of the models shared a common upstream or downstream cross-section and boundary condition to insure continuity between the reaches. Iterations were required until the downstream stage hydrograph and flow hydrograph from the upstream model matched the upstream stage and flow hydrograph of the downstream model. Table 15 provides the upstream and downstream boundary conditions for each model used in the overflow hydraulic analysis.

Table 15. HEC-RAS Overflow Model Boundaries

Reach/Model	Stream	Frequency	Upstream Boundary	Downstream Boundary
Wharton Upper	Colorado	2-100 Year	Garwood Flow Hydrograph from FDEP	Stage Hydrograph from Wharton Lower
Wharton Upper	Peach Creek	2-100 Year	Baseflow Hydrograph and Outflow from PC Storage Area	Stage Hydrograph from Wharton Lower
Wharton Lower	Colorado	2-100 Year	Flow Hydrograph from Wharton Upper	Stage Hydrograph from Bay City
Wharton Lower	Colorado	500-Year *	Garwood Flow Hydrograph from FDEP	Normal Depth at Bay City Gauge
Wharton Lower	Peach Creek	2-100 Year	Flow Hydrograph from Wharton Upper	Normal Depth Below Baughman Slough Confluence
Bay City	Colorado	2-100 Year	Flow Hydrograph from Wharton Lower	Stage Hydrograph at Bay City Gauge from FDEP

* The 500-Year Overflow Model reach is from Garwood to Bay City (Peach Creek modeled with storage areas to maintain stability).

The 500-year simulation would not remain stable with Peach Creek treated as a riverine environment due to the extensive overflow occurring from the Colorado River. As a result of this instability, Peach Creek was modeled as a series of storage areas (similar to Caney Creek) for the 500-year event. This model change allowed the 500-year Colorado River overflow event to be simulated with one model as opposed to three.

MODEL SET-UP/FEATURES

Due to the complexity associated with the Colorado River overflows, each of the three individual HEC-RAS models incorporated some unique modeling techniques. The features of each of the HEC-RAS models allowed for an accurate simulation of the complex system of overflows within and near the City of Wharton.

Wharton Upper

The most upstream Colorado River overflow HEC-RAS model is named *Wharton_Upper.prj*. This model extends from the Garwood gauge to cross-section 401626.8, just upstream of Glen Flora, Texas. Thirty-one Colorado River miles are included in this model. Overflow first begins to escape the Colorado River near Egypt, Texas, near the headwaters of Peach Creek. Caney Creek flows between the Colorado River and Peach Creek in this area upstream of Glen Flora. However, Caney Creek outfalls into the Colorado River near Glen Flora. The two water bodies share a common channel for approximately one mile, before Caney Creek splits from the Colorado channel just south of Glen Flora (See the *Caney Creek* section). Since water that escapes the Colorado River into Caney Creek upstream of Glen Flora flows back into the Colorado River, the divide for the upper model was placed along the Peach Creek and Caney Creek (Colorado River) watershed boundary.

Overflow escaping the Colorado River into the Peach Creek watershed must cross FM 102 before flowing downstream along Peach Creek. FM 102 is raised slightly above the natural ground and only one major structure exists to allow water to pass through FM 102. This structure is at the Peach Creek crossing of FM 102 near Egypt, Texas, and consists of 3 - 10'x9' boxes. To model this overflow phenomenon, the area bounded by FM 102 and the Peach Creek/Colorado River drainage divide was modeled as a storage area. Sub-basin PC-1 shown on Figure 4 provides the location of the Peach Creek/FM 102 storage area. An elevation-volume relationship for the area was computed in GIS and input into the HEC-RAS model.

Almost eight miles of lateral weirs along the divide between the Colorado River and Peach Creek were included on the left overbank of the Colorado River to allow overflow to escape into the Peach Creek storage area. Almost three miles of lateral weirs along the top-of-road profile of FM 102 were included on the right overbank of Peach Creek to allow overflow to escape from the storage area and flow downstream along Peach Creek. The 3 - 10'x9' box culverts were included in the lateral weir representing the FM 102 crossing. Initially, this structure was modeled as a culvert, but the model became unstable regardless of time step. To overcome this limitation of the model, the structure was modeled as gates. Gate dimensions were input to match the culvert dimensions and flowline. The "gates" were fully opened during the peak of the Colorado River overflow to allow the water to escape into Peach Creek through the same opening area as the actual culverts. A check was made to ensure that the gate rating computed in HEC-RAS was comparable to the actual culvert rating based on culvert hydraulics. Although the gates are not a perfect representation of the Peach Creek structure at FM 102, the downstream flow hydrograph produced with HEC-RAS is comparable to actual conditions. The Wharton Interim Feasibility Study is primarily concerned with flooding in the City of Wharton. The Peach Creek FM 102 crossing was only modeled because overflow from the Colorado River at this location flows just north of Wharton and impacts flooding within the City of Wharton.

No bridges or culverts were included on this upper reach of Peach Creek other than FM 102. Peach Creek was modeled in a riverine environment from FM 102 to CR 232/CR249 (3.1

miles) in the Wharton Upper model. There were only a few structures along this reach, and most of these were small private driveway structures. The only public road crossing this reach is CR 232/CR 249, which consists of a small wooden bridge. Stability was maintained for this model by removing the small structures along Peach Creek. No significant impacts to computed water surface elevations resulted from the omission of bridges and culverts. Once again, the focus of this study is within the City of Wharton, and minor changes in the upper reaches of Peach Creek should not affect conditions within the city.

Wharton Lower

The Wharton Lower model is the most complex of the three overflow simulations. This model includes the City of Wharton, the entire length of Baughman Slough, Peach Creek to the confluence with Baughman Slough, Caney Creek, and the Colorado River. River and stream miles for this model are shown in Table 16.

Table 16. Wharton Lower Model Stream Miles

River/Stream	Length in Model
Colorado River	11.2 miles
Baughman Slough	11.3 miles (entire stream)
Peach Creek	12.5 miles

Colorado River overflows spill into the Caney Creek storage areas, which spill into Baughman Slough. For the larger storms, significant flow in Peach Creek from the upper storage area spills into Baughman Slough as well. A junction was included in the model at the Peach Creek and Baughman Slough confluence to simulate the interactions of these two streams at this point. HEC-RAS was able to simulate this complex interaction of overflows, storage areas, a junction, and riverine hydraulics that occurs within the City of Wharton. All the bridges and culverts along Peach Creek, Baughman Slough, and the Colorado River within this reach were included in the models. These bridges were modeled with the energy method for both high and low flows. This helped to keep the model stable during the simulations. Since many of the bridges are overtopped (See *Frequency Results* section) during larger flood events, HEC-RAS would automatically utilize the energy method for high flow computations once the bridge was highly submerged.

Over 5.3 miles of lateral weirs were included along the Peach Creek and Baughman Slough drainage divide in this model to allow for the interchange of flow between these watersheds. Over 3.2 miles of lateral weirs were incorporated in the model to simulate the flow interactions between the Caney Creek storage areas and Baughman Slough. Over seven miles of lateral structures were included along the left overbank of the Colorado River in this model to allow for overflow of the mainstream.

Bay City Model

The Bay City model includes only the Colorado River. It extends from downstream of the Business Highway 59 Bridge in Wharton to the Bay City gauge. This reach covers a distance of 33.3 river miles along the Colorado River. The Bay City model was needed to generate water surface elevations along the Colorado River in the downstream portion of the City of Wharton. Boundary conditions existed at the Bay City gauge from the FDEP study, so this location was used as the downstream limit of the model. No lateral weirs exist within this model since the Colorado River is the only stream included.

CALIBRATION TO OCTOBER 1998 EVENT

Prior to frequency storm simulations, calibration of the HEC-RAS model was required. The most comprehensive historical data for Wharton flooding was the October 1998 flood event (See *Historic Floods* section). Flow and stage hydrographs existed for this event at the Garwood, Wharton, and Bay City gauges. A few highwater marks also existed along the Colorado River and in the West End neighborhood. Manning's "n" values in the Colorado River channel were raised to 0.055 in areas near the City of Wharton, and remained at 0.05 for other areas of the mainstream as a result of the calibration to the 1998 event. Table 17 provides a comparison of the calibrated high water marks and observed high water marks for the 1998 flood.

Table 17. HEC-RAS Model Calibration to 1998 Highwater Marks

Location	Observed 1998 HW Mark	Computed WSEL
Cross-section 477128	133.9'	134.0'
Cross-section 466130	131.3'	131.3'
Cross-section 441622.5	124.9'	125.5'
Cross-section 408861	120.0'	120.0'
FM 960	115.4'	115.2'
Cross-section 377751.8	107.3'	108.7'
Hwy. 59	105.0'	104.5'
CC Outfall	103.6' (Dawson Elementary)	103.3'
Wharton Gauge	101.1' (74,800 cfs)	101.1' (72,245 cfs)

Further verification of the accuracy of the HEC-RAS model was obtained by aerial photographs and video of the 1998 event. Flow did not spill into the Peach Creek storage area near FM 102 in 1998 nor in the model. In 1998 flow spilled over Richmond Road into the CC-Wharton storage area. Flow also backed-up through the Alabama Box culvert into the CC-Wharton area. Photographs reveal that the water surface elevation along Caney Creek within the CC-Wharton storage area was approximately 100.0' during the 1998 event. The HEC-RAS model computed a maximum CC-Wharton storage area water surface elevation of 100.0'.

FREQUENCY RESULTS

Following the calibration of the overflow models to the 1998 event, Colorado River frequency storm events were simulated in the Wharton area. Flow hydrographs at Garwood and stage hydrographs at Bay City developed in the FDEP study were used as the extreme upstream and downstream boundary conditions, respectively. Several iterations of the three overflow models were required before common boundary condition stage and flow hydrographs matched.

Table 18 provides maximum Colorado River overflow water surface elevations at selected locations within the study area. Flooding and water surfaces along Caney Creek, Baughman Slough, and Peach Creek are strictly the result of Colorado River overflows. No local rainfall and runoff were modeled simultaneously with the overflows.

Table 18. Overflow Event Water Surface Elevations (ft)

	Frequency	

Location	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year	Top-of-Road Elevation
Colorado River								
Garwood Gauge	149.3	152. 7	153.5	154.4	155.1	155.8	158.9	-
FM 960	99.6	107.2	111.6	115.9	117.8	118.4	119.1	121.9
Hwy. 59	90.3	97.2	101.0	104.3	105.9	106.2	106.7	115.0
RR	86.1	93.3	97.4	100.9	102.4	102.9	103.3	106.5
Bus. 59	86.1	93.2	97.3	100.7	102.2	102.7	103.1	110.0
Peach Creek								
FM 640	-	-	-	-	105.0	113.6	116.5	103.4
CR 239	-	-	-	-	96.8	106.6	112.1	94.7
CR 235	-	-	-	-	89.6	103.1	109.1	89.6
Hwy. 59	-	-	-	-	85.4	100.7	107.1	103.5
RR	-	-	-	-	85.1	100.2	106.5	102.5
Bus. 59	-	-	-	-	85.1	100.1	106.4	100.7
CR 135	-	-	-	-	84.2	97.8	101.8	95.2
CR 129	-	-	-	-	80.3	92.0	98.5	91.7
BS Confluence	-	-	-	-	80.1	91.2	98.0	-
Baughman Slough								
FM 640	-	-	-	-	-	-	-	112.6
CR 239	-	-	-	-	-	-	-	109.1
CR 235	-	-	-	-	101.5	103.1	104.3	107.2
Hwy. 59	-	-	-	-	101.4	103.1	104.3	104.5
CR 231	-	-	-	-	101.1	102.9	103.7	101.5
RR	-	-	-	-	100.7	102.2	102.8	102.4
Bus. 59	-	-	-	-	99.7	101.5	102.9	101.2
Fulton Rd.	-	-	-	-	97.9	99.2	101.8	98.7
Alabama Rd.	-	-	-	-	95.0	98.8	101.8	96.2
CR 150	-	-	-	-	93.6	96.2	100.0	91.8
CR 129	-	-	-	-	82.3	92.0	98.6	91.3
PC Confluence	-	-	-	-	80.1	91.2	98.0	-
Caney Creek								
CC-US59	-	-	-	102.7	106.6	107.0	107.4	-
CC-59to102	-	-	-	102.4	104.6	105.0	105.3	-
CC-OutfalltoCR	-	-	98.1	102.3	104.2	104.5	104.8	-
CC-Wharton	-	-	-	-	102.0	102.1	102.2	-
CC-HEB	-	-	-	-	102.0	102.1	102.1	-
CC-Crestmont	-	-	-	-	102.0	102.1	102.1	-

Analysis of Table 18 indicates the locations and frequency of overflow along the Colorado River. In the upper reaches above Glen Flora, the Colorado River does not spill into the Peach Creek storage area at FM 102 until the 50-year event. Although the 10-year event backs up through the Hughes Street outfall pipe into Caney Creek, significant overflows between the Colorado River, Caney Creek, and Baughman Slough near the City of Wharton do not occur until the 25-year event along the mainstream. There is no interchange of flow between Baughman Slough and Peach Creek until the 50-year Colorado River flood. Table 19 provides locations of flow interchanges (overflows) for the various synthetic Colorado River floods.

Table 19. Overflow Locations for Colorado River Event

Frequency	Locations of Overflow*
2-year	None
5-year	None
10-year	None
25-year	Colorado River to CC just upstream of Highway 59 Caney Creek to Baughman Slough just downstream of Highway 59
50-year	Colorado River to Peach Creek Storage Area Colorado River to Peach Creek upstream of Glen Flora Colorado River to CC between Highway 59 and Railroad Caney Creek to Baughman Slough at Multiple Locations from CR 235 to Alabama Road
100-year	Baughman Slough to Peach Creek between Highway 59 and RR Interchanges Between Peach Creek & Baughman Slough from Richmond Road to Alabama Road

*Overflow Locations are listed for the smallest event (most frequent). Less frequent storms include the overflow areas listed for smaller storms.

FINAL BASELINE STAGE-FREQUENCY RELATIONSHIPS

The local hydraulics were modeled as well as the overflow hydraulics and both analyses produced water surface elevations for the various frequency storm events. In order to determine one final water surface elevation at each cross-section for each frequency event, a probabilistic approach was used considering both local floods and the Colorado River. The methodology and reasoning behind this probabilistic approach is described below with applications to the Wharton Interim Feasibility Study.

PROBALISTIC METHODOLOGY

A method used to determine frequency elevations in areas subject to both storm surge and riverine flooding in coastal areas was used for the City of Wharton. This methodology is based on total independence of the two events. The equation used to calculate the frequency of an event considering both local flooding and overflow flooding is:

$$T_{R\ Combined} = 1/(1-(1-1/T_{R\ Local})*(1-1/T_{R\ Overflow}))\quad ,\text{where}$$
$$T_{Rx} = \text{Return Period (years)}$$

In order to use this procedure, the local and overflow events must be totally independent. A historical analysis of Colorado River flows and local rainfall indicated that these two events are relatively independent in the Wharton area (See *Tailwater Effects from the Colorado River* section). Independence is further supported by considering the 30,600 contributing square miles of Colorado River drainage area above Wharton and the reservoir regulation above Wharton. A local flood in Wharton is not dependent on high flows along the Colorado River or vice versa.

As an example of the probabilistic approach, suppose the 100-year Colorado River OVERFLOW at a selected point of interest produces a WSEL of 99'. Suppose a LOCAL water surface elevation of 99' is produced by a 50-year local event at that same location. By applying the above equation, a WSEL of 99' at the selected point corresponds to a 33.6-year frequency.

For the Colorado River water surface elevations, the probabilistic analysis was not needed. The probabilistic analysis was utilized for points of interest within the study area including: each of the Caney Creek storage areas and the upstream and downstream faces of all bridges along Baughman Slough and Peach Creek.

The probabilistic analysis was rather lengthy, and was therefore applied only at selected points of interest and the results interpolated to other cross-sections. Since the overflow and local water surface elevations were known at each cross-section, the interpolation of the combined probabilistic frequencies produced lower elevations in some cases. This is not possible, as the probabilistic water surface elevation should be equal to or greater than both the overflow and local water surface elevations. If the probabilistic analysis had been applied at each cross-section, this would not have been an issue. Since the complete probabilistic calculation was only applied at points of interest, the maximum of the overflow, local, and interpolated probabilistic water surface elevation was considered the final answer at the intermediate cross-sections.

RESULTS

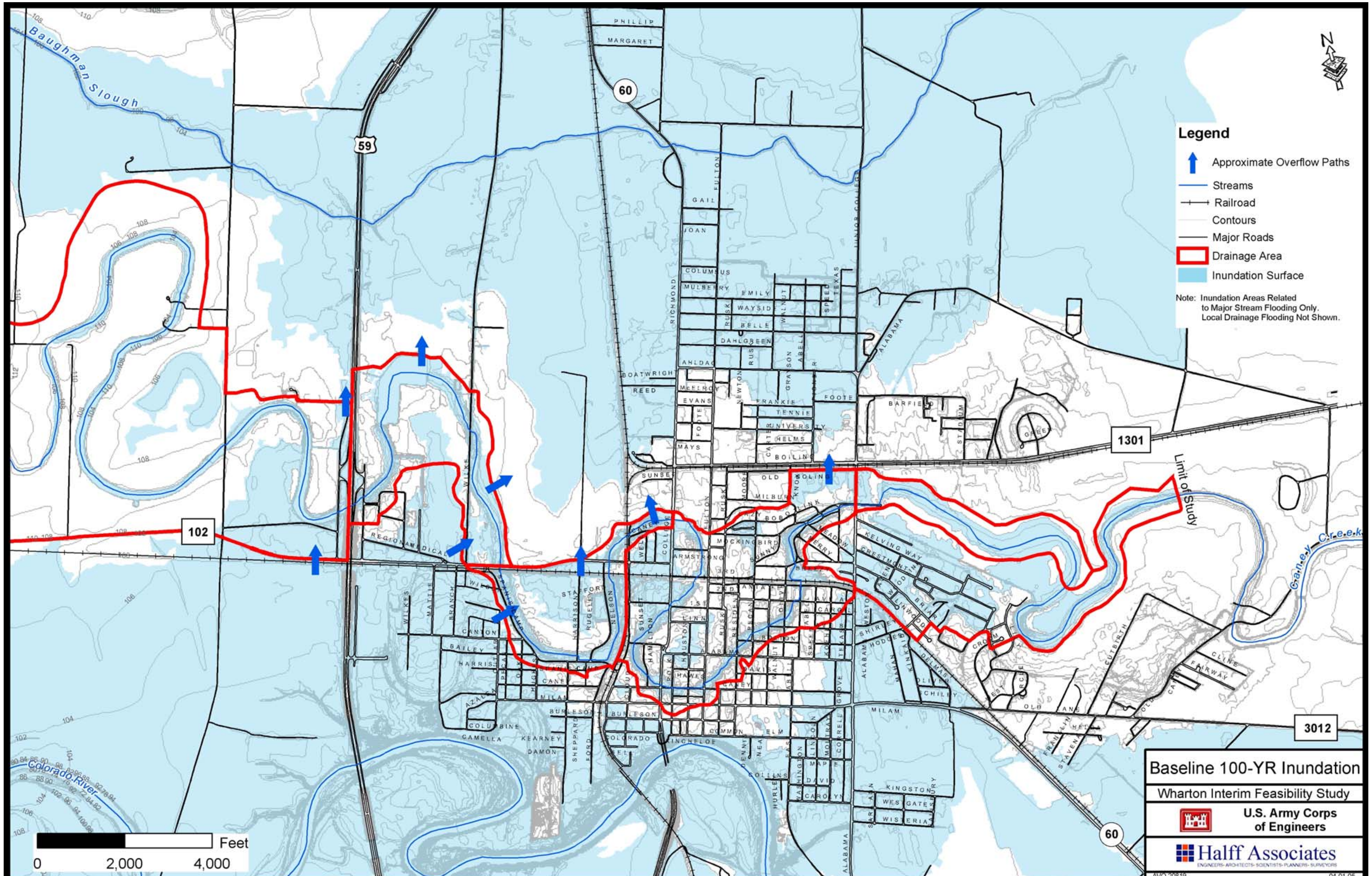
Results of the probabilistic analysis at selected points of interest are shown in Table 20 and indicate the stage-frequency relationships considering both local storm events and overflow from the Colorado River. The 100-year baseline conditions inundation map for the study area is shown in Figure 12. Baseline condition water surface elevations for all cross-sections and storage areas in the study area are provided in Attachment B.

Table 20. Existing Conditions Water Surface Elevations (ft)

	Frequency						
Location	**2-Year**	**5-Year**	**10-Year**	**25-Year**	**50-Year**	**100-Year**	**500-Year**
Colorado River							
Garwood Gauge	149.3	152.7	153.5	154.4	155.1	155.8	158.9
FM 960	99.6	107.2	111.6	115.9	117.8	118.4	119.1
Hwy. 59	90.3	97.2	101.0	104.3	105.9	106.2	106.7
RR	86.1	93.3	97.4	100.9	102.4	102.9	103.3
Bus. 59	86.1	93.2	97.3	100.7	102.2	102.7	103.1
Peach Creek							
FM 640	105.9	107.6	108.6	109.6	110.7	113.6	116.5
CR 239	98.3	100.6	101.6	102.6	103.6	106.6	112.1
CR 235	91.3	93.7	95.2	96.8	98.8	103.1	109.1
Hwy. 59	87.8	91.0	92.9	94.7	96.9	100.7	107.1
RR	87.5	90.8	92.6	94.5	96.6	100.2	106.5
Bus. 59	87.5	90.7	92.6	94.4	96.6	100.1	106.4
CR 135	86.3	89.3	91.0	92.8	94.7	97.8	101.8

	Frequency						
Location	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
CR 129	81.8	84.5	86.2	87.6	89.3	92.0	98.5
BS Confluence	81.6	84.1	85.8	87.0	88.4	91.2	98.0
Baughman Slough							
FM 640	112.1	112.5	112.5	112.5	112.6	112.7	112.8
CR 239	108.1	108.9	109.3	109.6	109.9	110.1	110.6
CR 235	104.5	105.4	105.8	106.1	106.3	106.5	107.1
Hwy. 59	102.2	103.1	103.5	103.8	104.0	104.1	104.3
CR 231	100.3	100.8	101.2	101.7	102.2	102.9	103.7
RR	99.0	99.6	99.9	100.7	101.3	102.2	102.9
Bus. 59	98.4	99.0	99.3	99.5	100.0	101.5	102.9
Fulton Rd.	96.8	97.6	97.8	97.8	98.1	99.2	101.8
Alabama Rd.	93.6	93.9	94.1	94.2	95.7	98.8	101.8
CR 150	92.3	92.9	93.0	93.2	94.3	96.2	100.0
CR 129	84.8	86.0	86.6	87.6	88.8	92.0	98.6
PC Confluence	81.6	84.1	85.8	87.0	88.4	91.2	98.0
Caney Creek							
CC-US59	103.2	104.6	105.1	105.5	106.6	107.0	107.4
CC-59to102	102.3	103.2	103.6	104.0	104.6	105.0	105.3
CC-Outfall to CR	101.3	102.6	103.2	103.4	104.2	104.5	104.8
CC-Wharton	99.6	100.3	100.8	101.1	102.0	102.1	102.2
CC-HEB	100.2	100.6	101.0	101.1	102.0	102.1	102.1
CC-Crestmont	100.2	100.6	101.0	101.1	102.0	102.1	102.1

Figure 12. 100-Year Baseline Conditions Inundation Area



CURRENT EFFECTIVE FIS WSELs VERSUS CURRENT STUDY WSELs

A comparison between the current effective Flood Insurance Study (2001) 100-year water surface elevations and those computed in the Wharton Interim Feasibility Study is shown in Table 21. The Interim Feasibility Study produced both higher and lower water surface elevations than the current effective FIS.

Table 21. Current Effective FIS (2001) vs. Interim Feasibility Study 100-Year Water Surface Elevations

Location	Current Effective WSEL (ft)	Interim Feasibility Study WSEL (ft)	Difference (Interim Study - Current Effective)
Colorado River – Business Highway 59	103.2	102.7	-0.5
Colorado River – RR	103.5	102.9	-0.6
Colorado River – U.S. Highway 59	107.0	106.2	-0.8
Colorado River – FM 960	119.0	118.4	-0.6
Baughman Slough – CR 129/Montgomery	91.0	92.0	1.0
Baughman Slough – CR 150/Moers Rd.	92.8	96.2	3.4
Baughman Slough – Junior College Blvd.	94.9	98.8	3.9
Baughman Slough – Fulton Street	98.2	99.2	1.0
Baughman Slough – Business Highway 59	98.8	101.5	2.7
Baughman Slough – Railroad	102.0	102.2	0.2
Baughman Slough – Wilke Road/ CR 231	102.8	102.9	0.1
Baughman Slough – U.S. Highway 59	104.0	104.1	0.1
Baughman Slough – Owens Road/CR 235	106.5	106.5	0.0

There are many reasons for the elevation results of the Wharton Interim Feasibility Study to differ from the current effective values. Table 22 provides differences between the two studies.

Table 22. Study Differences (Current Effective FIS and Interim Feasibility Study)

Criteria	Current Effective FIS	Wharton Interim Feasibility Study
Local Hydrology	Snyder's Unit Hydrograph Method for 3 points along Baughman Slough. Peak flows at other locations interpolated based on ratios of drainage areas. No routing and combining of sub-basin hydrographs	SCS Unit Hydrograph. Hydrographs routed and combined in HEC-HMS.
Topographic Data	Surveys, COE data, TXDOT data	2' Aerial Topo; Bridge Surveys; TXDOT plans
Hydraulic Model	Steady State – "Water-Surface Profiles," Generalized Computer Program 723-X6-L202A, HEC, 1979.	Unsteady State – HEC-RAS 3.1.1, HEC, 2003.
Colorado River Overflows	Manually subtracted for the Colorado River and added to Baughman Slough or Peach Creek based on profile of drainage divide and Colorado River water surface profile. Based on $Q=CLH^{3/2}$ No tailwater effects of receiving streams considered.	Computed by HEC-RAS. Considered any tailwater effects of receiving streams.
Overflows Between Other Streams	Not Considered	Computed by HEC-RAS. Considered any tailwater effects of receiving streams.
Caney Creek Modeling	Riverine	Storage Areas
Combined Frequencies	Assumed that maximum flow rate between Colorado River overflow and local event controlled.	Probabilistic Analysis to Compute Frequency Water Surface Elevation.
Peach Creek and Baughman Slough Confluence	Peach Creek not Modeled. Confluence not Considered.	Confluence Modeled. Backwater effects accounted for.
Colorado River Flows Prior to Overflow Escape	Higher than Wharton Interim Feasibility Study	Lower than Current Effective FIS Study

As evidenced by Table 22, there are numerous reasons for the results of the two studies to be different. Changes in methodology, technology, and data sources between the two studies have resulted in different water surface elevations around the City of Wharton. The Wharton Interim Feasibility Study technology and data sources enabled a much more refined and detailed model to be generated for the Wharton area and produce corresponding frequency water surface elevations.

SUMMARY OF BASELINE CONDITIONS FLOODING

A detailed study of existing flooding conditions within the City of Wharton and surrounding area was conducted and resulted in the establishment of baseline conditions frequency water surface elevations. Table 23 provides a summary of flooding conditions for each frequency storm event for both local events and Colorado River overflows. These results reflect current conditions without the implementation of any flood control/reduction alternatives.

Table 23. Summary of Baseline Conditions Flooding

Frequency	Local Event	Colorado River Overflow
2-Year	Baughman Slough CR 150 Overtopped. Peach Creek FM 640, CR 239, CR 235 Overtopped.	No Flooding
5-Year	Caney Creek spills to Colorado River just upstream of Highway 59 (FM 102 overtopped).	No Flooding
10-Year	Water at top of Baughman Slough FM 640.	Camella & Outlar Intersection Flooded. Flow backs up from Colorado River through Hughes Street Pipe into Caney Creek. Flow backs up through Alabama Box into Park Area at Santa Fe & Alabama.
25-Year	Baughman Slough CR 239 and CR 231 Overtopped. Caney Creek spills to Baughman Slough between Highway 59 and CR 231. Caney Creek overtops Richmond Road. Caney Creek overtops Alabama Road. Caney Creek spills to Colorado River just upstream of Railroad.	FM 102 Overtopped just upstream of Highway 59. Flow Under Highway 59 at FM 102 Intersection. Caney Creek spills to Baughman Slough between Highway 59 and CR 231. Caney Creek overtops Richmond Road and Alabama Road. Inundation of Caney Creek through Wharton begins between Richmond Road and Alabama Road.
50-Year	No Additional Overflows or Overtoppings.	Colorado River spills to Peach Creek upstream of Glen Flora. Colorado River spills to Caney Creek in West End neighborhood. Caney Creek spills to Baughman Slough at Multiple Locations from CR 235 to Alabama Road. Peach Creek FM 640, CR 239, and CR 235 overtopped.

Frequency	Local Event	Colorado River Overflow
100-Year	Baughman Slough spills to Peach Creek Between Highway 59 and Railroad.	Baughman Slough spills to Peach Creek between Highway 59 and Railroad. Peach Creek and Baughman Slough interchange flow between Richmond Road and Alabama Road. Peach Creek CR 135 overtopped. Baughman Slough Alabama Road and CR 150 overtopped.

FLOOD CONTROL ALTERNATIVES

Following the establishment of existing baseline conditions for the City of Wharton and surrounding area, potential flood control alternatives were investigated by the study team. Several potential alternatives were noted during a June 17, 2003, meeting involving the entire study team. These alternatives were not evaluated for economic feasibility nor for their ability to reduce flood damages within the City of Wharton at that time. Several of these initial alternatives were refined and modeled hydraulically with the HEC-RAS models to evaluate the potential reductions in flood water surface elevations.

OVERVIEW

The potential alternatives theorized by the team in June ranged from non-structural buyouts of flood prone property to levees, channel clearing, and new diversion channels. The Colorado River was divided into two reaches through the City of Wharton. The first reach was downstream of Business Highway 59 (Richmond Road) and included the downtown area and southeastern portions of the city. The second reach was upstream of Business Highway 59 and included the West End neighborhood which is highly susceptible to flood damages for events as frequent as the 25-year flood. Potential Caney Creek, Baughman Slough, and Peach Creek flood control alternatives were also noted at the June team meeting. Table 24 provides a list of alternatives that were noted at this initial Alternatives Planning Meeting.

Table 24. Potential Wharton Flood-Control Alternatives

River/Stream	Potential Alternative
Colorado River Downstream of Bus. 59	1. Non-structural: Floodplain evacuation at the 25-year flood level or below. 2. Flood wall/levee combination
Colorado River Upstream of Bus. 59	1. Non-structural: Floodplain evacuation at the 25-year flood level or below. 2. Levee 1: Build Around the Old Oxbow in West End neighborhood. No Sump. 3. Levee 2: Build Close to the Main Channel, and use Old Oxbow as Sump. 4. Levee 3: Extend Levee around U.S. 59 preventing water from entering Caney Creek 5. Combination of Levee and Buyout. Levee would encircle industrial area (FM 102 and U.S. Hwy. 59) 6. Diversion Channel

River/Stream	Potential Alternative
Caney Creek	1. Lower Hughes Street to provide conveyance 2. Enlarge Wal-Mart channel to increase conveyance 3. Reroute drainage from the Crestmont subdivision 4. Remove downstream dams
Baughman Slough	1. Diversion to Peach Creek along Bus. 59 2. Diversion to Peach Creek slough upstream of CR 235 (Owens Road) 3. Channel Improvement from Alabama Road to Peach Creek confluence 4. Channel Improvement from Business Highway 59 to Peach Creek confluence
Peach Creek	1. No Modifications – Leave in Natural State

COLORADO RIVER OVERFLOW FLOODING

Flood damages resulting from Colorado River overflows into the City of Wharton were the first to be addressed with a potential alternative. Due to the flat terrain of the Wharton area, once water is out of the banks of the Colorado River, a large increase in flow is needed to result in a significant water surface elevation rise. In many areas near Wharton, the increase in water surface elevation from a 25-year flood to a 100-year flood is less than two feet. This relatively small rise in water surface elevation indicates that a slight increase in levee or floodwall height has the potential to provide much higher levels of flood protection and economic benefit. Therefore, the first alternative investigated was a levee (or floodwall in space restricted areas) along the Colorado River. The levee alternative modeled extended along FM 102 upstream of US 59 to downstream of Wharton near the intersection of Southeast Avenue and FM 1299.

Colorado River Levee Alignments

The total length of proposed levee or floodwall along the Colorado River is approximately four miles. In most areas the levee can be less than five feet in height and still provide one-foot of freeboard above the 100-year Colorado River water surface elevation. Another benefit of this levee alternative is that it can be used in combination with existing embankments and levees at highways, railroads, and the landfill. A description of each segment of the proposed levee is discussed below.

Based on some of the comments received from those attending the Public Workshop in Wharton in January 2004, a desire was expressed to move the proposed levees away from the roads and closer to the left bank of the Colorado River. This alternative levee alignment would protect more areas, utilize a less expensive right-of-way and allow for property along FM 102 to be used as envisioned by local citizens. The two primary locations for these proposed alignment changes were along FM 102 upstream of Highway 59 near the new Wal-Mart and on the southeast side of town near Carolyn and Alabama. Moving the levee alignment in the southeast portion of Wharton will not raise downstream water surface elevations more than the original proposed alignment. A realignment of the levee in this area would result in approximately 800’ less total length of levee and could also protect the Wastewater Treatment Plant (WWTP) from Colorado River flooding. The alignment of the proposed Colorado River levee upstream of Highway 59 was also moved away from FM 102 and closer to the Colorado River to allow the area around Wal-Mart to be developed. These two levee alignments are shown on Figure 13.

Segment 1

This portion of the levee extends for approximately 4,800 feet along the abandoned railroad right-of-way along FM 102 west of Highway 59 to the FM 102 and Highway 59 intersection. The abandoned railroad grade is still raised in some locations which will help in reducing the amount of soil required to raise the levee to the required height in this location. A flap gate structure will be needed to allow the Wal-Mart and Caney Creek drainage channel to pass through the levee en route to the Colorado River. The proposed levee will tie-into the Highway 59 overpass embankment just south of the FM 102 intersection. This will prevent water from flowing through the FM 102 underpass and flooding the West End neighborhood from the northwest. Water did pass under the Highway 59 overpass at FM 102 during the October 1998 flood.

This levee alignment and height will also prevent Colorado River flows from overtopping FM 102 and flowing into Caney Creek and the Wal-Mart property just upstream of Highway 59.

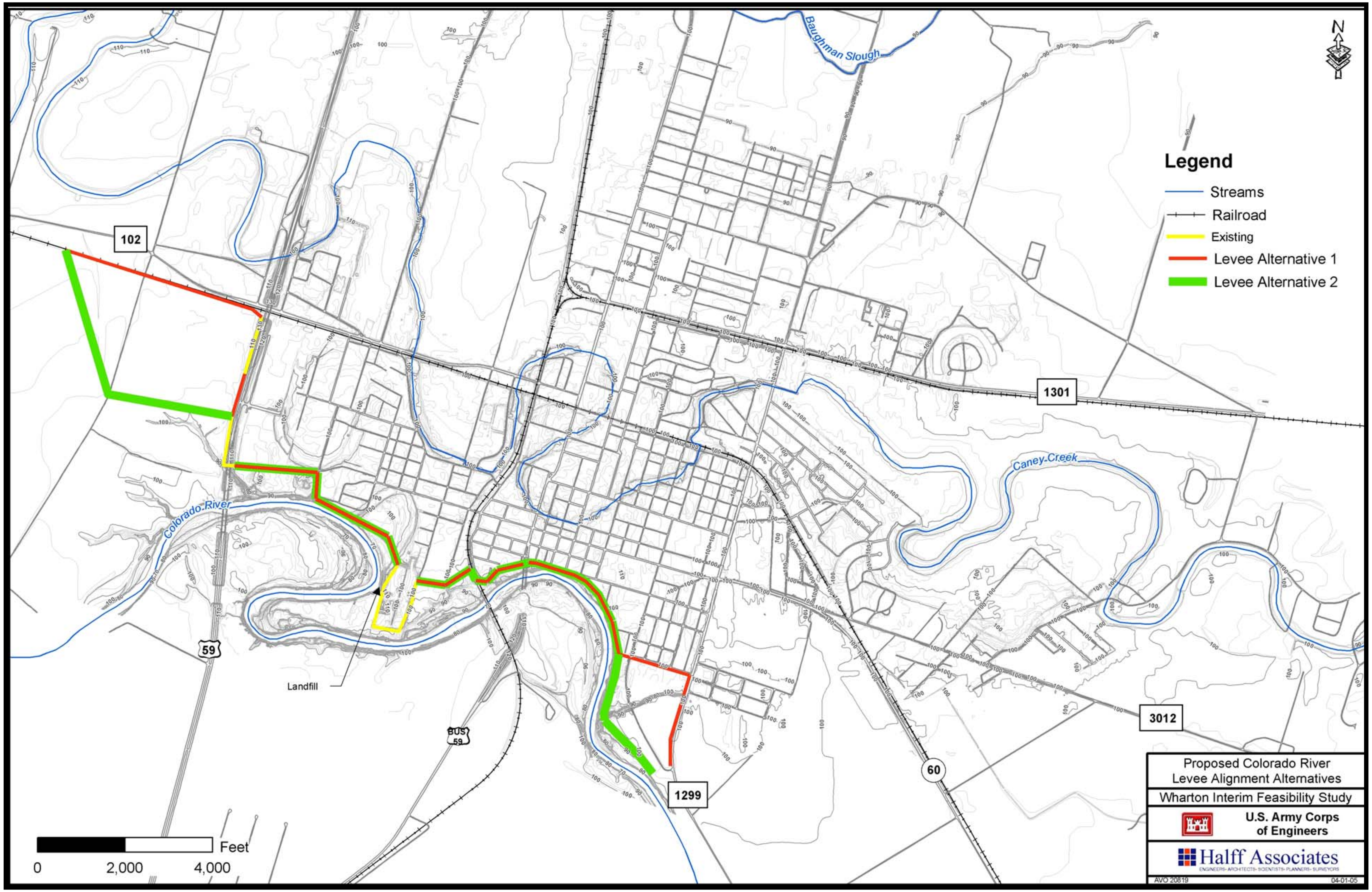
The second alignment for Segment 1 would run along the bank of the Colorado River and allow the area southwest of the FM 102 and Highway 59 intersection to remain untouched. This alignment would add approximately 1,500’ to the levee length along Segment 1, but would require less expensive real estate.

The existing 100-year water surface elevation along the Colorado River is near the highest point (abandoned railroad grade or FM 102) on the left overbank extending from upstream of Wharton to Glen Flora. Any changes to proposed levee alignment or other downstream alternatives during final design could raise the 100-year water surface elevation over FM 102 and allow spills into Caney Creek. Figure 14 provides a profile of the 100-year water surface elevations along the Colorado River between Highway 59 and upstream of Glen Flora with the highest point on the left overbank depicted by the lateral structure (gray shading). The current proposed Colorado River levee extends from upstream of Highway 59 to river stations 364418.5 and 371425.3. The City of Glen Flora is near river station 397111.4. As Figure 14 indicates, in many locations upstream of Wharton, there is little freeboard between the existing 100-year water surface elevation and the top of FM 102 or top of abandoned railroad grade. An extension of the levee upstream of river station 371425.3 may need to be considered during final design.

Segment 2

The Highway 59 embankment will serve as the primary protection of the West End neighborhood from FM 102 to the Colorado River relief bridge channel. The 100-year Colorado River flood does not overtop Highway 59. At the low point of Highway 59, the top of road elevation is equal to the 100-year Colorado River water surface. At this location, approximately 950 feet of levee would need to be constructed on the upstream face of the road embankment to ensure that 100-year flood water does not overtop Highway 59 and flood the industrial area and residential sections of west Wharton. The second alignment for Segment 1 will eliminate the need for Segment 2 of the levee.

Figure 13. Proposed Colorado River Levee Alignment Alternatives



Segment 3

The next section of proposed levee will extend from the downstream face of the Highway 59 embankment (just north of the relief channel bridge) to the abandoned railroad embankment just west of Business Highway 59. For this analysis, the levee was assumed to follow the main channel of the Colorado River. This alignment allows the oxbows to be used as interior sump storage areas if needed. The downfall to treating the oxbow areas as interior drainage areas is that more soil will be needed to fill the oxbow channel and create the levee through these areas. The levee analyzed for this analysis will extend from Highway 59 to the park area near the intersection of Camella and West Milam. The levee will then turn south along Camella to the banks of the Colorado River and follow the river across the old oxbow before connecting to the northwest corner of the existing landfill levee. The existing landfill levee offers 100-year flood protection and was utilized for this analysis. A levee will have to be constructed on the east side of the landfill levee near South Sheppard Street and will extend to South Ford Street before tying into the abandoned railroad grade. Total length of levee to be constructed in this segment is over 6,200 feet. This segment of levee will offer Colorado River flood protection to the West End neighborhood of Wharton. Flap/sluice gates will be needed to drain the interior areas and old oxbows to the Colorado River.

Segment 4

This short segment of levee will extend from the abandoned railroad embankment to the Business Highway 59 embankment. The proposed levee will pass between the Colorado River and Sunset Road to offer protection to the homes along Sunset. The length of this levee segment will be approximately 1,225 feet.

Segment 5

The final levee segment will extend from Business Highway 59 along the banks of the Colorado River and Elm Street to the intersection of Carolyn Street and Southeast Avenue. At this point, the proposed levee will turn east along Carolyn to South Alabama, before turning south along Alabama (FM 1299) to its termination at the intersection of Southeast Avenue and FM 1299. This area between Elm Street and the Colorado River may require a floodwall instead of a levee due to limited space. Currently, the proposed alignment is along Elm Street. The restaurant on the left bank of the Colorado River at Elm and Polk Street was not offered protection by the levee. The alignment in this area may be altered slightly to protect the restaurant area from Colorado River flooding.

The second alignment for Segment 5 would have the levee continue along the left bank of the Colorado River instead of following Carolyn and Alabama Streets (See Figure 13). Table 25 provides a summary of the proposed Colorado River levees.

Figure 14. Colorado River 100-Year WSEL Profile Upstream of Wharton with High Left Overbank

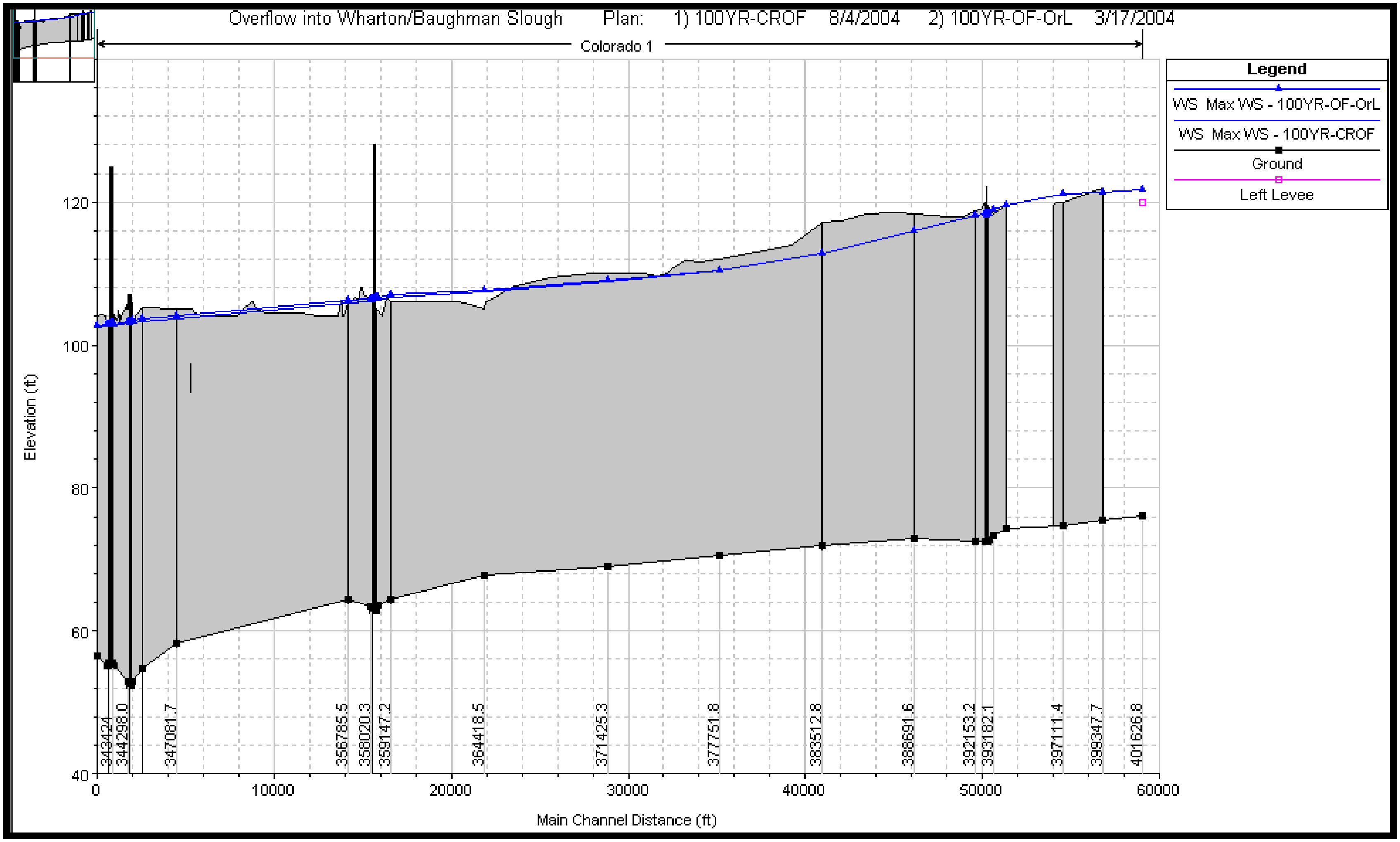


Table 25. Summary of Proposed Colorado River Levee Alternatives

Levee Segment	Original Alignment Length	Secondary Alignment Length	Flap/Sluice Gate Structures
CR-1	4,800’	6,300’	1 (Wal-Mart/CC Channel)
CR-2	950’	N/A	0
CR-3	6,200’	6,200’	3-4
CR-4	1,225’	1,225’	0
CR-5	7,000’	6,200’	1 (Alabama Box Outfall)
Approximate Total	**18,950’**	**19,925’**	**6**

Colorado River Levee Modeling & Results

Based on the proposed levee alignment noted above, corresponding stations along the Colorado River cross-sections were found and a levee elevation entered in the HEC-RAS model. Levee elevations were set and adjusted following simulations to provide a relatively constant levee slope along the river and a minimum of one-foot of freeboard above the Colorado River 100-year water surface elevation. The levee heights can be adjusted once detailed levee design begins, dependent upon the design frequency selected. HEC-RAS simulations were made to determine the effect of the proposed levee alignment on the left overbank of the Colorado River. One concern was the increase in water surface elevation as a result of the levee and its effects on the land and property owners downstream and on the right overbank of the Colorado River. A comparison of existing Colorado River 100-year water surface elevations was made with those resulting from the proposed levee. Table 26 provides water surface elevations at various locations along the Colorado River for both existing 100-year conditions and with the proposed levee alignments. Protection levels exceeding a 100-year frequency would require an extension of the proposed levees west of Wharton to prevent overflow into Caney Creek.

Areas upstream of the levee experienced basically no rise in 100-year water surface elevation. The rise for the cross-sections just upstream of the levee was hundredths of a foot. The rise in water surface within the extents of the levee (through the City of Wharton) ranged from 0.16’ to 0.44’ with an average rise throughout this reach of 0.33’. Although this rise has no impact on the City of Wharton since it is protected by the proposed levee, this rise will impact areas on the right overbank of the Colorado River not protected by a levee or other measure. This area is primarily agriculture and pasture land on the right overbank of the Colorado River opposite of the City of Wharton.

Table 26. Comparison of Colorado River Levee 100-Year WSEL

Cross-section	Existing 100-YR WSEL (ft)	Levee 100-YR WSEL (ft)	Δ Existing (ft)
172237.2	47.6	47.7	0.1
190389.8	53.9	54.2	0.3
213668.4	60.7	61.2	0.5
258531.2	71.6	72.4	0.8
301429.3	85.4	85.9	0.5
335290.4	99.7	100.1	0.4
341031.5	102.0	102.4	0.4

Cross-section	Existing 100-YR WSEL (ft)	Levee 100-YR WSEL (ft)	∆ Existing (ft)
342574	102.4	102.7	0.3
343158.2	102.7	102.9	0.2
343254.8	102.7	102.9	0.2
343370.4	102.7	103.0	0.3
343385.8	102.7	103.0	0.3
343463.4	102.8	103.1	0.3
343515.6	102.7	103.0	0.3
344298	103.1	103.4	0.3
344406.1	102.9	103.2	0.3
344502.1	103.0	103.3	0.3
344557.2	103.1	103.4	0.3
345172.7	103.3	103.7	0.4
347081.7	103.6	104.0	0.4
356785.5	105.9	106.3	0.4
358020.3	106.1	106.5	0.4
358095.3	106.1	106.5	0.4
358233.8	106.0	106.4	0.4
358243.9	106.1	106.5	0.4
358386.2	106.2	106.6	0.4
358435.6	106.3	106.7	0.4
359147.2	106.7	107.0	0.3
364418.5	107.5	107.8	0.3
371425.3	108.9	109.1	0.2
377751.8	110.4	110.5	0.1
399347.7	121.4	121.4	0.0

Cross-Sections in Bold Within Extents of Proposed Levee

Larger water surface elevation rises were experienced downstream of Wharton and the levee. A rise of at least 0.1’ was experienced to the State Highway 35 bridge in Matagorda County. A major reason for this rise is that the proposed levee prevents approximately 4,000 cfs of flow from escaping into Caney Creek. With the levee preventing this overflow, this additional water is carried by the Colorado River downstream and impacts water surface elevations accordingly. A detailed analysis of overflows between Caney Creek and the Colorado River downstream of the City of Wharton was beyond the scope of this study. A more comprehensive analysis of downstream overflows may indicate that the levee impact is not as severe as some of the overflow to Caney Creek under existing conditions may re-enter the Colorado River downstream of the City of Wharton.

The secondary alignment resulted in a 0.13’ rise in water surface elevation over the original proposed alignment at Cross-section 371425.3 for the 100-year event. No further downstream rise was experienced below Highway 59. The secondary alignment may need to be altered slightly due to areas in which it encroaches into the Colorado River floodway.

Based on the original levee alignment, a comparison of the 50-year, 100-year, and 500-year water surface profiles was made along the Colorado River levee section to provide some indication of the required levee heights for various frequency events. The Colorado River water surface profile increased an average of 0.56' through the levee section from the 50-year event to the 100-year event. Table 27 provides the water surface elevation results through the proposed Colorado River levee section.

Table 27. Colorado River WSEL Comparisons for Levee Alignment

Cross-section	50-Year (ft)	100-Year (ft)	500-Year (ft)
377751.8	110.0	110.5	111.0
371425.3	108.7	109.1	109.6
364418.5	107.4	107.8	108.3
359147.2	106.5	107.0	107.7
358435.6	106.2	106.7	107.5
358386.2	106.1	106.6	107.5
358243.9	106.0	106.5	107.4
358233.8	105.9	106.4	107.4
358095.3	105.9	106.5	107.3
358020.3	106.0	106.5	107.3
356785.5	105.7	106.3	107.1
347081.7	103.3	104.0	104.8
345172.7	103.0	103.7	104.5
344557.2	102.8	103.4	104.3
344502.1	102.7	103.3	104.2
344406.1	102.6	103.2	104.0
344298.0	102.7	103.4	104.2
343515.6	102.4	103.0	103.8
343463.4	102.5	103.1	103.8
343385.8	102.4	103.0	103.7
343370.4	102.4	103.0	103.7
343254.8	102.4	102.9	103.6
343158.2	102.3	102.9	103.6
342574	102.1	102.7	103.4
341031.5	101.8	102.4	103.1

Baughman Slough Alternatives for Colorado River Overflows

For Colorado River events larger than the 25-year storm, some flow escapes into Peach Creek near Egypt, Texas, well upstream of the City of Wharton. For the 100-year event, this flow escape from the Colorado River into Peach Creek is approximately 15,000 cfs. This water flows down Peach Creek passing to the north of the City of Wharton. Current 100-year Colorado flows in Peach Creek are contained by the channel to Business Highway 59. Between Business Highway 59 and County Road 135 (Lee's Lane), the Colorado River overflow in Peach Creek spills into the right overbank overtopping County Road 146 and moving south to the City of Wharton before overtopping County Road 144 (Brooks Road) on the left overbank of Baughman Slough. This flow that escapes from Peach Creek into Baughman Slough exceeds the channel capacity of Baughman Slough and creates flooding problems within the Ahldag subdivision of northern Wharton. Although the levee along the Colorado River addressed flooding as a direct result of the River, the flow that escapes to Peach Creek and then into Baughman Slough aggravates flooding in the northern neighborhoods of Wharton.

As a result of this problem created by the 100-year Colorado River overflow, levee alternatives for the northern section of Wharton were also investigated. Although no action along Peach Creek was foreseen at the June 2003 meeting, flooding conditions in the northern sections of Wharton needed to be addressed. The banks of Peach Creek are built up several feet above the surrounding ground creating a perched condition just north of the City of Wharton. The high banks of Peach Creek provided a logical location for constructing a small levee along the right bank of Peach Creek to prevent water from escaping and flooding Baughman Slough. A proposed levee along the Peach Creek right bank from Business Highway 59 to County Road 135 was input into HEC-RAS. This levee was successful in preventing Baughman Slough and Ahldag subdivision flooding as a result of Colorado River “backdoor” flooding, but it raised water surfaces along Peach Creek by almost two feet in some locations. This large rise is attributable to the narrow drainage area along Peach Creek in this location due to the high banks. This rise pushed water into the left overbank of Peach Creek and into the West Bernard Creek watershed.

In an attempt to add storage and increase conveyance along Peach Creek, the proposed levee was moved to the Peach Creek and Baughman Slough drainage divide near County Road 146. This levee option did prevent flooding of Baughman Slough, but water surface elevations along Peach Creek rose by almost 1.5 feet in some areas creating the same problem as the proposed levee alignment along the right bank.

The negative impacts to the West Bernard watershed and upstream locations along Peach Creek as a result of the proposed right bank levees, indicated the need to allow the natural overflow of Peach Creek into Baughman Slough to continue. The Ahldag subdivision is located on the right overbank of Baughman Slough. Aerial photographs and field observations indicated that there is little development in the area bounded by Peach Creek, Baughman Slough, County Road 135, and Business Highway 59. Allowing flow to escape from Peach Creek into Baughman Slough does not create major flooding problems due to the little development in this area. The significant damages occur when Baughman Slough overflows and floods the Ahldag subdivision on the right overbank. As a result of this analysis, a proposed levee was placed on the right bank of Baughman Slough between the abandoned railroad and County Road 135.

This levee alignment allowed Peach Creek to overflow into Baughman Slough and remain in a natural state without eliminating additional storage and conveyance along Peach Creek. A decrease in existing Colorado River overflow 100-year water surface elevations was actually noted for Baughman Slough and Peach Creek as a result of this levee alignment. With existing conditions, Colorado River water spills into Caney Creek and Caney Creek spills to Baughman Slough. The proposed levees prevent the overflow from the Colorado River to Baughman Slough via Caney Creek. The flow in Baughman Slough with the proposed levees is significantly less than existing conditions as a result of this overflow blockage, and thus the water surface elevation is actually lower than existing even with less conveyance as a result of the levees. The levee along the right bank of Baughman Slough would only need to be a few feet in height to prevent overflow into the Ahldag subdivision. Table 28 provides Colorado River overflow water surface elevations for various frequency events along Baughman Slough with the proposed levee in place.

Table 28. Baughman Slough Colorado River Overflow WSEL Comparisons for Levee Alignment on Right Bank

Cross-section	50-Year (ft)		100-Year (ft)		500-Year (ft)	
	Existing	Levee	Existing	Levee	Existing	Levee
19918	99.8	N/A	101.5	98.1	103.0	101.7
19400.5	99.7	N/A	101.5	98.1	103.0	101.7
18814	99.7	N/A	101.5	98.1	103.0	101.7
18175	99.7	N/A	101.5	98.0	102.9	101.7
18033	98.3	N/A	99.2	98.0	101.8	101.5
17629	98.1	N/A	99.2	98.0	101.8	101.5
17260	98.0	N/A	99.2	98.0	101.8	101.5
16961	97.9	N/A	99.2	98.0	101.8	101.5
16869	97.5	N/A	98.9	97.9	101.8	101.5
16564	97.0	N/A	98.9	97.8	101.8	101.5
16153	96.6	N/A	98.9	97.8	101.8	101.5
15465	96.5	N/A	98.9	97.8	101.8	101.5
14730	95.8	N/A	98.8	97.8	101.8	101.5
14040	95.4	N/A	98.8	97.8	101.8	101.5
13613	95.3	N/A	98.8	97.7	101.8	101.5
12982	95.0	N/A	98.8	97.7	101.8	101.5

PROPOSED SUMP/SLUICE

Upon completion of the preliminary alternative analysis (including the levees), interior drainage and sump/sluice structures needed to be evaluated. Specific tasks associated with the proposed sump/sluice analysis included: a coincident tailwater analysis for sump/sluice gate ratings, addressing existing railroad openings between Baughman Slough and Caney Creek, and the sizing and placement of sumps/sluices along the proposed Colorado River and Baughman Slough levees.

The Colorado River and Baughman Slough sump and sluice sizing is preliminary in nature. The sump/sluice analysis was based on a March 2006 Corps of Engineers' levee alignment that is slightly different than the levee alignment used for the remainder of the study and presented in Figure 13. The minor change in alignment will not significantly alter water surface elevations. Some of the sumps were sized to store the entire 100-year local runoff volume, while some of the sluices discharged during the local storm event due to lower tailwater conditions. The time required to evacuate water from the sump areas was not evaluated as part of this feasibility level study. If evacuation time is critical, pumps may be required for a few of the sumps.

Tailwater Analysis

Three local (interior) storm frequencies were analyzed as part of this study (25-, 50-, and 100-year). In order to size the sumps/sluice structures, it was important to have an appropriate coincidental tailwater elevation to develop rating curves for the analysis. U.S. Army Corps of Engineers Engineer Manual (EM) No. 1110-2-1413, *Hydrologic Analysis of Interior Areas, January 1987*, was used as a reference for the tailwater and interior drainage area analysis.

EM 1110-2-1413 suggested several interior drainage analysis procedures, including continuous record analysis and coincident frequency methods. The continuous record analysis involves a period-of-record simulation or stochastically generated continuous record. Due to the feasibility level of the current study, lack of sufficient detailed rainfall or runoff records, and short project duration, the continuous record analysis was not used for this study.

The coincident frequency method requires independence between interior conditions and exterior (tailwater) conditions and involves application of the total probability theorem. For Baughman Slough, due to the small drainage area of both exterior and interior areas, independence does not exist. For areas along the Colorado River, interior (local) storms are more independent of high tailwater conditions due to the large difference in drainage areas between the City of Wharton local areas and the Colorado River watershed. However, due to the project time constraints, multiple/iterative simulations required, and feasibility level of the current study, coincident frequency methods were not utilized for this analysis.

Colorado River Tailwater

Interior local events behind the proposed Colorado River levee are relatively independent of the Colorado River tailwater, due to the large difference in drainage areas. Historic rainfall records and Colorado River flows were researched in order to determine an appropriate Colorado River tailwater for the sump/sluice analysis. Over 57 years of daily rainfall records were obtained from the EarthInfo Database for the Wharton station covering a period-of-record from September 1946 through December 2003. Daily rainfall totals, two-day totals, and storm totals were analyzed. Table 29 provides maximum daily and two-day totals for the Wharton station during the 57 years of available data. The computed local rainfall frequency is based on TP-40 and TP-49 data. Table 30 provides storm totals and maximum Colorado River flows at the Wharton gauge. Table 31 shows the maximum annual peak Colorado River flows at the Wharton gauge since 1946 and coincidental local rainfall.

Table 29. Wharton Maximum 1-Day and 2-Day Rainfall Totals (1946-2003)

24-Hour Rainfall Totals			2-Day Rainfall Totals		
Date	Rainfall (in)	Frequency	Date	Rainfall (in)	Frequency
Oct. 18, 1994	11.58	50-100 YR	Oct. 18-19, 1994	12.08	25-50 YR
Sept. 12, 1961	9.69	25+ YR	Oct. 17-18, 1994	11.87	25-50 YR
Sept. 7, 2002	8.87	10-25 YR	Sept. 11-12, 1961	11.69	25-50 YR
Sept. 19, 1983	7.55	<10 YR	June 25-26, 1960	11.58	25-50 YR
June 26, 1960	7.48	5-10 YR	May 30-31, 1975	10.81	25 YR
Aug. 30, 1953	6.90	5-10 YR	Sept. 19-20, 1979	10.48	10-25 YR
Sept. 11, 1998	6.63	5-10YR	Sept. 12-13, 1961	10.25	10-25YR
Oct. 12, 1970	6.53	5-10 YR	Sept. 7-8, 2002	9.47	10-25 YR
May 28, 1952	6.15	<5 YR	Sept. 19-20, 1983	9.18	10-25 YR

Table 30. Wharton Storm Totals (1946-2003)

Date	Rainfall (in)	CR Max Mean Daily Flow (cfs)	CR Max Peak Flow (cfs)	Frequency Flow
May 24-31, 1975	15.25	50,300	50,800	5-10 YR
June 24-27, 1960	13.48	51,600	53,000	5-10 YR
Sept 11-14, 1961	13.05	55,000	59,600	10 YR
Oct. 15-19,1994	13.05	48,400	49,600	5-10 YR
Oct. 3-12, 1949	11.73	4,740	N/A	N/A
Sept. 8-17, 1998	11.47	5,910	N/A	N/A
May 6-13, 1972	11.45	22,900	24,500	2 YR
Sept 18-20, 1979	11.30	11,300	N/A	N/A

Table 31. Colorado River at Wharton Maximum Annual Peak Flows (1946-2003)

Date	CR Peak Flow (cfs)	Frequency Flow	Local Rainfall (in)
Oct. 23, 1998	74,800	25 YR	0
Dec. 27, 1991	61,900	10-25 YR	1.95" on 12/26
Sept. 15, 1961	59,600	10-25 YR	13.05" Prior 4 Days
June 15, 1973	59,400	10-25 YR	0
Oct. 17, 1957	58,500	10-25 YR	7.11" Prior 2 Days
June 26, 1968	55,400	10 YR	8.95" Prior 2 Weeks
April 30, 1957	54,200	<10 YR	3.85" Prior 2 Days

The results of Tables 29-31 indicate that the wettest season for the Wharton area is May through October. Analysis of both local Wharton rainfall events and peak Colorado River flows, and consideration of the difference in magnitude of drainage areas and the timing of runoff, indicates that a 5-10 year Colorado River tailwater elevation would be appropriate for the analysis of the proposed Wharton sumps/sluices for 25-, 50-, and 100-year interior events. A 10-year Colorado River tailwater determined with the HEC-RAS Unsteady model developed as part of this Wharton Interim Feasibility Study was selected for the sluice analysis. A 10-year Colorado River flow at the Wharton gauge is approximately 55,000 cfs.

Baughman Slough Tailwater

The Baughman Slough watershed has a drainage area of approximately 11.5 square miles at Alabama Road (downstream extent of proposed levee). Due to this overall small drainage area, local (interior) storm events behind the proposed Baughman Slough levee are not independent of high Baughman Slough tailwater elevations (exterior condition). For this analysis, the local frequency water surface elevation in Baughman Slough was used as a tailwater for the sump/sluice sizing. The local frequency water surface elevations were computed with a HEC-RAS Unsteady hydraulic model developed is part of this Wharton Interim Feasibility Study. Table 32 provides local frequency water surface elevations for Baughman Slough with the proposed levee in place.

Table 32. Baughman Slough Local Event WSELs with Proposed Levee

Location	25-Year Local	50-Year Local	100-Year Local
D/S of RR (19400.54)	99.68'	99.82'	99.94'
D/S of Richmond Road (17629.22)	98.32'	98.32'	98.32'
D/S of Fulton Road (16564.54)	97.11'	97.16'	97.19'
U/S of Alabama Road (13613.13)	94.09'	94.25'	94.39'

Railroad Openings

The abandoned railroad running north to south through the City of Wharton, parallel to Richmond Road, will be utilized as a "levee" to prevent Baughman Slough water from inundating the City of Wharton from the west. A field inspection was made of the railroad embankment between Baughman Slough and Caney Creek. The embankment is in relatively good shape and remains elevated above the surrounding ground. Several openings along the railroad embankment were identified and need to be addressed in order to prevent Baughman Slough overflow in the right overbank from inundating properties east of the railroad.

Two bridge openings and four culverts were identified along the 9,100 feet of railroad embankment between Baughman Slough and Caney Creek, not including the bridges over these two creeks. Photographs of the openings are in Attachment D. All six openings convey flow from the west side of the railroad into a channel that runs along the east side of the railroad embankment with an outfall into Baughman Slough. The upstream limit of the drainage channel on the east side of the railroad is near Bernstein Street, approximately 7,500 feet south of the Baughman Slough outfall.

It is proposed that all six openings be closed to prevent runoff from the west side of the railroad from crossing to the east side. During a five-year event on Baughman Slough, flow exceeds the channel capacity upstream of the railroad and begins to spill into the right overbank. By closing these openings, “backdoor” flooding behind the proposed Baughman Slough levee of areas east of the railroad will be prevented. The flowline of Baughman Slough at the railroad crossing is approximately 90.6’. The existing ditch along the west railroad right-of-way can be graded to convey runoff from west of the railroad into Baughman Slough upstream of the proposed levee. Figure 1 in Attachment D shows the approximate locations of the openings along the railroad, the existing east side drainage channel, and the proposed west side channel. Figure 2 in Attachment D shows the proposed typical cross-sections for the west side railroad ditch. This channel grading work can be completed within the railroad right-of-way.

Proposed Colorado River Sumps/Sluices

Seven Colorado River levee sump areas were identified and analyzed as part of this study. The proposed levee extends upstream of Highway 59 to downstream of the intersection of Alabama Road and S. East Avenue. Figure 3 in Attachment D shows the proposed levee alignment and sump drainage areas. HEC-HMS models were developed for each sump area to generate 2-day 25-, 50-, and 100-year interior runoff hydrographs. Rainfall depths and durations were obtained from TP-40, TP-49, and HYDRO-35. The SCS curve number method was utilized for loss rate computations. Proposed sump area elevation-volume relationships were developed, as well as a rating curve for each sluice considering the 10-year Colorado River tailwater elevation discussed previously. Each individual Colorado River sump is discussed in detail in this section. Additional figures and details related to each sump are in Attachment D.

Wal-Mart/Caney Creek Sump

The area approximately bounded by Highway 59, the proposed levee, and Caney Creek drains south toward the Colorado River via overland sheet flow or in the 18’ bottom width Caney Creek Outfall Ditch along the west side of Wal-Mart. Total drainage of this sump site is approximately 450 acres with the primary land use being pasture and farmland. The proposed sump and sluice structure will be located at the downstream end of the Caney Creek Outfall Ditch. Some excavation/grading will be required in order to increase the storage volume of the sump to prevent flooding of nearby businesses. Two – 7’ x 7’ box culverts were used for the analysis and would require flap gates to prevent Colorado River flow from backing up into the Wal-Mart sump area. Currently, there are 2 – 7’ x 7’ box culverts with flap gates located approximately 1,000 feet downstream of FM 102 along the Caney Creek Outfall Ditch. These existing flap gates and structures could be moved downstream to serve as a sluice through the proposed levee. The Colorado River 10-year tailwater elevation at the sluice outfall is approximately 101.3’. The 100-year interior storm required sump storage is just over 249 acre-feet. Table 33 provides a summary of the interior 100-year frequency storm.

Table 33. Wal-Mart/Caney Creek Sump Summary

100-Year Peak Inflow	1,980 cfs
100-Year Total Inflow Volume	383.8 ac-ft
100-Year Peak Outflow	235 cfs
100-Year Peak Storage	249.5 ac-ft
100-Year Peak WSEL	101.6 ft

Flows within Caney Creek were not considered in the Wal-Mart sump analysis. It was assumed the effects of the upstream Caney Creek dams and ponds would prevent additional flow other than the 450-acre watershed from entering the Wal-Mart sump. Further investigation into the impacts of Caney Creek upstream of the Wal-Mart and Outfall Ditch is recommended for a final design. A detailed hydrologic analysis of the Caney Creek watershed upstream of Wal-Mart may be required at this time. A swale will need to be constructed along the interior side of the levee on the east side of the sump to convey overland sheet flow to the sump and sluice area. The proposed sump excavation/grading will impact one property based on the City of Wharton parcel map. A figure and table showing the results of the proposed Wal-Mart sump analysis are in Attachment D.

Nanya Plastics Sump

The area approximately bounded by Highway 59, West Milam, Oak Street, and Caney Creek drains south toward the Colorado River via overland sheet flow or in a drainage ditch near Wilkes Street. A large portion of the Nanya Plastics Factory drains south along Highway 59 towards the Colorado River, and will need to be directed toward the proposed sump via a swale. Total drainage of this sump site is approximately 440 acres with the primary land use being residential and commercial. The proposed sump and sluice structure will be located near the natural draw to the Colorado River just downstream of the existing oxbow. Some excavation/grading will be required in order to increase the storage volume of the sump to prevent flooding of homes and the elementary school located in the drainage area. Two 60" pipes were used for the analysis and would require flap gates to prevent Colorado River flow from backing up into the Nanya Plastics sump area. The Colorado River 10-year tailwater elevation at the sluice outfall is approximately 100.5'. The 100-year interior storm required sump storage is approximately 355 acre-feet. Table 34 provides a summary of the interior 100-year frequency storm.

Table 34. Nanya Plastics Sump Summary

100-Year Peak Inflow	2,055 cfs
100-Year Total Inflow Volume	430.9 ac-ft
100-Year Peak Outflow	85 cfs
100-Year Peak Storage	356.5 ac-ft
100-Year Peak WSEL	100.8 ft

A swale will need to be constructed along the interior side of the levee to convey the flow from Nanya Plastics Factory to the sump and sluice. Some additional grading will also be required at the sluice outfall to drain this flow into the Colorado River. The proposed sump excavation/grading will impact approximately 21 properties based on the City of Wharton parcel map. A figure and table showing the results of the proposed Nanya Plastics sump analysis are in Attachment D.

Hughes Street Sump

The area approximately bounded by FM 102, Richmond Road, and Caney Creek drains south toward Hughes Street via Caney Creek and is noted as the upper basin. The runoff from this basin is collected in a 48" RCP that runs under Hughes Street and outfalls into an oxbow that drains to the Colorado River. This existing 48" RCP will be replaced with 3-60" pipes as noted in this Wharton Interim Feasibility Study. The area bounded by Azalea Street, Oak Street, Caney Creek, Richmond Road, S. Sheppard Street, and the Wharton landfill is noted as the lower basin and drains via overland sheet flow to the oxbow and eventually to the Colorado River. Total drainage of this sump site is approximately 475 acres with the primary land use being residential and open space. The proposed sump and sluice structure will be located near the natural draw to the Colorado River on the west side of the existing oxbow. Some excavation/grading will be required in order to increase the storage volume of the sump to prevent flooding of homes on Camellia Street and Milam Street. The existing Caney Creek storage capacity upstream of the Hughes Street pipe will also be utilized. Two – 7' x 7' box culverts were used for the analysis and would require flap gates to prevent Colorado River flow from backing up into the Hughes Street sump area. The Colorado River 10-year tailwater elevation at the sluice is approximately 100.0'.

The 100-year interior storm required sump storage is just over 350 acre-feet. Table 35 provides a summary of the interior 100-year frequency storm.

Table 35. Hughes Street Sump Summary

100-Year Peak Inflow	2,110 cfs
100-Year Total Inflow Volume	422.5 ac-ft
100-Year Peak Outflow	100 cfs
100-Year Peak Storage	353.6 ac-ft
100-Year Peak WSEL	100 ft

A swale will need to be constructed along the interior side of the levee to intercept the Camellia Street ditch flow and overland sheet flow and convey it to the sump and sluice area. There is an existing 48” RCP outlet structure located on the east end of the oxbow that will need to be plugged and abandoned. The landfill has a 36” pipe with a flap gate that conveys flow out of the landfill to the 48” outlet structure. The landfill pipe and flap gate will remain. The proposed sump excavation/grading will impact nine properties based on the City of Wharton parcel map. A figure and table showing the results of the proposed Hughes Street sump analysis are in Attachment D.

Ford Street Sump

The area approximately bounded by Sheppard Street, Milam Street, and the railroad embankment drains south toward the Colorado River via overland sheet flow or in the roadside ditches along Ford Street. Total drainage of this sump site is approximately 36 acres with the primary land use being residential and open space. The proposed sump and sluice structure will be located near the natural draw to the Colorado River just upstream of the railroad. Some excavation/grading will be required in order to increase the storage volume of the sump to prevent flooding of homes on Ford Street. A 54” pipe was used for the analysis and would require a flap gate to prevent Colorado River flow from backing up into the Ford Street sump area. The Colorado River 10-year tailwater elevation at the sluice is approximately 97.4’. The 100-year interior storm required sump storage is just over nine acre-feet. Table 36 provides a summary of the interior 100-year frequency storm.

Table 36. Ford Street Sump Summary

100-Year Peak Inflow	250 cfs
100-Year Total Inflow Volume	33.9 ac-ft
100-Year Peak Outflow	135 cfs
100-Year Peak Storage	9.2 ac-ft
100-Year Peak WSEL	99.9 ft

A swale will need to be constructed along the interior side of the levee to intercept the Ford Street ditch flow and overland sheet flow and convey it to the sump and sluice area. Some additional grading will also be required at the sluice outfall to drain this flow into the Colorado River. The proposed sump excavation/grading will impact two properties based on the City of Wharton parcel map. A figure and table showing the results of the proposed Ford Street sump analysis are in Attachment D.

Sunset Street Sump/Richmond Road Storm Drain

The area approximately bounded by Richmond Road, the railroad embankment, and Milam Street drains south toward the Colorado River via overland sheet flow or in the ditch between Sunset Street and the railroad embankment. Total drainage of this sump site is approximately 21 acres with the primary land use being residential. The proposed sump and sluice structure will be located near the natural draw to the Colorado River along the proposed levee. Some excavation/grading will be required in order to increase the storage volume of the sump to prevent flooding of homes on Sunset and Bell Street. A 48" pipe was used for the analysis and would require a flap gate to prevent Colorado River flow from backing up into the Sunset Street sump area. The Colorado River 10-year tailwater elevation at the sluice is approximately 97.2'. The 100-year interior storm required sump storage is 13.6 acre-feet. Table 37 provides a summary of the interior 100-year frequency storm.

Table 37. Sunset Street Sump Summary

100-Year Peak Inflow	190 cfs
100-Year Total Inflow Volume	22.0 ac-ft
100-Year Peak Outflow	30 cfs
100-Year Peak Storage	13.6 ac-ft
100-Year Peak WSEL	97.6 ft

There is an existing storm drain system in the Sunset sump area with a 42" outfall pipe into the Colorado River just upstream of Richmond Road. A flap gate needs to be installed on this 42" outfall to prevent Colorado River flow from backing up into the Sunset area. The drainage area delineation for the Sunset Street sump considered overflow from the Richmond Road storm sewer system. The outfall into the Colorado River will be into a natural swale located on the exterior side of the proposed levee. The proposed sump excavation/grading will impact eight properties based on the City of Wharton parcel map. A figure and table showing the results of the proposed Sunset Street sump analysis are in Attachment D.

Black/Rusk Street Sump

The area approximately bounded by S. East Avenue, Black/Brietling Street, and Elm Street drains west toward the Colorado River via overland sheet flow or in the roadside ditch along the west side of S. East Avenue and is noted as the Black Street sub-basin. The area approximately bounded by Brietling/Black Street, S. East Avenue, Milam Street, and Richmond Road drains south toward the Colorado River via overland sheet flow or in the streets leading to Elm Street and is noted as the Rusk Street sub-basin. A storm drain network located in the Rusk Street sub-basin was assumed to be flowing full with runoff from the old Caney Creek. All runoff within the Rusk Street sub-basin area was assumed to flow via the streets to Elm Street and the proposed swale leading to the proposed sump structure. The Rusk Street sub-basin sump drainage area was delineated considering overland and street flow to Elm Street and the Colorado River.

Total drainage of the Black Street sub-basin sump site is approximately 24 acres with the primary land use being residential and industrial. Total drainage of the Rusk Street sub-basin sump site is approximately 58 acres with the primary land use being commercial and open space. The proposed sump and sluice structure will be located near the natural draw to the Colorado River near Black/Brietling Street. Some excavation/grading will be required, primarily in the park area, in order to increase the storage volume of the sump to prevent flooding of homes and businesses. A 66" pipe was used for the analysis and would require a flap gate to prevent Colorado River flow from backing up into the sump area. A flap gate will also need to be installed on the existing 54" pipe outfall under Rusk Street to prevent Colorado River flow from backing up into the storm drain network. The Colorado River 10-year tailwater elevation at the sluice is approximately 96.6'. The 100-year interior storm required sump storage is just over 20 acre-feet. Table 38 provides a summary of the interior 100-year frequency storm.

Table 38. Black/Rusk Street Sump Summary

100-Year Peak Inflow	625 cfs
100-Year Total Inflow Volume	79.1 ac-ft
100-Year Peak Outflow	300 cfs
100-Year Peak Storage	20.9 ac-ft
100-Year Peak WSEL	100.7 ft

Currently, the runoff within the S. East Avenue west side borrow ditch is discharged into the Colorado River via a storm drain pipe located just south of the Carolyn Street/S. East Avenue intersection. This outfall pipe will be plugged and abandoned with the construction of the levee. Three swales will need to be constructed to convey runoff to the proposed sump. A swale will parallel the levee on the interior, another swale will intercept the S. East Avenue west side ditch flow near Carolyn Street after the removal of the existing storm drain pipe to the Colorado River, and a final swale will intercept and direct flow from Elm Street to the proposed sump area. The proposed sump excavation/grading will impact 11 properties based on the City of Wharton parcel map. A figure and table showing the results of the proposed Black/Rusk Street sump analysis are in Attachment D.

Alabama Road Sump

The area approximately bounded by S. East Avenue, Abell Street, and Alabama Road drains southwest toward the Colorado River via the 12’ x 8’ box culvert located under Alabama Road and a channel/ravine located just downstream of the 12’ x 8’ box culvert. Total drainage of this sump site is approximately 345 acres with the primary land use being residential. The proposed sump and sluice structure will be located at the natural draw to the Colorado River between Alabama Road and the proposed levee, south of Carolyn Street. Some excavation/grading will be required, primarily in the open area at the outfall of the 12’ x 8’ box culvert, in order to increase the storage volume of the sump to prevent flooding of homes and businesses. Two – 7’ x 7’ box culverts were used for the analysis and would require flap gates to prevent Colorado River flow from backing up into the Alabama Road sump area. The Colorado River 10-year tailwater elevation at the sluice is approximately 95.9’. The 100-year interior storm required sump storage is just over 185 acre-feet. Table 39 provides a summary of the interior 100-year frequency storm.

Table 39. Alabama Road Sump Summary

100-Year Peak Inflow	2,120 cfs
100-Year Total Inflow Volume	334.9 ac-ft
100-Year Peak Outflow	665 cfs
100-Year Peak Storage	185.1 ac-ft
100-Year Peak WSEL	97.8 ft

Two swales will need to be constructed to convey runoff to the proposed sump. Both swales will parallel the levee and convey runoff that is not in the Alabama Road box culvert outfall channel to the sluice structure. The proposed sump excavation/grading will impact one property based on the City of Wharton parcel map. A figure and table showing the results of the proposed Alabama Road sump analysis are in Attachment D.

Proposed Baughman Slough Sumps/Sluices

Two Baughman Slough levee sump areas were identified and analyzed as part of this study. The proposed Baughman Slough levee extends along the south bank of Baughman Slough between the railroad and Alabama Road. Figure 3 in Attachment D shows the proposed sump drainage areas. HEC-HMS models were developed for each sump area to generate 2-day 25-, 50-, and 100-year interior runoff hydrographs. Rainfall depths and durations were obtained from TP-40, TP-49, and HYDRO-35. The SCS curve number method was utilized for loss rate computations. The Baughman Slough tailwater is higher than portions of the interior ground surface.

Baughman Slough-RR Sump

The area approximately bounded by Richmond Road and the railroad embankment drains north toward Baughman Slough via overland sheet flow and a trapezoidal channel running along the east side of the railroad embankment. Total drainage of this sump site is approximately 184 acres with the land use split between residential and open pasture. The proposed sump and sluice structure will be located at the outfall of the existing trapezoidal channel. Excavation/grading will be required in order to increase the storage volume of the sump to prevent flooding of homes and businesses located within the drainage area. A 66” pipe was used for the analysis and would require a flap gate to prevent Baughman Slough flow from backing up into the Baughman Slough-RR sump area. The Baughman Slough 25-, 50-, and 100-year tailwater elevations at the sluice are approximately 99.68’, 99.82’, and 99.94’, respectively. Since the tailwaters for each frequency event were higher than the existing ground elevation on the interior of the proposed levee, the sump storage area was designed to contain the entire 100-year inflow volume. The 100-year interior storm required sump storage is approximately 182 acre-feet. Table 40 provides a summary of the interior 100-year frequency storm.

Table 40. Baughman Slough-RR Sump Summary

100-Year Peak Inflow	835 cfs
100-Year Total Inflow Volume	181.9 ac-ft
100-Year Peak Outflow	0 cfs
100-Year Peak Storage	181.9 ac-ft
100-Year Peak WSEL	97.0 ft

The proposed sump excavation/grading will impact one property based on the City of Wharton parcel map. A figure and table showing the results of the proposed Baughman Slough-RR sump are in Attachment D.

Baughman Slough-Ahldag Sump

The Baughman Slough – Ahldag Sump encompasses the Ahldag neighborhood and areas draining to Baughman Slough near Alabama Road. The Ahldag sump area includes several man-made channels that direct runoff into Baughman Slough. The area approximately bounded by Alabama Road, East Wayside Street, and Richmond Road is noted as the lower basin. This area drains north toward Baughman Slough via a new trapezoidal channel flowing between Speed Street and N. Texas Street. The area approximately bounded by Alabama Road, Boling Highway, and Richmond Road is noted as the upper basin. This area drains toward Baughman Slough via a trapezoidal channel that diverts flow under Alabama Road just south of Mulberry Street.

Total drainage area of the lower basin is approximately 361 acres with the land use split between residential and open space. The total drainage area of the upper basin is approximately 444 acres with the primary land use being residential. The proposed sump and sluice structure will be located at the outfall of the existing new trapezoidal channel located in the lower basin. Two – 7’ x 7’ box culverts requiring flap gates will replace the existing opening under Alabama Road at the outfall of the upper basin channel.

Excavation/grading will be required in order to increase the storage volume of the sump to prevent flooding of homes and businesses located on the north and west portions of the drainage area. The storage capacities of the existing channels were included in the analysis. Two – 7’ x 7’ box culverts along with the culverts under Alabama Road were used for the analysis and would require flap gates to prevent Baughman Slough flow from backing up into the Baughman Slough-Ahldag sump area. The Baughman Slough 25-, 50-, and 100-year tailwater elevations at the pond sluice (Sluice 1) are approximately 94.09’, 94.25’, and 94.39’, respectively. The 25-, 50-, and 100-year tailwater elevations at the Alabama Road sluice (Sluice 2) are approximately 93.9’, 94.1’, and 94.2’, respectively. Preliminary calculations and designs were completed for four proposed pond alternatives and one alternative assuming no excavation. Table 41 provides a summary of the interior 100-year frequency storm for each of the proposed pond designs. Figures and tables showing the Baughman Slough-Ahldag sump alternatives are in the Attachment D.

Table 41. Baughman Slough-Ahldag Sump Summary

100-Year Peak Inflow	2,850 cfs
100-Year Total Inflow Volume	818.6 ac-ft
POND 1	
Approximate Required Sump Excavation Volume	1,331,480 CY
100-Year Peak Outflow	0 cfs
100-Year Peak Storage	818.6 ac-ft
100-Year Peak WSEL	93.9 ft
POND 2	
Approximate Required Sump Excavation Volume	662,270 CY
100-Year Peak Outflow	755 cfs
100-Year Peak Storage	524.0 ac-ft
100-Year Peak WSEL	94.8 ft
POND 3	
Approximate Required Sump Excavation Volume	551,280 CY
100-Year Peak Outflow	985 cfs
100-Year Peak Storage	479.6 ac-ft
100-Year Peak WSEL	95.1 ft
POND 4	
Approximate Required Sump Excavation Volume	411,080 CY
100-Year Peak Outflow	1,225 cfs
100-Year Peak Storage	436.0 ac-ft
100-Year Peak WSEL	95.3 ft
NO POND	
Approximate Required Sump Excavation Volume	0 CY
100-Year Peak Outflow	1,370 cfs
100-Year Peak Storage	258.5 ac-ft
100-Year Peak WSEL	95.8 ft

The proposed sump excavation/grading will impact six properties for Ponds 1 and 2 and one property for Ponds 3 and 4, based on the City of Wharton parcel map.

The Corps of Engineers is investigating another option for the Baughman Slough – Ahldag Sump area. The preliminary option proposes a much smaller excavation area and also a relocation of the 2 – 7' x 7' box culverts (outfall of the lower basin) to direct flow under Alabama Road. Figure 4 in Attachment D shows the proposed Corps of Engineers' excavation area and new location of proposed culverts. No 100-year sump water surface elevation or inundation area is available at this time for the Corps' alternative.

Sump/Sluice Analysis Summary

Several sump areas and sluice structures will be needed in the City of Wharton with the construction of the proposed levees. Seven sumps are proposed for the Colorado River levee system. A 10-year Colorado River tailwater was utilized for the sump/sluice analysis. Two sumps are proposed for the Baughman Slough levee system. Several alternatives were presented for the Baughman Slough-Ahldag sump.

In addition to construction of the sumps/sluices, six openings along the abandoned railroad between Baughman Slough and Caney Creek need to be closed. A ditch is proposed within the railroad right-of-way on the west side of the railroad embankment to convey flow north to Baughman Slough upstream of the proposed levee.

The sump/sluice analysis is preliminary and is intended only for a feasibility level study. Time required to evacuate the sump areas was not considered, and may affect the final design of the sump/sluice structures. Areas along Caney Creek upstream of the City of Wharton need to be studied in more detail during the design phase of the project to better understand the impacts at the Wal-Mart/Caney Creek sump.

LOCAL EVENT FLOODING

Following the completion of addressing flooding problems related to overflow of the Colorado River within Wharton, local storm event flood control alternatives were analyzed. Local events independent of the Colorado River have created serious flooding problems as well, especially along Baughman Slough in the northern portions of Wharton. Local storms as frequent as the 2-year and 5-year events escape the Baughman Slough channel, inundating portions of Wharton such as the Ahldag subdivision. Flow escaping into the right overbank of Baughman Slough between the abandoned railroad and Alabama Road (Junior College Road) impacts many neighborhoods and businesses. The Baughman Slough stream length between the railroad and Alabama Road is approximately 1.35 miles. Although this analysis focused on local flooding, it is referring to local in the sense of Wharton County as opposed to the Colorado River. This analysis was focused on preventing flooding from Peach Creek, Caney Creek, and Baughman Slough. It does not address extremely localized flooding such as the channels through the Ahldag subdivision or other localized problem areas within the City of Wharton.

Caney Creek Local Events

Caney Creek areas within the City of Wharton are impacted by local storm events. A large portion of businesses and homes within the City of Wharton are located within the Caney Creek watershed. Potential alternatives were analyzed for the CC-Outfall, CC-Wharton, and CC-Crestmont areas to address flooding issues. Combinations of Caney Creek alternatives were also analyzed to assess the relative impacts of each alternative.

CC-Outfall to Colorado and CC-Wharton Areas

The three most upstream Caney Creek storage areas (CC-US59, CC-59to102, and CC-Outfall) all drain to a 48” pipe (CMP) under Hughes Street for discharge into the Colorado River. The construction of the Caney Creek/Wal-Mart Channel drains a portion of the CC-US59 storage area to the Colorado River as well. During major events, flow overtops Richmond Road and allows water to spill from CC-Outfall to CC-Wharton. The CC-Wharton area is drained by a couple of storm sewer systems with the largest system having an outfall to the Colorado River near Rusk and Elm Streets. A box culvert under Alabama Road drains low lying areas near Santa Fe and Alabama.

No invert elevation or plans exist for the Rusk Street storm sewer system through the CC-Wharton (downtown) storage area. Based on small and inconsistent pipe sizes, age of the system, and most likely undersized inlets, this system was assumed to offer little or no benefits to the CC-Wharton area in the analysis. One alternative investigated was an improvement to the Hughes Street drainage with the construction of a box or larger multiple pipes. The goal was to prevent overflow from occurring across Richmond Road and impacting CC-Wharton. However, investigation into the local storm event hydrographs revealed that the peak water surface elevation for CC-Wharton occurs prior to any Richmond Road overflow. The peak is entirely driven by the local rainfall on the CC-Wharton storage area. Although Hughes Street improvements will lower the water surface elevation in CC-Outfall, they would not impact CC-Wharton peak local water surface elevations.

Improvements to the 48” CMP under Hughes Street that currently drains the CC-Outfall storage area were investigated. The pipe currently extends over 1000’ from near the Hughes Street/Spanish Camp Road intersection to the outfall channel near the Hughes Street/Milam Street intersection. Current maximum capacity of the 48” outfall pipe is approximately 90 cfs with the headwater elevation remaining below the top of Spanish Camp Road.

Before any improvements to the 48" pipe were made, an analysis of the outfall channel capacity insured that additional water would not aggravate flooding problems in the homes along Burleson, Kearney, and Damon Streets along the channel. The channel is capable of passing a flow of 500 cfs with a water surface elevation at the upstream section of less than 100.0'. This maximum water surface elevation will not flood homes along the outfall channel.

The first Hughes Street alternative analyzed was the replacement of the 48" CMP with 3-60" concrete pipes. A maximum height of 60" was dictated by the current flowline of the outfall channel near the Milam/Hughes Street intersection. The surveyed flowline at this point is 93.35'. The top of road elevation at the intersection of Milam/Hughes Street is 100.9'. Two feet of cover was assumed plus the pipe thickness. This alternative assumed that any other utilities under Hughes Street could be relocated within the available right-of-way. The 3-60" pipes can pass up to 400 cfs without the headwater overtopping Spanish Camp Road.

The local event water surface elevations were lowered by nearly two feet in the CC-Outfall storage area with the replacement of the 48" pipe with 3-60" pipes. Table 29 provides a comparison of the LOCAL event water surface elevations under existing conditions and with the proposed Hughes Street alternative.

Replacement of the 48" pipe with 5-60" pipes was simulated to analyze the sensitivity of the CC-Outfall storage area water surface elevation to the outlet structure size. Although 5-60" pipes have the ability to drop the CC-Outfall water surface elevation more, the discharge would exceed the outfall channel capacity and aggravate flooding along Burleson, Kearney, and Damon Streets. The placement of 5-60" pipes under Hughes Street in addition to any existing utilities could pose right-of-way problems as well. Also investigated was the replacement of the 48" pipe with 2-60" pipes instead of three. Results for the local events with 2-60" pipes are shown in Table 30.

Since the proposed Colorado River levee would prevent overflow into the Caney Creek storage areas, the final probabilistic water surface elevations for Caney Creek with the Hughes Street improvements would be equal to the local event elevations summarized in Tables 42 and 43.

Table 42. Caney Creek Local Event Frequency WSELs (ft) with 3-60" Hughes Street Pipes

Frequency	Condition	CC-US59	CC-59to102	CC-Outfall	CC-Wharton
2-Year	Existing	103.1	102.3	101.2	99.6
	Proposed	103.1	102.3	99.4	99.6
	Difference	0	0	1.8	0
5-Year	Existing	104.4	103.1	102.4	100.3
	Proposed	104.4	103.1	100.2	100.3
	Difference	0	0	2.2	0
10-Year	Existing	104.9	103.4	103.0	100.8
	Proposed	104.9	103.4	100.7	100.8
	Difference	0	0	2.3	0
25-Year	Existing	105.2	103.7	103.3	101.1
	Proposed	105.2	103.6	101.1	101.1
	Difference	0	0.1	2.2	0
50-Year	Existing	105.4	103.9	103.4	101.4

Frequency	Condition	CC-US59	CC-59to102	CC-Outfall	CC-Wharton
	Proposed	105.4	103.7	101.5	101.4
	Difference	0	0.2	1.9	0
100-Year	Existing	105.6	104.0	103.4	101.6
	Proposed	105.6	103.9	101.8	101.6
	Difference	0	0.1	1.6	0

Table 43. Caney Creek Local Event Frequency WSELs (ft) with 2-60" Hughes Street Pipes

Frequency	Condition	CC-US59	CC-59to102	CC-Outfall	CC-Wharton
2-Year	Existing	103.1	102.3	101.2	99.6
	Proposed	103.1	102.3	99.8	99.6
	Difference	0	0	1.4	0
5-Year	Existing	104.4	103.1	102.4	100.3
	Proposed	104.4	103.1	100.9	100.3
	Difference	0	0	1.5	0
10-Year	Existing	104.9	103.4	103.0	100.8
	Proposed	104.9	103.4	101.6	100.8
	Difference	0	0	1.4	0
25-Year	Existing	105.2	103.7	103.3	101.1
	Proposed	105.2	103.6	102.2	101.1
	Difference	0	0.1	1.1	0
50-Year	Existing	105.4	103.9	103.4	101.4
	Proposed	105.4	103.8	102.4	101.4
	Difference	0	0.1	1.0	0
100-Year	Existing	105.6	104.0	103.4	101.6
	Proposed	105.6	103.9	102.7	101.6
	Difference	0	0.1	0.7	0

The CC-Wharton storage area (downtown Wharton) was not impacted by the proposed improvement to the Hughes Street drainage system. Due to the fill areas along and within the old Caney Creek channel, a defined conveyance path does not exist through CC-Wharton. A detailed look at the topography through this area reveals a series of depression areas separated by high points formed by roadways or other manmade improvements. A storm sewer system exists in the CC-Wharton area, but it is undersized and no detailed data are available concerning flowlines and inlet sizes. The storm sewer system was ignored for the existing conditions analysis. In order to perform a detailed analysis of the system and its capacity, a survey and inventory would be needed noting the flowlines of all pipes and the location and size of inlets. Even with a detailed analysis, and the inclusion of the system as an outlet for CC-Wharton, flooding would most likely still occur for the larger storm events.

One possible alternative to alleviate local flooding within the CC-Wharton storage area was a connection to the CC-Outfall storage area through the existing railroad grade acting as the boundary between these two storage areas. The connection could occur at the low point of the old Caney Creek channel near the intersection of Sunset and Bolton Streets. The connection would require approximately 300 feet of culvert pipe passing under Sunset Street and the abandoned railroad with an outfall into the old Caney Creek channel near Nelson Street. Some channel improvements would be required to Caney Creek from the railroad outfall to the Hughes Street pipe inlet. These improvements would involve both the Nelson and Harrison Street crossings. Total channel length from the Hughes Street pipe inlet to the proposed railroad outfall is approximately 2,000 feet. Assuming the Hughes Street inlet flowline elevation of 94.40' cannot be adjusted due to the Hughes Street outfall constraints, the channel flowline at the railroad outfall would be 96.40' at a 0.1% slope. Assuming the 300 feet of culvert is at a 0.1% slope as well, the flowline at the Caney Creek inlet near Sunset and Bolton Streets would be 96.7'. Current minimum elevation at this location in the Caney Creek channel is 93.2'. The top-of-road elevation of Sunset over the culvert is 102.3'. The maximum culvert height (diameter) would be four feet to provide adequate cover.

This CC-Wharton alternative assumes that the City of Wharton could take the necessary steps to direct flow to this proposed culvert. This would most likely require some grading and culvert construction to eliminate the series of depression areas along old Caney Creek created by fill and roadways. A preliminary investigation of the topographic map for this area indicates that a culvert at Bolton Road, Richmond Road, Third Street, and Armstrong Road, and some grading would enable approximately 110 acres of area upstream of the Richmond Road and Caney Street intersection to be conveyed to the proposed culvert.

With the addition of the CC-Wharton flow into the CC-Outfall area and ultimately the Hughes Street pipes, an alternative outfall greater than 3-60" pipes would be needed at Hughes Street. The total capacity of the Hughes Street outfall channel is 500 cfs (discussed previously). Three 7'x5' box culverts were rated to replace the current 48" pipe under Hughes Street. Three 7'x5' boxes can pass 450 cfs of flow with a headwater elevation of 101.85'. One potential problem with this alternative is that water currently ponds in the CC-Outfall area within the Caney Creek channel as the outflow is controlled by the Hughes Street pipe size and ultimately the maximum capacity of the outfall channel. Even with maximum outflow allowed from CC-Outfall (500 cfs), water still ponds within the channel. These ponding elevations in CC-Outfall range from 99.0' to 102.0' depending on the frequency storm event. Although this is a decrease in existing water surface elevations for CC-Outfall, these ponding depths would cause a very high tailwater on the proposed railroad outfall culvert, and possibly submergence of the structure. Not only would this greatly restrict the outfall capacity from the CC-Wharton area through the proposed railroad culvert, but flow could actually spill back into the CC-Wharton area from the CC-Outfall area further aggravating flooding problems. Heavy rainfall and flooding conditions would be coincident for both of these Caney Creek sub-basins, so it may be difficult to experience much benefit from an outfall through the railroad connecting CC-Wharton to CC-Outfall. The City of Wharton owns the outfall channel location downstream of Hughes and Milam. This capacity of this channel could be increased if needed, but an increase in Hughes Street box size greater than 3 - 7'x5' boxes may not be feasible due to right-of -way limits and re-location of existing utilities.

Another alternative investigated was a direct connection from CC-Wharton to the Colorado River. A diversion under Richmond Road from the Caney Street/Richmond Road intersection to the Colorado River was analyzed. Approximately 1,350 feet of culvert would be needed under Richmond Road. The same area that could be drained to the CC-Wharton to CC-Outfall connection discussed previously could be collected by this Richmond Road alternative. In addition, the low area near Polk Street and Caney Street could be diverted into the Richmond Road culvert with some inlets and laterals. Channelization of the Caney Creek channel through the CC-Outfall area would not be needed with this alternative. The downside to this proposed Richmond Road culvert is the 1,350 feet of culvert that would be needed and construction along a major thoroughfare in the City of Wharton.

Two-60" pipes were added to the HEC-RAS model to simulate this Richmond Road diversion. Flap gates would be needed to prevent Colorado River flows from backing up into the culverts. Tables 44 and 45 show the Caney Creek local event water surface elevations for the combination Hughes Street improvements and CC-Wharton improvements.

Table 44. Caney Creek Local Event WSELs (ft) w/ 3-60" Hughes Street Pipes and Richmond Road Culverts

Storage Area	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
CC-US59	103.1	104.4	104.9	105.2	105.4	105.6	106.0
CC-59to102	102.3	103.1	103.4	103.6	103.7	103.9	104.1
CC-OutfalltoCR	99.4	100.2	100.7	101.1	101.5	101.8	102.6
CC-Wharton	97.9	99.1	99.7	100.2	100.5	100.8	101.3
CC-HEB	100.2	100.6	101.0	101.1	101.4	101.6	101.8
CC-Crestmont	100.2	100.6	101.0	101.1	101.4	101.6	101.8

Table 45. Caney Creek Local Event WSELs (ft) w/ 3 - 7'x 5' Hughes Street Boxes and Railroad Culverts

Storage Area	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
CC-US59	103.1	104.4	104.9	105.2	105.4	105.6	106.0
CC-59to102	102.3	103.1	103.4	103.6	103.7	103.9	104.1
CC-OutfalltoCR	98.0	99.1	99.7	100.1	100.5	100.8	101.4
CC-Wharton	98.7	99.5	100.1	100.5	100.8	101.1	101.5
CC-HEB	100.2	100.6	101.0	101.1	101.4	101.6	101.8
CC-Crestmont	100.2	100.6	101.0	101.1	101.4	101.6	101.8

One other alternative analyzed for CC-Wharton was a channel paralleling the old railroad right-of-way (ROW) from the low point near Bolton and Sunset Streets to the Colorado River (3,450'). Problems with this alternative include limited ROW between Sunset Streets and the railroad grade, as well as the issue of historic structures in the proposed channel path such as the Wharton Depot. If the channel was only constructed from the intersection of Caney and Sunset Streets to the Colorado River (2,300') to avoid the historic structures, water would still need to be conveyed under Caney Street to the channel from the intersection of Richmond Road and Caney Streets. This distance of storm sewer pipe (1,200') is not significantly less than the required length for the Richmond Road alternative (1,350'), so the Richmond Road alternative would be a shorter distance overall since a channel would not need to be constructed.

CC-Crestmont

The Crestmont neighborhood on the eastside of Wharton has experienced flooding problems from local rainfall events. The Crestmont subdivision is drained by a storm sewer system with an outfall into Caney Creek. At the outfall location, Caney Creek is ponded by a downstream dam, and the outfall pipe remains 75-100% submerged under normal (non-storm) conditions. This high tailwater greatly reduces the hydraulic capacity of the storm sewer system during a storm event. In addition to this problem, the box culvert under Alabama Road also overflows during heavy local events, and this additional water spills into the Crestmont and Mahan subdivisions. For both the existing conditions and alternatives analyses of CC-Crestmont, a starting water surface elevation of 98.0' was used in the HEC-RAS models. This starting water surface elevation simulated the effects of the Caney Creek channel being near capacity at the on-set of a storm due to the downstream impoundments.

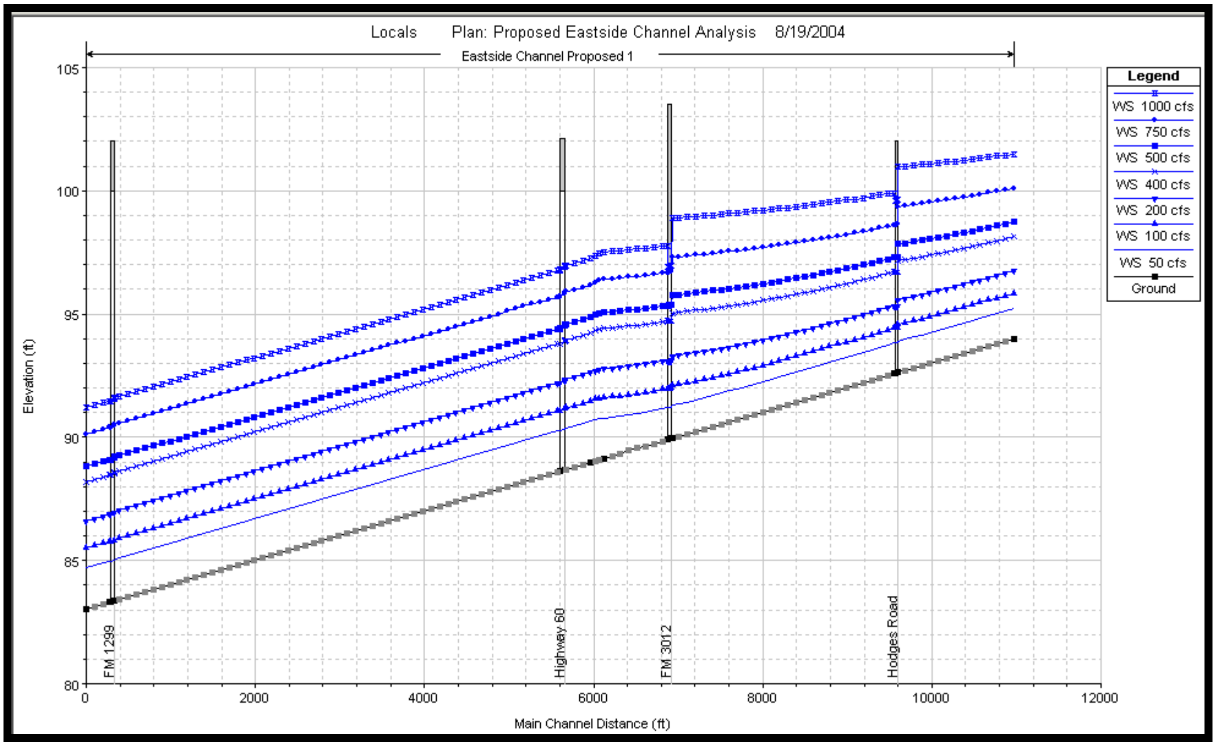
It was proposed that a channel be constructed along the old railroad right-of-way from near Santa Fe and Alabama Roads to the Colorado River. This channel (named Santa Fe Ditch by local Wharton officials) would alleviate some flooding problems related to the Alabama Box and within the Crestmont and Mahan subdivisions. It could also make land southeast of the intersection of Highway 60 and Alabama Road less susceptible to flooding if a storm drain outfall existed. The proposed channel would be approximately 11,000-13,000 feet long depending on a final route alignment. The existing ground elevation at both the upstream and downstream ends of the channel is approximately 100.0'. A flowline of 94.0' was chosen at the upstream end of the proposed channel to provide a total depth of six feet. At a 0.1% channel slope, the flowline at the outfall of the channel into the Colorado River would be 83.0'. The 2-year Colorado River water surface elevation at this location is approximately 83.0'. The proposed channel was assumed to have a 25' bottom width and 3:1 side slopes from the upstream end near Alabama Road and Santa Fe Road to a point 4,850 feet downstream. The lower 6,150 feet of the channel was assumed to have a 15' bottom width with 2:1 side slopes.

Four road crossings would be required with the proposed channel. The FM 2199 and Highway 60 crossings were assumed to be small bridges due to the depth of the channel at these locations. Three 7'x7' box culverts were assumed at both the FM 3012 (Old Lane City Road) and Hodges Road crossings. A preliminary (un-refined) HEC-RAS model with the channel dimensions, elevations, and crossings noted above was created. The model results indicated that this proposed channel has the capability to pass approximately 700 cfs of flow without exceeding the channel capacity near Alabama Road and Santa Fe Road (upstream end). Although this channel geometry and alignment is slightly different than the city's preliminary design, the capacity is comparable at the upstream end of the channel. Figure 15 shows the computed water surface profiles for various flows in the proposed channel.

The construction and benefits of this proposed channel assume that it is feasible for the City of Wharton to direct local drainage and runoff to the proposed channel. No detailed hydrologic analysis or grading plan was completed to determine the best method of directing local runoff into the proposed channel. The excavated soil from the channel construction could be used for the proposed levees if the soil properties are suitable for levee applications. An additional 55 acres along Caney Creek from the Fulton/Caney Street intersection to the Santa Fe/Abell Road intersection from the CC-Wharton storage area could be conveyed to the ponding area and inlet to the proposed Sante Fe Ditch at Alabama/Santa Fe Roads. This assumption of an additional 55 acres would require some grading on the part of the city to convey flow in this reach of Caney Creek.

The current opening from the ponding area to the proposed Santa Fe Ditch is an 8'x 6' box under Alabama Road. This box culvert can currently pass a flow of approximately 320 cfs with a headwater elevation of 100.0' and a corresponding tailwater depth based on the proposed City of Wharton channel geometry (6' bottom width, 4:1 side slopes). Total drainage area to this inlet box is 160 acres (City of Wharton drainage area) bounded by FM 1301, Alabama Road, Santa Fe Road, Fulton, Lazy, and Milburn Roads. If the 55-acre drainage area along Caney Creek in the CC-Wharton storage area is conveyed to this point, the total drainage area to the box would be 215 acres. Some detention is possible in the ponding area, but an elevation above 100.0' does not need to be exceeded due to structure impacts and overtopping of Alabama Road in some locations. The approximate capacity of this ponding area in the City Park is approximately 42 acre-feet at elevation 100.0'. The 8'x 6' box may need to be upsized to prevent ponding depths greater than 100.0' based on the 215 contributing acres of runoff.

Figure 15. Proposed Santa Fe Ditch Approximate WSEL Profiles



In the current overall Wharton hydraulic model, the Caney Creek area is represented as a series of storage areas, including the CC-Crestmont area. There is not an easy and direct method for incorporating the proposed channel into the overall hydraulic model. The Santa Fe Ditch alternative was simulated in the overall Unsteady HEC-RAS model using a storage area connection attached to CC-Crestmont with a weir rated to discharge 500 cfs when the CC-Crestmont water surface elevation reaches 100.0'. This diversion simulates an estimate of the flow that could be carried from the Crestmont storage area via the proposed channel. A flap gate structure would be needed to prevent Colorado River flows from backing into the proposed channel.

A simulation was made with the Santa Fe Ditch capacity noted above and a slightly larger channel (690 cfs capacity at elevation 100.0'). The two Santa Fe Ditch alternatives were simulated in conjunction with 3-7'x5' boxes under Hughes Street and the railroad connection culverts. Tables 46 and 47 provide the LOCAL event results along Caney Creek with the proposed alternatives in place. Assuming that the Colorado River levee will be constructed and prevent Colorado River overflows from inundating the Caney Creek storage areas, these local event water surface elevations would also represent the final probabilistic frequency water surface elevations (except for the 500-year).

Table 46. Caney Creek Local Event WSELs (ft) w/ 3 - 7'x 5' Hughes Street Boxes, Railroad Culverts, & Original Santa Fe Ditch

Storage Area	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
CC-US59	103.1	104.4	104.9	105.2	105.4	105.6	106.0
CC-59to102	102.3	103.1	103.4	103.6	103.7	103.9	104.1
CC-OutfalltoCR	98.0	99.1	99.7	100.1	100.5	100.8	101.4
CC-Wharton	98.7	99.5	100.1	100.5	100.8	101.1	101.5
CC-HEB	99.0	99.3	99.6	99.9	100.1	100.2	100.8
CC-Crestmont	99.0	99.3	99.6	99.9	100.1	100.2	100.8

Table 47. Caney Creek Local Event WSELs (ft) w/ 3 - 7'x 5' Hughes Street Boxes, Railroad Culverts, & Larger Santa Fe Ditch

Storage Area	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
CC-US59	103.1	104.4	104.9	105.2	105.4	105.6	106.0
CC-59to102	102.3	103.1	103.4	103.6	103.7	103.9	104.1
CC-OutfalltoCR	98.0	99.1	99.7	100.1	100.5	100.8	101.4
CC-Wharton	98.7	99.5	100.1	100.5	100.8	101.1	101.5
CC-HEB	99.0	99.3	99.5	99.8	100.0	100.1	100.6
CC-Crestmont	98.9	99.3	99.5	99.8	100.0	100.1	100.6

BAUGHMAN SLOUGH LOCAL EVENTS

As discussed previously, Baughman Slough flooding impacts areas of northern Wharton, especially the Ahldag subdivision. Several Baughman Slough alternatives were analyzed including diversion channels to Peach Creek, channelization, levees, and a combination of alternatives. The goal of these alternatives was to not only prevent Baughman Slough flooding of neighborhoods in northern Wharton, but to also lower the Baughman Slough water surface elevation near Junior College Boulevard for the more frequent events. A lower Baughman Slough water surface elevation at this point will improve the drainage of localized channels that outfall into Baughman Slough such as the Ahldag channels.

Diversion Channel

One of the first alternatives analyzed for Baughman Slough local events was the construction of a diversion channel to Peach Creek. The Peach Creek channel in the area just north of Wharton has the capacity to contain the 100-year local flow along Peach Creek, and could therefore theoretically carry diverted flow from Baughman Slough. The first alternative analyzed was a diversion channel from Baughman Slough to Peach Creek between the abandoned railroad and Business Highway 59. This area is ideal for a channel due to ease of right-of-way acquisition and lack of development. A positive head differential exists between Baughman Slough and Peach Creek for the 100-year local event. The peak Baughman Slough 100-year local water surface elevation is approximately 99.8', while the Peach Creek 100-year local water surface maximum is 96.3'.

A diversion design flow rate of 600 cfs was selected to alleviate some Baughman Slough flooding. A major constraint to a channel alternative in this location is the limited space between the railroad and Business Highway 59. The distance between the centerline of the railroad and the centerline of Business Highway 59 is 100' feet or less in some locations. Based on these constraints a channel with a maximum top width of 50' was selected.

A lateral weir would be needed along Baughman Slough to divert a controlled flow into the diversion channel. Assuming a maximum weir crest length of 50 feet based on channel width constraints, a head of 2.64' would be needed to pass 600 cfs over the weir. With a maximum water surface elevation of 99.82' in Baughman Slough for the 100-year event, the crest of the weir would have to be at a maximum of 97.18'. Therefore, the maximum tailwater elevation (headwater in the diversion channel) could not exceed the crest of the weir. The diversion channel distance from Baughman Slough to Peach Creek is 5,500'. Even at an extremely mild slope of 0.1%, a drop of 5.5' is needed along the channel flowline. A HEC-RAS model was created assuming a channel with a slope of 0.1%, 15' bottom width, and 3:1 side slopes, lined with grass. With the Peach Creek tailwater elevation of 96.3', a flowline of 90.7' was needed at the upstream portion of the channel to obtain a water surface elevation of 97.17' (maximum allowable to prevent tailwater effects on weir) for 600 cfs. The resulting channel flowline near Peach Creek was nearly 15' below the natural ground surface, and could not be tied back into the natural ground with a 3:1 side slope respecting the top width constraint of 50'. A design flow rate of less than 600 cfs does not significantly lower water surface elevations along Baughman Slough. This design was eliminated from further consideration.

Another possible location for the diversion channel discussed at the June meeting was upstream of Wharton along County Road 235 to the old slough that flows into Peach Creek downstream of County Road 235. No detailed channel surveys exist for the old slough, but based on topographic data to the edge of water, the flowline at County Road 235 was assumed to be 102.5'. The distance along County Road 235 to Baughman Slough is 3,300 feet. At a mild slope of 0.1%, the flowline of the diversion channel at Baughman Slough would be 105.8' (102.5' + 3.3'). The 100-year maximum local stage at County Road 235 and Baughman Slough is 106.05'. Considering tailwater effects and required head on a lateral diversion weir, this option does not seem feasible. If a desirable flow was able to be diverted to the old slough, the capacity of the old slough itself would have to be analyzed. The drainage area to the old slough is approximately 525 acres. Any diverted flow would have to be carried by the old slough in addition to the local drainage area flow.

The next alternative analyzed was moving the proposed channel from the downstream side of the railroad to the upstream side of the railroad. This eliminated the limited space associated with a channel between the railroad and Business Highway 59. A top width of 70 feet was assumed for a channel on the upstream face of the railroad. This 70' top width is in conjunction with a total right-of-way acquisition of 100 feet. Once again, problems occurred with tailwater conditions and tying into natural grade near the Peach Creek outlet.

Instead of placing the lateral weir on the banks of Baughman Slough and constructing a channel for 5,500' at 0.1% to Peach Creek, the idea of locating the lateral diversion weir on the overbank was investigated. The upstream face of the railroad was selected again to allow more space for the diversion channel. An inspection of water surface elevations upstream of the railroad revealed that County Road 222 is the drainage divide between Peach Creek and Baughman Slough. This road is not overtopped by the 100-year local event and the maximum water surface elevation just upstream of the railroad is approximately 101.0' to the County Road 222 embankment. Since County Road 222 acts as a levee preventing overflow to Peach Creek and Baughman Slough water is ponded on the left overbank at this point, the idea of constructing an inlet structure and diversion channel at this location was analyzed. This location reduces the diversion channel length from 5,500' to 2,100', and prevents the channel from becoming too deep below the natural ground. An inlet structure with a weir crest length of 50 feet and weir crest elevation of 98.50' would allow 600 cfs to enter the culverts that would need to be constructed under County Road 222. Two 10'x 6' culverts with a flowline of 92.80' would pass the 600 cfs diversion with a headwater elevation of 98.5'. The diversion channel downstream of the culvert was assumed to have a 15' bottom width with 3:1 side slopes at a 0.1% slope with grass lining. The flowline of the channel at the outfall into Peach Creek would be 90.70'. This alternative would not provide relief for the smaller more frequent events along Baughman Slough as the water may not be able to reach the inlet weir for the diversion channel.

Each of these diversion channel alternatives discussed above were analyzed in a conservative manner (no tailwater impacts on weir allowed and perfect coincident peaks on both Baughman Slough and Peach Creek). These diversion alternatives only considered the 100-year event as well.

Another Baughman Slough diversion channel alternative was analyzed that included a lower weir crest elevation in which the tailwater could exceed the crest. The channel was assumed to extend along the upstream face of the abandoned railroad from the left bank of Baughman Slough to the right bank of Peach Creek. The existing flowline of Baughman Slough at this location is approximately 90.6' and the Peach Creek flowline is approximately 75.7'. The diversion channel would span 5,400' at a slope of 0.1% with a 25' bottom width and 3:1 side slopes. The weir crest was assumed to be thirty feet in length with a crest elevation of 93.0'. The upstream invert of the diversion channel was assumed to be at elevation 92.0', with an invert elevation of 86.6' at the Peach Creek confluence (5,400' at 0.1% slope). Two 10'x10' boxes would be needed under CR 222 to pass the flow along the diversion channel. A HEC-RAS steady-state model of the proposed diversion channel was used to determine the water surface elevation at the upstream portion of the channel (tailwater on weir) for various flows. The weir was then rated for various headwater elevations (Baughman Slough) considering the tailwater impacts on the weir. Table 48 provides the rating computed for the weir using U.S. Army Corps of Engineers design charts for submerged weirs. The tailwater elevation was computed by the HEC-RAS model for the given flow rate assuming normal depth in the channel. For the time periods when the Peach Creek water surface elevation is near its peak, the diverted flow may be slightly less than shown due to increased tailwater on the weir.

Table 48. Rating of Baughman Slough Diversion Channel Weir U/S of Railroad to Peach Creek

Headwater Elevation (Baughman Slough WSEL)	Tailwater Elevation (Diversion Channel U/S)	Diverted Flow (cfs)
93.0'	N/A	0
94.0'	93.3'	80
95.0'	94.5'	210
96.0'	95.3'	360
97.0'	96.2'	535
98.0'	97.1'	735
99.0'	97.9'	940
100.'0	98.7'	1,155

An Unsteady HEC-RAS model was then used to simulate the diversions over time from Baughman Slough to Peach Creek for the local storm events. Maximum diversions from Baughman Slough were approximately 1,000 cfs. The diversion channel reduced the local water surface elevations along Baughman Slough from the railroad to Junior College Boulevard (CR 135). Despite the drops in water surface elevations, the flow was still not contained within the Baughman Slough channel at many cross-sections in this reach for events as frequent as the 5-year or 10-year storms. Also, a significant drop in water surface elevation at Junior College Boulevard was not experienced with the diversion channel. This location along Baughman Slough represents the outfall for the Ahldag channels. The additional flow diverted to Peach Creek results in an increase in water surface elevations in that stream. Most of the additional flow is still contained within the Peach Creek channel, but the 100-year local water surface elevation along Peach Creek with the diverted flow does spill into the right overbank at cross-section 62145 between Business Highway 59 and CR 135.

The analysis of the proposed diversion channel between Baughman Slough and Peach Creek indicates that the diversion channel can lower the Baughman Slough local event water surface elevations between the railroad and Junior College Boulevard. However, the diversion channel cannot fully contain the Baughman Slough local events and prevent flooding. The diversion channel also does not significantly lower the Baughman Slough water surface elevation near Junior College Boulevard and the Ahldag outfall channels to aid in local drainage.

Baughman Slough Channelization Options

In addition to a diversion channel alternative for Baughman Slough, channelization options were also investigated. Baughman Slough channelization alternatives were investigated for the area primarily downstream of Business Highway 59 (Richmond Road) to the confluence with Peach Creek. Various combinations of channel bottom width and channelization extents were modeled to gain a better understanding of channelization sensitivity and impacts on water surface elevations. In most locations throughout this area, the existing channel is approximately ten feet deep. Initially, six channelization options along Baughman Slough were analyzed for the 100-year local event. This analysis was made using an older version of HEC-RAS models, so the existing conditions water surface elevations may be slightly different than the latest models. However, this analysis still provides a general sense of the relative effectiveness and sensitivity of the channelization alternatives. Table 49 provides a description of the channel alternatives initially analyzed for hydraulic benefits. Table 50 provides a comparison of 100-year local water surface elevations along Baughman Slough with the various channelization alternatives in place.

Table 49. Baughman Slough Channelization Alternatives Analyzed

Alternative	Option	Baughman Slough Channelization Length
50' Bottom Width; 3:1 Side Slopes; 0.1% Grade from Peach Creek Confluence to Fulton Road	A	3.2 miles
50' Bottom Width; 3:1 Side Slopes; 0.1% Grade from CR 129 to Fulton Road	B	2.9 miles
50' Bottom Width; 3:1 Side Slopes; 0.1% Grade from CR 150 to Fulton Road	C	1.25 miles
25' Bottom Width; 3:1 Side Slopes; 0.1% Grade from Peach Creek Confluence to Fulton Road	D	3.2 miles
25' Bottom Width; 3:1 Side Slopes; 0.1% Grade from Business Highway 59 to CR 150	E	1.5 miles
75' Bottom Width; 3:1 Side Slopes; 0.1% Grade from Business Highway 59 to CR 150	F	1.5 miles

Table 50. Baughman Slough Channelization 100-Year Local WSEL (ft) Comparisons

River Station	Location	Existing	Option A	Option B	Option C	Option D	Option E	Option F	Right Bank Elevation
									-
414.614		88.2	88.8	88.6	88.3	88.4	88.2	88.3	-
1082.54		88.3	89.0	88.9	88.4	88.7	88.3	88.4	-
1702.774	CR 129	88.4	89.1	89.3	88.5	88.8	88.4	88.5	-
1721.793	CR 129	88.5	89.1	89.4	88.5	88.9	88.5	88.5	-
1894.304		88.7	89.3	89.6	88.8	89.1	88.8	88.8	-
2839.493		89.4	89.6	89.9	89.4	89.6	89.4	89.5	-
4213.426		90.0	90.0	90.3	90.0	90.1	90.0	90.0	-
5127.07		90.2	90.2	90.5	90.2	90.4	90.2	90.2	-
6131.145		90.4	90.4	90.7	90.4	90.6	90.4	90.5	-
7105.406		91.1	90.6	91.0	91.1	91.1	91.1	91.2	-
8237.41		92.3	91.2	91.7	92.4	92.0	92.4	92.5	-
8959.325		92.8	91.7	92.0	92.9	92.3	92.8	92.9	-
9489.47		92.8	92.0	92.2	92.9	92.4	92.9	93.0	-

River Station	Location	Existing	Option A	Option B	Option C	Option D	Option E	Option F	Right Bank Elevation
9948.687		92.8	92.1	92.3	92.9	92.5	92.9	93.0	-
10209.35	CR 150	92.8	92.1	92.3	93.0	92.5	92.9	93.0	-
10280.11	CR 150	93.5	93.4	93.5	93.8	93.5	93.8	94.0	92.0
10513.92		93.5	93.4	93.5	93.8	93.5	93.8	94.0	93.4
10993.92		93.7	93.5	93.5	93.8	93.5	93.8	94.0	93.5
11614.54		93.9	93.6	93.6	93.8	93.7	93.9	94.1	93.6
11968.98		94.0	93.6	93.7	93.9	93.8	94.0	94.1	93.6
12515.14		94.2	93.8	93.9	93.9	94.0	94.1	94.1	93.7
12855.04	Alabama	94.3	94.0	94.1	94.1	94.2	94.3	94.1	92.0
12982.9	Alabama	94.5	94.2	94.2	94.3	94.4	94.5	94.5	94.1
13613.13		94.7	94.3	94.3	94.3	94.5	94.7	94.6	94.0
14040.2		94.9	94.3	94.4	94.4	94.6	94.8	94.6	94.0
14730.8		95.5	94.4	94.5	94.5	94.8	95.1	94.8	96.0
15465.96		96.5	94.5	94.6	94.7	95.1	95.7	95.0	96.1
16153.77		96.6	94.7	94.8	94.9	95.6	96.1	95.3	97.3
16564.54		97.0	94.8	94.9	95.1	96.0	96.5	95.5	97.6
16869.3	Fulton	97.5	94.9	95.0	95.2	96.2	96.8	95.6	97.9
16961.81	Fulton	98.0	95.4	95.5	95.8	96.9	97.1	96.9	98.0
17260.74		98.1	96.6	96.7	96.9	97.4	97.3	97.1	97.9
17629.22		98.1	98.1	98.1	98.0	98.0	97.6	97.2	97.5
18033.22	Bus. 59	98.3	98.3	98.3	98.3	98.3	98.0	97.4	97.0
18175.58	Bus. 59	99.8	98.8	98.8	99.8	99.8	99.5	99.1	-
18814.77		99.8	99.8	99.8	99.8	99.8	99.5	99.2	-

The 100-year local water surface elevation dropped by up to two feet in some locations between Alabama Road and Fulton Road. Although the channelization options lowered Baughman Slough water surface elevations, the water still exceeded the right bank elevation in many locations.

The lower water surface elevations could aid in the drainage of extremely localized storms via the Ahldag channels into Baughman Slough, but 100-year levels of protection do not appear evident for the areas along the right bank of Baughman Slough between Business Highway 59 and Alabama Road with channelization alone. Currently, runoff is collected in the Ahldag subdivision via two channels that converge near Alabama Road and flow in a ditch along Alabama to an outfall into Baughman Slough. One proposed city improvement would be the construction of additional diversion channels from the Ahldag subdivision to Baughman Slough just upstream of Alabama Road. One of the proposed diversions would connect to the northern channel in the Ahldag subdivision along Mulberry Street and divert flow to Alabama Road prior to the confluence with the southern Ahldag channel. The other proposed diversion would simply parallel Alabama road and provide additional drainage to Baughman Slough from both of the Ahldag channels. Existing topography of the area and current flowlines of the Ahldag channels would require approximately 2,500’ of channel at a 0.1% slope. The flowline of the outfall at

Baughman Slough would be approximately 84.0’. Preliminary 25-year and 100-year diversion channel designs from the Ahldag subdivision channels to Baughman Slough were developed by the City of Wharton based on a normal depth of 6.5’-7.0’. This normal depth is equal to an approximate water surface elevation at the Baughman Slough confluence of 91.0’ (84.0’ + 7.0’). Should either of these channel layout alternatives be implemented, the 2-year normal depth would be less than 7.0’ and the water surface elevation at the Baughman Slough confluence would be less than 91.0’.

Existing conditions analysis of the Baughman Slough channel indicates that the 2-year water surface elevation at the proposed channel outfall is approximately 93.90’ (cross-section 14040.2). Natural ground elevations along the proposed channel are below this elevation and therefore, the proposed channel would be of little benefit if the Ahldag and Baughman Slough peaks are coincident under existing conditions. Channelization of Baughman Slough would lower the water surface elevation at the proposed channel outfall and would improve the drainage from the Ahldag subdivision during both coincident peaks and Ahldag subdivision peaks. The lowering of the Baughman Slough tailwater would improve Ahldag subdivision drainage along the existing Alabama Road ditch as well.

A preliminary analysis revealed that channelization along Baughman Slough does not have a significant impact on the 100-year local event water surface elevations. However, channelization improvements downstream of Alabama Road can lower the water surface elevations near the Ahldag subdivision existing and proposed outfalls for the smaller, more frequent events. To examine the sensitivity of various channelization options (size and extent) along Baughman Slough, several simulations of the local 2-year storm were made in HEC-RAS. For these sensitivity analyses, existing bridge structures were not altered although widening of the channel would require bridge work. The Manning’s “n” value of the channelized reach was kept equal to existing conditions and all slopes were set at 0.1% projected upstream of the existing downstream cross-section flowline. All side slopes were set at 3:1.

Table 51 provides a summary of the various Baughman Slough channelization options analyzed for sensitivity with the 2-year local storm. All water surface elevations presented are at cross-section 14040.2 (just upstream of Alabama Road near the proposed Ahldag channel outfalls). The existing 2-year local Baughman Slough peak water surface elevation at this location is 93.90’.

Table 51. 2-Year Local Storm Event Results at Cross-Section 14040.2 with Baughman Slough Channelization Alternatives

Bottom Width	Upstream Extent	Downstream Extent	Distance (miles)	# of Bridges	2-Year WSEL	Decrease from Existing WSEL
25’	14730.8	CR 150 (10280.1)	0.85	1	93.5’	0.4’
50’	14730.8	8237	1.25	2	92.8’	1.1’
50’	14730.8	CR 129 (1894.3)	2.45	2	91.9’	2.0’
75’	14730.8	CR 150 (10280.1)	0.85	1	92.8’	1.1’
75’	14730.8	8237	1.25	2	92.4’	1.5’
75’	14730.8	CR 129 (1894.3)	2.45	2	90.1’	3.8’

Tables 50 and 51 indicate that the Baughman Slough channelization alternatives alone cannot prevent flooding. However, the channelization options have the ability to lower the water surface elevation at the Ahldag channel outfalls to aid with local drainage.

Baughman Slough Levee

A small levee would be required along the right bank of Baughman Slough between Business Highway 59 and Alabama Road to prevent Peach Creek from flooding Baughman Slough and the Ahldag subdivision as a result of 100-year Colorado River overflow that entered Peach Creek near Egypt, Texas (See *Baughman Slough Alternatives for Colorado River Overflows* section). Assuming this levee would be in place to prevent Colorado River "backdoor" flooding, it could also have benefit during local storm events along Baughman Slough. As was the case with most sections of the Colorado River levee, the required levee along Baughman Slough would only be a few feet in height to provide 100-year local protection for the northern portions of Wharton. Although the levee would create a rise in water surface elevation along Baughman Slough due to the loss of conveyance area, it would prevent overflow from Baughman Slough from entering the Ahldag subdivision and northern portions of Wharton.

With the levee in place, inundation areas on the left overbank will not change upstream of Business Highway 59. County Road 222 prevents water from overflowing into Peach Creek except for a small amount just upstream of County Road 231. This small overflow occurs for existing conditions as well. The area bounded by Business Highway 59, Baughman Slough, County Road 135, and Peach Creek is the location that would experience the most significant impacts from a levee along the right overbank of Baughman Slough. The levee would prevent flooding on the right overbank of Baughman Slough, but it forces water to spill over County Road 144 which served as a "levee" that did not overtop with existing conditions. However, for both existing conditions and proposed conditions, the 100-year local event causes water to escape from Peach Creek just downstream of Business Highway 59 and flow into this area. This is a complex area to analyze with the current models. The actual drainage divide between Baughman Slough and Peach Creek in this area is along the right bank of Peach Creek. County Road 146 and County Road 144 are high points (barriers to flow) in the area separating Peach Creek and Baughman Slough. A channel exists between these two county roads that carries flow to County Road 135, along County Road 135 to County Road 150, and along County Road 150 to its outfall into Baughman Slough. Although the changes in water surface elevation are minor between existing conditions and proposed conditions, the inundation area could be impacted significantly due to the county road overtopping and effects of the small channel. This area may behave like a storage area or sump during heavy rainfall events. There is currently little development east of County Road 137 to Alabama Road in this area.

The placement of a levee along the right overbank of Baughman Slough may result in the need for sump storage for interior drainage. This analysis did not consider the size or location for this sump storage. Also, flooding already existed for the areas downstream of CR 135 (Alabama Road) along Baughman Slough prior to any levee alternatives. This area outside of the city limits has some development. Water surface elevations will increase by hundredths of a foot at CR 150 with the proposed levee alternatives along Baughman Slough. Table 52 provides a summary of Baughman Slough local event water surface elevations with a levee in place.

Table 52. Baughman Slough Local Storm Event WSELs (ft)with Levees

Cross-Section	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
19918.83 (RR)	99.0	99.4	99.6	99.8	99.9	100.0
19400.54	98.9	99.3	99.5	99.7	99.8	100.0
18814.77	98.8	99.3	99.5	99.7	99.8	99.9
18175.58 Bus 59	98.7	99.2	99.4	99.6	99.8	99.9

Cross-Section	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
18033.22 Bus 59	98.1	98.4	98.4	98.4	98.4	98.5
17629.22	97.8	98.2	98.3	98.3	98.3	98.4
17260.74	97.5	98.1	98.3	98.3	98.3	98.3
16961.81 (Fulton)	97.3	98.1	98.3	98.3	98.3	98.3
16869.3 (Fulton)	97.1	97.8	97.9	97.9	98.0	98.0
16564.54	96.8	97.3	97.5	97.5	97.6	97.6
16153.77	96.5	96.9	97.0	97.0	97.1	97.1
15465.96	96.0	96.4	96.5	96.5	96.6	96.6
14730.8	95.0	95.3	95.4	95.4	95.5	95.6
14040.2	94.1	94.6	94.7	94.9	95.0	95.1
13613.13	93.9	94.3	94.5	94.7	94.8	95.0
12982.9 (Alabama)	93.7	94.1	94.3	94.5	94.6	94.7
12855.04 (Alabama)	93.6	94.0	94.1	94.3	94.4	94.5

These levee options will be used in conjunction with the existing railroad grade to provide protection from Baughman Slough floodwaters entering the City of Wharton. Inspection of the railroad indicates that the lowest elevation is 101.5' between Baughman Slough and the Caney Creek drainage divide. The 100-year local Baughman Slough water surface elevation at the upstream face of the railroad is 101.1'. Any culverts or other openings in the railroad embankment would need to be addressed to prevent Baughman Slough water from flowing behind the proposed Baughman Slough levee.

Baughman Slough Combination Alternatives

The Baughman Slough diversion channel and Baughman Slough channelization alternatives alone do not prevent flooding along the right bank of Baughman Slough for the larger storm events. The channelization alternative does however decrease the Baughman Slough water surface elevation near Alabama Road which could aid in the Ahldag channel drainage during more frequent storms. Since a levee along the right bank of Baughman Slough will be needed to protect the northern portions of Wharton from both the Colorado River "backdoor" flooding and larger local events, a combination alternative was analyzed including both channelization and the levee. With the levee alternative alone, the 100-year water surface elevation in Baughman Slough exceeds the highest point of Junior College Boulevard for the levee connection. The diversion channel and levee combination alternative was not analyzed because the diversion channel did not lower the Baughman Slough water surface elevation at Alabama Road to aid in the Ahldag subdivision drainage. With a levee in place protecting northern sections of Wharton, the diversion channel combination does not provide much additional benefit.

A channelization project of 1.25 miles along Baughman Slough extending from cross-section 14730.8 (between Fulton Road and Alabama Road) to cross-section 8237 (downstream of CR 150) was selected based on the results presented in Table 51. Channel bottom widths of both 75' and 85' were analyzed. Tables 53 and 54 provide the Baughman Slough local event water surface elevations with the combination levee and channelization alternatives in place for the 75' and 85' channel bottom widths, respectively.

Table 53. Baughman Slough Local Storm Event WSELs (ft) with Levee & 75' Channelization Alternative

Cross-Section	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
19918.83 (RR)	99.0	99.5	99.6	99.8	99.9	100.0
19400.54	98.9	99.4	99.5	99.7	99.8	99.9
18814.77	98.7	99.3	99.5	99.7	99.8	99.9
18175.58	98.6	99.3	99.4	99.6	99.8	99.9
18033.22	98.0	98.4	98.5	98.4	98.4	98.5
17629.22	97.5	98.0	98.3	98.3	98.3	98.3
17260.74	96.7	97.6	98.1	98.2	98.3	98.3
16961.81 (Fulton)	96.3	97.3	98.0	98.2	98.3	98.3
16869.3 (Fulton)	96.0	96.9	97.6	97.8	97.8	97.9
16564.54	95.6	96.5	96.9	97.1	97.2	97.2
16153.77	94.8	95.6	96.0	96.2	96.3	96.4
15465.96	92.9	94.0	94.6	94.9	95.1	95.2
14730.8	92.2	93.4	94.0	94.3	94.5	94.6
14040.2	92.0	93.3	93.8	94.2	94.3	94.5
13613.13	92.0	93.2	93.8	94.1	94.3	94.4
12982.9 (Alabama)	91.9	93.1	93.7	94.0	94.2	94.3
12855.04 (Alabama)	91.9	93.1	93.6	93.9	94.1	94.2

Table 54. Baughman Slough Local Storm Event WSELs (ft) with Levee & 85' Channelization Alternative

Cross-Section	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
19918.83 (RR)	99.0	99.5	99.6	99.8	99.9	100.0
19400.54	98.9	99.4	99.5	99.7	99.8	99.9
18814.77	98.7	99.3	99.5	99.7	99.8	99.9
18175.58	98.6	99.3	99.4	99.6	99.8	99.9
18033.22	98.0	98.4	98.5	98.4	98.4	98.4
17629.22	97.5	98.0	98.3	98.3	98.3	98.3
17260.74	96.7	97.6	98.1	98.2	98.3	98.3
16961.81 (Fulton)	96.4	97.3	98.0	98.2	98.3	98.3
16869.3 (Fulton)	96.1	96.9	97.6	97.8	97.8	97.9
16564.54	95.6	96.5	96.9	97.1	97.2	97.2
16153.77	94.9	95.6	96.0	96.2	96.2	96.4
15465.96	92.8	93.9	94.5	94.8	95.0	95.1
14730.8	92.0	93.3	93.8	94.2	94.4	94.5
14040.2	91.9	93.2	93.7	94.1	94.3	94.4
13613.13	91.8	93.1	93.7	94.0	94.2	94.3
12982.9 (Alabama)	91.8	93.0	93.6	94.0	94.1	94.3
12855.04 (Alabama)	91.7	93.0	93.5	93.9	94.0	94.1

The Baughman Slough channelization alternatives create a rise in water surface elevation (compared to existing conditions) downstream of the City of Wharton up to one-foot in some locations. The water surface rise can be decreased through a well-designed channel transition from the 75' or 85' bottom width to natural channel. Although this rise can be decreased through a transition, it is doubtful that a zero rise above existing conditions can be achieved downstream due to the small existing natural channel capacity below the channelization alternative. The channelization would need to be extended to the Peach Creek confluence to eliminate the rise in downstream Baughman Slough water surface elevations. A channel transition and acceptable level of downstream water surface elevation rise can be investigated during final design.

SUMMARY OF ALTERNATIVES ANALYSIS

A levee along the left overbank of the Colorado River from upstream of Highway 59 to the southeastern side of the City of Wharton, and a levee on the right bank of Baughman Slough between the railroad and County Road 135 have the potential to provide protection to virtually all development within the City of Wharton for both a 100-year Colorado River event and a 100-year local event on Baughman Slough. Areas within the Caney Creek storage areas will not be impacted by these levees for local events, but will experience significantly lower water surface elevations for the Colorado River overflows. The levees will not have to be extremely high to offer 100-year levels of protection. The negative impact associated with the levees is the rise in water surface elevations along the bank opposite the levee and downstream. Acceptable levels of rise will need to be noted at all areas impacted to ensure that the proposed levees do not cause problems in other locations outside of the city limits of Wharton.

It is important to note historic elevations at the Wharton gauge. The 500-year event in Wharton is controlled by a Lake Travis release. The peak flow in Austin for this Mansfield Dam release is 365,000 cfs. In 1913 and 1935 (both prior to Mansfield Dam construction), Wharton experienced the highest Colorado River water surface elevations since 1869. In 1913, the water surface elevation at the Wharton gauge was 104.3'. In 1935, the water surface elevation peaked at 103.6' in Wharton. The measured flow at Austin in 1935 was 481,000 cfs (over 120,000 cfs higher than the current 500-year event with Lake Travis in place). These historic water surface elevations along the Colorado River further justify the higher levels of protection that may be offered by a minor increase in levee height, although the upstream extent of the levee may need to increase to prevent overtopping of FM 102 and spills to Caney Creek west of Wharton.

The proposed Colorado River levee will prevent overflows from impacting the Caney Creek storage areas. However, local events may still cause flooding with Caney Creek. Improvements to the Hughes Street outfall pipe were analyzed, and water surface elevations in the CC-Outfall storage area can be dropped by one to two feet with the replacement of the single 48" Hughes Street pipe with two or three 60" pipes. The CC-Wharton storage area storm drain system was ignored in the analysis due to inconsistent pipe sizes and undersized inlets. Alternatives to reduce local flooding in the CC-Wharton area included a connection to the CC-Outfall storage area and further upgrades to the Hughes Street pipes. Another potential alternative for reducing localized flooding within CC-Wharton would be diversion culverts under Richmond Road to the Colorado River. Both of these CC-Wharton alternatives assume that the City of Wharton can restore conveyance of flow to the inlets of these proposed diversions.

The Crestmont subdivision and other neighborhoods on the eastside of Wharton have experienced localized flooding problems as well. One potential alternative analyzed for this area is the construction of a channel along the abandoned railroad right-of-way from near the Alabama and Santa Fe Road intersection to the Colorado River. The Santa Fe Ditch has the capability to drain the eastside of Wharton if local runoff can be collected in the proposed channel.

Several alternatives to address localized flooding along Baughman Slough were investigated. Several Baughman Slough to Peach Creek diversion channel alternatives were analyzed. The diversion channel has the potential to drop the Baughman Slough water surface elevations between the railroad and Alabama Road, but the flow is still not contained within the Baughman Slough channel for the larger storm events. The Baughman Slough water surface elevation at Alabama Road near the Ahldag channel outfall is also not lowered significantly to aid in local drainage with the diversion channel in place.

Several combinations of Baughman Slough channelization extents and channel sizes were analyzed. Channelization alone cannot prevent Baughman Slough flooding, but does have the ability to lower the water surface elevation at the Ahldag channel outfalls by one to two feet.

Construction of a levee along the right bank of Baughman Slough between the railroad and Alabama Road to prevent Colorado River "backdoor" flooding could also protect northern sections of Wharton from localized Baughman Slough flooding. Although the levee can prevent Baughman Slough flooding on the right overbank, the water surface in the creek actually rises slightly over existing conditions due to loss of conveyance resulting from the levees. Channelization would be needed to lower the water surface elevation at the tie-in of the levee and Alabama Road to prevent overtopping. In order to offer protection to the right overbank of Baughman Slough and aid in the localized drainage of the Ahldag subdivision, a combination of levee and Baughman Slough channelization option was analyzed. This combination alternative has the ability to prevent Baughman Slough overflows into the northern section of Wharton, as well as lower tailwater elevations on the Ahldag outfall channels near Alabama Road

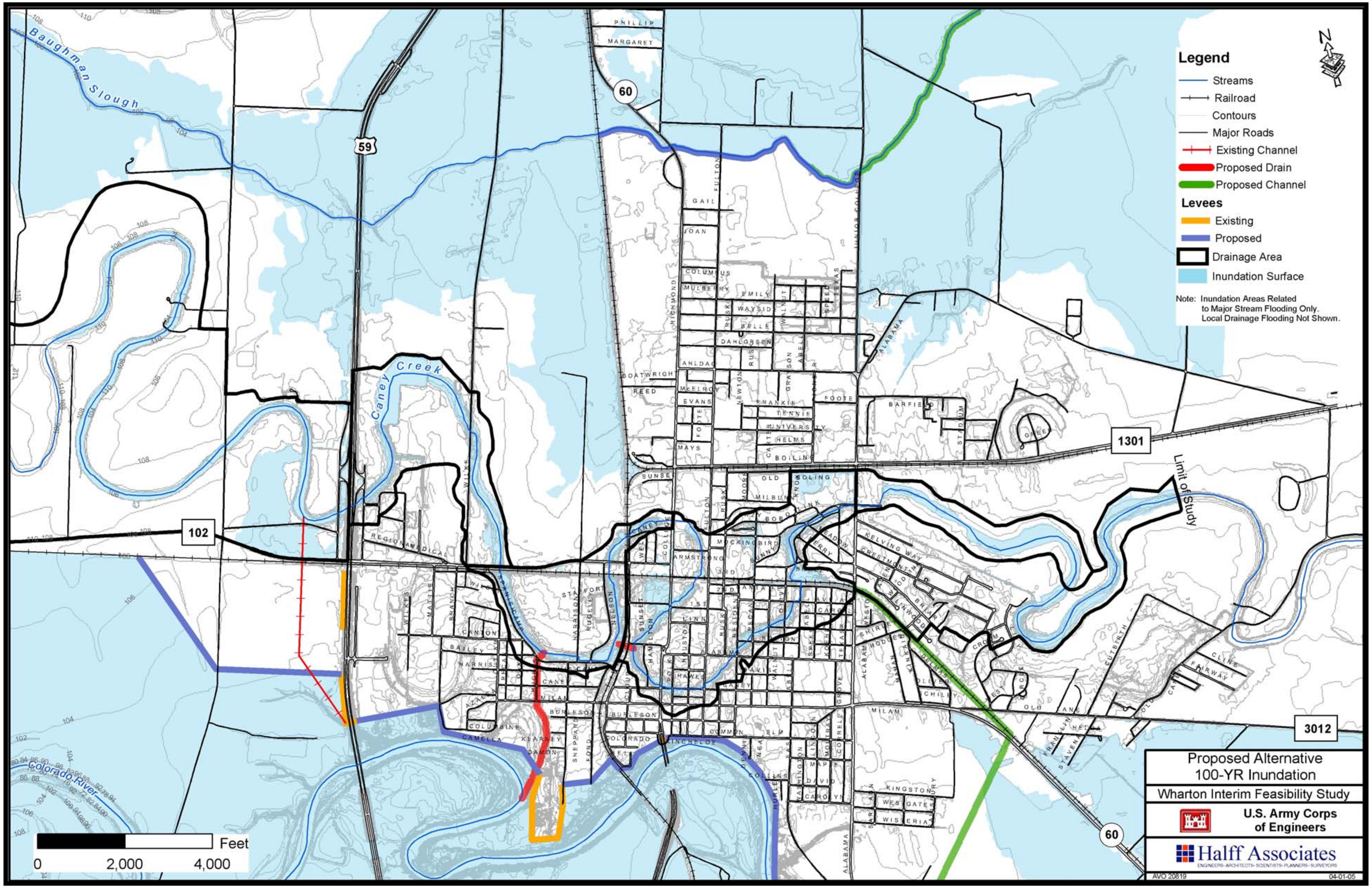
The hydraulic analysis of potential flood control alternatives in the Wharton area is preliminary in nature. This analysis does indicate that the potential exists to reduce flooding up to a 100-year event level in the Wharton area. Figure 16 shows the proposed 100-year inundation area with several of the alternatives in place. Figure 16 does not include the proposed sump inundation areas. This inundation assumes the Colorado River levees, Baughman Slough levees, 75' Baughman Slough channelization, Santa Fe Ditch, 2-60" Richmond Road culverts, and 3 – 60" Hughes Street culverts are in place. Various alternatives' results for all cross-sections within the study area are in Attachment C.

CONCLUSIONS

Wharton, Texas is subject to overflow flooding from the Colorado River, as well as local event flooding along Caney Creek, Baughman Slough, and Peach Creek. The Wharton Interim Feasibility Study included an extensive analysis and evaluation of baseline existing conditions within and near the City of Wharton. Frequency water surface elevations were computed for the Colorado River, Caney Creek, Baughman Slough, and Peach Creek considering the complex hydraulics of the area for both local events and Colorado River overflow flooding.

A wide range of flood control/reduction alternatives were evaluated as part of the Wharton Interim Feasibility Study analysis. The recommended plan includes a combination of levees, channelization, diversion channels, and pipe upgrades. If implemented, the alternatives have the potential to reduce flooding and associated damages for a range of varying frequency storm events in the City of Wharton.

Figure 16. Approximate 100-Year Inundation with Proposed Alternatives



REFERENCES

Asquith, William H. *Depth-duration frequency of precipitation for Texas.* WRI98-4044. USGS, Austin, Texas. 1998.

Clay, Comer. *The Colorado River Raft.* Lower Colorado River Authority. 1948.

CNN. New Archives. Oct. 1998.

Federal Emergency Management Agency. *Wharton County Flood Insurance Study*. November 2001.

Halff Associates, Inc. *Colorado River Flood Damage Evaluation Project Phase I*. July 2002.

Halff Associates, Inc. *Mapping the Colorado River, Technical Support Data Notebook, Wharton County, Texas (Partial)*. Fort Worth, Texas. May 2003.

Houston Chronicle. Newspaper Article Archives. Oct. 1998 and Nov. 2004.

Lower Colorado River Authority. Press Releases. Oct. 1998 and Nov. 2004.

National Oceanic and Atmospheric Administration. *NOAA Technical Memorandum NWS HYDRO-35 Five-to-Sixty Minute Precipitation Frequency for the Eastern and Central United States*. Silver Spring, Maryland. June 1977.

Texas Almanac, 2004-2005. Dallas Morning News, Texas. 2003.

Turk, Kehle, & Associates. *Colorado River Flood Plain at Wharton, Texas*. Austin, Texas. June 1977.

U.S. Army Corps of Engineers, Galveston District. *Flood Plain Information Colorado River and Baughman Slough Wharton, Texas*. December 1970.

U.S. Army Corps of Engineers, Galveston District. *Reconnaissance Report San Bernard River Watershed, Texas.* March 1991.

U.S. Army Corps of Engineers, Galveston District. *Survey Report on San Bernard River, Texas Flood Control*. July 1971.

United States Department of Agriculture. *Technical Paper No. 40 Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years*. Washington, D.C. May 1961.

United States Geological Survey. *Water Resources Data Texas*. Colorado River Basin.

Wharton County and City Officials. Interviews. 2002-2004.

Wharton Fresh Water Resources Conservation & Development Commission. *Streambed Variations Study*. 197